Run-on and Run-off Control System Plan

GenOn Northeast Management Company
Keystone Station Disposal Site
Shelocta, Pennsylvania

GAI Project Number: C151611.01, Task 001

October 2016

Prepared by: GAI Consultants, Inc.
Murrysville Office
4200 Triangle Lane
Export, Pennsylvania 15632-1358

Prepared for: GenOn Northeast Management Company
121 Champion Way, Suite 300
Canonsburg, Pennsylvania 15317

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Report Authors:

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Senior Engineer-in-Training

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Director, Engineering

Table of Contents

Professional Engineer's Certification

The Run-on and Run-off Control System Plan for the Keystone Station Disposal Site was prepared by GAI Consultants, Inc. The Plan was based on certain information that, other than for information GAI originally prepared, GAI has relied on but not independently verified. Therefore, this Professional Engineer's Certification is limited to the information available to GAI at the time the Plan was written. On the basis of and subject to the foregoing, it is my professional opinion as a Professional Engineer licensed in the Commonwealth of Pennsylvania, that the Plan has been prepared in accordance with good and accepted engineering practices as exercised by other engineers practicing in the same discipline(s), under similar circumstances and at the time and in the same locale. It is my professional opinion that the Plan was prepared consistent with the requirements of Section 257.81 of the United States Environmental Protection Agency's "Disposal of Coal Combustion Residuals from Electric Utilities," published in the Federal Register on April 17, 2015 with an effective date of October 19, 2015.

The use of the words "certification" and/or "certify" in this document shall be interpreted and construed as a Statement of Professional Opinion and is not and shall not be interpreted or construed as a guarantee, warranty, or legal opinion.

Kent C. Cockley, P.E.
Engineering Director / Senior Associate

COMMONWEALTH OF PENNSYLVANIA
REGISTERED PROFESSIONAL
Kent C. Cockley
ENGINEER

1.0 Introduction

The Keystone Generating Station is a steam electric generating station located along Crooked Creek in Plumcreek Township, Shelocta, Pennsylvania (PA). The station is jointly owned by the utilities in the Keystone Owners Group, and operated by GenOn Northeast Management Company, a subsidiary of NRG Energy, Inc. The station consists of two 900-megawatt coal-fired units.

The current Keystone Station Disposal Site (disposal site) is permitted under PA Department of Environmental Protection Solid Waste Permit No. 300837 and has been used for the disposal of Keystone Generating Station's Coal Combustion Residuals (CCRs) and coal refuse since 1984. The currently permitted site (233 acres) is a lined disposal area.

The facility consists of four stages of the contiguous East Valley and West Valley:

- Stage I of the East Valley (northern side) was constructed first and became operational in 1984 (soil covered and vegetated);
- Stage II of the East Valley, specifically Stage IIC (southern side), is currently active and completes the existing permitted development in the East Valley;
- Stage III of the West Valley (northern side) is currently active; and
- Stage IV of the West Valley (southern side) is permitted and in construction. Stage IV is an existing CCR unit since it is approved under the current solid waste permit (originally permitted April 2000) and construction activities have been on-going since March 2015.

In accordance with applicable permits, all off-site stormwater run-on and stormwater run-off from soil-covered and vegetated areas will be diverted around the working areas and discharged into unnamed tributaries of Crooked Creek and Plum Creek. All run-off from the active portions of Stage IIC and Stage III, and the future Stage IV area, will be collected in run-off channels and conveyed to equalization ponds prior to treatment at the on-site treatment plant.

2.0 Run-on and Run-off Control System Plan

This Run-on and Run-off Control System Plan (RRCSP) (§257.81) sets forth the techniques that are utilized to minimize stormwater run-on, and divert or collect stormwater run-off during operation of the disposal site. The purpose of the Run-on and Run-off Control System is to limit flow of stormwater run-on from a 25-year, 24-hour storm onto the active portion of the disposal site; and to divert or collect run-off from the soil-covered and vegetated portions and the active portions of the disposal site (resulting from a 25-year, 24-hour storm) during operation. Stormwater controls include the following:

- Temporary/permanent stormwater diversion and collection channels;
- Culverts;
- Slope Drains; and
- Stormwater Equalization Ponds.

All surface run-on along the perimeter of the soil-covered and vegetated Stage I area is combined with stormwater run-off from the soil-covered and vegetated Stage I area and discharged into a stormwater diversion channel that discharges to Plum Creek. The surface run-on along the perimeter of Stage II, Stage III, and the Stage IV area is or will be conveyed to diversion channels that are directed to unnamed tributaries of the Crooked Creek (west side) or Plum Creek (east side).

The run-off channels consist of collection channels for stormwater run-off from the active portions of Stage IIC, Stage III, and Stage IV, and diversion channels for stormwater run-off from soil-covered and vegetated areas. All run-off from active areas is conveyed to the existing West Valley Equalization

Pond south of the disposal site for subsequent treatment, while run-off from soil-covered and vegetated areas is directed to the West Stormwater Management Pond or unnamed tributaries of the Crooked Creek or Plum Creek.

2.1 Stormwater Run-on Control

Stormwater run-on to the disposal site is controlled via diversion features such as diversion channels and culverts.

Most existing East Valley (Stage I and Stage II) diversion channels and culverts were designed for the 100-year, 24-hour storm event. The existing Stage IIC and Stage III drainage facilities and the proposed Stage IV drainage facilities are designed for the 25-year, 24-hour storm event. Stages IIC, III, and IV only slightly changed the drainage to a number of the existing East diversion channels and culverts. However, the existing and permanent facilities will maintain the capacity to pass the 25-year, 24-hour event. Therefore, all diversion channels and culverts have been designed to meet §257.81 of the Federal CCR Rule.

2.1.1 Run-on Channel and Culvert Design

2.1.1.1 Existing Stage I Features

Stage I is the first phase of the East Valley development and is currently soil covered and vegetated. The existing diversion features capture some off-site run-on and stormwater run-off from the soil-covered and vegetated areas. The existing Stage I drainage features are discussed in Section 2.2.1.1.

2.1.1.1 Existing Stage IIC and III Features

The existing run-on control drainage features for Stage IIC and Stage III are designed for the peak flow from a 25-year, 24-hour storm event. Below is a list of the temporary and permanent features designed for the Stage IIC and Stage III developments. The Stages IIC and III temporary features will be buried by development of the Stage IV area:

Temporary Run-on Features

- The Stage III southwest ditch is a Type C-2 channel which diverts discharge from vegetated upland areas to discharge to the stream through a pipe. This channel will be buried by subsequent construction – Drawing No. D-728-1055 and Appendix A (July 1996 Form I, Appendix A).
- Diversion Ditch D33 is a Type A-2 channel and diverted run-on from the western side of the site to discharge to the southwest ditch – Drawing No. D-728-1055 and Appendix A (July 1996 Form I, Appendix A).

Permanent Run-on Features

- The portion of the Southeast ditch developed during Stage III is a Type C-1 channel and diverts run-on for ultimate discharge to an unnamed tributary of Crooked Creek – Drawing Nos. D-728-1055, D-728-1056 and D-728-1058, and Appendix A (July 1996 Form I, Appendix A).
- Haul Road diversion ditch Part 2 is a Type C-3 channel which diverts water from work areas upstream of the southeastern portion of the Stage III Haul Road to discharge through Culvert No. 1 for ultimate discharge to an unnamed tributary of Crooked Creek – Drawing Nos. D-728-1055, D-728-1056 and D-728-1058, and Appendix A (July 1996 Form I, Appendix A).

- The pond diversion channels are sub-divided into two parts:
  - Pond diversion ditch Part 1 is a Type A-2 channel and diverts flow to the Pond diversion ditch Part 2 for ultimate discharge to an unnamed tributary of Crooked Creek – Drawing Nos. D-728-1055, D-728-1056, and D-728-1058, and Appendix A (July 1996 Form I, Appendix A).
  - Pond diversion ditch Part 2 is a Type C-2 channel and diverts flow through Culvert No. 13 for ultimate discharge to an unnamed tributary of Crooked Creek – Drawing Nos. D-728-1055, D-728-1056, and D-728-1058, and Appendix A (July 1996 Form I, Appendix A).

2.1.1.3 Stage IV Disposal Area Development

Construction work to initiate Stage IV started in March 2015. The following sections include drainage features to be constructed for Stage IV subgrade work up to when Stage IV reaches ultimate configuration.

The proposed run-on control drainage features for Stage IV are designed for the peak flow from a 25-year, 24-hour storm event. Below is a list of the temporary and permanent features to be designed for the Stage IV development:

Temporary Run-on Features

- Diversion ditch D41 is a Type A-7 channel that will convey run-on from the southwestern working area to discharge to a spring – Drawing No. D-728-1056, Appendix A (July 1996 Form I, Appendix A).

Permanent Run-on Features

- Culvert Nos. 18 and 19 divert flows from the western side of the Stage III haul road under the proposed Stage IV access road to ultimately discharge to unnamed tributary of Crooked Creek – Drawing No D-728-1058, and Appendix B (Form I Supplemental Calculations for 2013 Stage IV Minor Permit Modification Application).
- The Stage IV Southwest Access Road Diversion Ditch is a Type C-8 channel and diverts flow from the Stage IV Southwest Access Road to discharge to the stream – Drawing Nos. D-728-1056 and D-728-1058, and Appendix B.

2.2 Stormwater Run-off Control

Stormwater run-off from soil-covered and vegetated areas is diverted around the active areas of the site. Stormwater run-off from active areas is collected and treated prior to off-site discharge through a National Pollutant Discharge Elimination System (NPDES) outfall. All stormwater run-off will be managed by run-off controls, such as diversion or collection channels, slope drains, culverts, and equalization ponds.

Most existing East Valley (Stage I and Stage II) run-off channels and culverts were designed for the 100-year, 24-hour storm event. The existing Stages IIC and III drainage facilities, and the proposed Stage IV drainage facilities are designed for the 25-year, 24-hour storm event. Stages IIC, III, and IV only slightly changed the drainage to a number of the existing East run-off channels and culverts. However, the existing facilities will maintain the capacity to pass the 25-year, 24-hour event. Therefore, all run-off channels (diversion and collection) and culverts have been designed to meet Section 257.81 of the Federal CCR Rule.

2.2.1 Run-off Channel and Slope Drain Design

2.2.1.1 Existing Stage I Features

Stage I is the first phase of the East Valley development and is soil covered and vegetated.

The East Peripheral Drainage Channel was developed during the East Valley development for the 100-year, 24-hour storm event and remains the primary diversion feature of stormwater run-off from the existing soil-covered and vegetated East Valley areas. The channel was intended to carry run-off from the top of the East/West valley areas, portions of the landfill benches, and areas within the immediate vicinity of the channel; ultimately discharging into Plum Creek as permitted under NPDES Permit No. PA0002062.

An existing east slope drain is located to the east of the completed Stage I benched area and drains the swale on the completed top of Stage IIC and the Stage I benched area to discharge to the East Peripheral Drainage Channel. The existing East Valley stormwater run-off diversion ditches have been designed to carry the 100-year, 24-hour storm and will continue to be used during the West Valley development.

All existing Stage I collection ditches drain run-off from soil-covered and vegetated areas, and have also been designed to manage the 100-year, 24-hour storm event.

2.2.1.2 Existing Stage IIC and III Features

The existing run-off control drainage features for Stage IIC and Stage III are designed for the peak flow from a 25-year, 24-hour storm event. Below is a list of the temporary and permanent features designed for the Stage IIC and Stage III developments. The Stage IIC and III temporary features will be buried by development of the Stage IV area:

Temporary Run-off Controls from Soil-Covered and Vegetated Areas

- The southeast "top of pile" swale is a Type A-4 channel which diverts drainage from the completed top of Stage IIC to the northeast ditch for ultimate discharge to existing East Valley ditches – Drawing No. D-728-1055 and Appendix A (July 1996 Form I, Appendix A).
- The southeast ditch (during Stage IIC development) is a Type B-2 channel which diverts discharge from the Stage IIC slope drain to the south ditch for ultimate discharge to an unnamed tributary of Crooked Creek. This channel will be buried by subsequent construction – Drawing No. D-728-1055 and Appendix A (July 1996 Form I, Appendix A).
- The south ditch is a Type C-2 channel which diverts flow from the southeastern side of the Stage III Haul Road to the Haul Road diversion ditch for ultimate discharge to an unnamed tributary to Crooked Creek. This channel will be buried by subsequent construction – Drawing No. D-728-1055 and Appendix A (July 1996 Form I, Appendix A).
- The north temporary diversion channel is a Type A-6 channel that conveys drainage through the north temporary diversion culvert to a stream – Drawing No. D-728-1055 and Appendix A (July 1996 Form I, Appendix A).
- The east temporary diversion ditch is a Type C-1 channel which diverts flows from the eastern side of the work area through Culvert No. 2 that discharges to an unnamed tributary of Crooked Creek – Drawing No. D-728-1055 and Appendix A (July 1996 Form I, Appendix A).

- Diversion Ditch D31 is a Type A-7 and C-6 channel that conveys flow from an existing slope drain on the completed benches of Stage II to discharge to the West Stormwater Management (SWM) Pond. The C-6 channel portion of the diversion ditch will be a permanent feature of the ultimate configuration – Drawing Nos. D-728-1055, D-728-1056, and D-728-1058, and Appendix A (July 1996 Form I, Appendix A).
- Diversion Ditch D32 is a Type A-2 channel which diverts flow from the western edge of liner of the East Valley to discharge to the stream – Drawing No. D-728-1055 and Appendix A (July 1996 Form I, Appendix A).
- The east slope drain conveys drainage north of the east ditch to the northeast ditch for ultimate discharge to existing East Valley ditches – Drawing No. D-728-1056 and Appendix A (July 1996 Form I, Appendix A).
- The northeast ditch is a Type C-2 channel which diverts flow from the completed top of Stage IIC to the east ditch for ultimate discharge to existing East Valley ditches. This channel will be buried by subsequent construction – Drawing No. D-728-1056 and Appendix A (July 1996 Form I, Appendix A).
- The east ditch is a Type A-5 channel which conveys flow from the northeast ditch to existing East Valley ditches. This channel will be buried by subsequent construction – Drawing No. D-728-1056 and Appendix A (July 1996 Form I, Appendix A).
- Haul Road diversion ditch Part 1 is a Type C-1 channel which diverts flow from the upstream work areas located northwest of the Stage III Haul Road to the west ditch for ultimate discharge to an unnamed tributary of Crooked Creek. This channel will be buried by subsequent construction – Drawing No. D-728-1056 and Appendix A (July 1996 Form I, Appendix A).

<u>Temporary Run-off Controls from Active Areas</u>

- A Stage IIC collection ditch along the reversed bench was designed to convey runoff from the Stage IIC area (through Culvert No. 11) to the existing East Valley Haul Road Ditch. – Drawing No. D-728-1055 and Appendix A (July 1996 Form I, Appendix H).
- The north collection water ditch is a Type C-2 channel. Its flow is conveyed through the west collection water ditch for ultimate discharge to the West Valley Equalization Pond – Drawing Nos. D-728-1055 and D-728-1056 and Appendix A (July 1996 Form I, Appendix A).
- The west collection water ditch is a Type D-2 channel which conveys discharge to the West Valley Equalization Pond – Drawing No. D-728-1055 and Appendix A (July 1996 Form I, Appendix A).
- The haul road collection water ditch Part 1 is a Type C-5 channel placed in Stage 3. The drainage is conveyed adjacent to the existing Stage III haul road for ultimate discharge to the West Valley Equalization Pond – Drawing No. D-728-1056 and Appendix A (July 1996 Form I, Appendix A).

Permanent Run-off Controls from Soil-Covered and Vegetated Areas

- The west ditch is a Type C-2 channel and diverts water from the west side of the site to the West SWM Pond – Drawing Nos. D-728-1055, D-728-1056, and D-728-1058, and Appendix A (July 1996 Form I, Appendix A).
- The existing East Valley West Side Collection Channel is a Type C-6 channel and conveys flow around the southeast toe of the ultimate landfill development, to the existing east valley ditches – Drawing Nos. D-728-1055, D-728-1056, and D-728-1058, and Appendix A (July 1996 Form I, Appendix A).
- The existing East Valley Haul Road Ditch is a Type C-2 channel and diverts flow from the western side of the existing Stage II Haul Road for ultimate discharge to existing East Valley ditches – Drawing Nos. D-728-1055, D-728-1056, and D-728-1058, and Appendix A (July 1996 Form I, Appendix A).

Permanent Run-off Controls from Active Areas

- The haul road collection water ditch Part 2 is a Type D-4 channel completed during Stage III subgrade development but will be a permanent feature through Stage IV. The ditch will drain runoff from the Stage III Haul Road during Stage III and Stage IV developments for ultimate discharge to the West Valley Equalization Pond – Drawing Nos. D-728-1055, D-728-1056, D-728-1058, and Appendix A (July 1996 Form I, Appendix A).

2.2.1.3 Stage IV Disposal Area Development

Construction work to initiate Stage IV started in March 2015. The following sections include drainage features to be constructed for Stage IV subgrade work up to when Stage IV reaches ultimate configuration.

The proposed run-off control drainage features for Stage IV are designed for the peak flow from a 25-year, 24-hour storm event. Below is a list of the temporary and permanent features to be designed for the Stage IV development:

Temporary Run-off Features from Active Areas

- Temporary Culvert No. 16 will convey run-off from the Stage IV haul road for ultimate discharge to the West Valley Equalization Pond – Appendix B.

Permanent Run-off Features from Soil-Covered and Vegetated Areas

- The north "top of pile" swale is a Type A-4 channel and diverts flow from the northern top of the disposal site to ultimately discharge to the existing East Valley East Peripheral Drainage Ditch – Drawing Nos. D-728-1056 and D-728-1058, and Appendix A (July 1996 Form I, Appendix A).
- The north ditch will be a subdivided into three parts:
    - North ditch Part 1 will be a Type A-1 channel and will divert flow from the north side of the site to the existing East Valley East Peripheral Ditch – Drawing Nos. D-728-1056 and D-728-1058, and Appendix A (July 1996 Form I, Appendix A).
    - North ditch Part 2 will be a Type C-1 channel and will divert flow from the north side of the site to the existing East Valley East Peripheral Ditch – Drawing Nos. D-728-1056 and D-728-1056, and Appendix A (July 1996 Form I, Appendix A).

  - North ditch Part 3 will be a Type C-2 channel and will divert flow from the north side of the site to the existing East Valley East Peripheral Ditch – Drawing Nos. D-728-1056 and D-728-1058, and Appendix A (July 1996 Form I, Appendix A).
- The north-west ditch will be a Type A-2 channel and will divert flow from a north-western bench of the site to the existing East Valley Peripheral Ditch – Drawing Nos. D-728-1056 and D-728-1058, and Appendix A (July 1996 Form I, Appendix A).
- The south-west ditch is sub-divided into two parts:
  - South-west ditch Part 1 will be a Type A-3 channel and will divert flow from the benched area located south of the haul road, through Culvert No. 15 to discharge to an unnamed tributary of Crooked Creek – Drawing No. D-728-1058 and Appendix A (July 1996 Form I, Appendix A).
  - South-west ditch Part 2 will be a Type C-2 channel and will divert flow from the benched area located south of the haul road to an unnamed tributary of Crooked Creek – Drawing No. D-728-1056 and Appendix A (July 1996 Form I, Appendix A).
- At Stage IV disposal area development closure, the run-off channel at the south "top of pile" swale, designed to collect run-off from active areas, will be converted to a diversion Type B-1 channel designed to divert run-off from soil-covered and vegetated areas. The channel will divert run-off across the top of the Stage IV disposal area for ultimate discharge to an unnamed tributary at the Stage IV embankment toe – Drawing No. D-728-1058 and Appendix A (July 1996 Form I, Appendix A).
- The south-east slope drain will be located southeast of the completed Stage IV top and will discharge to existing east valley ditches – Drawing No. D-728-1058 and Appendix A (July 1996 Form I, Appendix A).
- The west slope drain will be located on the western side of the landfill to drain the benches for ultimate discharge to an unnamed tributary of Crooked Creek – Drawing No. D-728-1058 and Appendix A (July 1996 Form I, Appendix A).
- Southeast ditch to be developed during Stage IV is a Type C-1 channel and diverts flow around the West Valley Equalization Pond for ultimate discharge to an unnamed tributary of Crooked Creek – Drawing Nos. D-728-1056, and D-728-1058, and Appendix A (July 1996 Form I, Appendix A).

<u>Permanent Run-off Features from Active Areas</u>

- The haul road collection water ditch Part 1 completed in Stage IV will be a Type C-4 channel and will be conveyed under the existing Stage IV haul road to the south collection ditch – Drawing No. D-728-1058 and Appendix A (July 1996 Form I, Appendix A).
- The south collection water ditch is a Type C-2 channel and discharges through Culvert No. 14 to ultimate discharge to the West Valley Equalization Pond – Drawing Nos. D-728-1056 and D-728-1058, and Appendix A (July 1996 Form I, Appendix A).

2.3 Pond Designs

The existing West SWM Pond is permitted by NPDES Permit No. PA0002062. The West SWM Pond is designed to control stormwater that flows to the west to a culvert beneath Route 210 and to reduce the post-development flows to pre-development flows for the two-year, 10-year, 25-year, and 100-year, 24-hour storm events. The outlet structures have also been designed to manage flow for the 25-year, 24-hour storm event and provide one-foot of freeboard for pre-development and post-development conditions (refer to Appendix A – Form I, Appendix B).

The West Valley Equalization Pond was designed to store the 10-year, 24-hour year storm and handles the 25-year, 24-hour storm volume through the principal spillway (Refer to Form I, Appendix E).

The existing East Valley Equalization Pond is currently idle and unused, but was designed to store the runoff from two 10-year, 24-hour storm events separated by a 24-hour pumping period. Its emergency spillway was designed to pass the peak discharge from the 100-year, 24-hour storm (refer to Appendix A – Form I, Appendix F).

Therefore, the designs for these existing features comply with the federal requirement of handling the 25-year, 24-hour storm as stated in Section 257.81 of the CCR Rule.

2.4 Plan Amendment

The initial RRCSP can be amended (257.81(c)(2)) at any time, and must be amended whenever there is a change in conditions that would substantially affect the written plan. In addition, a plan must be prepared every five years (257.81(c)(4)). Plan must be included into the facilities operating record (257.105(g)(3)).

3.0 References

GAI Consultants, Inc., Keystone Generating Station West Valley Disposal Site, West Side Pump Station and Stage IV-A Liner Construction, May 2015.

Minor Permit Modification Residual Waste Permit # 300837, Stage IV Leachate Improvements. Form I Supplemental Calculations, July 2013.

Pennsylvania Department of Environmental Protection, Residual Waste Major Permit Modification, Keystone Station Disposal Site, Form I Soil Erosion and Sedimentation Controls, July 1996.

United States Environmental Protection Agency (USEPA) 40 CFR Parts 257 and 261 Hazardous and Solid Waste Management Disposal System; Disposal of Coal Combustion Residual from Electric Utilities, Final Rule April 2015.

DRAWINGS

STAGE III HAUL ROAD CURVE DATA

CURVE NO.	P.I. COORDINATES		△	D	R	T	L	P.C. STA.	P.T. STA.
	NORTH	EAST							
BEGIN	489800.27	1527965.25	–	–	–	–	–	–	0+00
1	490151.89	1528183.22	57°17'43"	19°05'54"	300.00	163.89	300.00	2+49.99	5+49.99
2	490609.35	1527922.90	10°44'23"	14°19'10"	400.13	37.61	75.00	8+74.98	9+49.98
3	490708.55	1527830.43	15°53'37"	17°39'35"	324.45	45.29	90.00	10+02.72	10+92.72
4	490830.85	1527766.99	31°49'07"	17°39'30"	324.47	92.48	180.19	10+92.72	12+72.91
5	492216.17	1527873.55	74°57'16"	19°05'56"	299.99	230.00	392.45	23+39.83	27+32.28
6*	492555.02	1526913.72	118°21'31"	31°56'40"	179.36	300.63	370.51	32+19.54	35+90.05
7*	493035.93	1527444.16	07°24'33"	10°20'39"	553.90	35.86	71.63	39+69.53	40+41.16
8*	493579.72	1527906.87	35°49'34"	18°33'07"	308.84	99.83	193.11	46+19.48	48+12.59
END	493654.40	1528211.39	–	–	–	–	–	50+26.31	–

*SHOWN ON DRAWING D-728-1056

LEGEND

- STREAM ENCLOSURE PIPE
- FABRIC FORM SLOPE DRAIN — SLOPE DRAIN
- EXISTING COLLECTION DITCH
- PROPOSED COLLECTION DITCH
- EXISTING DIVERSION DITCH
- PROPOSED DIVERSION DITCH
- RIGHT OF WAY LINES
- PROPERTY LINES (KEYSTONE OWNERS)
- PROPERTY LINES (OTHERS)
- EXISTING PERMIT AREA (STAGE I AND II)
- NEW PERMIT AREA (STAGE III AND IV)
- EXISTING PERMIT LINER FOOTPRINT
- L — EXISTING EAST VALLEY LEACHATE PIPELINE
- K-58 1291.26 — SURVEY MONUMENT AND ELEVATION
- BUILDING IN USE
- X / XXXX — DETAIL NUMBER OR SECTION LETTER / SHOWN ON DRAWING NUMBER

REFERENCE:
MAPPING COMPILED BY TVGA, INC. LANSE PENNSYLVANIA USING STEREOPHOTOGRAMMETRIC METHODS FROM 1"=1333' SCALE AERIAL PHOTOGRAPHY 04/08/94.

HORIZONTAL CONTROL IS BY A SURVEYED TIE TO THE PENNSYLVANIA STATE COORDINATE SYSTEM, SOUTH ZONE (1983 DATUM). VERTICAL CONTROL IS BASED ON THE NATIONAL GEODETIC VERTICAL DATUM.

CAD FILE: KE-F4012.DWG 6/09/98

NOTES:

1. CONTOUR INTERVALS SHOWN AT TEN(10) FEET.
2. FINISHED GRADE CONTOURS DEPICT TOP OF SOIL COVER OR FINISHED ROAD SURFACE.
3. SEE DRAWING D-728-1065 FOR TYPE A, B, C, AND D CHANNEL DETAILS AND SLOPE DRAIN DETAILS.
4. SEE DRAWING D-728-1061 FOR CHANNELS ON EDGE OF LINER DETAILS.
5. SEE DRAWING D-728-1064 FOR CHANNEL DETAILS AT EDGE OF HAUL ROAD AND DRAWING D-728-1063 FOR HAUL ROAD PROFILES.
6. SEE DRAWING D-728-1066 FOR RIPRAP OUTLET PROTECTION DETAILS.
7. SEE DRAWING D-728-1068 FOR CULVERT DETAILS AND SCHEDULE.
8. SEE DRAWING D-728-1057 FOR LINER LIMITS, SUBGRADE DRAIN PIPES, LEAK DETECTION PIPES AND LEACHATE COLLECTION PIPES.
9. FOR LEAK DETECTION, LEACHATE COLLECTION PIPES HEADERS AND LEACHATE PIPELINE TO STATION IWT SEE DRAWINGS D-728-1056, D-728-1071, D-728-1075, D-728-1076, AND D-728-1077.
10. SEE DRAWING D-728-1070 FOR STAGE IIC ROAD CURVE DATA.

REV	DATE	MADE	ENGR	SUPV/MGR	DESCRIPTION	
2	5/27/98	FJC	JMJ	JDP	REVISED STAGE IIC AND EQUALIZATION POND	GAI CONSULTANTS, INC. 413472
1	4/25/97	FJC	JMJ	JDP	ADDED STAGE IIC DEVELOPMENT	GAI CONSULTANTS, INC. 413470

GAI CONSULTANTS
570 BEATTY ROAD
MONROEVILLE, PA 15146
(412)856-6400

GAI Dwg. File No. 92-220-F4012

	DRAFTING MADE	DRAFTING CHECKED	ENGINEERING ENG.	ENGINEERING MGR.
	JPS	JMJ	JMJ	JDP
DATE	7-26-96	7-30-96	7-30-96	7-30-96

Genco GPU Generation Corp. 1001 BROAD STREET JOHNSTOWN, PA. 15907

KEYSTONE STATION UNITS 1 & 2
KEYSTONE STATION DISPOSAL SITE
PLUMCREEK TOWNSHIP

DWG. TYPE: CIVIL

TITLE: RESIDUAL WASTE PERMIT DRAWING
STAGE IIC PILE DEVELOPMENT
AND STAGE III SUBGRADE PLAN

PROJECT CONTROL NO	SCALE	DRAWING NUMBER	REV
413330	1"=200'	D-728-1055	2

STAGE III HAUL ROAD CURVE DATA

CURVE NO.	P.I. COORDINATES NORTH	P.I. COORDINATES EAST	△	D	R	T	L	P.C. STA.	P.T. STA.
BEGIN	489800.27	1527965.25	–	–	–	–	–	–	0+00
1	490151.89	1528183.22	57°17'43"	19°05'54"	300.00	163.89	300.00	2+49.99	5+49.99
2	490609.35	1527922.90	10°44'23"	14°19'10"	400.13	37.61	75.00	8+74.98	9+49.98
3	490708.55	1527830.43	15°53'37"	17°39'35"	324.45	45.29	90.00	10+02.72	10+92.72
4	490830.85	1527766.99	31°49'07"	17°39'30"	324.47	92.48	180.19	10+92.72	12+72.91
5	492216.17	1527873.55	74°57'16"	19°05'56"	299.99	230.00	392.45	23+39.83	27+32.28
6	492555.02	1526913.72	118°21'31"	31°56'40"	179.36	300.63	370.51	32+19.54	35+90.05
7	493035.93	1527444.16	07°24'33"	10°20'39"	553.90	35.86	71.63	39+69.53	40+41.16
8	493579.72	1527906.87	35°49'34"	18°33'07"	308.84	99.83	193.11	46+19.48	48+12.59
END	493654.40	1528211.39	–	–	–	–	–	50+26.31	–

LEGEND

- SLOPE DRAIN
- EXISTING COLLECTION DITCH
- PROPOSED COLLECTION DITCH
- EXISTING DIVERSION DITCH
- PROPOSED DIVERSION DITCH
- RIGHT OF WAY LINES
- PROPERTY LINES (KEYSTONE OWNERS)
- PROPERTY LINES (OTHERS)
- EXISTING PERMIT AREA (STAGE I AND II)
- NEW PERMIT AREA (STAGE III AND IV)
- EXISTING PERMIT LINER FOOTPRINT
- EXISTING EAST VALLEY LEACHATE PIPELINE
- K-58 1291.26 SURVEY MONUMENT AND ELEVATION
- STREAM ENCLOSURE PIPE
- TEMPORARY 8" PVC SOLID WALL LEACHATE HEADER PIPE AND TEMPORARY 6" PVC SOLID WALL LEAK DETECTION PIPE (BUILT DURING STAGE IIIA LINER CONSTRUCTION)
- BUILDING IN USE
- LEACHATE FORCE MAIN (3)

REFERENCE:
MAPPING COMPILED BY TVGA, INC. LANSE PENNSYLVANIA USING STEREOPHOTOGRAMMETRIC METHODS FROM 1"=1333' SCALE AERIAL PHOTOGRAPHY 04/08/94.

HORIZONTAL CONTROL IS BY A SURVEYED TIE TO THE PENNSYLVANIA STATE COORDINATE SYSTEM, SOUTH ZONE (1983 DATUM). VERTICAL CONTROL IS BASED ON THE NATIONAL GEODETIC VERTICAL DATUM.

NOTES:

1. CONTOUR INTERVALS SHOWN AT TEN(10) FEET.
2. FINISHED GRADE CONTOURS DEPICT TOP OF SOIL COVER OR FINISHED ROAD SURFACE.
3. SEE DRAWING D-728-1065 FOR TYPE A, B, C, AND D CHANNEL DETAILS AND SLOPE DRAIN DETAILS.
4. SEE DRAWING D-728-1061 FOR CHANNELS ON EDGE OF LINER DETAILS.
5. SEE DRAWING D-728-1064 FOR CHANNEL DETAILS AT EDGE OF HAUL ROAD AND DRAWING D-728-1063 FOR HAUL ROAD PROFILES.
6. SEE DRAWING D-728-1066 FOR RIPRAP OUTLET PROTECTION DETAILS.
7. SEE DRAWING D-728-1068 FOR CULVERT DETAILS AND SCHEDULE.
8. SEE DRAWING D-728-1057 FOR LINER LIMITS, SUBGRADE DRAIN/LEAK DETECTION PIPES AND LEACHATE COLLECTION PIPES.
9. TEMPORARY SCHEDULE 40 LEACHATE AND LEAK DETECTION/SUBGRADE DRAIN HEADER PIPES TO BE GROUTED SHUT AND ABANDONED UPON INSTALLATION OF PERMANENT PIPES IN STAGE IV.
10. THE EXISTING 5' LONG x 18' WIDE OVERFLOW WEIR CREST SHALL BE RECONSTRUCTED WITH CULVERT 12 AND A TYPE C-2 OUTLET CHANNEL AS SHOWN ON DRAWING D-728-1071 REV. 5, TO ALLOW NEW CLEANOUT ACCESS ROAD CROSSING. (3)

REV	DATE	MADE	ENGR	SUPV/MGR	DESCRIPTION		
3	7-9-13	JCN	DMD	JMJ	ADDED STAGE IV CLEANOUT ACCESS ROAD AND CULVERTS 12-17	GAI CONSULTANTS, INC.	–
2	4-25-97	FJC	JMJ	JDP	REVISED NORTHEAST STAGE III DEVELOPMENT	GAI CONSULTANTS, INC.	413470
1	4-25-97	FJC	JMJ	JDP	REVISED TO REFLECT STAGE IIC DEVELOPMENT	GAI CONSULTANTS, INC.	413470

GAI CONSULTANTS
570 BEATTY ROAD
MONROEVILLE, PA 15146
(412)856-6400

GAI Dwg. File No. 92-220-F4013

	DRAFTING MADE	DRAFTING CHECKED	ENGINEERING ENG.	ENGINEERING MGR.
	JPS	JMJ	JMJ	JDP
DATE	7-11-96	7-30-96	7-30-96	7-30-96

Genco GPU Generation Corp. 1001 BROAD STREET JOHNSTOWN, PA. 15907

KEYSTONE STATION UNITS 1 & 2
KEYSTONE STATION DISPOSAL SITE
PLUMCREEK TOWNSHIP

DWG. TYPE: CIVIL

TITLE: RESIDUAL WASTE PERMIT DRAWING
STAGE III PILE DEVELOPMENT AND
STAGE IV SUBGRADE PLAN

PROJECT CONTROL NO	SCALE	DRAWING NUMBER	REV
413330	1"=200'	D-728-1056	3

CAD FILE: P:\MUR\2011\C110408\Dash 00\Task 001\CAD Department\Production Drawings\KE-F4013REV3.dwg 7/9/2013

STAGE IV HAUL ROAD CURVE DATA

CURVE NO.	P.I. COORDINATES NORTH	P.I. COORDINATES EAST	△	D	R	T	L	P.C. STA.	P.T. STA.
BEGIN	490637.71	1527898.20	–	–	–	–	–	–	0+00
1	490716.26	1527829.55	29°13'36"	14°19'10"	400.13	104.32	204.10	0+00.00	2+04.10
2	491080.68	1526800.00	44°35'16"	14°19'26"	400.00	164.00	311.28	10+27.92	13+39.20
3	491615.00	1526540.00	71°06'00"	19°05'55"	300.00	214.38	372.28	15+55.03	19+27.31
4	492075.03	1527002.58	03°39'24"	07°17'12"	786.30	25.10	50.18	23+40.22	23+90.40
END	492427.01	1527417.68	–	–	–	–	–	29+09.54	–

LEGEND

FABRIC FORM SLOPE DRAIN — SLOPE DRAIN
EXISTING COLLECTION DITCH
PROPOSED COLLECTION DITCH
EXISTING DIVERSION DITCH
PROPOSED DIVERSION DITCH
RIGHT OF WAY LINES
PROPERTY LINES (KEYSTONE OWNERS)
PROPERTY LINES (OTHERS)
EXISTING PERMIT AREA (STAGE I AND II)
NEW PERMIT AREA (STAGE III AND IV)
EXISTING PERMIT LINER FOOTPRINT
EXISTING EAST VALLEY LEACHATE PIPELINE
K-58 1291.26 — SURVEY MONUMENT AND ELEVATION
BUILDING IN USE
LEACHATE FORCE MAIN

REFERENCE:
MAPPING COMPILED BY TVGA, INC. LANSE PENNSYLVANIA USING STEREOPHOTOGRAMMETRIC METHODS FROM 1"=1333' SCALE AERIAL PHOTOGRAPHY 04/08/94.
HORIZONTAL CONTROL IS BY A SURVEYED TIE TO THE PENNSYLVANIA STATE COORDINATE SYSTEM, SOUTH ZONE (1983 DATUM). VERTICAL CONTROL IS BASED ON THE NATIONAL GEODETIC VERTICAL DATUM.

NOTES:

1. CONTOUR INTERVALS SHOWN AT TEN(10) FEET.
2. FINISHED GRADE CONTOURS DEPICT TOP OF SOIL COVER OR FINISHED ROAD SURFACE.
3. SEE DRAWING D-728-1065 FOR TYPE A, B, C, AND D CHANNEL DETAILS AND SLOPE DRAIN DETAILS.
4. SEE DRAWING D-728-1061 FOR CHANNELS ON EDGE OF LINER DETAILS.
5. SEE DRAWING D-728-1064 FOR CHANNEL DETAILS AT EDGE OF HAUL ROAD AND DRAWING D-728-1063 FOR HAUL ROAD PROFILES.
6. SEE DRAWING D-728-1066 FOR RIPRAP OUTLET PROTECTION DETAILS.
7. SEE DRAWING D-728-1068 FOR CULVERT DETAILS AND SCHEDULE.
8. SEE DRAWING D-728-1057 FOR LINER LIMITS, SUBGRADE DRAIN PIPES, LEAK DETECTION PIPES AND LEACHATE COLLECTION PIPES.
9. AT STAGE IV PILE DEVELOPMENT CLOSURE, THE COLLECTION CHANNEL ACROSS THE TOP OF STAGE IV AND DOWN THE STAGE IV HAUL ROAD WILL BE CONVERTED TO A DIVERSION CHANNEL AND DIVERTED TO THE UNNAMED TRIBUTARY AT THE STAGE IV EMBANKMENT TOE.
10. THE EXISTING 5' LONG x 18' WIDE OVERFLOW WEIR CREST SHALL BE RECONSTRUCTED WITH CULVERT 12 AND A TYPE C-2 OUTLET CHANNEL AS SHOWN ON DRAWING D-728-1071 REV. 5, TO ALLOW NEW CLEANOUT ACCESS ROAD CROSSING.

REV	DATE	MADE	ENGR	SUPV/MGR	DESCRIPTION	
3	7-9-13	JCN	DMD	JMJ	ADDED STAGE IV CLEANOUT ACCESS ROADS AND CULVERTS 12-19	–
2	5-27-98	FJC	JMJ	JDP	REVISED EQUALIZATION POND AND NOTE 8	413472
1	4-25-97	FJC	JMJ	JDP	REVISED TO REFLECT STAGE IIC DEVELOPMENT	413470

GAI CONSULTANTS, INC.

GAI CONSULTANTS
570 BEATTY ROAD
MONROEVILLE, PA 15146
(412)856-6400

GAI Dwg. File No. 92-220-F4015

	DRAFTING MADE	DRAFTING CHECKED	ENGINEERING ENG.	ENGINEERING MGR.
	JPS	JMJ	JDP	JDP
DATE	7-11-96	7-30-96	7-30-96	7-30-96

Genco GPU Generation Corp. 1001 BROAD STREET JOHNSTOWN, PA. 15907

KEYSTONE STATION UNITS 1 & 2
KEYSTONE STATION DISPOSAL SITE
PLUMCREEK TOWNSHIP

DWG. TYPE: CIVIL

TITLE: RESIDUAL WASTE PERMIT DRAWING
STAGE IV (ULTIMATE) PILE DEVELOPMENT

PROJECT CONTROL NO	SCALE	DRAWING NUMBER	REV
413330	1"=200'	D-728-1058	3

CAD FILE: P:\MUR\2011\C110408\Dash 00\Task 001\CAD Department\Production Drawings\KE-F4015_Rev 3.dwg 7/9/2013

APPENDIX A

Calculations from July 1996 Keystone Station Disposal Site West Valley Form I

APPENDIX I-1-A

FORM I

PROPOSED WEST VALLEY DRAINAGE FACILITIES
(EXCEPT PONDS) - DESIGN CALCULATIONS

SUBJECT

BY DATE PROJ. NO.

CHKD. BY DATE SHEET NO. OF

GAI CONSULTANTS, INC.
Engineers • Geologists • Planners
Environmental Specialists

Keystone West Valley

PROPOSED DRAINAGE FACILITIES (except Ponds)

DESIGN CALCULATIONS

TABLE OF CONTENTS

* Note - Stage 3C Drainage Calcs are in Appendix 3-2-11 *

WORKSHEETS

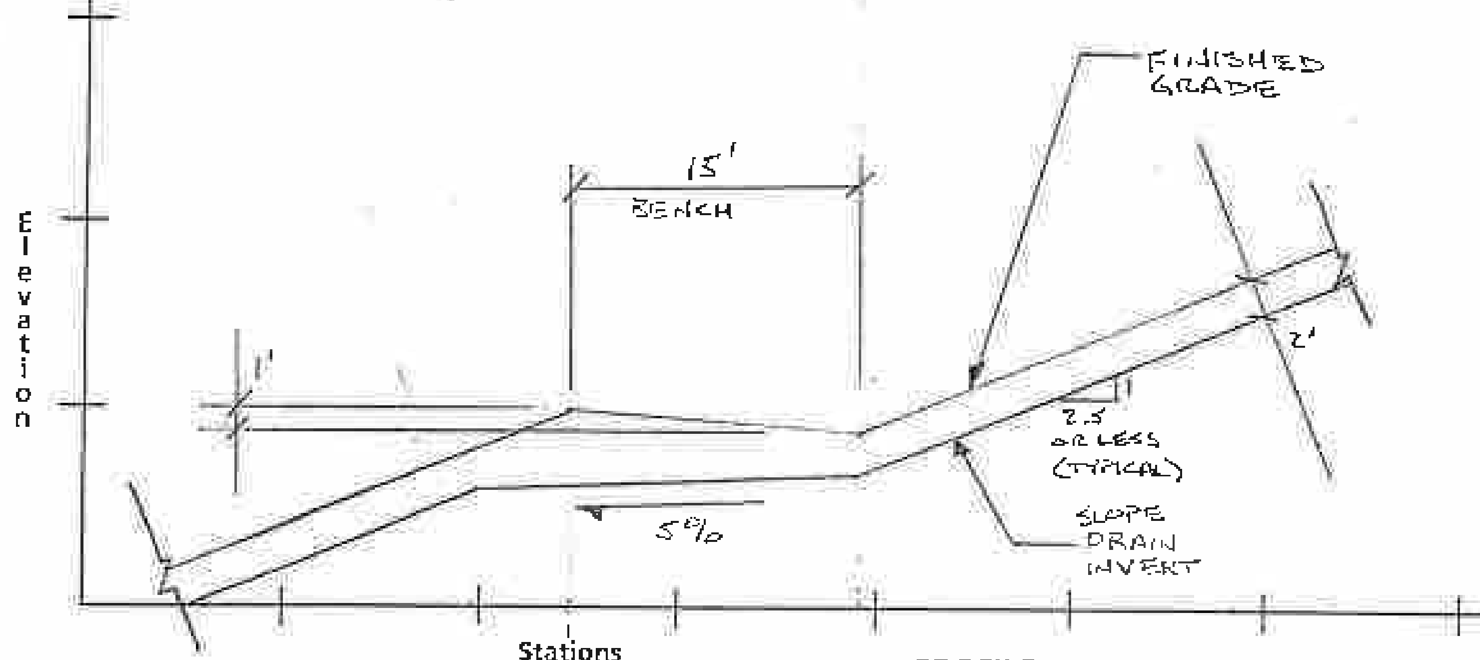
FORM I
DIVERSION/COLLECTION DITCH DATA SHEET

Title: Slope Drains | Site: Keystone West Valley
Prepared by: OMK / SER | Telephone Number: | Date: 7/25/96 | Sheet 1 of 30

Estimated Peak Storm Intensity: ______ (In./hr.) *Design Calculations:* ______

Station: Start End	Station: Elevation	Drainage Area (acres)	Design Storm (yrs.)	Average Watershed Slope (%)	Curve Number	Peak Discharge Q (cfs)	Channel Bed Slope (%)	Freeboard (ft.)	Channel Lining	Manning's Coefficient (n)	Channel Bottom Width (ft)	Channel Side Slopes (%)	Flow Area (sq/ft.)	Flow Depth (ft.)	Top Flow Width (ft.)	Flow Velocity (ft/sec)	Q Available (cfs)	With Freeboard: Channel Depth (ft.)	With Freeboard: Top Channel Width (ft.)	With Freeboard: Q Available (cfs)
Bench		23.2	25		78	71	5.0 (Min)	0.5	USM	0.015	2	50	4.3	1.0	6.2	16.5	71	2.0	10	282
Bench		23.2	25		78	71	40.0 (Max)	0.9	USM	0.015	2	50	2	0.6	4.5	35.3	71	2.0	10	802

SEE ULTIMATE CONDITIONS CALC FOR DESIGN.



PROFILE
TYPICAL SLOPE DRAIN

Vertical Scale 1" - 10'
Horizontal Scale 1" = 10'

FORM I

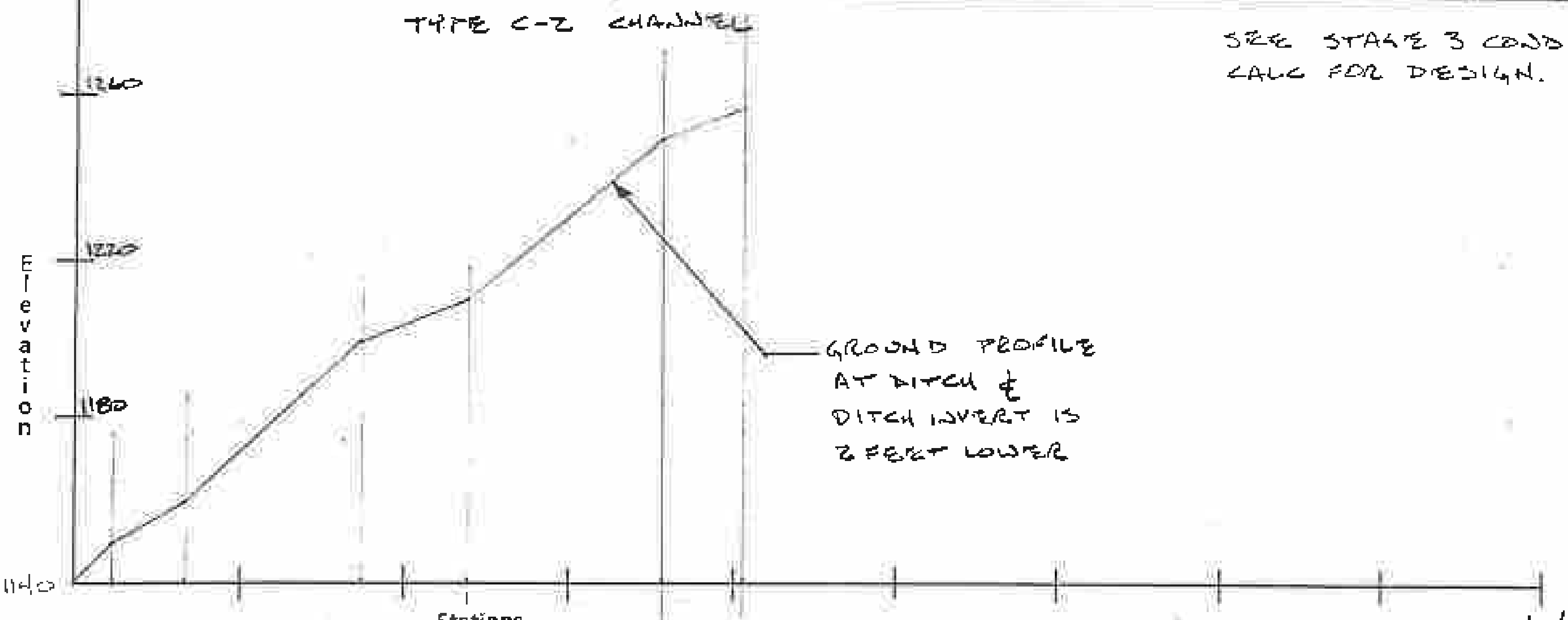
DIVERSION/COLLECTION DITCH DATA SHEET

Title: STAGE 4 (ULTIMATE CASE) WEST DITCH	Site: Keystone West Valley		
Prepared by: PMK/SER	Telephone Number:	Date: 7/25/94	Sheet 2 of 30

Estimated Peak Storm Intensity: ________ (in./hr.) **Design Calculations:** ________

Station: Start End	Station: Elevation	Drainage Area (acres)	Design Storm (yrs.)	Average Watershed Slope (%)	Curve Number	Peak Discharge Q (cfs)	Channel Bed Slope (%)	Freeboard (ft.)	Channel Lining	Manning's Coefficient (n)	Channel Bottom Width (ft)	Channel Side Slopes (%)	Flow Area (sq/ft.)	Flow Depth (ft)	Top Flow Width (ft.)	Flow Velocity (ft/sec.)	Q Available (cfs)	With Freeboard: Channel Depth (ft.)	With Freeboard: Top Channel Width (ft.)	With Freeboard: Q Available (cfs)
ULTIMATE CONDITIONS		12.3	25		78	29	4.5 (Min)	0.6	GROUTED ROCK	0.025	2	50	3.4	0.9	5.6	8.6	29	2	10	162
"		12.3	25		78	29	27.8 (Max)	0.9	"	0.025	2	50	1.7	0.6	4.2	16.7	29	2	10	401
STAGE 3 CONDITIONS		20.9	25		78	51	4.5 (Min)	0.8	"	0.025	2	50	5.1	1.2	6.7	10	51	2	10	162
"		20.9	25		78	51	27.8 (Max)	1.3	"	0.025	2	50	2.6	0.7	5	19.5	51	2	10	401

SEE STAGE 3 CONDITIONS CALC FOR DESIGN.



PROFILE

Vertical Scale 1" - 40'
Horizontal Scale 1" = 200'

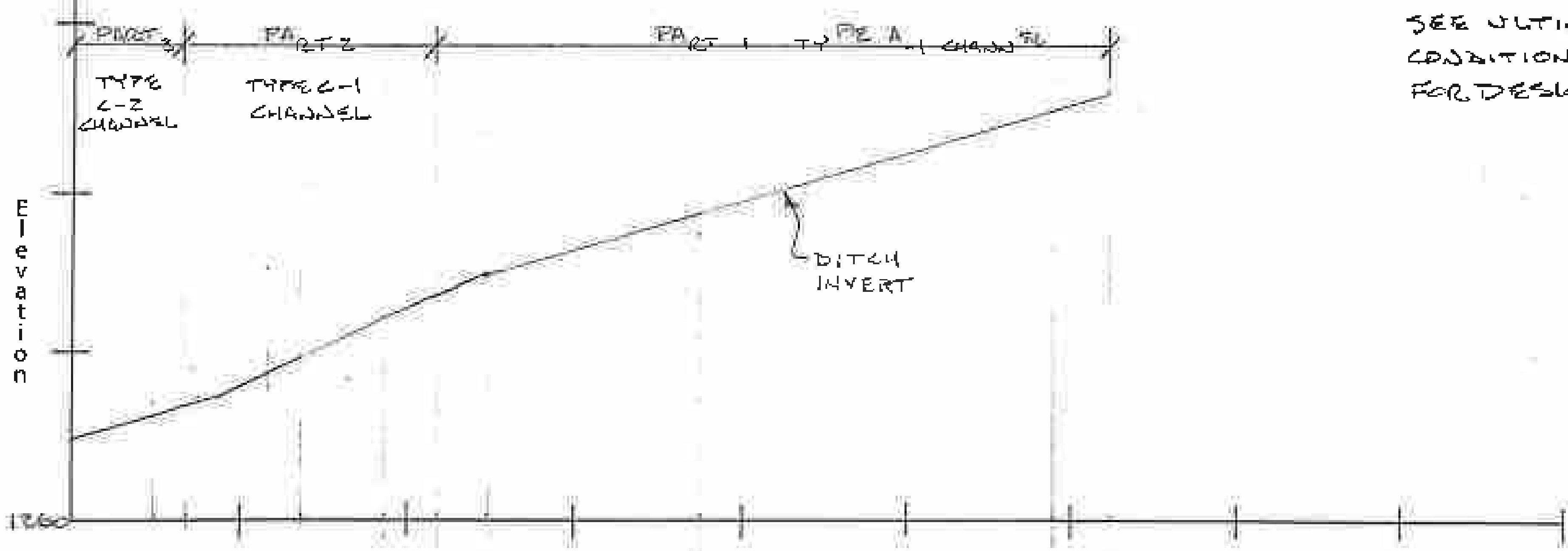
FORM I

DIVERSION/COLLECTION DITCH DATA SHEET

Title: North Ditch (Ult. Cond.) | Site: Keystone ... Valley
Prepared by: DAK/SSR | Telephone Number: | Date 7/25/96 | Sheet 3 of 30

Estimated Peak Storm Intensity: ________ (in./hr.) ***Design Calculations:*** ________

Station Start End	Station Elevation	Drainage Area (acres)	Design Storm (yrs)	Average Watershed Slope (%)	Curve Number	Peak Discharge Q (cfs)	Channel Bed Slope (%)	Freeboard (ft.)	Channel Lining	Manning's Coefficient (n)	Channel Bottom Width (ft)	Channel Side Slopes (%)	Flow Area (sq.ft.)	Flow Depth (ft.)	Top Flow Width (ft.)	Flow Velocity (ft/sec.)	Q Available (cfs)	With Freeboard: Channel Depth (ft.)	With Freeboard: Top Channel Width (ft.)	With Freeboard: Q Available (cfs)
Part 1		7.3	25		78	7	5.4 (MIN)	0.5	GRASS	0.045	2	50	1.7	0.5	4.2	4.2	7	1	6	23
Part 1		7.3	25		78	7	7.1 (MAX)	0.5	"	0.045	2	50	1.6	0.5	4.1	4.5	7	1	6	26
Part 2		6.9	25		79	20	10.0 (MIN)	0.9	GROUTED RIPS	0.025	2	50	1.9	0.6	4.4	10.4	20	1.5	8	128
Part 2		6.9	25		79	20	20.0 (MAX)	1.0	"	0.025	2	50	1.6	0.5	4.0	13.3	20	1.5	8	181
PART 3		32.5	25		76	69	6.0	0.7	"	0.025	2	50	5.7	1.3	7.1	12.0	69	2.0	10.0	186



SEE ULTIMATE CONDITIONS CALC FOR DESIGN.

PROFILE

Vertical Scale 1" - 40'
Horizontal Scale 1" = ~~200'~~

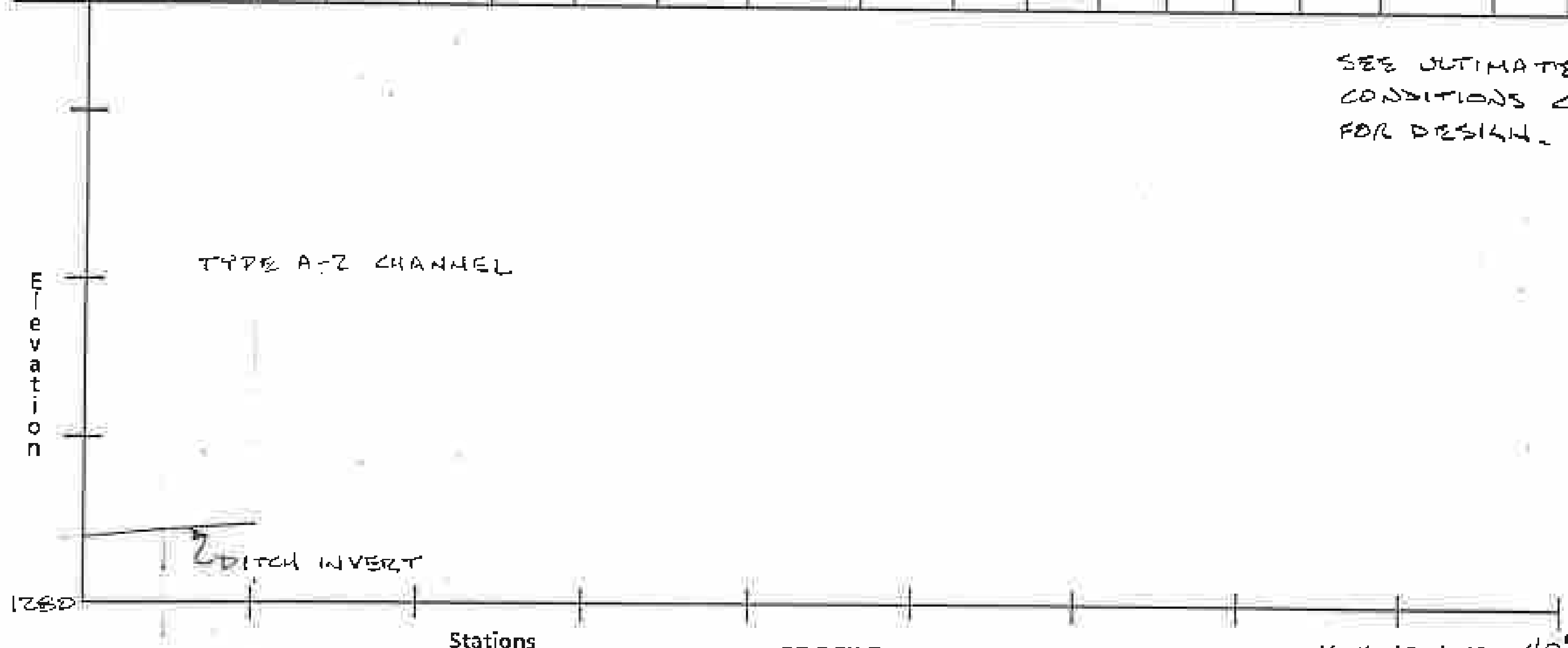
FORM I
DIVERSION/COLLECTION DITCH DATA SHEET

Title: Northwest Ditch (Ult. Cond.)	Site: Keystone (West Valley		
Prepared by: PHK / SSR	Telephone Number:	Date: 7/25/94	Sheet 4 of 30

Estimated Peak Storm Intensity: ________ (in./hr.) **Design Calculations:** ________

Station: Start End	Station: Elevation	Drainage Area (acres)	Design Storm (yrs.)	Average Watershed Slope (%)	Curve Number	Peak Discharge Q (cfs)	Channel Bed Slope (%)	Freeboard (ft.)	Channel Lining	Manning's Coefficient (n)	Channel Bottom Width (ft)	Channel Side Slopes (%)	Flow Area (sq/ ft.)	Flow Depth (ft.)	Top Flow Width (ft.)	Flow Velocity (ft/sec.)	Q Available (cfs)	With Freeboard: Channel Depth (ft.)	With Freeboard: Top Channel Width (ft.)	With Freeboard: Q Available (cfs)
Pt. #1		4.6	25		79	13	1.9	0.5	Grass	0.045	2	50	4	1.0	6	3.3	13	1.5	8	31

SEE ULTIMATE CONDITIONS CALC FOR DESIGN.



PROFILE

Vertical Scale 1" - 40'
Horizontal Scale 1" = 200'

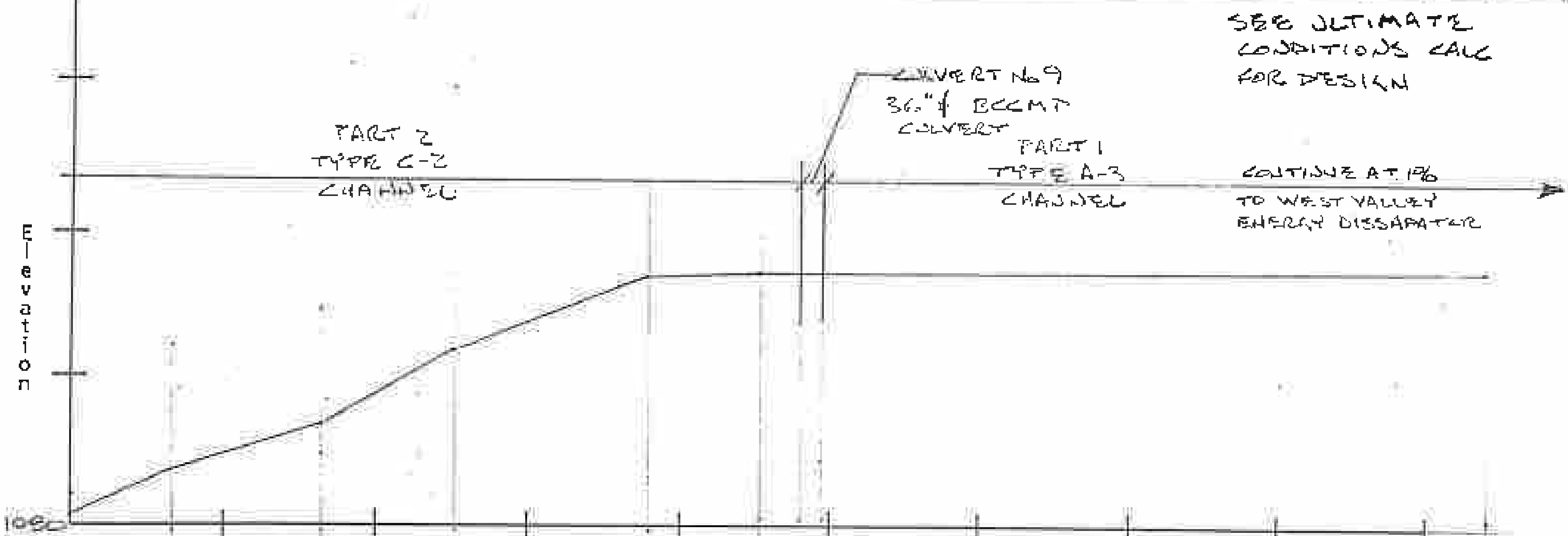
FORM I
DIVERSION/COLLECTION DITCH DATA SHEET

Title: SOUTHWEST DITCH (ULTIMATE)	Site: Keystone West Valley		
Prepared by: PMK/SER	Telephone Number:	Date: 7/25/96	Sheet 5 of 30

Estimated Peak Storm Intensity: ________ (in./hr.) **Design Calculations:** ________

Station Start End	Station Elevation	Drainage Area (acres)	Design Storm (yrs.)	Average Watershed Slope (%)	Curve Number	Peak Discharge Q (cfs)	Channel Bed Slope (%)	Freeboard (ft.)	Channel Lining	Manning's Coefficient (n)	Channel Bottom Width (ft)	Channel Side Slopes (%)	Flow Area (sq/ft.)	Flow Depth (ft.)	Top Flow Width (ft.)	Flow Velocity (ft/sec.)	Q Available (cfs)	With Freeboard: Channel Depth (ft.)	With Freeboard: Top Channel Width (ft.)	With Freeboard: Q Available (cfs)
Part 2		33.6	25		78	90	14.3 (Min)	0.8	GROUTED RIPRAP	0.025	2	50	5.1	1.2	6.7	17.8	90	2	10	287
Part 2		33.6	25		78	90	33.3 (Max)	1.0	"	0.025	2	50	3.7	1.0	5.8	24.3	90	2	10	439
PART 1		33.6	25		78	90	1.0	1.2	GRASS	0.045	2	50	21.1	2.8	13.2	4.3	90	4	18	211

SEE ULTIMATE CONDITIONS CALC FOR DESIGN



Stations

PROFILE

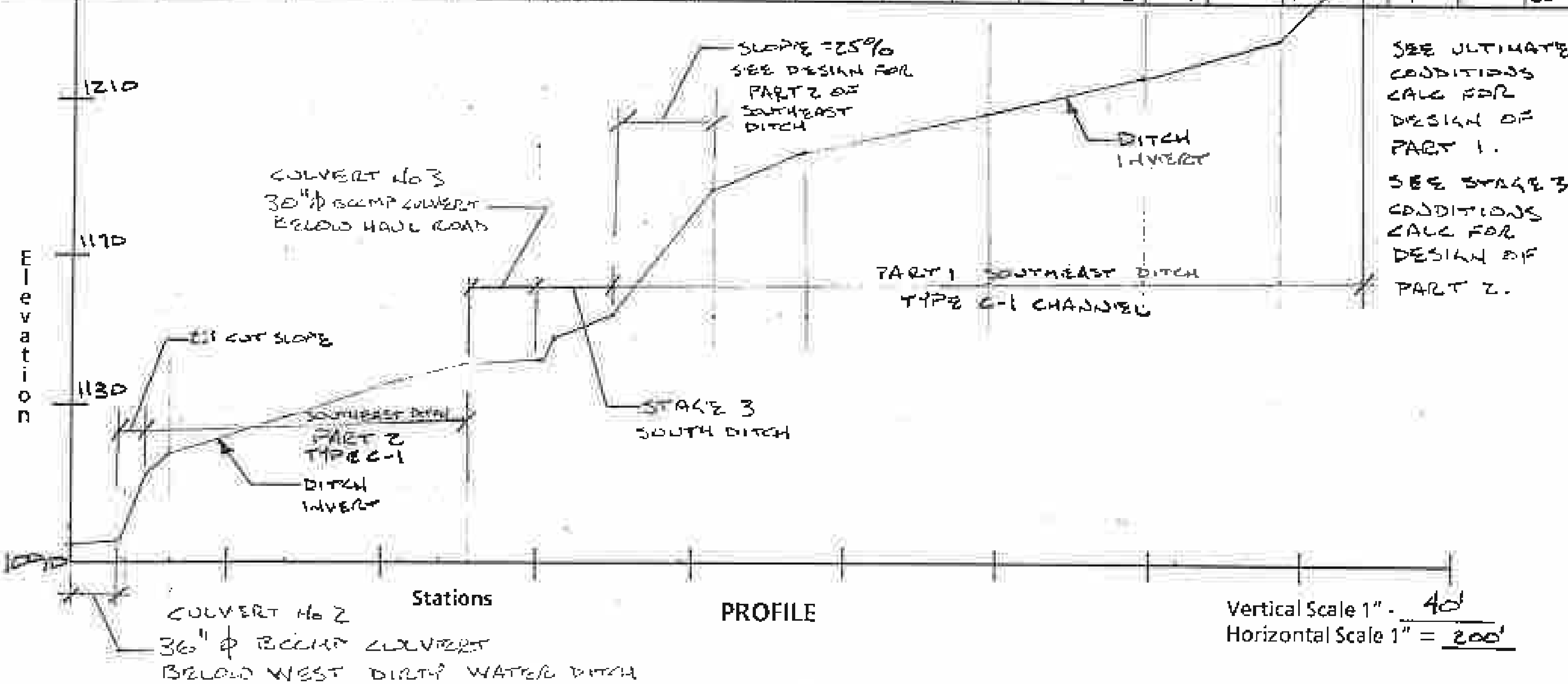
Vertical Scale 1" - 100'
Horizontal Scale 1" = 200'

FORM I
DIVERSION/COLLECTION DITCH DATA SHEET

Title: ULTIMATE CONDITIONS SOUTHEAST DITCH	Site: KEYSTONE WEST VALLEY		
Prepared by: SBR	Telephone Number:	Date: 7/25/96	Sheet 6 of 30

Estimated Peak Storm Intensity: ________ (in./hr.)   **Design Calculations:** ________

Station		Drainage Area (acres)	Design Storm (yrs.)	Average Watershed Slope (%)	Curve Number	Peak Discharge Q (cfs)	Channel Bed Slope (%)	Freeboard (ft.)	Channel Lining	Manning's Coefficient (n)	Channel Bottom Width (ft)	Channel Side Slopes (%)	Flow Area (sq/ft.)	Flow Depth (ft.)	Top Flow Width (ft.)	Flow Velocity (ft/sec.)	Q Available (cfs)	With Freeboard		
Start End	Elevation																	Channel Depth (ft.)	Top Channel Width (ft.)	Q Available (cfs)
PART 1		8.4	25		78	22	3.3 (Min)	0.7	GROUTED ROCK	0.025	2	50	3.1	0.8	5.3	7.2	22	1.5	8.0	74.0
PART 1		8.4	25		78	22	15.6 (Max)	0.9	"	0.025	2	50	1.8	0.6	4.2	12.5	22	1.5	8.0	160.0
PART 2		17.7	25		80	51	6.3 (Min)	0.4	"	0.025	2	50	4.5	1.1	6.3	11.3	51	1.5	8.0	101.0
PART 2		17.7	25		80	51	25.0 (Max)	0.7	"	0.025	2	50	2.7	0.8	5.1	18.8	51	1.5	8.0	202.0



PROFILE

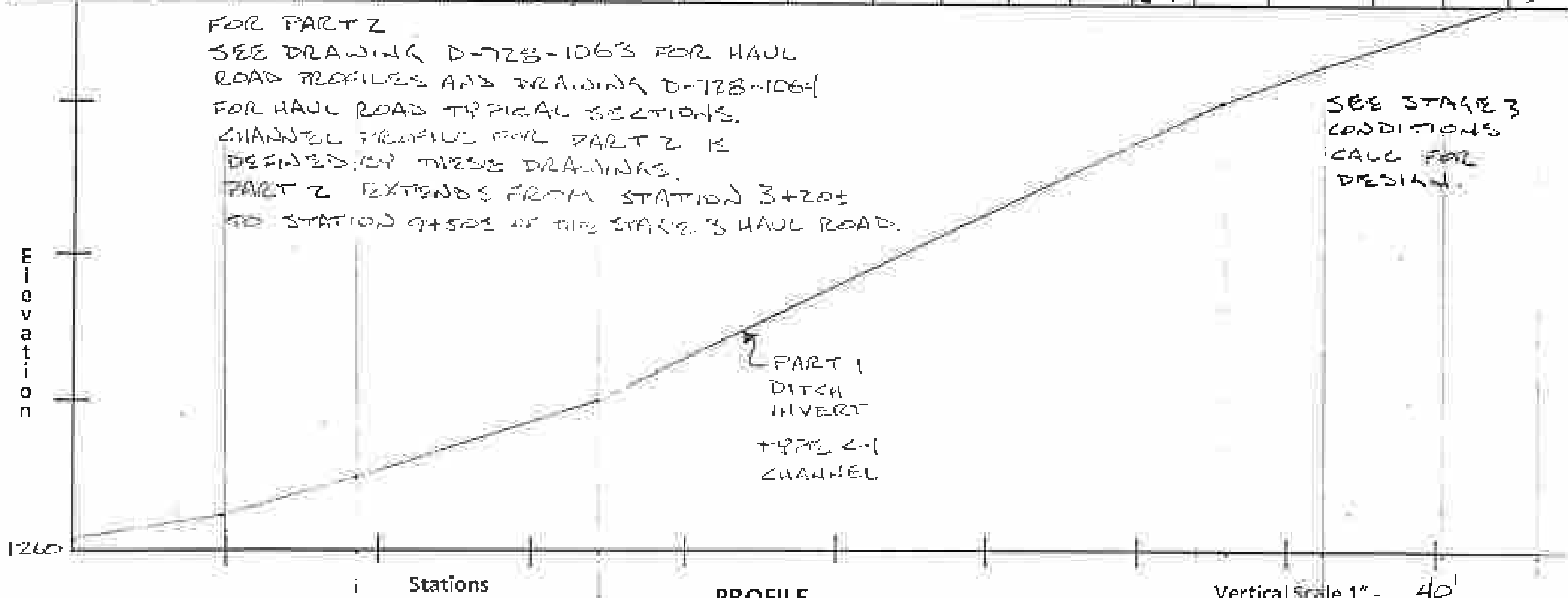
Vertical Scale 1" - 40'
Horizontal Scale 1" = 200'

FORM I
DIVERSION/COLLECTION DITCH DATA SHEET

Title: HAUL ROAD CLEAN WATER DITCH (Stage 3)	Site: Keystone West Valley		
Prepared by: PMK/SER	Telephone Number:	Date: 7/25/96	Sheet 7 of 30

Estimated Peak Storm Intensity: ______ (in./hr.) **Design Calculations:** ______

Station																		With Freeboard		
Start End	Elevation	Drainage Area (acres)	Design Storm (yrs.)	Average Watershed Slope (%)	Curve Number	Peak Discharge Q (cfs)	Channel Bed Slope (%)	Freeboard (ft.)	Channel Lining	Manning's Coefficient (n)	Channel Bottom Width (ft)	Channel Side Slopes (%)	Flow Area (sq/ft.)	Flow Depth (ft.)	Top Flow Width (ft.)	Flow Velocity (ft/sec.)	Q Available (cfs)	Channel Depth (ft.)	Top Channel Width (ft.)	Q Available (cfs)
Part 1		8.6	25		78	23	5.4 (Min)	0.8	GROUTED ROCK	0.025	2	50	2.6	0.7	5	8.9	23	1.5	8	98
Part 1		8.6	25		78	23	10.0 (Max)	0.9	"	0.025	2	50	2.1	0.6	4.6	10.8	23	1.5	8	128
Part 2		3.0	25		80	10	5.0 (Min)	0.5	"	0.025	2	50	1.5	0.5	4	6.7	10	1	6	39
Part 2		3.0	25		80	10	10.0 (Max)	0.6	"	0.025	2	50	1.2	0.4	3.7	8.5	10	1	6	55



PROFILE

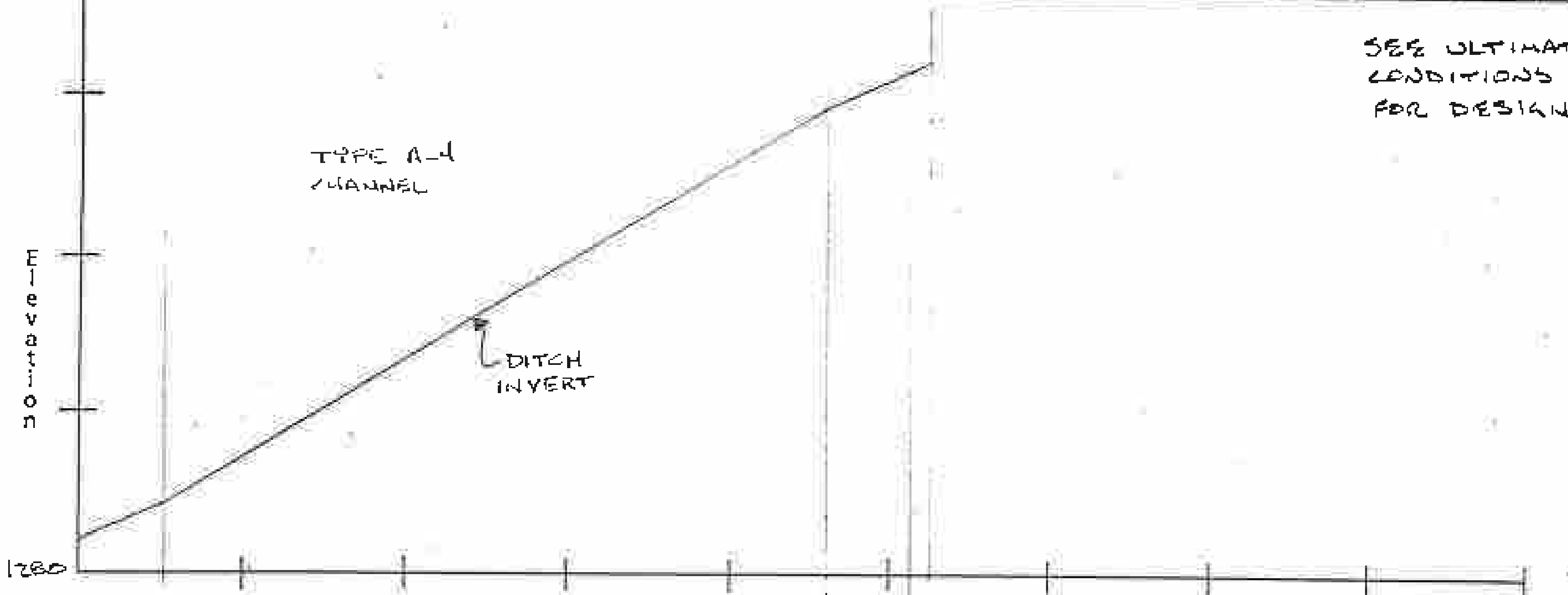
Vertical Scale 1" - 40'
Horizontal Scale 1" = 200'

FORM I
DIVERSION/COLLECTION DITCH DATA SHEET

Title: North Top of Pile Slope (Ult. Des)	Site: Keystone West Valley		
Prepared by: PMK / SER	Telephone Number:	Date: 7/25/96	Sheet 8 of 30

Estimated Peak Storm Intensity: ______ (in./hr.)   *Design Calculations:* ______

Station Start End	Station Elevation	Drainage Area (acres)	Design Storm (yrs)	Average Watershed Slope (%)	Curve Number	Peak Discharge Q (cfs)	Channel Bed Slope (%)	Freeboard (ft.)	Channel Lining	Manning's Coefficient (n)	Channel Bottom Width (ft)	Channel Side Slopes (%)	Flow Area (sq. ft.)	Flow Depth (ft.)	Top Flow Width (ft.)	Flow Velocity (ft/sec)	Q Available (cfs)	With Freeboard: Channel Depth (ft.)	With Freeboard: Top Channel Width (ft.)	With Freeboard: Q Available (cfs)
Still		25.6	25		75	53	3.7 (Min)	1.2	Grass	0.045	0	33.3	9.4	1.8	10.6	5.7	53	3	18	218
Bust		25.6	25		75	53	6.0 (Max)	1.4	"	0.045	0	33.3	7.8	1.6	9.7	6.8	53	3	18	278



PROFILE

SEE ULTIMATE CONDITIONS CALC FOR DESIGN.

Vertical Scale 1" = 40'
Horizontal Scale 1" = 400'

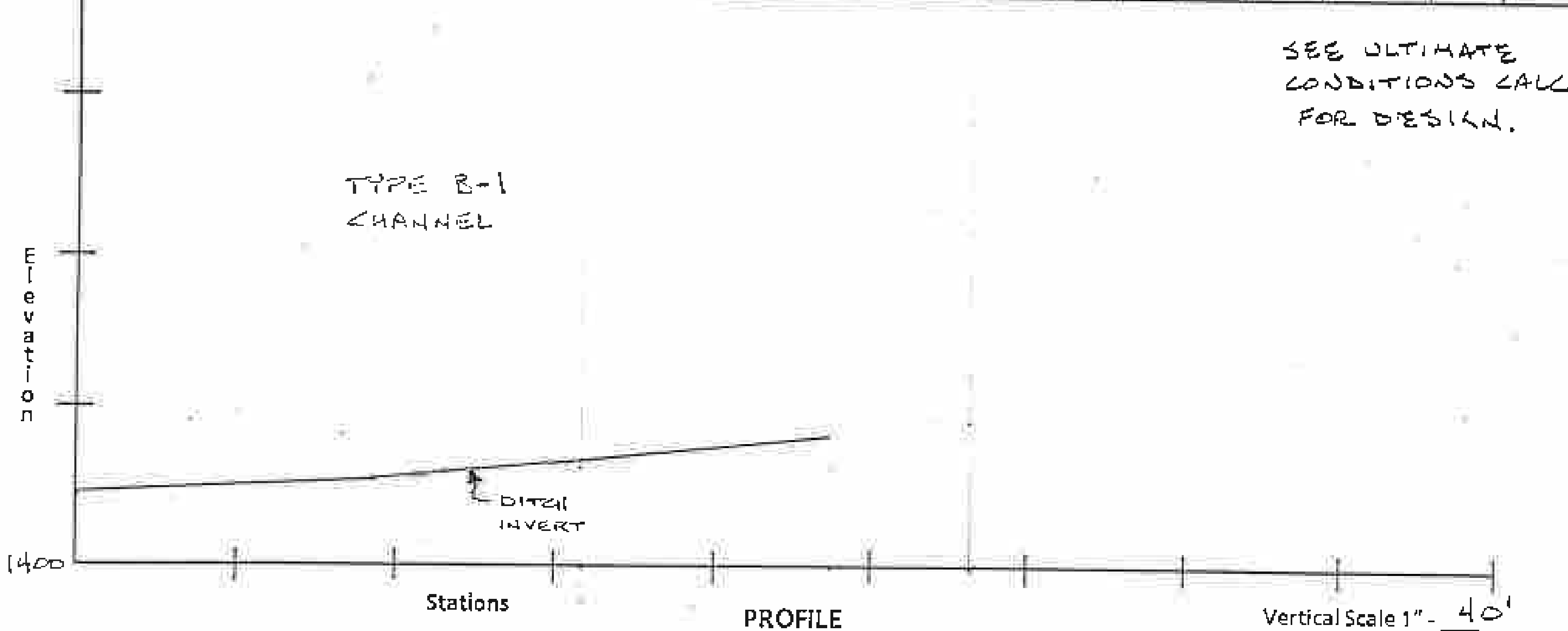
FORM I

DIVERSION/COLLECTION DITCH DATA SHEET

Title: SOUTH TOP OF PILE SLOPE (ULT. COND.)	Site: Keystone West Valley		
Prepared by: PAR	Telephone Number:	Date: 7/25/96	Sheet 9 of 30

Estimated Peak Storm Intensity: ________ (in./hr.) **Design Calculations:** ________

Station																		With Freeboard		
Start End	Elevation	Drainage Area (acres)	Design Storm (yrs.)	Average Watershed Slope (%)	Curve Number	Peak Discharge Q (cfs)	Channel Bed Slope (%)	Freeboard (ft.)	Channel Lining	Manning's Coefficient (n)	Channel Bottom Width (ft)	Channel Side Slopes (%)	Flow Area (sq. ft.)	Flow Depth (ft.)	Top Flow Width (ft.)	Flow Velocity (ft/sec.)	Q Available (cfs)	Channel Depth (ft.)	Top Channel Width (ft.)	Q Available (cfs)
Bull		42.2	25		77	85	1.5 (Min)	0.5	GRASS W/ NECM	0.045	0	33.3	18.6	2.5	15	4.6	85	3	18	139
Bull		42.2	25		77	85	4.5 (Max)	1.0	"	0.045	0	33.3	12.4	2.0	12.2	6.9	85	3	18	241



Vertical Scale 1" - 40'
Horizontal Scale 1" = 200'

FORM J

DIVERSION/COLLECTION DITCH DATA SHEET

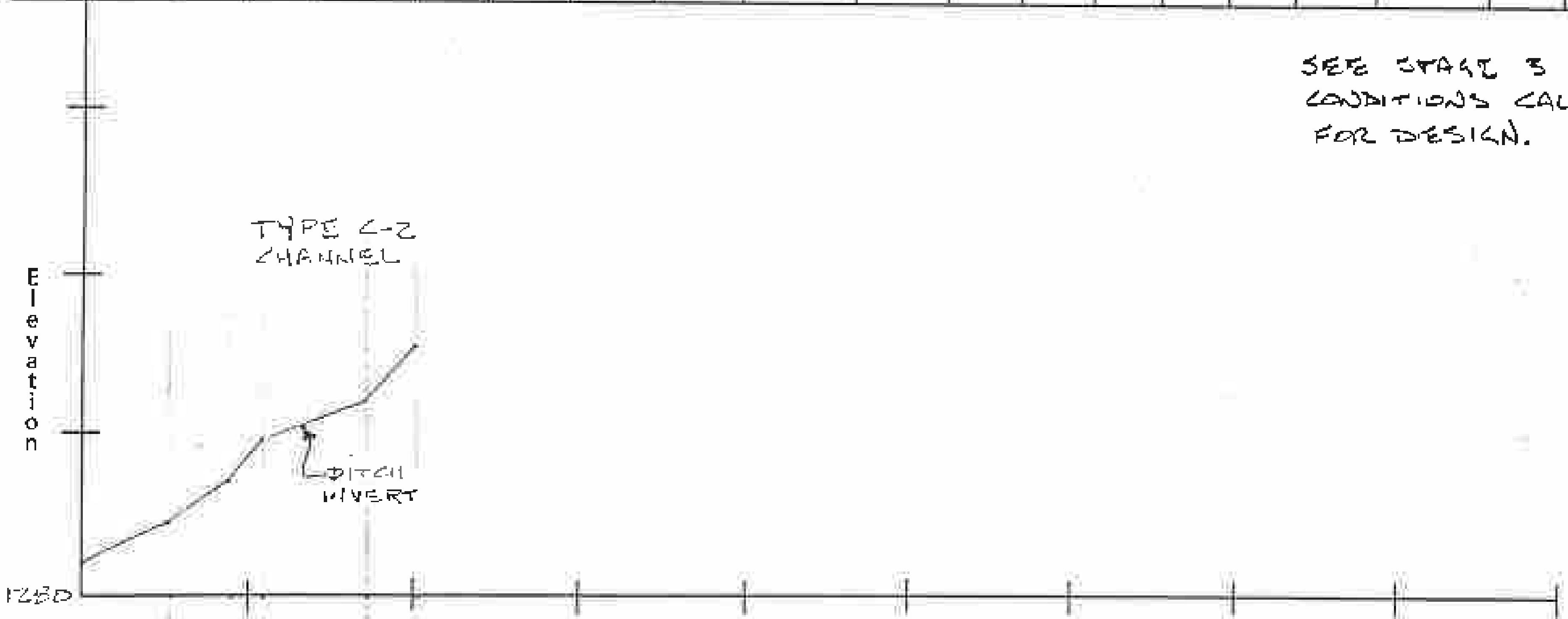
Title: Northern Ditch (Stage 3) Site: Keystone West Valley

Prepared by: PMK/SER Telephone Number: Date: 7/25/96 Sheet 10 of 30

Estimated Peak Storm Intensity: ______ (in./hr.) **Design Calculations:** ______

Station Start End	Station Elevation	Drainage Area (acres)	Design Storm (yrs.)	Average Watershed Slope (%)	Curve Number	Peak Discharge Q (cfs)	Channel Bed Slope (%)	Freeboard (ft.)	Channel Lining	Manning's Coefficient (n)	Channel Bottom Width (ft)	Channel Side Slopes (%)	Flow Area (sq/ft)	Flow Depth (ft.)	Top Flow Width (ft.)	Flow Velocity (ft/sec)	Q Available (cfs)	With Freeboard: Channel Depth (ft)	With Freeboard: Top Channel Width (ft.)	With Freeboard: Q Available (cfs)
Part 1		24.8	25		77	65	7.1 (Min)	0.8	Grouted Rock	0.025	2	50	5.1	1.2	6.7	12.6	65	2	10	203
Part 1		24.8	25		77	65	29.4 (Max)	1.2	"	0.025	2	50	3.1	0.8	5.3	21.3	65	2	10	412

SEE STAGE 3 CONDITIONS CALC FOR DESIGN.



PROFILE

Vertical Scale 1" = 40'
Horizontal Scale 1" = 200'

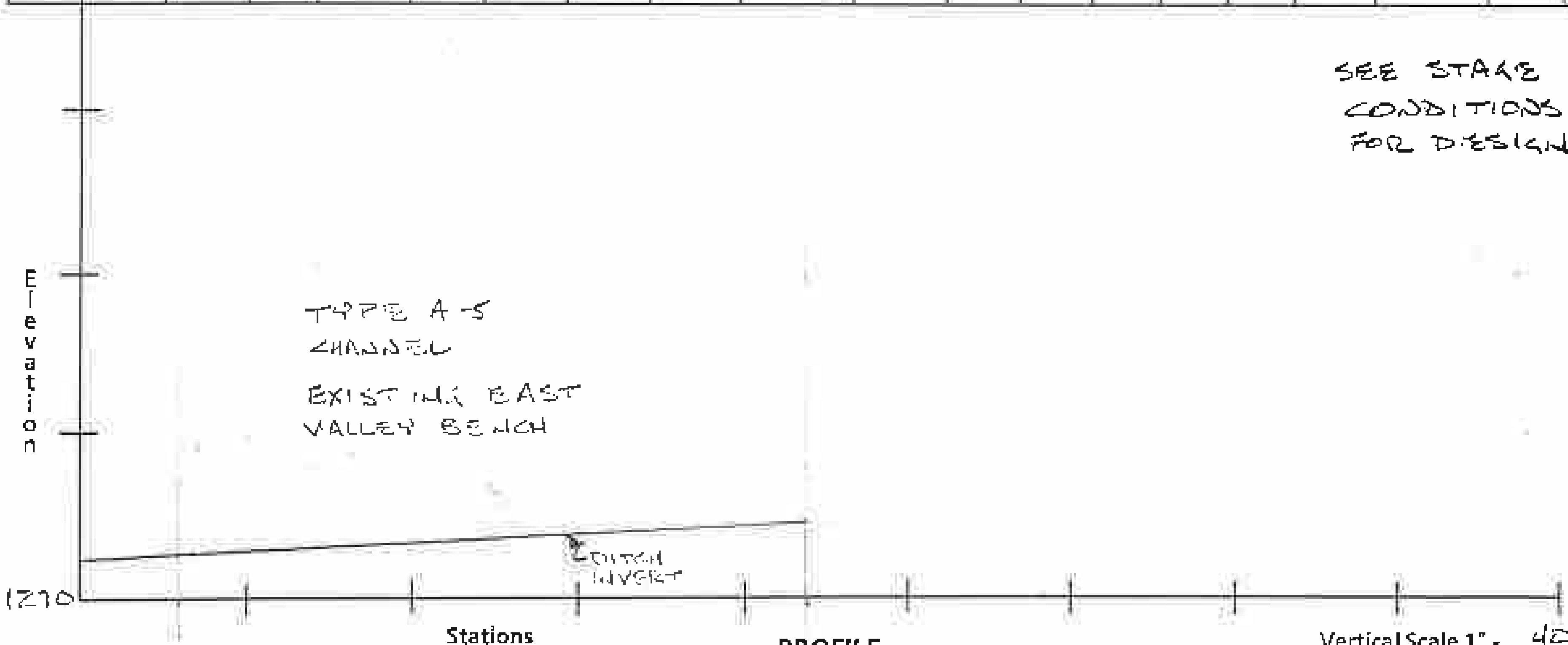
FORM I
DIVERSION/COLLECTION DITCH DATA SHEET

Title: East Ditch (Stage 3)	Site: Keystone West Valley		
Prepared by: PHK / SBR	Telephone Number:	Date: 7/25/96	Sheet 11 of 30

Estimated Peak Storm Intensity: ________ (in./hr.) **Design Calculations:** ________

Station: Start End	Station: Elevation	Drainage Area (acres)	Design Storm (yrs.)	Average Watershed Slope (%)	Curve Number	Peak Discharge Q (cfs)	Channel Bed Slope (%)	Freeboard (ft.)	Channel Lining	Manning's Coefficient (n)	Channel Bottom Width (ft)	Channel Side Slopes (%)	Flow Area (sq/ft.)	Flow Depth (ft.)	Top Flow Width (ft.)	Flow Velocity (ft/sec)	Q Available (cfs)	With Freeboard: Channel Depth (ft.)	With Freeboard: Top Channel Width (ft.)	With Freeboard: Q Available (cfs)
Part 1		26.1	25		77	69	1.0	0.7	Grass	0.045	13.5	50	20.1	1.3	18.5	3.4	69	2	21.5	156

SEE STAKE 3 CONDITIONS CALC FOR DESIGN.



PROFILE

Vertical Scale 1" = 40'
Horizontal Scale 1" = 200'

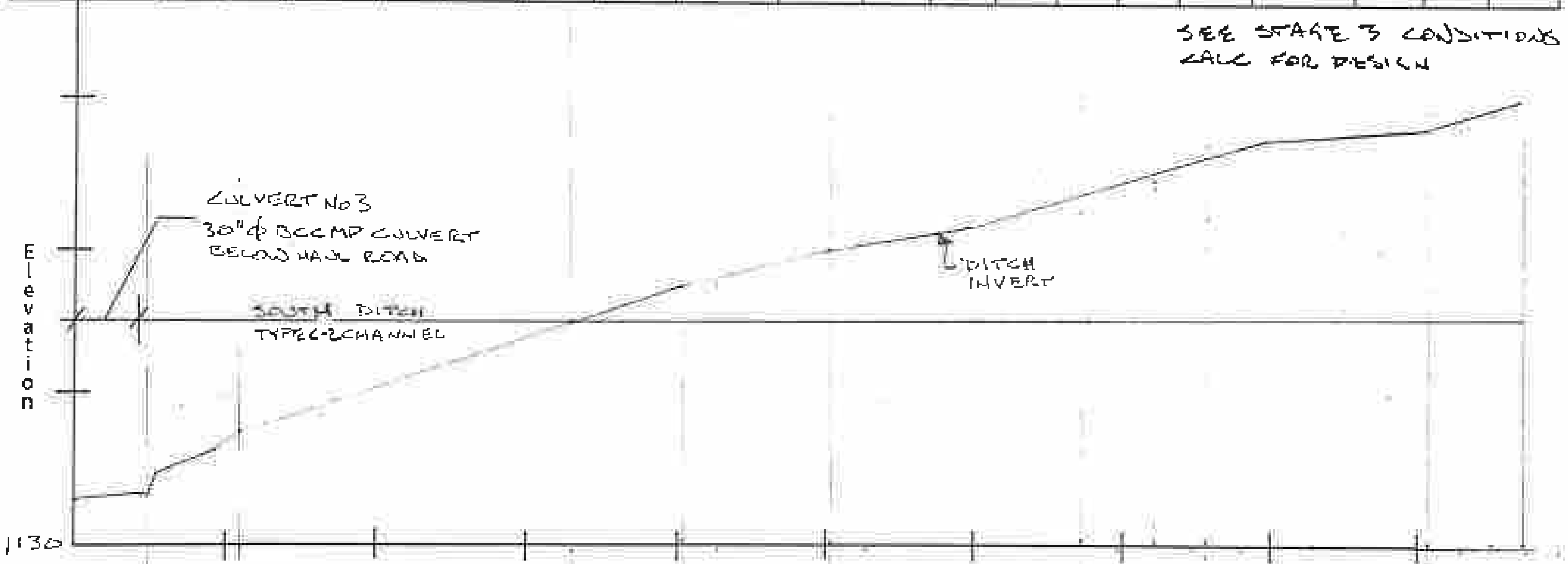
FORM I
DIVERSION/COLLECTION DITCH DATA SHEET

Title: South Ditch (Stage 3)	Site: Keystone West Valley		
Prepared by: PMK/SER	Telephone Number:	Date: 7/25/96	Sheet 12 of 30

Estimated Peak Storm Intensity: ________ (in./hr.) **Design Calculations:** ________

Station		Drainage Area (acres)	Design Storm (yrs.)	Average Watershed Slope (%)	Curve Number	Peak Discharge Q (cfs)	Channel Bed Slope (%)	Freeboard (ft.)	Channel Lining	Manning's Coefficient (n)	Channel Bottom Width (ft)	Channel Side Slopes (%)	Flow Area (sq/ft.)	Flow Depth (ft.)	Top Flow Width (ft.)	Flow Velocity (ft/sec.)	Q Available (cfs)	With Freeboard		
Start End	Elevation																	Channel Depth (ft.)	Top Channel Width (ft.)	Q Available (cfs)
Part 1		17.3	25		80	51	1.4 (Min)	0.5	Grouted rock	0.025	2	50	7.8	1.5	8.1	6.5	51	2	10	91
Part 1		17.3	25		80	51	6.3 (Max)	0.9	"	0.025	2	50	4.5	1.1	6.3	11.3	51	2	6.3	190

SEE STAGE 3 CONDITIONS CALC FOR DESIGN



PROFILE

Vertical Scale 1" = 40'
Horizontal Scale 1" = 200'

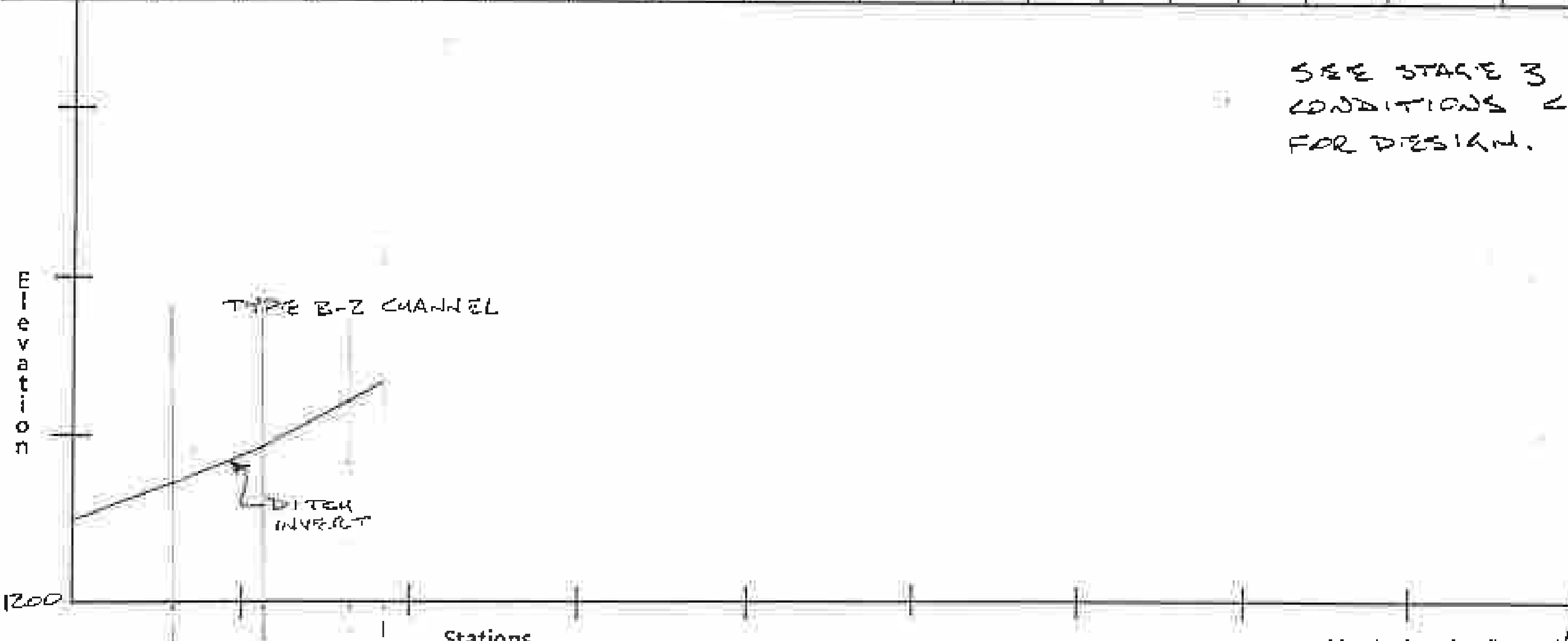
FORM I
DIVERSION/COLLECTION DITCH DATA SHEET

Title: Southeast Ditch (Stage 3)	Site: Keystone West Valley		
Prepared by: PMK/SER	Telephone Number:	Date: 7/25/96	Sheet 13 of 30

Estimated Peak Storm Intensity: ________ (in./hr.)　　**Design Calculations:** ________

Station Start End	Station Elevation	Drainage Area (acres)	Design Storm (yrs.)	Average Watershed Slope (%)	Curve Number	Peak Discharge Q (cfs)	Channel Bed Slope (%)	Freeboard (ft.)	Channel Lining	Manning's Coefficient (n)	Channel Bottom Width (ft)	Channel Side Slopes (%)	Flow Area (sq/ft.)	Flow Depth (ft.)	Top Flow Width (ft.)	Flow Velocity (ft/sec.)	Q Available (cfs)	With Freeboard: Channel Depth (ft.)	With Freeboard: Top Channel Width (ft.)	With Freeboard: Q Available (cfs)
Pael1		3.5	25		79	12	5.6 (Min)	0.8	GRASS	0.045	2	50	2.5	0.7	4.9	4.8	12	1.5	8	53
Pael1		3.5	25		79	12	12.5 (Max)	0.9	"	0.045	2	50	1.9	0.6	4.4	6.4	12	1.5	8	79

SEE STAGE 3 CONDITIONS CALC FOR DESIGN.



PROFILE

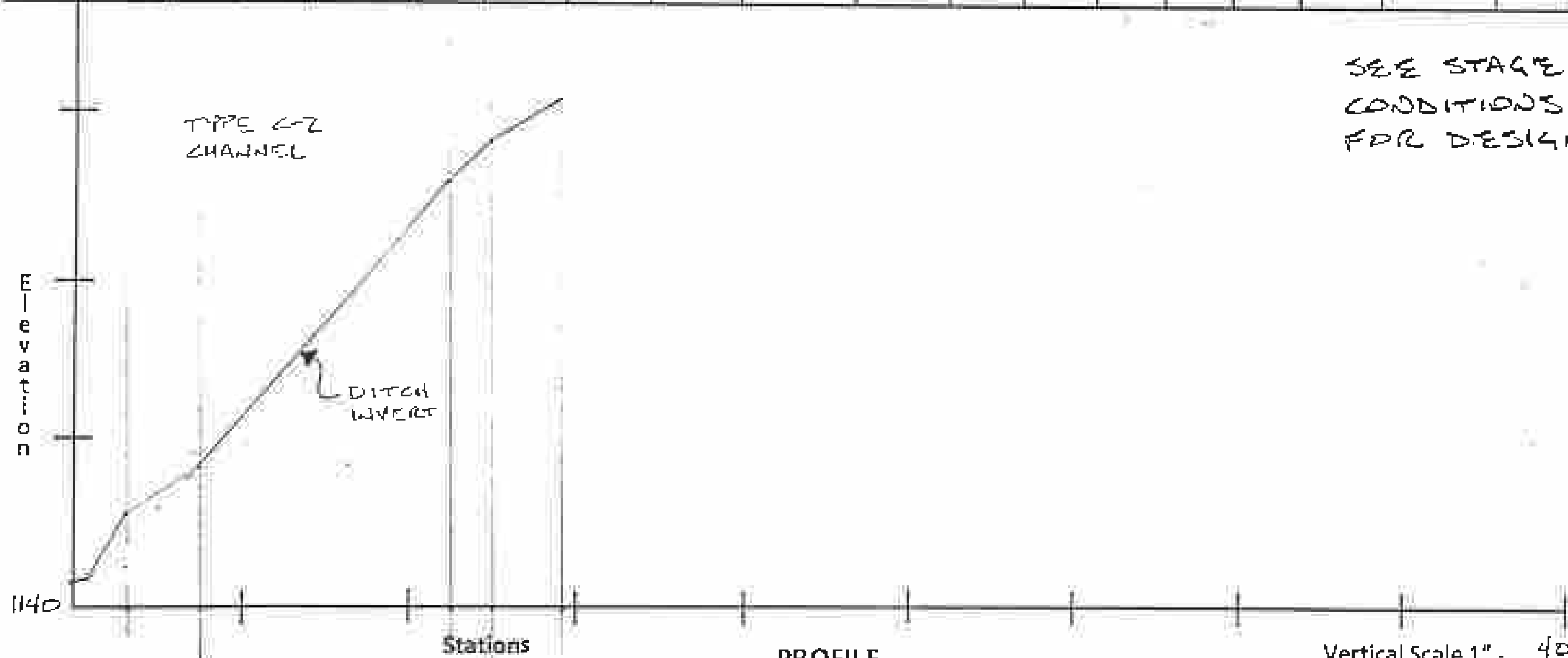
Vertical Scale 1" - 40'
Horizontal Scale 1" = 200'

FORM I
DIVERSION/COLLECTION DITCH DATA SHEET

Title: Southwest Ditch (stage 3)	Site: Keystone west Valley		
Prepared by: PHK/SER	Telephone Number:	Date: 7/25/96	Sheet 14 of 30

Estimated Peak Storm Intensity: ________ (in./hr.) ***Design Calculations:*** ________

Station																			With Freeboard		
Start End	Elevation	Drainage Area (acres)	Design Storm (yrs)	Average Watershed Slope (%)	Curve Number	Peak Discharge Q (cfs)	Channel Bed Slope (%)	Freeboard (ft.)	Channel Lining	Manning's Coefficient (n)	Channel Bottom Width (ft)	Channel Side Slopes (%)	Flow Area (sq/ft.)	Flow Depth (ft)	Top Flow Width (ft.)	Flow Velocity (ft/sec)	Q Available (cfs)	Channel Depth (ft.)	Top Channel Width (ft.)	Q Available (cfs)
Part1		15.0	25		78	42	6.3 (Min)	1	GROUTED ROCK	0.025	2	50	3.9	1.0	5.9	10.7	42	2	10	190
Part1		15.0	25		78	42	35.3 (Max)	1.4	"	0.025	2	50	2.1	0.6	4.6	19.8	42	2,	10	437



SEE STAGE 3
CONDITIONS CALC
FOR DESIGN.

PROFILE

Vertical Scale 1" - 40'
Horizontal Scale 1" = 200'

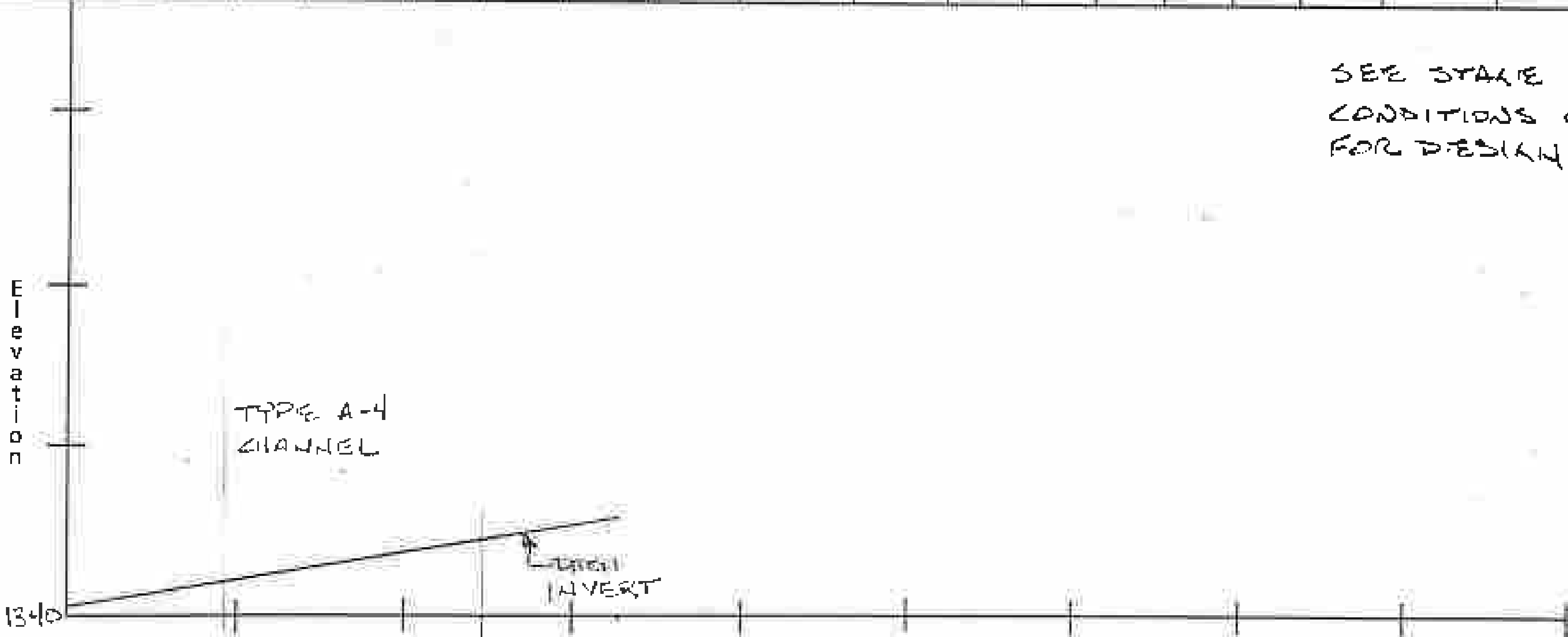
FORM I
DIVERSION/COLLECTION DITCH DATA SHEET

Title: SOUTHEAST TOP OF Pile Swale (Stage 3)	Site: Keystone West Valley		
Prepared by: PMK/SEE	Telephone Number:	Date: 7/25/96	Sheet 15 of 30

Estimated Peak Storm Intensity: ________ (in./hr.) **Design Calculations:** ________

Station																			With Freeboard		
Start End	Elevation	Drainage Area (acres)	Design Storm (yrs.)	Average Watershed Slope (%)	Curve Number	Peak Discharge Q (cfs)	Channel Bed Slope (%)	Freeboard (ft.)	Channel Lining	Manning's Coefficient (n)	Channel Bottom Width (ft)	Channel Side Slopes (%)	Flow Area (sq/ft.)	Flow Depth (ft.)	Top Flow Width (ft.)	Flow Velocity (ft/sec)	Q Available (cfs)	Channel Depth (ft.)	Top Channel Width (ft.)	Q Available (cfs)
Part 1		4.8	25		75	11	3.1	2.0	GRASS	0.045	0	33.3	3.1	1.0	6.1	3.6	11	3	18	200

SEE STAGE 3 CONDITIONS CALC FOR DESIGN.



PROFILE

Vertical Scale 1" - 40'
Horizontal Scale 1" = 200'

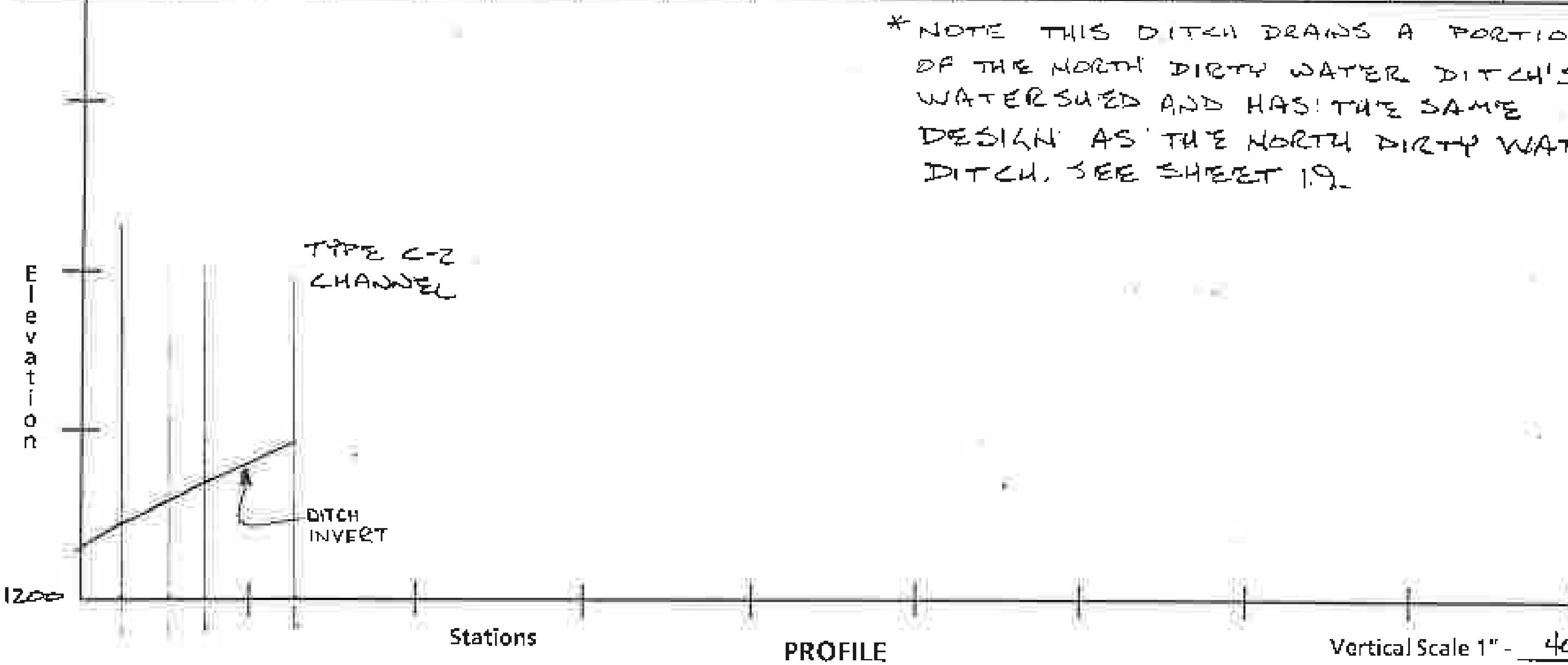
FORM I
DIVERSION/COLLECTION DITCH DATA SHEET

Title: NORTHEAST TEMPORARY DIVERSION DITCH	Site: KEYSTONE WEST VALLEY		
Prepared by: SER	Telephone Number:	Date: 7/31/96	Sheet 16 of 30

Estimated Peak Storm Intensity: ________ (in./hr.) ***Design Calculations:*** ________

Station																		With Freeboard		
Start End	Elevation	Drainage Area (acres)	Design Storm (yrs.)	Average Watershed Slope (%)	Curve Number	Peak Discharge Q (cfs)	Channel Bed Slope (%)	Freeboard (ft.)	Channel Lining	Manning's Coefficient (n)	Channel Bottom Width (ft)	Channel Side Slopes (%)	Flow Area (sq/ ft.)	Flow Depth (ft.)	Top Flow Width (ft.)	Flow Velocity (ft/sec.)	Q Available (cfs)	Channel Depth (ft.)	Top Channel Width (ft.)	Q Available (cfs)
*																				

*NOTE THIS DITCH DRAINS A PORTION OF THE NORTH DIRTY WATER DITCH'S WATERSHED AND HAS THE SAME DESIGN AS THE NORTH DIRTY WATER DITCH, SEE SHEET 19.



Vertical Scale 1" - 40'
Horizontal Scale 1" = 200'

FORM I

DIVERSION/COLLECTION DITCH DATA SHEET

Title: HAUL ROAD DIRTY WATER DITCH	Site: Keystone West Valley		
Prepared by: PMK/SER	Telephone Number:	Date: 7/25/96	Sheet 17 of 30

Estimated Peak Storm Intensity: ________ (in./hr.) **Design Calculations:** ________

Station Start End	Station Elevation	Drainage Area (acres)	Design Storm (yrs.)	Average Watershed Slope (%)	Curve Number	Peak Discharge Q (cfs)	Channel Bed Slope (%)	Freeboard (ft.)	Channel Lining	Manning's Coefficient (n)	Channel Bottom Width (ft)	Channel Side Slopes (%)	Flow Area (sq/ft.)	Flow Depth (ft.)	Top Flow Width (ft.)	Flow Velocity (ft/sec.)	Q Available (cfs)	With Freeboard: Channel Depth (ft.)	With Freeboard: Top Channel Width (ft.)	With Freeboard: Q Available (cfs)
Stage 4 Pt 1		74.2	25		84	91	7.1 (Min)	0.7	GROUTED ROCK	0.025	2	33.3/40	7	1.3	9	13	91	2	13	254
Stage 4 Pt 1		74.2	25		84	91	10.0 (Max)	0.8	"	0.025	2	33.3/40	6.2	1.2	8.5	14.8	91	2	13	300
Stage 3 Pt 1		62.5	25		83	72	7.1 (Min)	0.6	"	0.025	4	33.3/40	6.1	0.9	9.1	11.9	72	1.5	12.3	188
Stage 3 Pt 1		62.5	25		83	72	10.0 (Max)	0.7	"	0.025	4	33.3/40	5.4	0.8	8.7	13.4	72	1.5	12.3	223

SEE DRAWING D-728-1063 FOR HAUL ROAD PROFILES AND DRAWING D-728-1064 FOR HAUL ROAD TYPICAL SECTIONS. CHANNEL PROFILE IS DEFINED BY THESE DRAWINGS.

STAGE 4 PART 1 HAUL ROAD DIRTY WATER DITCH EXTENDS FROM STATION 1+40 TO END ON THE STAGE 4 HAUL ROAD.

STAGE 3 PART 1 HAUL ROAD DIRTY WATER DITCH EXTENDS FROM STATION 27+00 TO END ON THE STAGE 3 HAUL ROAD.

SEE DIRTY WATER DITCHES AND RELATED FACILITIES CALC FOR DESIGN.

Elevation

Stations

PROFILE

Vertical Scale 1" - ________
Horizontal Scale 1" = ________

FORM I
DIVERSION/COLLECTION DITCH DATA SHEET

Title: HAUL ROAD DIRTY WATER DITCH	Site: Keystone West Valley		
Prepared by: PMK / SGR	Telephone Number:	Date: 7/25/96	Sheet 18 of 30

Estimated Peak Storm Intensity: ________ (in./hr.) **Design Calculations:** ________

Station Start End	Station Elevation	Drainage Area (acres)	Design Storm (yrs.)	Average Watershed Slope (%)	Curve Number	Peak Discharge Q (cfs)	Channel Bed Slope (%)	Freeboard (ft.)	Channel Lining	Manning's Coefficient (n)	Channel Bottom Width (ft)	Channel Side Slopes (%)	Flow Area (sq/ft.)	Flow Depth (ft.)	Top Flow Width (ft.)	Flow Velocity (ft/sec.)	Q Available (cfs)	With Freeboard: Channel Depth (ft.)	With Freeboard: Top Channel Width (ft.)	With Freeboard: Q Available (cfs)
PART	2	4.5	25		98	24	1.0 (Min)	1.1	USM	0.015	2	40	3.6	0.9	6.3	6.6	24	2	12	148
PART	2	4.5	25		98	24	10.0 (Max)	1.5	USM	0.015	2	40	1.6	0.5	4.4	15.3	24	2	12	468

SEE DRAWING D-728-1063 FOR HAUL ROAD PROFILES AND DRAWING D-728-1064 FOR HAUL ROAD TYPICAL SECTIONS. CHANNEL PROFILE IS DEFINED BY THESE DRAWINGS.

PART 2 OF THE HAUL ROAD DIRTY WATER DITCH EXTENDS FROM STATION 4+00± TO STATION 26+50± ON THE STAGE 3 HAUL ROAD

SEE DIRTY WATER DITCHES AND RELATED FACILITIES FOR DESIGN CALC.

Elevation

Stations

PROFILE

Vertical Scale 1" - ________
Horizontal Scale 1" = ________

7

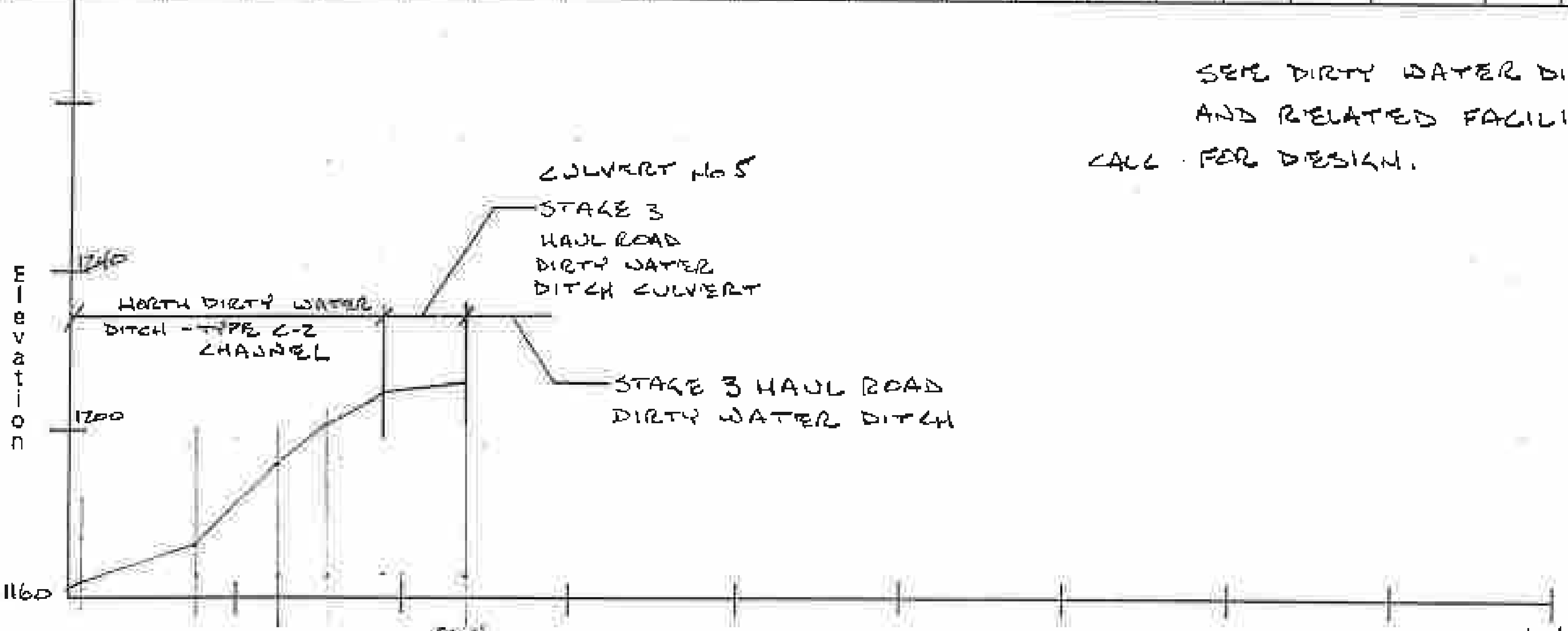
FORM I
DIVERSION/COLLECTION DITCH DATA SHEET

Title: North Dirty Water Ditch	Site: Keystone West Valley		
Prepared by: PMK/SSR	Telephone Number:	Date: 7/25/96	Sheet 19 of 30

Estimated Peak Storm Intensity: ________ (in./hr.) **Design Calculations:** ________

Station Start End	Station Elevation	Drainage Area (acres)	Design Storm (yrs.)	Average Watershed Slope (%)	Curve Number	Peak Discharge Q (cfs)	Channel Bed Slope (%)	Freeboard (ft.)	Channel Lining	Manning's Coefficient (n)	Channel Bottom Width (ft)	Channel Side Slopes (%)	Flow Area (sq/ft.)	Flow Depth (ft.)	Top Flow Width (ft.)	Flow Velocity (ft/sec.)	Q Available (cfs)	With Freeboard: Channel Depth (ft.)	With Freeboard: Top Channel Width (ft.)	With Freeboard: Q Available (cfs)
Both		66.0	25		83	75	7.1 (Min)	0.7	Grouted Rock	0.025	2	50	5.7	1.3	7.1	13.1	75	2	10	203
Both		66.0	25		83	75	25.0 (Max)	1.1	Grouted Rock	0.025	2	50	3.6	0.9	5.7	20.8	75	2	10	390

SEE DIRTY WATER DITCHES AND RELATED FACILITIES CALC FOR DESIGN.



PROFILE

Vertical Scale 1" = 40'
Horizontal Scale 1" = 200'

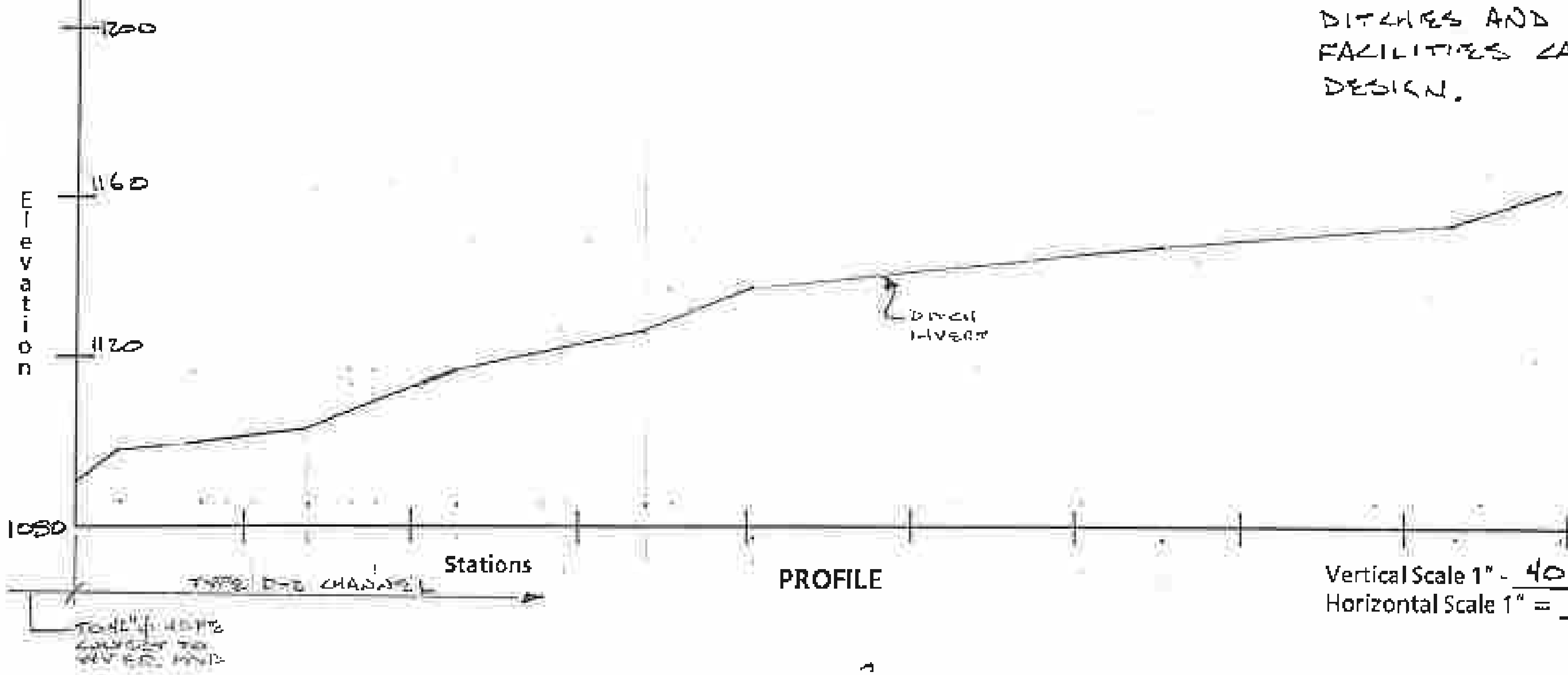
FORM I
DIVERSION/COLLECTION DITCH DATA SHEET

Title: West Dirty Water Ditch | Site: Keystone Bear Valley
Prepared by: PMK / SER | Telephone Number: | Date: 7/25/96 | Sheet 20 of 30

Estimated Peak Storm Intensity: ________ (in./hr.)   **Design Calculations:** ________

Station Start End	Station Elevation	Drainage Area (acres)	Design Storm (yrs.)	Average Watershed Slope (%)	Curve Number	Peak Discharge Q (cfs)	Channel Bed Slope (%)	Freeboard (ft)	Channel Lining	Manning's Coefficient (n)	Channel Bottom Width (ft)	Channel Side Slopes (%)	Flow Area (sq/ft)	Flow Depth (ft.)	Top Flow Width (ft)	Flow Velocity (ft/sec.)	Q Available (cfs)	With Freeboard: Channel Depth (ft.)	With Freeboard: Top Channel Width (ft.)	With Freeboard: Q Available (cfs)
Part I		74.7	25		83	91	1.0 (Min)	0.8	USM	0.015	2	50	9.4	1.7	8.9	9.7	91	2.5	12	210
Part I		74.7	25		83	91	25.0 (Max)	1.7	USM	0.015	2	50	2.9	0.8	5.2	31.9	91	2.5	12	1,000

SEE DIRTY WATER DITCHES AND RELATED FACILITIES CALL FOR DESIGN.



PROFILE

Vertical Scale 1" - 40'
Horizontal Scale 1" = 200'

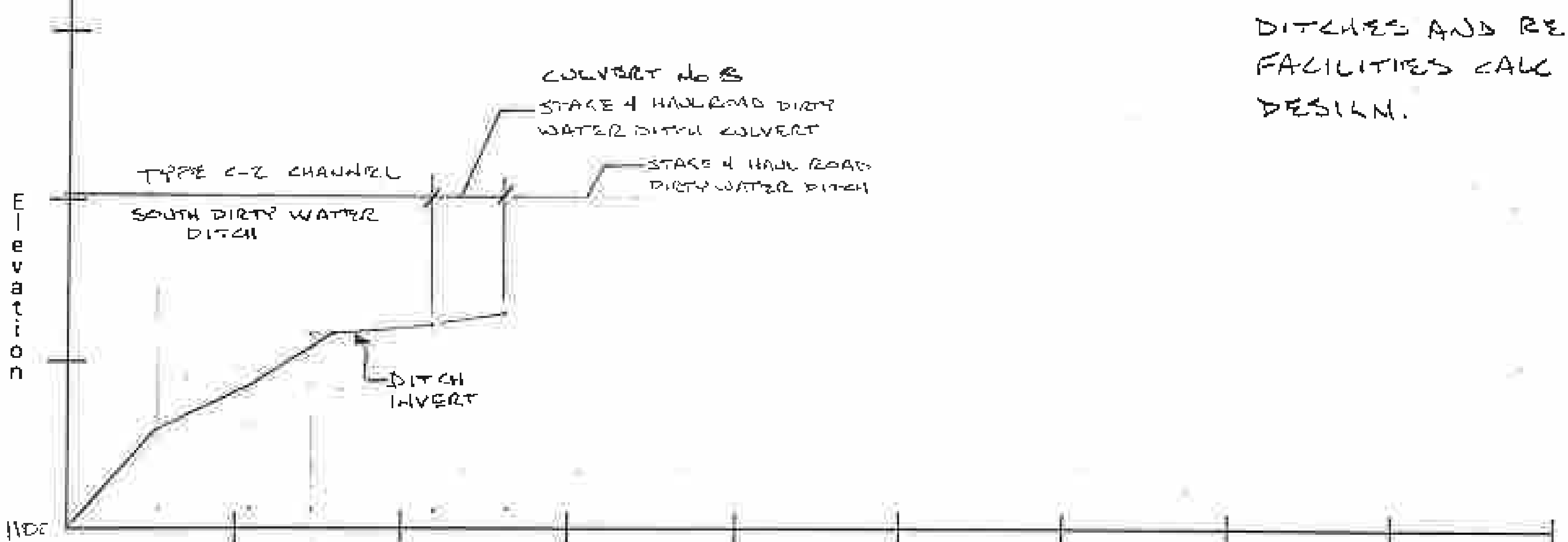
FORM I
DIVERSION/COLLECTION DITCH DATA SHEET

Title: South Dirty Water Ditch	Site: Keystone Warm Valley		
Prepared by: BHK / SER	Telephone Number:	Date: 7/25/96	Sheet 21 of 30

Estimated Peak Storm Intensity: ________ (in./hr.) ***Design Calculations:*** ________

Station		Drainage Area (acres)	Design Storm (yrs.)	Average Watershed Slope (%)	Curve Number	Peak Discharge Q (cfs)	Channel Bed Slope (%)	Freeboard (ft.)	Channel Lining	Manning's Coefficient (n)	Channel Bottom Width (ft)	Channel Side Slopes (%)	Flow Area (sq/ft.)	Flow Depth (ft.)	Top Flow Width (ft.)	Flow Velocity (ft/sec.)	Q Available (cfs)	With Freeboard		
Start End	Elevation																	Channel Depth (ft.)	Top Channel Width (ft.)	Q Available (cfs)
Part 1		74.7	25		83	91	6.3 (Min)	0.6	Grouted Rock	0.025	2	50	6.9	1.4	7.7	13.1	91	2	10	190
Part 1		74.7	25		83	91	25.0 (Max)	1.0	"	0.025	2	50	4.2	1.0	6.1	21.9	91	2	10	380

SEE DIRTY WATER DITCHES AND RELATED FACILITIES CALC FOR DESIGN.



Stations

PROFILE

Vertical Scale 1" - 40'
Horizontal Scale 1" = 200'

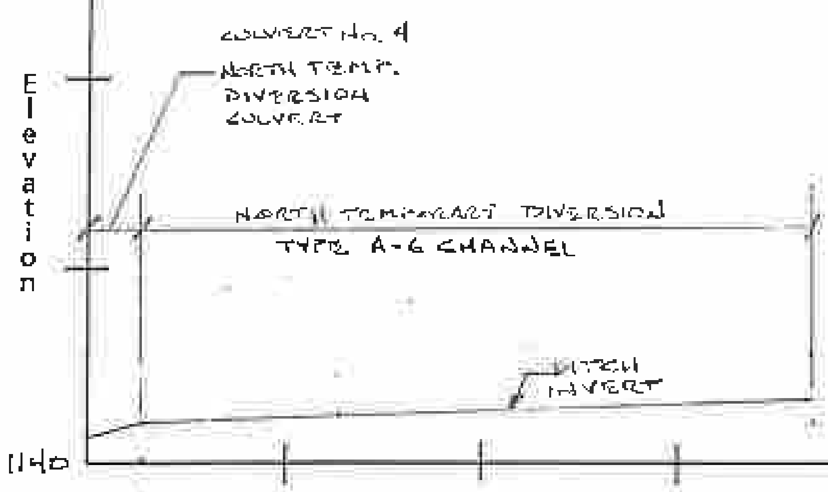
FORM I
DIVERSION/COLLECTION DITCH DATA SHEET

Title: NORTH TEMPORARY DIVERSION DITCH	Site: Keystone No - Valley		
Prepared by: PMK/SER	Telephone Number:	Date: 7/25/96	Sheet 22 of 30

Estimated Peak Storm Intensity: ______ (in./hr.) **Design Calculations:** ______

Station																			With Freeboard		
Start End	Elevation	Drainage Area (acres)	Design Storm (yrs)	Average Watershed Slope (%)	Curve Number	Peak Discharge Q (cfs)	Channel Bed Slope (%)	Freeboard (ft.)	Channel Lining	Manning's Coefficient (n)	Channel Bottom Width (ft)	Channel Side Slopes (%)	Flow Area (sq/ft.)	Flow Depth (ft.)	Top Flow Width (ft.)	Flow Velocity (ft/sec.)	Q Available (cfs)	Channel Depth (ft.)	Top Channel Width (ft.)	Q Available (cfs)
Part 1		6.6	25		80	19	1.0	0.6	GRASS	0.045	2	50	6.6	1.4	7.5	2.9	19	2	10	42

SEE DIRTY WATER DITCHES AND RELATED FACILITIES CALC FOR DESIGN.



PROFILE

Vertical Scale 1" - 40'
Horizontal Scale 1" = 200'

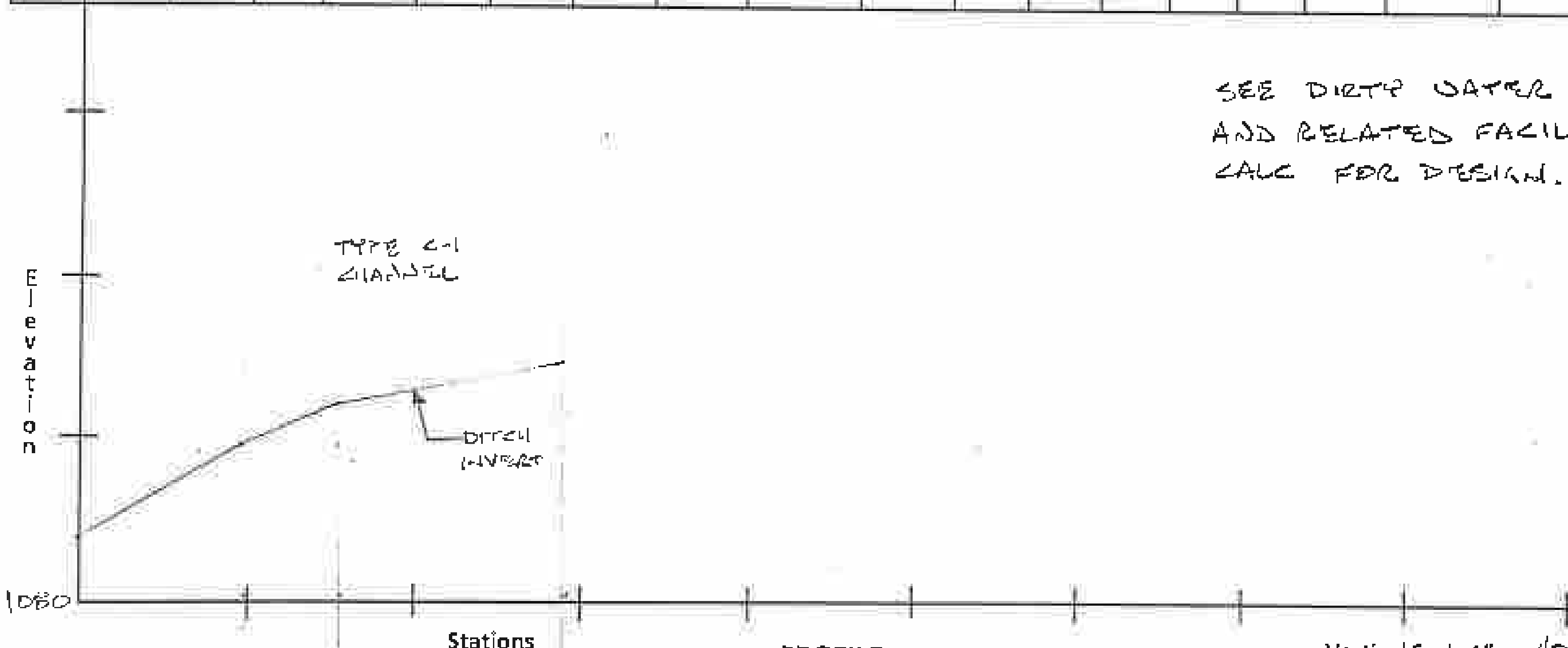
FORM I
DIVERSION/COLLECTION DITCH DATA SHEET

Title: EAST TEMPORARY DIVERSION DITCH	Site: Keystone [illegible] Valley		
Prepared by: PMK	Telephone Number:	Date: [illegible]	Sheet 23 of 30

Estimated Peak Storm Intensity: ________(in./hr.) **Design Calculations:** ________

Station																				With Freeboard		
Start End	Elevation	Drainage Area (acres)	Design Storm (yrs.)	Average Watershed Slope (%)	Curve Number	Peak Discharge Q (cfs)	Channel Bed Slope (%)	Freeboard (ft.)	Channel Lining	Manning's Coefficient (n)	Channel Bottom Width (ft)	Channel Side Slopes (%)	Flow Area (sq/ft.)	Flow Depth (ft.)	Top Flow Width (ft.)	Flow Velocity (ft/sec.)	Q Available (cfs)	Channel Depth (ft.)	Top Channel Width (ft.)	Q Available (cfs)
Part 1		3.5	25		80	10	1.0 (Min)	0.7	GROUTED ROCK	0.025	2	50	2.7	0.8	5	3.7	10	1.5	8	40
Part 1		3.5	25		80	10	25.0 (Max)	1.2	"	0.025	2	50	0.9	0.3	3.3	11.7	10	1.5	8	202

SEE DIRTY WATER DITCHES AND RELATED FACILITIES CALC FOR DESIGN.



PROFILE

Vertical Scale 1" - 40'
Horizontal Scale 1" = 200'

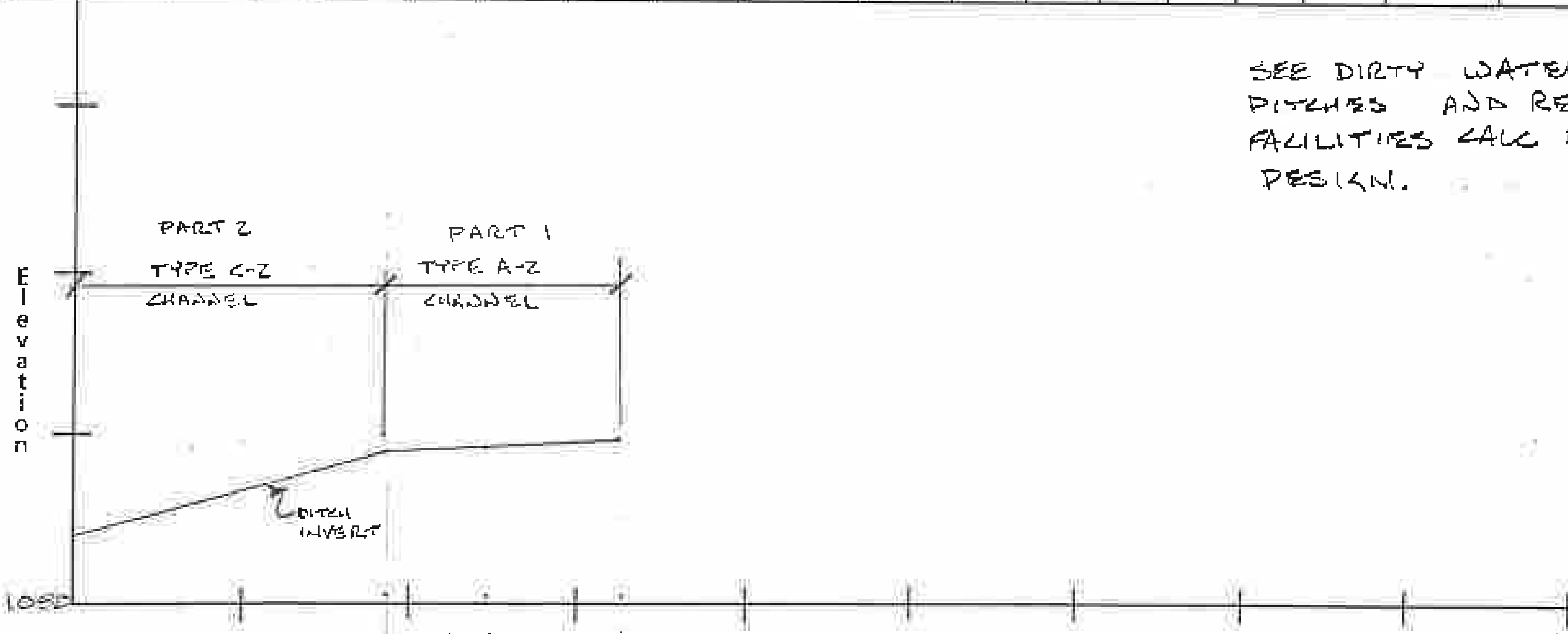
FORM I
DIVERSION/COLLECTION DITCH DATA SHEET

Title: Pond Diversion Ditch	Site: Keystone West Valley		
Prepared by: PMK/SER	Telephone Number:	Date: 7/25/96	Sheet 24 of 30

Estimated Peak Storm Intensity: ________ (in./hr.) ***Design Calculations:*** ________

Station																		With Freeboard		
Start End	Elevation	Drainage Area (acres)	Design Storm (yrs.)	Average Watershed Slope (%)	Curve Number	Peak Discharge Q (cfs)	Channel Bed Slope (%)	Freeboard (ft.)	Channel Lining	Manning's Coefficient (n)	Channel Bottom Width (ft)	Channel Side Slopes (%)	Flow Area (sq/ft.)	Flow Depth (ft.)	Top Flow Width (ft.)	Flow Velocity (ft/sec.)	Q Available (cfs)	Channel Depth (ft)	Top Channel Width (ft.)	Q Available (cfs)
Part 1		3.0	25		80	9	1.0	0.5	Grass	0.045	2	50	3.8	1.0	5.7	2.4	9	1.5	8	22
Part 2		3.0	25		80	9	1.0 (Min)	1.3	Grouted Rock	0.025	2	50	2.5	0.7	4.9	3.6	9	2	10	76
Part 2		3.0	25		80	9	10.0 (Max)	1.6	"	0.025	2	50	1.1	0.4	3.6	8.3	9	2	10	240

SEE DIRTY WATER DITCHES AND RELATED FACILITIES CALC FOR DESIGN.



PROFILE

Vertical Scale 1" - 40'
Horizontal Scale 1" = 200'

FORM I

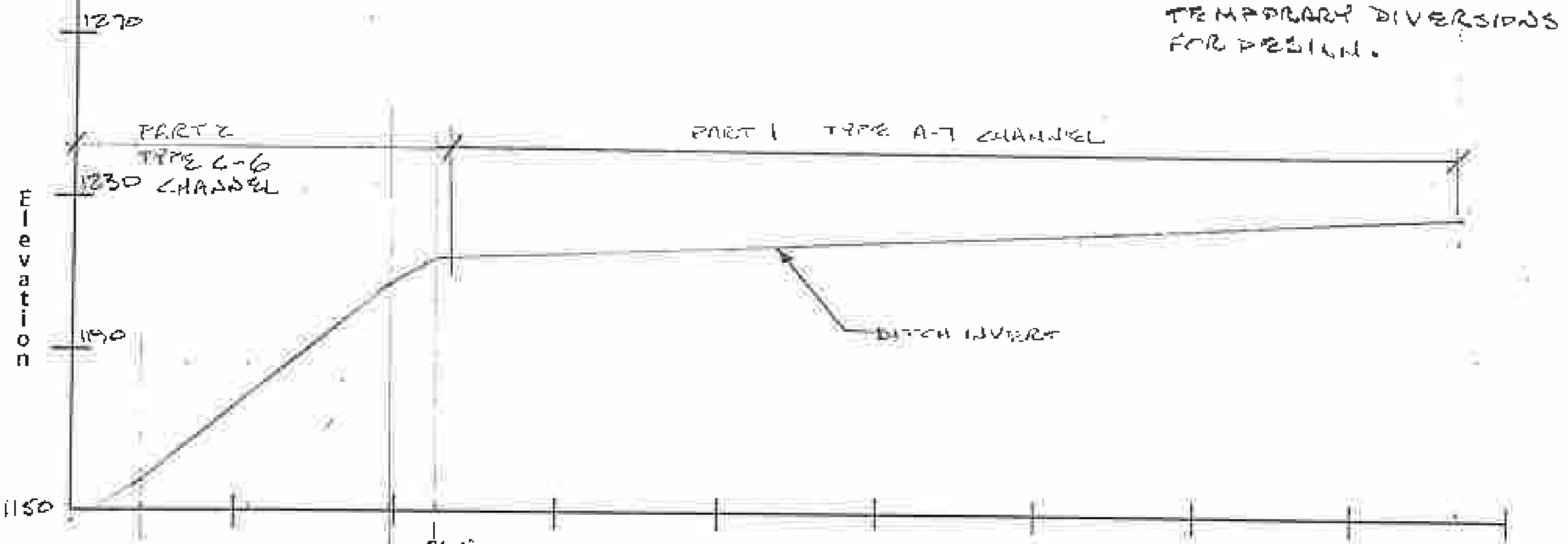
DIVERSION/COLLECTION DITCH DATA SHEET

Title: Diversion Ditch D31	Site: Keystone West Valley
Prepared by: PMK/SER	Telephone Number:
Date: 7/25/96	Sheet 25 of 30

Estimated Peak Storm Intensity: ______ (in./hr.) **Design Calculations:** ______

Station		Drainage Area (acres)	Design Storm (yrs.)	Average Watershed Slope (%)	Curve Number	Peak Discharge Q (cfs)	Channel Bed Slope (%)	Freeboard (ft.)	Channel Lining	Manning's Coefficient (n)	Channel Bottom Width (ft)	Channel Side Slopes (%)	Flow Area (sq. ft.)	Flow Depth (ft.)	Top Flow Width (ft.)	Flow Velocity (ft/sec)	Q Available (cfs)	With Freeboard		
Start End	Elevation																	Channel Depth (ft.)	Top Channel Width (ft.)	Q Available (cfs)
Part 1		28.5	25		79	54.5	1.0 (Min)	0.4	Grass	0.045	3	50	14.6	2.1	11.2	3.7	54.5	2.5	13	83
Part 1		28.5	25		79	54.5	16.1 (Max)	1.5	"	0.045	3	50	5.3	1.0	7.1	10.4	54.5	2.5	13	334
Part 2		28.5	25		79	54.5	1.0 (Min)	0.9	Grouted Rock	0.025	3	50	7.5	1.6	9.2	5.8	54.5	2.5	13	150
Part 2		28.5	25		79	54.5	16.1 (Max)	1.7	"	0.025	3	50	3.4	0.8	6	15.8	54.5	2.5	13	602

SEE STAGE 3 AND STAGE 4 TEMPORARY DIVERSIONS CALC FOR DESIGN.



PROFILE

Vertical Scale 1" = 40'
Horizontal Scale 1" = 200'

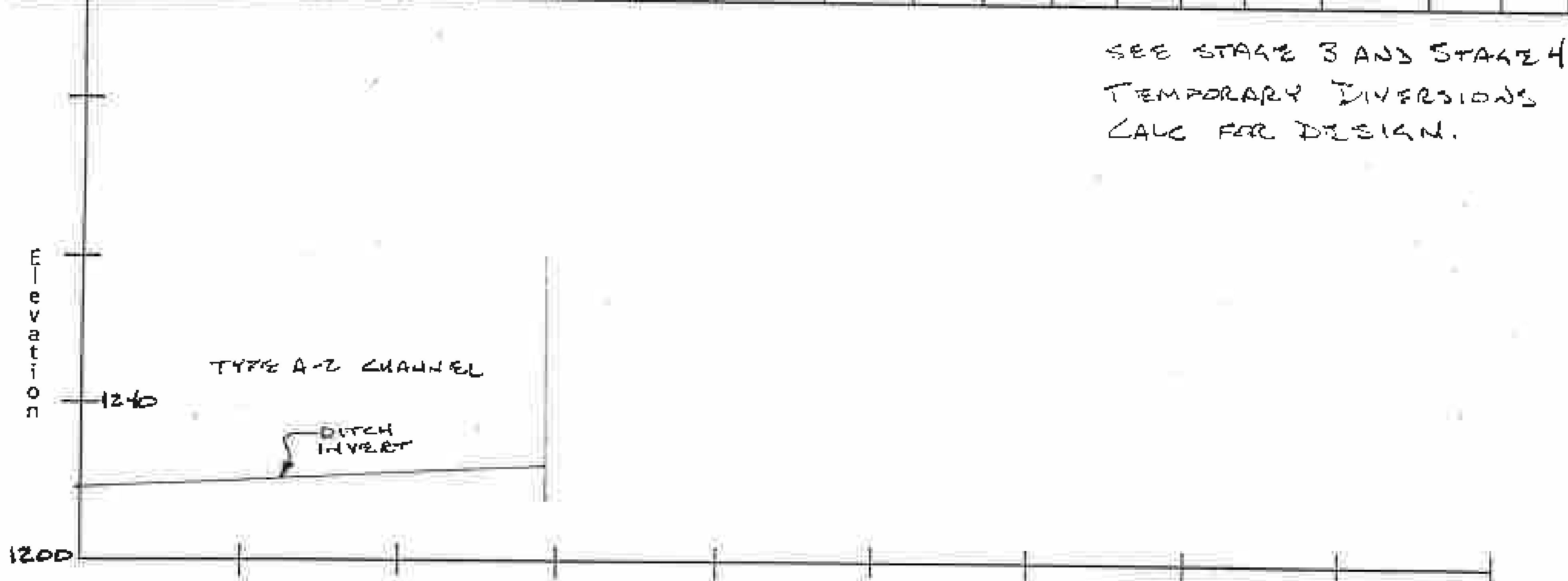
FORM I
DIVERSION/COLLECTION DITCH DATA SHEET

Title: Diversion Ditch D32	Site: Keystone Wes- Valley		
Prepared by: PMK/SGR	Telephone Number:	Date: 7/25/96	Sheet 26 of 30

Estimated Peak Storm Intensity: ______ (in./hr.) **Design Calculations:** ______

Station		Drainage Area (acres)	Design Storm (yrs.)	Average Watershed Slope (%)	Curve Number	Peak Discharge Q (cfs)	Channel Bed Slope (%)	Freeboard (ft.)	Channel Lining	Manning's Coefficient (n)	Channel Bottom Width (ft)	Channel Side Slopes (%)	Flow Area (sq. ft.)	Flow Depth (ft.)	Top Flow Width (ft.)	Flow Velocity (ft/sec.)	Q Available (cfs)	With Freeboard		
Start End	Elevation																	Channel Depth (ft.)	Top Channel Width (ft.)	Q Available (cfs)
Pond 1		2.4	25		80	6.6	1.0	0.7	Grass	0.045	2	50	3	0.8	5.3	2.2	6.6	1.5	8	22

SEE STAGE 3 AND STAGE 4 TEMPORARY DIVERSIONS CALC FOR DESIGN.



PROFILE

Vertical Scale 1" - 40'
Horizontal Scale 1" = 200'

FORM I
DIVERSION/COLLECTION DITCH DATA SHEET

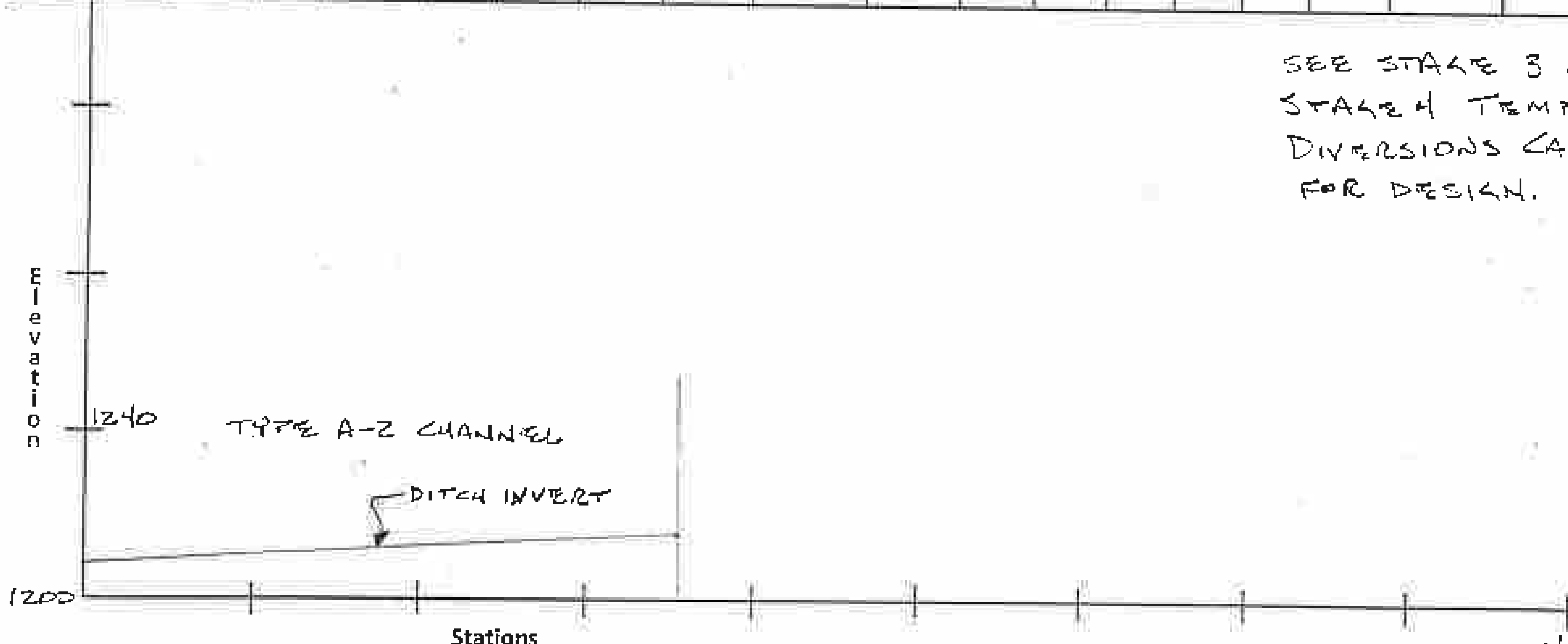
Title: Diversion Ditch D33 | Site: Keystone West Valley

Prepared by: PMK / SER | Telephone Number: | Date: 7/25/96 | Sheet 27 of 80

Estimated Peak Storm Intensity: ________ (in./hr.) *Design Calculations:* ________

Station: Start End	Station: Elevation	Drainage Area (acres)	Design Storm (yrs.)	Average Watershed Slope (%)	Curve Number	Peak Discharge Q (cfs)	Channel Bed Slope (%)	Freeboard (ft.)	Channel Lining	Manning's Coefficient (n)	Channel Bottom Width (ft)	Channel Side Slopes (%)	Flow Area (sq/ft.)	Flow Depth (ft.)	Top Flow Width (ft.)	Flow Velocity (ft/sec.)	Q Available (cfs)	With Freeboard: Channel Depth (ft.)	With Freeboard: Top Channel Width (ft.)	With Freeboard: Q Available (cfs)
Full		29	25		80	7.9	1.0	0.6	Grass	0.045	2	50	3.5	0.9	5.6	2.3	7.9	1.5	8	22

SEE STAGE 3 AND STAGE 4 TEMPORARY DIVERSIONS CALC FOR DESIGN.



PROFILE

Vertical Scale 1" = 40'
Horizontal Scale 1" = 200'

FORM I

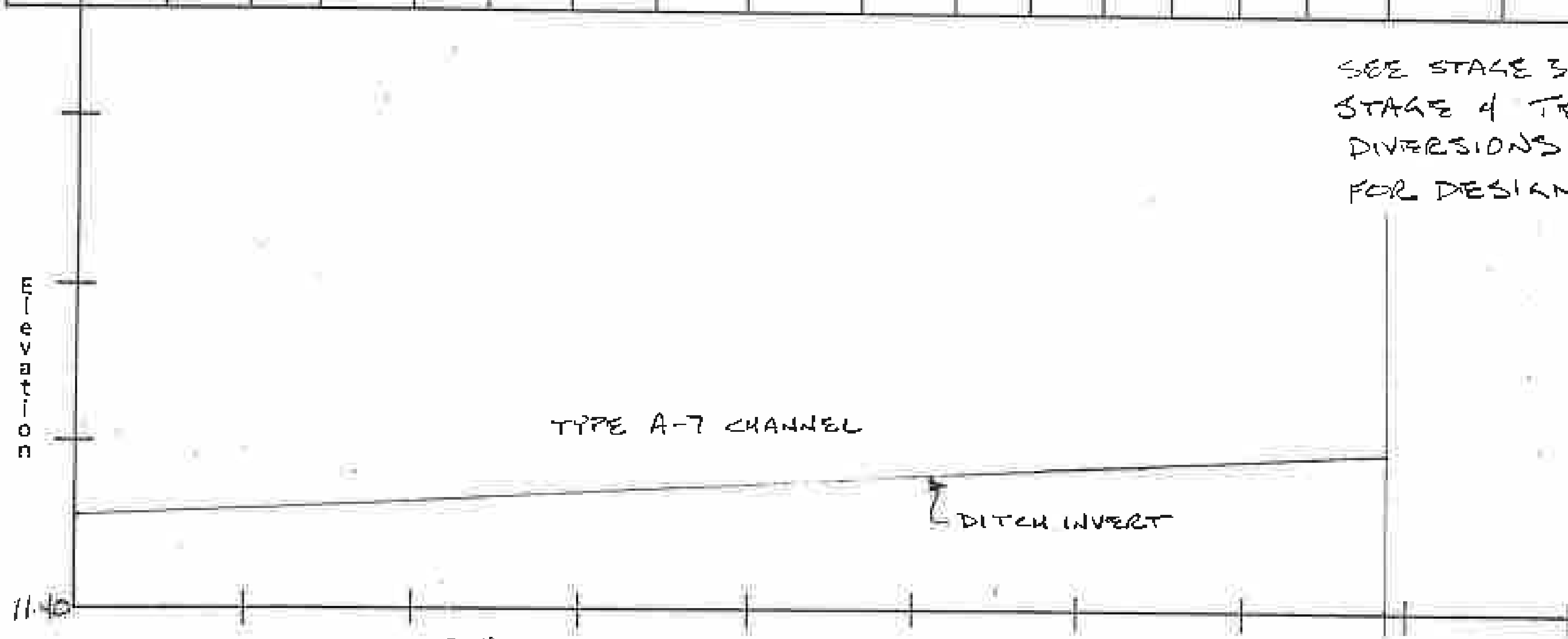
DIVERSION/COLLECTION DITCH DATA SHEET

Title: Diversion Ditch D41	Site: Keystone West Valley		
Prepared by: PMK/SER	Telephone Number:	Date: 7/25/16	Sheet 28 of 30

Estimated Peak Storm Intensity: ______ (in./hr.) **Design Calculations:** ______

Station Start End	Station Elevation	Drainage Area (acres)	Design Storm (yrs.)	Average Watershed Slope (%)	Curve Number	Peak Discharge Q (cfs)	Channel Bed Slope (%)	Freeboard (ft.)	Channel Lining	Manning's Coefficient (n)	Channel Bottom Width (ft)	Channel Side Slopes (%)	Flow Area (sq. ft.)	Flow Depth (ft.)	Top Flow Width (ft.)	Flow Velocity (ft/sec)	Q Available (cfs)	With Freeboard: Channel Depth (ft.)	With Freeboard: Top Channel Width (ft.)	With Freeboard: Q Available (cfs)
Part 1		26.5	25		80	52.8	1.0	0.5	Grass	0.045	3	50	14.3	2.0	11.1	3.7	52.8	2.5	13	83

SEE STAGE 3 AND STAGE 4 TEMPORARY DIVERSIONS CALC FOR DESIGN.



PROFILE

Vertical Scale 1" - 40'
Horizontal Scale 1" = 200'

FORM I
DIVERSION/COLLECTION DITCH DATA SHEET

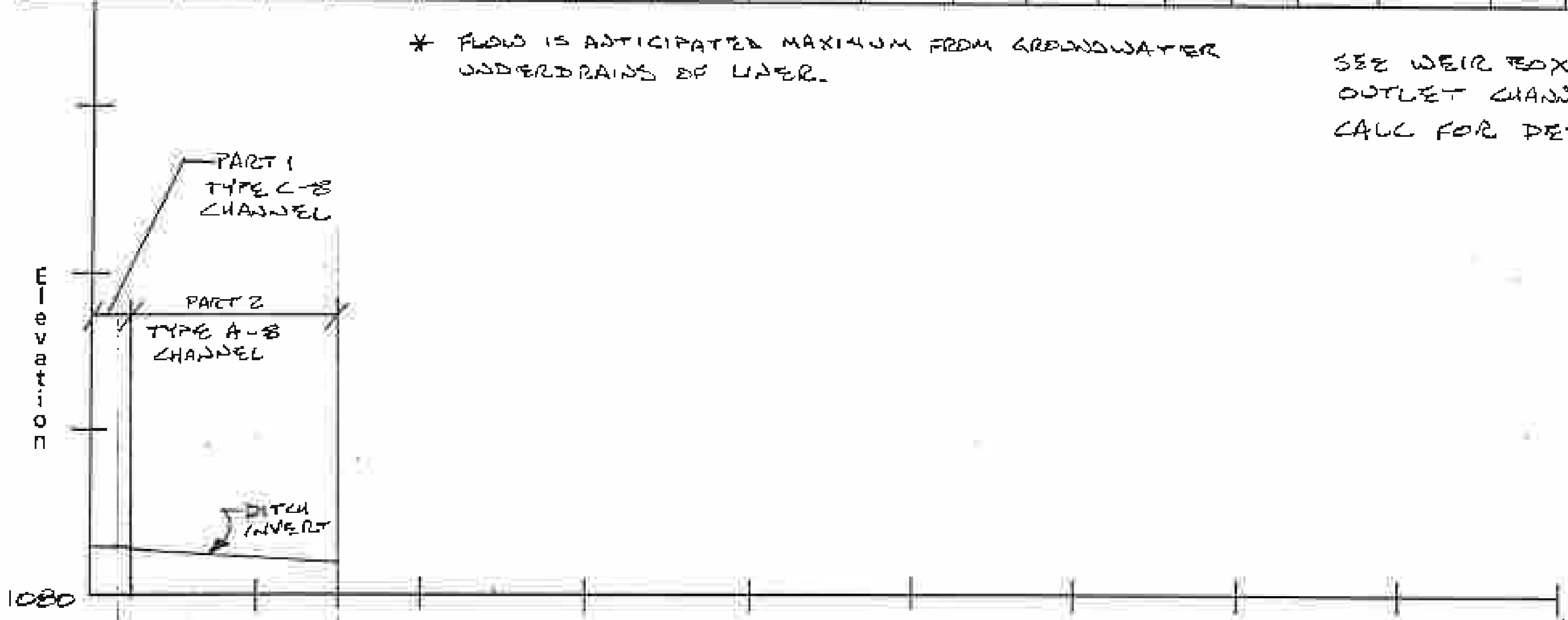
Title: WEIR BOX OUTLET CHANNEL	Site: KEYSTONE WEST VALLEY		
Prepared by: SER	Telephone Number:	Date: 7/31/96	Sheet 29 of 30

Estimated Peak Storm Intensity: ________ (in./hr.) ***Design Calculations:*** ________

Station																				
																		With Freeboard		
Start End	Elevation	Drainage Area (acres)	Design Storm (yrs)	Average Watershed Slope (%)	Curve Number	Peak Discharge Q (cfs)	Channel Bed Slope (%)	Freeboard (ft.)	Channel Lining	Manning's Coefficient (n)	Channel Bottom Width (ft)	Channel Side Slopes (%)	Flow Area (sq/ft.)	Flow Depth (ft.)	Top Flow Width (ft.)	Flow Velocity (ft/sec.)	Q Available (cfs)	Channel Depth (ft)	Top Channel Width (ft.)	Q Available (cfs)
PART 1		*	25		*	1*	1	0.5	GROUTED ROCK	0.025	0	50	0.5	0.5	1.9	2.1	1	1	4	7
PART 2		*	25		*	1*	6.7	0.6	GRASS	0.045	0	50	0.4	0.4	1.7	2.8	1	1	4	10

* FLOW IS ANTICIPATED MAXIMUM FROM GROUNDWATER UNDERDRAINS OF LINER.

SEE WEIR BOX OUTLET CHANNEL CALC FOR DESIGN



Stations

PROFILE

Vertical Scale 1" - 40'
Horizontal Scale 1" = 40'

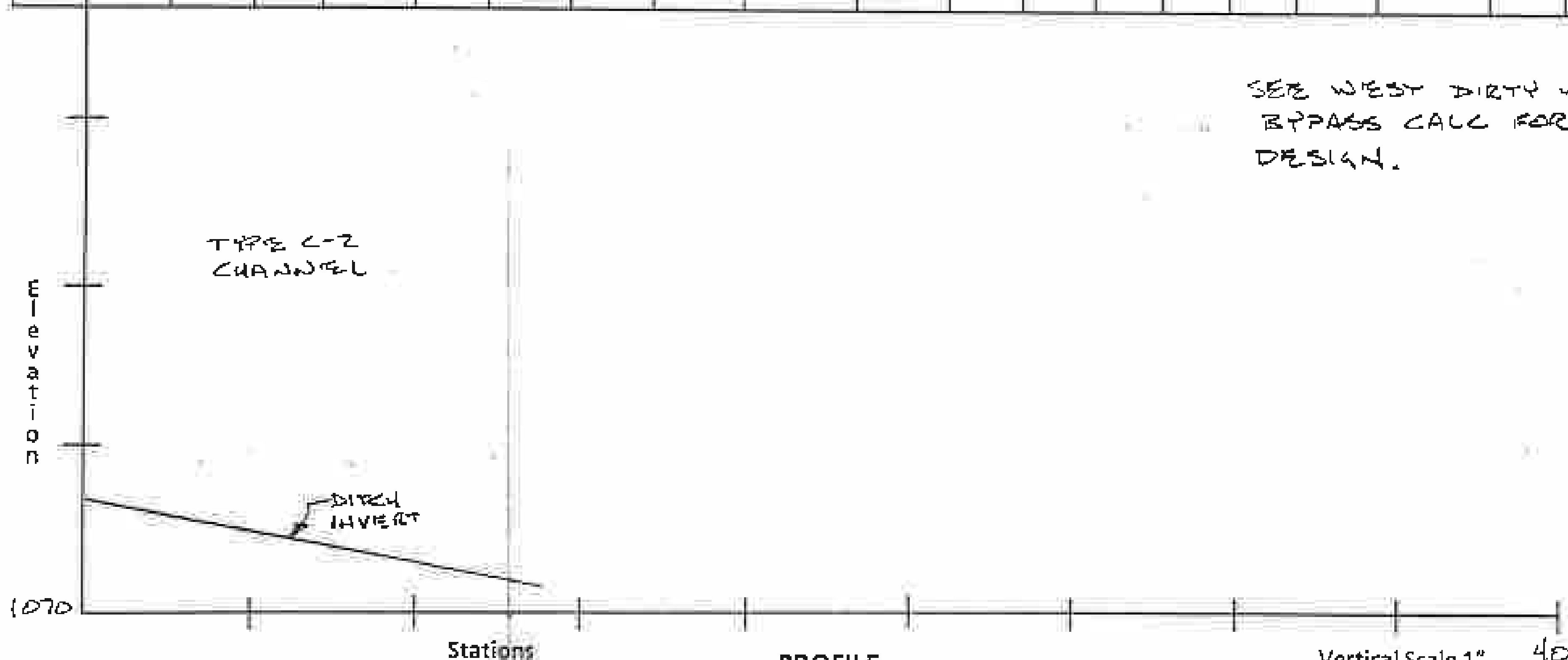
FORM I
DIVERSION/COLLECTION DITCH DATA SHEET

Title: WEST DIRTY WATER DITCH BYPASS CHANNEL	Site: KEYSTONE WEST VALLEY		
Prepared by:	Telephone Number:	Date: 7/31/96	Sheet 30 of 30

Estimated Peak Storm Intensity: ________ (in./hr.) **Design Calculations:** ________

Station		Drainage Area (acres)	Design Storm (yrs.)	Average Watershed Slope (%)	Curve Number	Peak Discharge Q (cfs)	Channel Bed Slope (%)	Freeboard (ft.)	Channel Lining	Manning's Coefficient (n)	Channel Bottom Width (ft)	Channel Side Slopes (%)	Flow Area (sq/ft.)	Flow Depth (ft.)	Top Flow Width (ft.)	Flow Velocity (ft/sec.)	Q Available (cfs)	With Freeboard		
Start End	Elevation																	Channel Depth (ft.)	Top Channel Width (ft.)	Q Available (cfs)
Part 1		74.7	25		83	91	20	0.9	GROUTED ROCK	0.025	2	50	4.5	1.1	6.3	20.2	91	2.0	10.0	340

SEE WEST DIRTY WATER BYPASS CALC FOR DESIGN.



PROFILE

Vertical Scale 1" - 40'
Horizontal Scale 1" = 40'

SUBJECT KEYSTONE WEST VALLEY
PHASE II PERMITTING

BY SER DATE 3/19/96 PROJ. NO. 92-220-73-7

CHKD. BY KMB DATE 4/10/96 SHEET NO. 1 OF 45

GAI CONSULTANTS, INC.
Engineers • Geologists • Planners
Environmental Specialists

ULTIMATE CONDITIONS - DRAINAGE FACILITIES

HYDROLOGY

PURPOSE: ESTIMATE THE DESIGN FLOWS FOR; THE PROPOSED AND EXISTING DRAINAGE FACILITIES WHICH WILL DRAIN THE PROPOSED WEST VALLEY, AND EXISTING FLOWS IN THE UNNAMED TRIBUTARIES OF CROOKED CREEK WHICH DRAIN TO THE SOUTH AND WEST OF THE SITE.

- DIRTY WATER DITCHES ARE DESIGNED IN A SEPARATE CALC. SET.

DESIGN STORMS: ALL DRAINAGE FACILITIES ARE TO BE DESIGNED TO PASS THE RUNOFF FROM THE 25-YEAR, 24-HOUR STORM AS REQUIRED IN CHAPTER 288.151 AND 288.242 SOIL EROSION AND SEDIMENTATION CONTROL PLAN.

NO STORMWATER MANAGEMENT REQUIREMENTS EXIST FOR PLUM CREEK OR ARMSTRONG COUNTY. THE RESIDUAL WASTE REGULATIONS HAVE NO SPECIFIC REQUIREMENTS.

AS PER CONVERSATIONS WITH ARMSTRONG COUNTY PERSONELL, ARMSTRONG COUNTY WILL REVIEW THE STORMWATER MANAGEMENT DESIGN.

USE THE 2-YEAR, 10-YEAR, AND 100-YEAR 24 HR STORMS FOR STORMWATER MANAGEMENT FACILITY DESIGN/ANALYSIS.

METHODOLOGY: TR-55, "URBAN HYDROLOGY FOR SMALL WATERSHEDS", SCS, JUNE 1986 AND TR-20

SUBJECT Keystone West Valley
Phase II Permitting

BY SER DATE 3/19/96 PROJ. NO. 92-220-73-7
CHKD. BY KMB DATE 6/10/96 SHEET NO. 3 OF 45
SHEET 2 OMITTED

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HYDROLOGY - ULTIMATE CONDITIONS

A SKETCH OF "ULTIMATE CONDITIONS DRAINAGE" IS SHOWN ON SHEET 4. A SCHEMATIC OF ULTIMATE CONDITIONS DRAINAGE CONDITIONS FACILITIES AND WATERSHEDS IS SHOWN ON SHEET 5.

THE ATTACHED WORKSHEET LABELLED "ULTIMATE CONDITIONS WORKSHEET" 92-220-7-SER1 SHOWS THE DITCHES, SLOPE DRAINS AND WATERSHEDS IN GREATER DETAIL.

NOTES:

1) TWO CULVERTS ARE REQUIRED TO CARRY THE SE DITCH AND THE PROPOSED HAUL ROAD DIRTY WATER DITCH UNDER THE HAUL ROAD NEAR THE LOCATION WHERE THE HAUL ROAD BEGINS CLIMBING THE PILE.

2) ALL DITCHES UNDER ULTIMATE CONDITIONS WILL BE CLEAN WATER DITCHES AND CARRY RUNOFF FROM STABILIZED AREAS. THE PROPOSED HAUL ROAD DIRTY WATER DITCH WILL CARRY DIRTY WATER TO THE SURGE POND UNTIL THE PILE IS COMPLETELY STABILIZED AT WHICH TIME IT WILL BE A CLEAN WATER DITCH AND WILL BE DIVERTED WITH A "BYPASS", SEE SHEET 5, TO THE SOUTH UNNAMED TRIBUTARY OF CROOKED CREEK.

3) STEEP CHANNELS AT OUTLETS OF DRAINAGE AREAS WILL NOT BE CONSIDERED FOR: TIME-OF-CONCENTRATION PATHS, TRAVEL TIMES, OR CHANNEL ROUTING IN TR-20, SINCE THE FLOW TIMES AND/OR CHANNEL ATTENUATIONS ASSOCIATED WITH THESE STEEP CHANNELS IS NEGLIGIBLE.

SUBJECT KEYSTONE WEST VALLEY
PHASE II PERMITTING

BY SER DATE 3/19/96 PROJ. NO. 92-220-73-7

CHKD. BY [illegible] DATE 6/10/96 SHEET NO. 4 OF 45

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ULTIMATE CONDITIONS

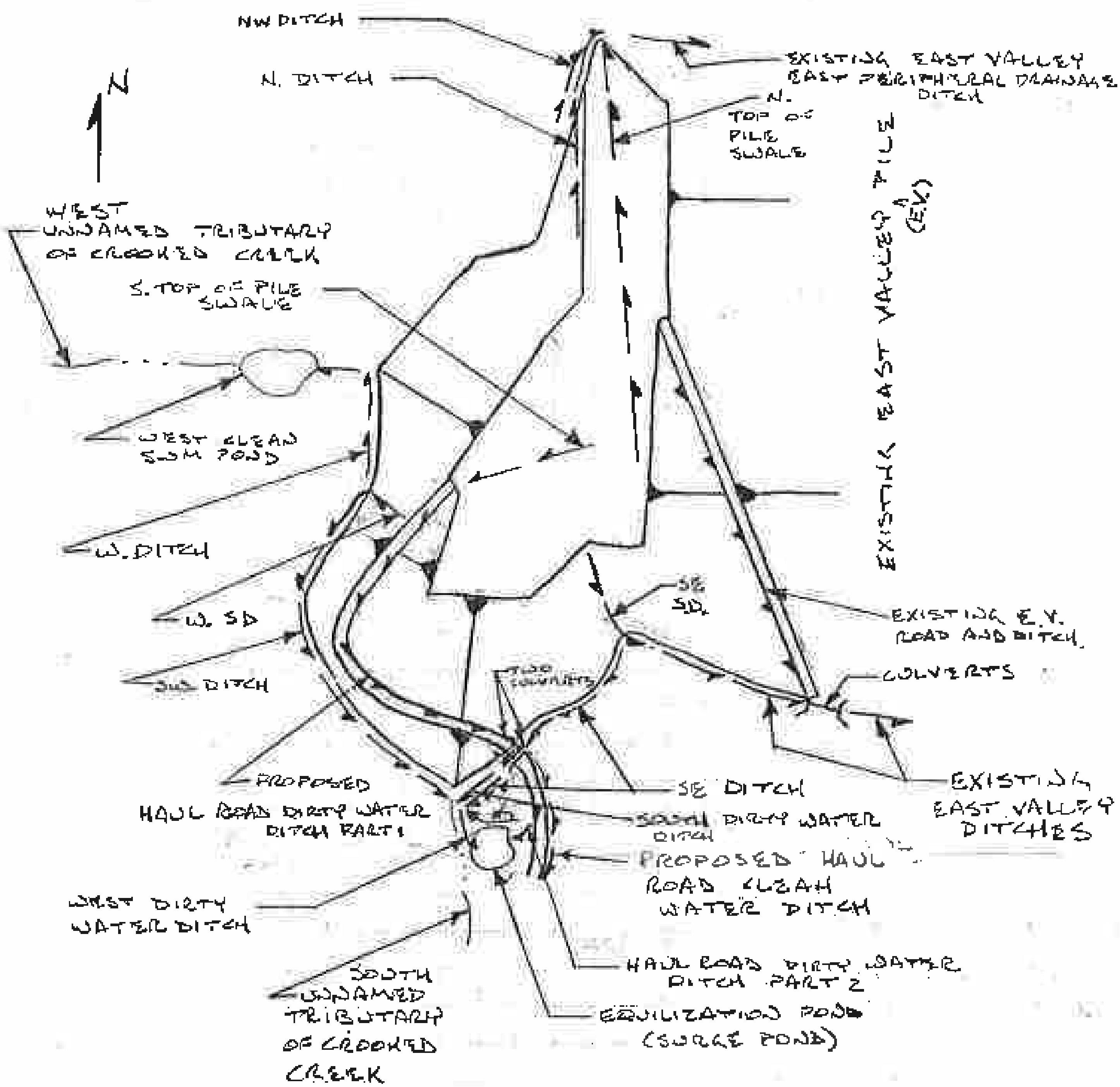
DRAINAGE SKETCH

N.T.S.

LEGEND

PILE SLOPE

DRAINAGE DITCH OR SLOPE DRAIN (S.D.)



* POND DIVERSION DITCH

SUBJECT: Keystone West Valley
Phase II Permitting

BY: SER DATE: 3/19/96 PROJ. NO. 92-220-73-7
CHKD. BY: KMB DATE: 6/10/96 SHEET NO. 5 OF 45

GAI CONSULTANTS, INC.
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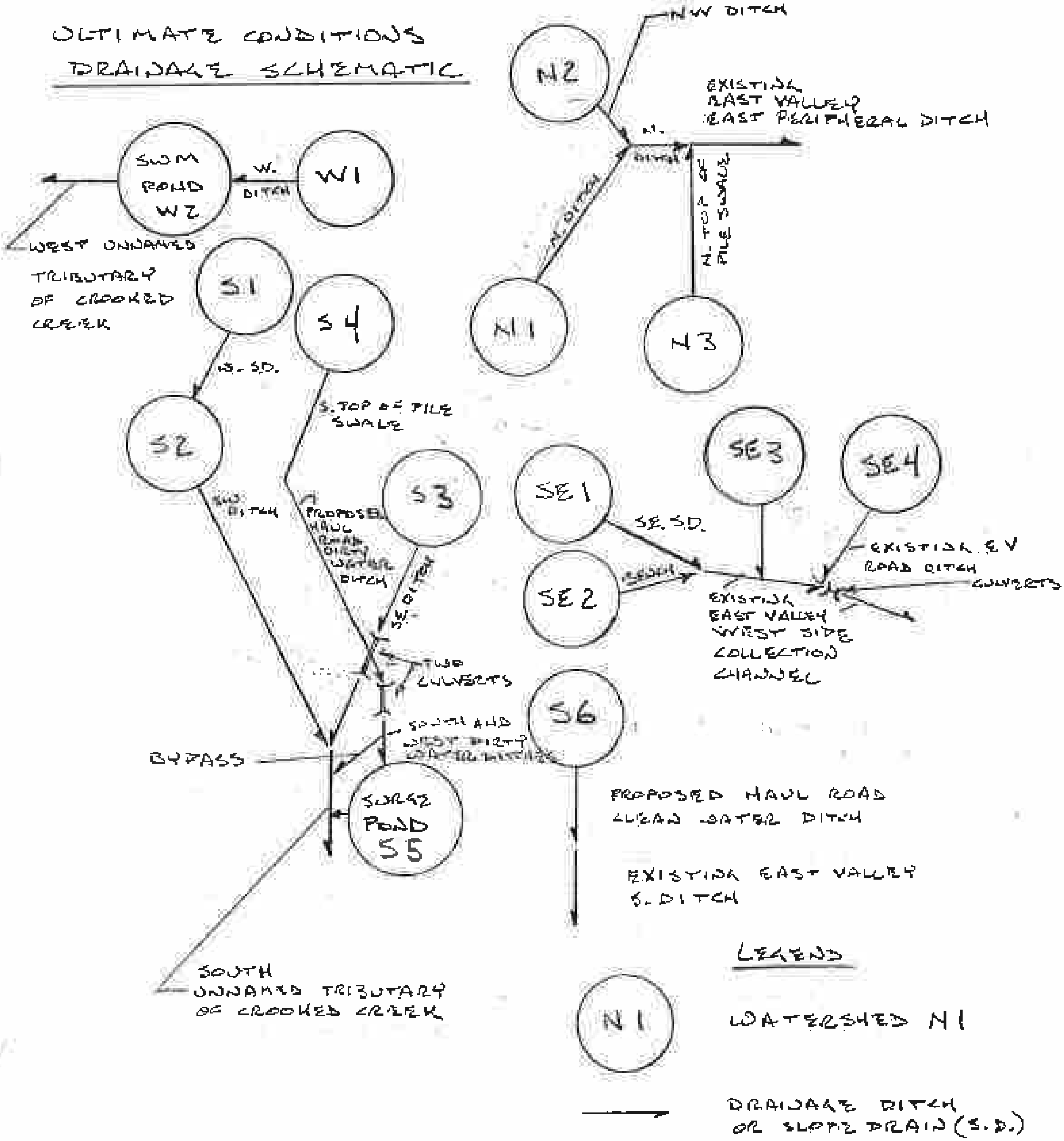
ULTIMATE CONDITIONS DRAINAGE SCHEMATIC



SUBJECT Keystone West Valley
Phase II Permitting

BY SER DATE 3/19/96 PROJ. NO. 92-220-73-7
CHKD. BY KMB DATE 6/10/96 SHEET NO. 6 OF 45

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CURVE NUMBERS, CN'S

REFERENCE: "PROJECT DESIGN PARAMETERS OUTLINE" KEYSTONE STATION, EAST VALLEY DISPOSAL AREA, 85-376-4, SEPT. 1987 HEREAFTER REFFERRED TO AS DES. PARAMETERS OUTLINE".

USE THE FOLLOWING CN'S

REVEGETATED PILE	
TOP SURFACE	CN = 75
BENCH FACES	CN = 78
ACTIVE DISPOSAL OR HAUL ROAD ON PILE	CN = 85
OFF SITE, FAIR PASTURE OR RANGE	CN = 80

ALSO USE CN = 100 FOR OPEN WATER OR PONDS

SUBJECT KEYSTONE WEST VALLEY
PHASE II PERMITTING

BY SER DATE 3/19/96 PROJ. NO. 92-220-73-7
CHKD. BY KMB DATE 6/10/96 SHEET NO. 7 OF 45

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TIME-OF-CONCENTRATION, t_c

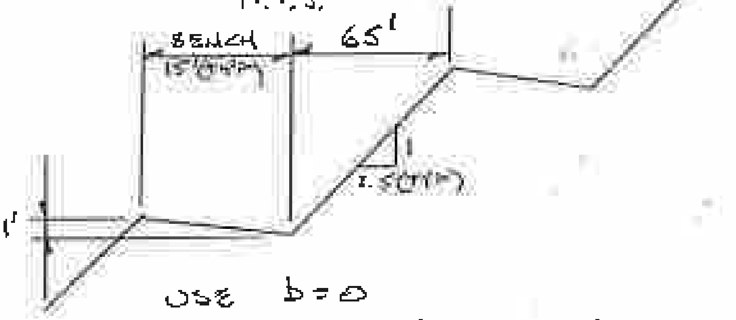
ESTIMATE t_c FOR EACH WATERSHED, SEE SHEETS 9 TO 23.
FLOW PATHS SHOWN ON WORKSHEET 92-220-73-7-SER1

t_c SUMMARY

WATERSHED	t_c, HRS
W1	0.31
W2	N/A
N1	0.15
N2	0.19
N3	0.32
S1	0.24
S2	0.17
S3	0.22
S4	0.39
S5	N/A
S6	0.24
SE1	0.25
SE2	0.17
SE3	0.16
SE4	0.21

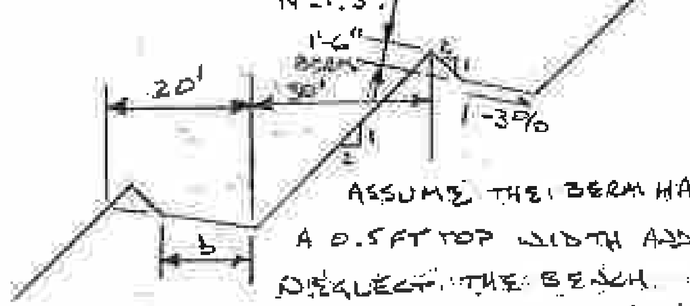
$P_{2,24}$ = 2.6 INCHES
$P_{10,24}$ = 3.9 "
$P_{25,24}$ = 4.4 "
$P_{100,24}$ = 5.2 "

PROPOSED WEST VALLEY
BENCH LAYOUT
N.T.S.



USE b = 0
$z = \frac{15+2.5}{2} = 8.75$
d = 1.0 FT.

EXISTING EAST VALLEY
BENCH LAYOUT
N.T.S.



ASSUME THE BERM HAS
A 0.5 FT TOP WIDTH AND
NEGLECT THE BENCH
BACKSLOPE, ∴ b = 13.5' AND z = 2, USE d = 1'
REF: DWG: 78-505-F6

n VALUES AS PER
DESIGN PARAMETERS OUTLINE
GROUTED ROCK n = 0.025
GRASS n = 0.045
FABRIFORM LINING n = 0.030

Keystone West Valley
Phase II Permitting

By : SER Date: 4/2/96
Chkd By: KrB Date: 6/14/96

Project No. 92-220-73-7
Sheet No. 8 of 45

Ultimate Conditions

Area and Curve Number Summary

Watershed	Total Area (Acres)	Total Area (SQ. MILES)	Composite CN		Areas of Individual Land Covers (Acres): Revegetated Pile Top	Revegetated Pile Bench Face	Active Area or Bottom Ash Haul Road	Paved Haul Road	Ponds	Pasture Offsite
				CN =	75	78	85	98	100	80
W1	12.3	0.0192	78		0.0	12.3	0.0	0.0	0.0	0.0
N1	2.3	0.0036	78		0.0	2.3	0.0	0.0	0.0	0.0
N2	4.6	0.0072	79		0.0	2.4	0.0	0.0	0.0	2.2
N3	25.6	0.0400	75		25.6	0.0	0.0	0.0	0.0	0.0
S1	23.2	0.0363	78		0.0	23.2	0.0	0.0	0.0	0.0
S2	10.4	0.0163	79		0.0	7.8	0.0	0.0	0.0	2.6
S3	8.4	0.013	78		0.0	7.6	0.0	0.0	0.0	0.8
S4	42.2	0.0659	77		33.4	3.0	5.8	0.0	0.0	0.0
S6	1.7	0.0027	80		0.0	0.0	0.0	0.0	0.0	1.7
SE1	28.7	0.0448	78		0.0	28.7	0.0	0.0	0.0	0.0
SE2	3.9	0.0061	78		0.0	3.9	0.0	0.0	0.0	0.0
SE3	10.6	0.0166	78		0.0	10.6	0.0	0.0	0.0	0.0
SE4	17.6	0.0275	80		0.0	13.6	4.0	0.0	0.0	0.0
Composite Area*										
W1	12.3	0.0192	78		0.0	12.3	0.0	0.0	0.0	0.0
W2	8.4	0.0131	84		0.0	0.0	0.0	0.0	1.5	6.9
West Pond	20.7	0.0323	80							

Note: Area S5 is used for design in a separate calc. set.

* Combine pond area W2 with the first upstream area W1 for use in West Clean SWM Pond routings.

d:\penelec\keystone\phase2\ksph2acn.wk3

SUBJECT: Penelec - Keystone West Valley
Phase II Permitting - Ultimate Conditions
BY: SER DATE: 4/9/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KMB DATE: 6\10\96 SHEET NO. 9 OF 45

Time of Concentration Worksheet - SCS Methods Reference: "Urban Hydrology for Small Watersheds", TR-55, Soil Conservation Service, June 1986
Watershed or Basin W1
Postdevelopment Conditions

SHEET FLOW	Flowpath: a-b	units
1. Surface description (table 3-1)	Dense Grass	
2. Manning's roughness coeff., n_{st} (table 3-1)	$n_{st} := 0.24$	
3. Flow length, L_{st} (total $L_{st} \le 150$ feet)	$L_{st} := 65$	feet
4. Two-year, 24-hour rainfall, P_2	$P_2 := 2.6$	inches
5. Land Slope, $S_{st} := 0.40$	$S_{st} = 0.4$	
6. Sheet Flow Time, $T_{st} := \dfrac{0.007\cdot(n_{st}\cdot L_{st})^{0.8}}{P_2^{0.5}\cdot S_{st}^{0.4}}$	$T_{st} = 0.056$	hours

SHALLOW CONCENTRATED FLOW	Flowpath: NA	
7. Surface description (paved or unpaved)		
8. Flow length, L_{sc}	$L_{sc} := 0$	feet
9. Watercourse Slope, $S_{sc} := 0$	$S_{sc} = 0$	
10. Average Velocity, $V_{sc} := 16.1345\cdot S_{sc}^{0.5}$	$V_{sc} = 0$	fps
11. Shallow Conc. Flow time, $T_{sc} := \left(\dfrac{L_{sc}}{3600\cdot V_{sc}}\right)$	$T_{sc} = 0$	hour

CHANNEL FLOW	Flowpath: b-c		Flowpath: c-d
12. Bottom width, b	$b := 0$	feet	$b_1 := 2$
13. Side slopes, z $\quad z := \dfrac{15 + 2.5}{2}$	$z = 8.75$		$z_1 := 2$
14. Flow depth, d	$d := 1$	feet	$d_1 := 1$
15. Cross sectional area, $a := (b + z\cdot d)\cdot d$	$a = 8.75$	ft^2	$a_1 := (b_1 + z_1\cdot d_1)\cdot d_1$ $\quad a_1 = 4$
16. Wetted perimeter, $P_w := \left[b + 2\cdot d\cdot(1 + z^2)^{0.5}\right]$	$P_w = 17.614$	feet	$P_{w1} := \left[b_1 + 2\cdot d_1\cdot(1 + z_1^2)^{0.5}\right]$ $\quad P_{w1} = 6.472$
17. Hydraulic radius, $r := \dfrac{a}{P_w}$	$r = 0.497$	feet	$r_1 := \dfrac{a_1}{P_{w1}}$ $\quad r_1 = 0.618$
18. Channel Length, L_{ch}	$L_{ch} := 2470$	feet	$L_{ch1} := 900$
19. Channel Slope, $S_{ch} := 0.02$	$S_{ch} = 0.02$		$S_{ch1} := \dfrac{1257 - 1150}{L_{ch1}}$ $\quad S_{ch1} = 0.119$
20. Channel lining	GRASS		Grouted Rock
21. Manning's roughness coeff., n	$n := 0.045$		$n_1 := 0.025$
22. Velocity, $V_{ch} := \left[\left(\dfrac{1.49}{n}\right)\cdot\left[r^{\left(\frac{2}{3}\right)}\right]\cdot S_{ch}^{\left(\frac{1}{2}\right)}\right]$	$V_{ch} = 2.937$	fps	$V_{ch1} := \left[\left(\dfrac{1.49}{n_1}\right)\cdot\left[r_1^{\left(\frac{2}{3}\right)}\right]\cdot S_{ch1}^{\left(\frac{1}{2}\right)}\right]$ $\quad V_{ch1} = 14.91$
22. Channel Flow time, $T_{ch} := \left(\dfrac{L_{ch}}{3600\cdot V_{ch}}\right)$	$T_{ch} = 0.234$	hour	$T_{ch1} := \left(\dfrac{L_{ch1}}{3600\cdot V_{ch1}}\right)$ $\quad T_{ch1} = 0.017$

Total Watershed Time-of-Concentration, $T_c := T_{st} + T_{sc} + T_{ch} + T_{ch1}$ $\quad T_c = 0.307$ hour

SUBJECT: Penelec - Keystone West Valley
Phase II Permitting - Ultimate Conditions
BY: SER DATE: 4/9/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KMB DATE: 10/10/96 SHEET NO. 10 OF 45

Time of Concentration Worksheet - SCS Methods Reference: "Urban Hydrology for Small Watersheds", TR-55, Soil Conservation Service, June 1986
Watershed or Basin N1
Postdevelopment Conditions

SHEET FLOW Flowpath: a-b units

1. Surface description (table 3-1) Dense Grass
2. Manning's roughness coeff., n_{st} (table 3-1) $n_{st} := 0.24$
3. Flow length, L_{st} (total $L_{st} \le 150$ feet) $L_{st} := 65$ feet
4. Two-year, 24-hour rainfall, P_2 $P_2 := 2.6$ inches
5. Land Slope, $S_{st} := 0.40$ $S_{st} = 0.4$
6. Sheet Flow Time, $T_{st} := \dfrac{0.007 \cdot (n_{st} \cdot L_{st})^{0.8}}{P_2^{0.5} \cdot S_{st}^{0.4}}$ $T_{st} = 0.056$ hours

SHALLOW CONCENTRATED FLOW Flowpath: NA

7. Surface description (paved or unpaved)
8. Flow length, L_{sc} $L_{sc} := 0$ feet
9. Watercourse Slope, $S_{sc} := 0$ $S_{sc} = 0$
10. Average Velocity, $V_{sc} := 16.1345 \cdot S_{sc}^{0.5}$ $V_{sc} = 0$ fps
11. Shallow Conc. Flow time, $T_{sc} := \left(\dfrac{L_{sc}}{3600 \cdot V_{sc}}\right)$ $T_{sc} = 0$ hour

CHANNEL FLOW Flowpath: b-c

The time for flowpath c-d is negligible. Assume t=0.

12. Bottom width, b $b := 0$ feet
13. Side slopes, z $z := \dfrac{15 + 2.5}{2}$ $z = 8.75$
14. Flow depth, d $d := 1.0$ feet
15. Cross sectional area, $a := (b + z \cdot d) \cdot d$ $a = 8.75$ ft^2
16. Wetted perimeter, $P_w := \left[b + 2 \cdot d \cdot (1 + z^2)^{0.5}\right]$ $P_w = 17.614$ feet
17. Hydraulic radius, $r := \dfrac{a}{P_w}$ $r = 0.497$ feet
18. Channel Length, L_{ch} $L_{ch} := 995$ feet
19. Channel Slope, $S_{ch} := 0.02$ $S_{ch} = 0.02$
20. Channel lining Grass
21. Manning's roughness coeff., n $n := 0.045$
22. Velocity, $V_{ch} := \left[\left(\dfrac{1.49}{n}\right) \cdot \left[r^{\left(\frac{2}{3}\right)}\right] \cdot S_{ch}^{\left(\frac{1}{2}\right)}\right]$ $V_{ch} = 2.937$ fps
22. Channel Flow time, $T_{ch} := \left(\dfrac{L_{ch}}{3600 \cdot V_{ch}}\right)$ $T_{ch} = 0.094$ hour

Total Watershed Time-of-Concentration, $T_c := T_{st} + T_{sc} + T_{ch}$ $T_c = 0.151$ hour

SUBJECT: Penelec - Keystone West Valley
Phase II Permitting - Ultimate Conditions
BY: SER DATE: 4/9/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KMS DATE: 6/10/96 SHEET NO. 11 OF 45

Time of Concentration Worksheet - SCS Methods Reference: "Urban Hydrology for Small Watersheds", TR-55, Soil Conservation Service, June 1986
Watershed or Basin N2
Postdevelopment Conditions

SHEET FLOW — Flowpath: a-b — units

1. Surface description (table 3-1) — Dense Grass
2. Manning's roughness coeff., n_{st} (table 3-1) — $n_{st} := 0.24$
3. Flow length, L_{st} (total $L_{st} \le 150$ feet) — $L_{st} := 65$ feet
4. Two-year, 24-hour rainfall, P_2 — $P_2 := 2.6$ inches
5. Land Slope, $S_{st} := 0.4$ — $S_{st} = 0.4$
6. Sheet Flow Time, $T_{st} := \frac{0.007 \cdot (n_{st} \cdot L_{st})^{0.8}}{P_2^{0.5} \cdot S_{st}^{0.4}}$ — $T_{st} = 0.056$ hours

SHALLOW CONCENTRATED FLOW — Flowpath: NA

7. Surface description (paved or unpaved) — ______
8. Flow length, L_{sc} — $L_{sc} := 0$ feet
9. Watercourse Slope, $S_{sc} := 0$ — $S_{sc} = 0$
10. Average Velocity, $V_{sc} := 16.1345 \cdot S_{sc}^{0.5}$ — $V_{sc} = 0$ fps
11. Shallow Conc. Flow time, $T_{sc} := \left(\frac{L_{sc}}{3600 \cdot V_{sc}}\right)$ — $T_{sc} = 0$ hour

CHANNEL FLOW — Flowpath: b-c — Flowpath: c-d

12. Bottom width, b — $b := 0$ feet — $b_1 := 2$
13. Side slopes, $z := \frac{15 + 2.5}{2}$ — $z = 8.75$ — $z_1 = 2$
14. Flow depth, d — $d := 1$ feet — $d_1 := 2$
15. Cross sectional area, $a := (b + z \cdot d) \cdot d$ — $a = 8.75$ ft^2 — $a_1 := (b_1 + z_1 \cdot d_1) \cdot d_1$ $a_1 = 12$
16. Wetted perimeter, $P_w := \left[b + 2 \cdot d \cdot (1 + z^2)^{0.5}\right]$ — $P_w = 17.614$ feet — $P_{w1} := \left[b_1 + 2 \cdot d_1 \cdot (1 + z_1^2)^{0.5}\right]$ $P_{w1} = 10.944$
17. Hydraulic radius, $r := \frac{a}{P_w}$ — $r = 0.497$ feet — $r_1 := \frac{a_1}{(P_{w1})}$ $r_1 = 1.096$
18. Channel Length, L_{ch} — $L_{ch} := 1170$ feet — $L_{ch1} := 600$
19. Channel Slope, $S_{ch} := 0.02$ — $S_{ch} = 0.02$ — $S_{ch1} := \frac{1299 - 1277}{L_{ch1}}$ $S_{ch1} = 0.037$
20. Channel lining — Grass — Grass
21. Manning's roughness coeff., n — $n = 0.045$ — $n_1 := 0.045$
22. Velocity, $V_{ch} := \left[\left(\frac{1.49}{n}\right) \cdot \left[r^{\left(\frac{2}{3}\right)}\right] \cdot S_{ch}^{\left(\frac{1}{2}\right)}\right]$ — $V_{ch} = 2.937$ fps — $V_{ch1} := \left[\left(\frac{1.49}{n_1}\right) \cdot \left[r_1^{\left(\frac{2}{3}\right)}\right] \cdot S_{ch1}^{\left(\frac{1}{2}\right)}\right]$ $V_{ch1} = 6.742$
22. Channel Flow time, $T_{ch} := \left(\frac{L_{ch}}{3600 \cdot V_{ch}}\right)$ — $T_{ch} = 0.111$ hour — $T_{ch1} := \left(\frac{L_{ch1}}{3600 \cdot V_{ch1}}\right)$ $T_{ch1} = 0.025$

Total Watershed Time-of-Concentration, $T_c = T_{st} + T_{sc} + T_{ch} + T_{ch1}$ $T_c = 0.192$ hour

SUBJECT: Penelec - Keystone West Valley
Phase II Permitting - Ultimate Conditions
BY: SER DATE: 4/9/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KMB DATE: 6/10/96 SHEET NO. 12 OF 45

Time of Concentration Worksheet - SCS Methods Reference: "Urban Hydrology for Small Watersheds", TR-55, Soil Conservation Service, June 1986
Watershed or Basin N3
Postdevelopment Conditions

SHEET FLOW	Flowpath: a-b	units
1. Surface description (table 3-1)	Dense Grass	
2. Manning's roughness coeff., n_{st} (table 3-1)	$n_{st} := 0.24$	
3. Flow length, L_{st} (total $L_{st} \le 150$ feet)	$L_{st} := 150$	feet
4. Two-year, 24-hour rainfall, P_2	$P_2 := 2.6$	inches
5. Land Slope, $S_{st} := \frac{1440.5 - 1424}{L_{st}}$	$S_{st} = 0.11$	
6. Sheet Flow Time, $T_{st} := \frac{0.007 \cdot (n_{st} \cdot L_{st})^{0.8}}{P_2^{0.5} \cdot S_{st}^{0.4}}$	$T_{st} = 0.185$	hours
SHALLOW CONCENTRATED FLOW	Flowpath: NA	
7. Surface description (paved or unpaved)		
8. Flow length, L_{sc}	$L_{sc} := 0$	feet
9. Watercourse Slope, $S_{sc} := 0$	$S_{sc} = 0$	
10. Average Velocity, $V_{sc} := 16.1345 \cdot S_{sc}^{0.5}$	$V_{sc} = 0$	fps
11. Shallow Conc. Flow time, $T_{sc} := \left(\frac{L_{sc}}{3600 \cdot V_{sc}}\right)$	$T_{sc} = 0$	hour
CHANNEL FLOW	Flowpath: b-c	
12. Bottom width, b	$b := 0$	feet
13. Side slopes, z	$z := 3$	
14. Flow depth, d	$d := 1$	feet
15. Cross sectional area, $a := (b + z \cdot d) \cdot d$	$a = 3$	ft^2
16. Wetted perimeter, $P_w := \left[b + 2 \cdot d \cdot (1 + z^2)^{0.5}\right]$	$P_w = 6.325$	feet
17. Hydraulic radius, $r := \frac{a}{P_w}$	$r = 0.474$	feet
18. Channel Length, L_{ch}	$L_{ch} := 2320$	feet
19. Channel Slope, $S_{ch} := \frac{1424 - 1294}{L_{ch}}$	$S_{ch} = 0.056$	
20. Channel lining	GRASS	
21. Manning's roughness coeff., n	$n := 0.045$	
22. Velocity, $V_{ch} := \left[\left(\frac{1.49}{n}\right) \cdot \left[r^{\left(\frac{2}{3}\right)}\right] \cdot S_{ch}^{\left(\frac{1}{2}\right)}\right]$	$V_{ch} = 4.767$	fps
22. Channel Flow time, $T_{ch} := \left(\frac{L_{ch}}{3600 \cdot V_{ch}}\right)$	$T_{ch} = 0.135$	hour

Total Watershed Time-of-Concentration, $T_c := T_{st} + T_{sc} + T_{ch}$ $T_c = 0.32$ hour

SUBJECT: Penelec - Keystone West Valley
Phase II Permitting - Ultimate Conditions
BY: SER DATE: 4/9/96 PROJ. NO.: 92-220-73-07
CHKD. BY KMB DATE: 4/10/96 SHEET NO.13 OF 45

Time of Concentration Worksheet - SCS Methods Reference: "Urban Hydrology for Small Watersheds", TR-55, Soil Conservation Service, June 1986
Watershed or Basin S1
Postdevelopment Conditions

SHEET FLOW	Flowpath: a-b	units
1. Surface description (table 3-1)	Dense Grass	
2. Manning's roughness coeff., n_{st} (table 3-1)	$n_{st} := 0.24$	
3. Flow length, L_{st} (total $L_{st} \le 150$ feet)	$L_{st} := 65$	feet
4. Two-year, 24-hour rainfall, P_2	$P_2 := 2.6$	inches
5. Land Slope, $S_{st} := 0.40$	$S_{st} = 0.4$	
6. Sheet Flow Time, $T_{st} := \frac{0.007 \cdot (n_{st} \cdot L_{st})^{0.8}}{P_2^{0.5} \cdot S_{st}^{0.4}}$	$T_{st} = 0.056$	hours
SHALLOW CONCENTRATED FLOW	Flowpath: NA	
7. Surface description (paved or unpaved)		
8. Flow length, L_{sc}	$L_{sc} := 0$	feet
9. Watercourse Slope, $S_{sc} := 0$	$S_{sc} = 0$	
10. Average Velocity, $V_{sc} := 16.1345 \cdot S_{sc}^{0.5}$	$V_{sc} = 0$	fps
11. Shallow Conc. Flow time, $T_{sc} := \left(\frac{L_{sc}}{3600 \cdot V_{sc}}\right)$	$T_{sc} = 0$	hour
CHANNEL FLOW	Flowpath: b-c	
12. Bottom width, b	$b := 0$	feet
13. Side slopes, z $z := \frac{15 + 2.5}{2}$	$z = 8.75$	
14. Flow depth, d	$d := 1.0$	feet
15. Cross sectional area, $a := (b + z \cdot d) \cdot d$	$a = 8.75$	ft^2
16. Wetted perimeter, $P_w := \left[b + 2 \cdot d \cdot (1 + z^2)^{0.5}\right]$	$P_w = 17.614$	feet
17. Hydraulic radius, $r := \frac{a}{P_w}$	$r = 0.497$	feet
18. Channel Length, L_{ch}	$L_{ch} := 1930$	feet
19. Channel Slope, $S_{ch} := 0.02$	$S_{ch} = 0.02$	
20. Channel lining	Grass	
21. Manning's roughness coeff., n	$n := 0.045$	
22. Velocity, $V_{ch} := \left[\left(\frac{1.49}{n}\right) \cdot \left[r^{\left(\frac{2}{3}\right)}\right] \cdot S_{ch}^{\left(\frac{1}{2}\right)}\right]$	$V_{ch} = 2.937$	fps
22. Channel Flow time, $T_{ch} := \left(\frac{L_{ch}}{3600 \cdot V_{ch}}\right)$	$T_{ch} = 0.183$	hour

Flowpath c-d is a slopedrain. Assume t = 0

Total Watershed Time-of-Concentration, $T_c := T_{st} + T_{sc} + T_{ch}$ $T_c = 0.239$ hour

SUBJECT: Penelec - Keystone West Valley
Phase II Permitting - Ultimate Conditions
BY: SER DATE: 4/8/96 PROJ. NO. 92-220-73-07
CHKD. BY: KWD DATE: 6/10/96 SHEET NO. 14 OF 45

Time of Concentration Worksheet - SCS Methods Reference: "Urban Hydrology for Small Watersheds", TR-55, Soil Conservation Service, June 1986
Watershed or Basin S2
Postdevelopment Conditions

SHEET FLOW	Flowpath: a-b	units
1. Surface description (table 3-1)	Dense Grass	
2. Manning's roughness coeff., n_{st} (table 3-1)	$n_{st} := 0.24$	
3. Flow length, L_{st} (total $L_{st} \le 150$ feet)	$L_{st} := 65$	feet
4. Two-year, 24-hour rainfall, P_2	$P_2 := 2.6$	inches
5. Land Slope, $S_{st} := 0.4$	$S_{st} = 0.4$	
6. Sheet Flow Time, $T_{st} := \frac{0.007 \cdot (n_{st} \cdot L_{st})^{0.8}}{P_2^{0.5} \cdot S_{st}^{0.4}}$	$T_{st} = 0.056$	hours
SHALLOW CONCENTRATED FLOW	Flowpath: NA	
7. Surface description (paved or unpaved)		
8. Flow length, L_{sc}	$L_{sc} := 0$	feet
9. Watercourse Slope, $S_{sc} := 0$	$S_{sc} = 0$	
10. Average Velocity, $V_{sc} := 16.1345 \cdot S_{sc}^{0.5}$	$V_{sc} = 0$	fps
11. Shallow Conc. Flow time, $T_{sc} := \left(\frac{L_{sc}}{3600 \cdot V_{sc}}\right)$	$T_{sc} = 0$	hour
CHANNEL FLOW	Flowpath: b-c	
12. Bottom width, b	$b := 0$	feet
13. Side slopes, z $z := \frac{15 + 2.5}{2}$	$z = 8.75$	
14. Flow depth, d	$d := 1$	feet
15. Cross sectional area, $a := (b + z \cdot d) \cdot d$	$a = 8.75$	ft^2
16. Wetted perimeter, $P_w := \left[b + 2 \cdot d \cdot (1 + z^2)^{0.5}\right]$	$P_w = 17.614$	feet
17. Hydraulic radius, $r := \frac{a}{P_w}$	$r = 0.497$	feet
18. Channel Length, L_{ch}	$L_{ch} := 520$	feet
19. Channel Slope, $S_{ch} = 0.02$	$S_{ch} = 0.02$	
20. Channel lining	Grass	
21. Manning's roughness coeff., n	$n := 0.045$	
22. Velocity, $V_{ch} := \left[\left(\frac{1.49}{n}\right) \cdot \left[r^{\left(\frac{2}{3}\right)}\right] \cdot S_{ch}^{\left(\frac{1}{2}\right)}\right]$	$V_{ch} = 2.937$	fps
22. Channel Flow time, $T_{ch} := \left(\frac{L_{ch}}{3600 \cdot V_{ch}}\right)$	$T_{ch} = 0.049$	hour

SUBJECT: Penelec - Keystone West Valley
Phase II Permitting - Ultimate Conditions
BY: SER DATE: 4/9/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KMB DATE: 6/10/96 SHEET NO. 15 OF 45

Time of Concentration Worksheet - SCS Methods Reference: "Urban Hydrology for Small Watersheds",
Watershed or Basin S2 Continued TR-55, Soil Conservation Service, June 1986
Postdevelopment Conditions

CHANNEL FLOW Flowpath: c-d

12. Bottom width, b $b_1 := 2$ feet
13. Side slopes, z $z_1 := 2$
14. Flow depth, d $d_1 := 2$ feet
15. Cross sectional area, $a_1 := (b_1 + z_1 \cdot d_1) \cdot d_1$ $a_1 = 12$ ft^2
16. Wetted perimeter, $P_{w1} := \left[b_1 + 2 \cdot d_1 \cdot \left(1 + z_1^2\right)^{0.5}\right]$ $P_{w1} = 10.944$ feet
17. Hydraulic radius, $r_1 := \frac{a_1}{P_{w1}}$ $r_1 = 1.096$ feet
18. Channel Length, L_{ch} $L_{ch1} := 1180$ feet
19. Channel Slope, $S_{ch} := 0.02$ $S_{ch1} := 0.01$
20. Channel lining Grouted Rock
21. Manning's roughness coeff., n $n_1 := 0.025$
22. Velocity, $V_{ch1} := \left[\left(\frac{1.49}{n_1}\right) \cdot \left[r_1^{\left(\frac{2}{3}\right)}\right] \cdot S_{ch1}^{\left(\frac{1}{2}\right)}\right]$ $V_{ch1} = 6.337$ fps
22. Channel Flow time, $T_{ch1} := \left(\frac{L_{ch1}}{3600 \cdot V_{ch1}}\right)$ $T_{ch1} = 0.052$ hour

CHANNEL FLOW Flowpath: d-e

12. Bottom width, b $b_2 := 2$
13. Side slopes, z $z_2 := 2$
14. Flow depth, d $d_2 := 1$
15. Cross sectional area, $a_2 := (b_2 + z_2 \cdot d_2) \cdot d_2$ $a_2 = 4$
16. Wetted perimeter, $P_{w2} := \left[b_2 + 2 \cdot d_2 \cdot \left(1 + z_2^2\right)^{0.5}\right]$ $P_{w2} = 6.472$
17. Hydraulic radius, $r_2 := \frac{a_2}{P_{w2}}$ $r_2 = 0.618$
18. Channel Length, L_{ch} $L_{ch2} := 900$
19. Channel Slope, $S_{ch2} := \frac{1260 - 1084}{L_{ch2}}$ $S_{ch2} = 0.196$
20. Channel lining Grouted Rock
21. Manning's roughness coeff., n $n_2 := 0.025$
22. Velocity, $V_{ch2} := \left(\frac{1.49}{n_2}\right) \cdot \left[r_2^{\left(\frac{2}{3}\right)}\right] \cdot S_{ch2}^{\left(\frac{1}{2}\right)}$ $V_{ch2} = 19.123$
22. Channel Flow time, $T_{ch2} := \left(\frac{L_{ch2}}{3600 \cdot V_{ch2}}\right)$ $T_{ch2} = 0.013$

Total Watershed Time-of-Concentration, $T_c := T_{st} + T_{sc} + T_{ch} + T_{ch1} + T_{ch2}$ $T_c = 0.17$ hour

SUBJECT: Penelec - Keystone West Valley
Phase II Permitting - Ultimate Conditions
BY: SER DATE: 4/9/98 PROJ. NO.: 92-220-73-07
CHKD. BY: KMB DATE: 6/12/98 SHEET NO. 16 OF 45

Time of Concentration Worksheet - SCS Methods Reference: "Urban Hydrology for Small Watersheds", TR-55, Soil Conservation Service, June 1986
Watershed or Basin S3
Postdevelopment Conditions

SHEET FLOW	Flowpath: a-b	units
1. Surface description (table 3-1)	Dense Grass	
2. Manning's roughness coeff., n_{st} (table 3-1)	$n_{st} := 0.24$	
3. Flow length, L_{st} (total $L_{st} \le 150$ feet)	$L_{st} := 65$	feet
4. Two-year, 24-hour rainfall, P_2	$P_2 := 2.6$	inches
5. Land Slope, $S_{st} := 0.40$	$S_{st} = 0.4$	
6. Sheet Flow Time, $T_{st} := \frac{0.007\cdot(n_{st}\cdot L_{st})^{0.8}}{P_2^{0.5}\cdot S_{st}^{0.4}}$	$T_{st} = 0.056$	hours
SHALLOW CONCENTRATED FLOW	Flowpath: NA	
7. Surface description (paved or unpaved)		
8. Flow length, L_{sc}	$L_{sc} := 0$	feet
9. Watercourse Slope, $S_{sc} := 0$	$S_{sc} = 0$	
10. Average Velocity, $V_{sc} := 16.1345\cdot S_{sc}^{0.5}$	$V_{sc} = 0$	fps
11. Shallow Conc. Flow time, $T_{sc} := \left(\frac{L_{sc}}{3600\cdot V_{sc}}\right)$	$T_{sc} = 0$	hour
CHANNEL FLOW	Flowpath: b-c	Flowpath c-d, assume t=0
12. Bottom width, b	$b := 0$	feet
13. Side slopes, z $\quad z := \frac{15+2.5}{2}$	$z = 8.75$	
14. Flow depth, d	$d := 1$	feet
15. Cross sectional area, $a := (b + z\cdot d)\cdot d$	$a = 8.75$	ft^2
16. Wetted perimeter, $P_w := \left[b + 2\cdot d\cdot\left(1+z^2\right)^{0.5}\right]$	$P_w = 17.614$	feet
17. Hydraulic radius, $r := \frac{a}{P_w}$	$r = 0.497$	feet
18. Channel Length, L_{ch}	$L_{ch} := 1720$	feet
19. Channel Slope, $S_{ch} := 0.02$	$S_{ch} = 0.02$	
20. Channel lining	GRASS	
21. Manning's roughness coeff., n	$n := 0.045$	
22. Velocity, $V_{ch} := \left[\left(\frac{1.49}{n}\right)\cdot\left[r^{\left(\frac{2}{3}\right)}\right]\cdot S_{ch}^{\left(\frac{1}{2}\right)}\right]$	$V_{ch} = 2.937$	fps
22. Channel Flow time, $T_{ch} := \left(\frac{L_{ch}}{3600\cdot V_{ch}}\right)$	$T_{ch} = 0.163$	hour

Total Watershed Time-of-Concentration, $T_c := T_{st} + T_{sc} + T_{ch}$ $\quad T_c = 0.219$ hour

SUBJECT: Penelec - Keystone West Valley
Phase II Permitting - Ultimate Conditions
BY: SER DATE: 4/9/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KYR DATE: 6/10/96 SHEET NO. 17 OF 45

Time of Concentration Worksheet - SCS Methods Reference: "Urban Hydrology for Small Watersheds", TR-55, Soil Conservation Service, June 1986
Watershed or Basin S4
Postdevelopment Conditions

SHEET FLOW	Flowpath: a-b	units
1. Surface description (table 3-1)	Dense Grass	
2. Manning's roughness coeff., n_{st} (table 3-1)	$n_{st} := 0.24$	
3. Flow length, L_{st} (total $L_{st} \le 150$ feet)	$L_{st} := 150$	feet
4. Two-year, 24-hour rainfall, P_2	$P_2 := 2.6$	inches
5. Land Slope, $S_{st} := \frac{1451 - 1444}{L_{st}}$	$S_{st} = 0.047$	
6. Sheet Flow Time, $T_{st} := \frac{0.007 \cdot (n_{st} \cdot L_{st})^{0.8}}{P_2^{0.5} \cdot S_{st}^{0.4}}$	$T_{st} = 0.26$	hours

SHALLOW CONCENTRATED FLOW	Flowpath: b-c	
7. Surface description (paved or unpaved)	unpaved	
8. Flow length, L_{sc}	$L_{sc} := 500$	feet
9. Watercourse Slope, $S_{sc} := \frac{1444 - 1433}{L_{sc}}$	$S_{sc} = 0.022$	
10. Average Velocity, $V_{sc} := 16.1345 \cdot S_{sc}^{0.5}$	$V_{sc} = 2.393$	fps
11. Shallow Conc. Flow time, $T_{sc} := \left(\frac{L_{sc}}{3600 \cdot V_{sc}}\right)$	$T_{sc} = 0.058$	hour

CHANNEL FLOW	Flowpath: c-d	
12. Bottom width, b	$b := 0$	feet
13. Side slopes, z	$z := 3$	
14. Flow depth, d	$d := 1.0$	feet
15. Cross sectional area, $a := (b + z \cdot d) \cdot d$	$a = 3$	ft^2
16. Wetted perimeter, $P_w := \left[b + 2 \cdot d \cdot (1 + z^2)^{0.5}\right]$	$P_w = 6.325$	feet
17. Hydraulic radius, $r := \frac{a}{P_w}$	$r = 0.474$	feet
18. Channel Length, L_{ch}	$L_{ch} := 820$	feet
19. Channel Slope, $S_{ch} := \frac{1433 - 1415}{L_{ch}}$	$S_{ch} = 0.022$	
20. Channel lining	Grass	
21. Manning's roughness coeff., n	$n := 0.045$	
22. Velocity, $V_{ch} := \left[\left(\frac{1.49}{n}\right) \cdot \left[r^{\left(\frac{2}{3}\right)}\right] \cdot S_{ch}^{\left(\frac{1}{2}\right)}\right]$	$V_{ch} = 2.984$	fps
22. Channel Flow time, $T_{ch} := \left(\frac{L_{ch}}{3600 \cdot V_{ch}}\right)$	$T_{ch} = 0.076$	hour

Flowpath d-e is a Steep channel, assume t=0

Total Watershed Time-of-Concentration, $T_c := T_{st} + T_{sc} + T_{ch}$ $T_c = 0.394$ hour

SUBJECT: Penelec - Keystone West Valley
Phase II Permitting - Ultimate Conditions
BY: SER DATE: 4/9/96 PROJ. NO.: 92-220-73-07
CHKD. BY: [handwritten initials] DATE: 6/10/96 SHEET NO. 18 OF 45

Time of Concentration Worksheet - SCS Methods Reference: "Urban Hydrology for Small Watersheds", TR-55, Soil Conservation Service, June 1986
Watershed or Basin S6
Postdevelopment Conditions

SHEET FLOW	Flowpath: a-b	units
1. Surface description (table 3-1)	Dense Grass	
2. Manning's roughness coeff., n_{st} (table 3-1)	$n_{st} := 0.24$	
3. Flow length, L_{st} (total $L_{st} \le 150$ feet)	$L_{st} := 60$	feet
4. Two-year, 24-hour rainfall, P_2	$P_2 := 2.6$	inches
5. Land Slope, $S_{st} := 0.01$ assumed	$S_{st} = 0.01$	
6. Sheet Flow Time, $T_{st} := \dfrac{0.007 \cdot (n_{st} \cdot L_{st})^{0.8}}{P_2^{0.5} \cdot S_{st}^{0.4}}$	$T_{st} = 0.231$	hours

SHALLOW CONCENTRATED FLOW	Flowpath: b-c	
7. Surface description (paved or unpaved)	unpaved	
8. Flow length, L_{sc}	$L_{sc} := 120$	feet
9. Watercourse Slope, $S_{sc} := \dfrac{1205 - 1175}{L_{sc}}$	$S_{sc} = 0.25$	
10. Average Velocity, $V_{sc} := 16.1345 \cdot S_{sc}^{0.5}$	$V_{sc} = 8.067$	fps
11. Shallow Conc. Flow time, $T_{sc} := \left(\dfrac{L_{sc}}{3600 \cdot V_{sc}}\right)$	$T_{sc} = 0.004$	hour

SHALLOW CONCENTRATED FLOW(cont)	Flowpath: c-d	
7. Surface description (paved or unpaved)	unpaved	
8. Flow length, L_{sc}	$L_{sc1} := 80$	feet
9. Watercourse Slope, $S_{sc1} = 0.5$	$S_{sc1} = 0.5$	
10. Average Velocity, $V_{sc1} := 16.1345 \cdot S_{sc1}^{0.5}$	$V_{sc1} = 11.409$	fps
11. Shallow Conc. Flow time, $T_{sc1} = \left(\dfrac{L_{sc1}}{3600 \cdot V_{sc1}}\right)$	$T_{sc1} = 0.002$	hour

The time for Flowpath d-e is negligible. Assume t=0.

Total Watershed Time-of-Concentration, $T_c := T_{st} + T_{sc} + T_{sc1}$ $T_c = 0.237$ hour

SUBJECT: Penelec - Keystone West Valley
Phase II Permitting - Ultimate Conditions
BY: SER DATE: 4/9/96 PROJ. NO.: 92-220-73-07
CHKD. BY: WB DATE: 6/10/96 SHEET NO. 19 OF 45

Time of Concentration Worksheet - SCS Methods Reference: "Urban Hydrology for Small Watersheds", TR-55, Soil Conservation Service, June 1986
Watershed or Basin SE1
Postdevelopment Conditions

SHEET FLOW	Flowpath: a-b	units	
1. Surface description (table 3-1)	Dense Grass		
2. Manning's roughness coeff., n_{st} (table 3-1)	$n_{st} := 0.24$		
3. Flow length, L_{st} (total $L_{st} \le 150$ feet)	$L_{st} := 65$	feet	
4. Two-year, 24-hour rainfall, P_2	$P_2 := 2.6$	inches	
5. Land Slope, $S_{st} := 0.40$	$S_{st} = 0.4$		
6. Sheet Flow Time, $T_{st} := \dfrac{0.007 \cdot (n_{st} \cdot L_{st})^{0.8}}{P_2^{0.5} \cdot S_{st}^{0.4}}$	$T_{st} = 0.056$	hours	
SHALLOW CONCENTRATED FLOW	Flowpath: NA		
7. Surface description (paved or unpaved)	______		
8. Flow length, L_{sc}	$L_{sc} := 0$	feet	
9. Watercourse Slope, $S_{sc} := 0$	$S_{sc} = 0$		
10. Average Velocity, $V_{sc} := 16.1345 \cdot S_{sc}^{0.5}$	$V_{sc} = 0$	fps	
11. Shallow Conc. Flow time, $T_{sc} := \left(\dfrac{L_{sc}}{3600 \cdot V_{sc}}\right)$	$T_{sc} = 0$	hour	
CHANNEL FLOW	Flowpath: b-c		
12. Bottom width, b	$b := 0$	feet	
13. Side slopes, z $\quad z := \dfrac{15 + 2.5}{2}$	$z = 8.75$		
14. Flow depth, d	$d := 1$	feet	
15. Cross sectional area, $a := (b + z \cdot d) \cdot d$	$a = 8.75$	ft^2	Flowpath c-d is a Slopedrain. Assume t=0
16. Wetted perimeter, $P_w := \left[b + 2 \cdot d \cdot (1 + z^2)^{0.5}\right]$	$P_w = 17.614$	feet	
17. Hydraulic radius, $r := \dfrac{a}{P_w}$	$r = 0.497$	feet	
18. Channel Length, L_{ch}	$L_{ch} := 2330$	feet	
19. Channel Slope, $S_{ch} := 0.02$	$S_{ch} = 0.02$		
20. Channel lining	GRASS		
21. Manning's roughness coeff., n	$n := 0.045$		
22. Velocity, $V_{ch} := \left[\left(\dfrac{1.49}{n}\right) \cdot \left[r^{\left(\frac{2}{3}\right)}\right] \cdot S_{ch}^{\left(\frac{1}{2}\right)}\right]$	$V_{ch} = 2.937$	fps	
22. Channel Flow time, $T_{ch} := \left(\dfrac{L_{ch}}{3600 \cdot V_{ch}}\right)$	$T_{ch} = 0.22$	hour	

Total Watershed Time-of-Concentration, $T_c = T_{st} + T_{sc} + T_{ch}$ $\quad T_c = 0.277$ hour

SUBJECT: Penelec - Keystone West Valley
Phase II Permitting - Ultimate Conditions
BY: SER DATE: 4/9/96 PROJ. NO.: 92-220-73-07
CHKD. BY: ___ DATE: 6/10/96 SHEET NO. 20 OF 45

Time of Concentration Worksheet - SCS Methods Reference: "Urban Hydrology for Small Watersheds", TR-55, Soil Conservation Service, June 1986
Watershed or Basin SE2
Postdevelopment Conditions

SHEET FLOW	Flowpath: a-b	units
1. Surface description (table 3-1)	Dense Grass	
2. Manning's roughness coeff., n_{st} (table 3-1)	$n_{st} := 0.24$	
3. Flow length, L_{st} (total $L_{st} \le 150$ feet)	$L_{st} := 65$	feet
4. Two-year, 24-hour rainfall, P_2	$P_2 := 2.6$	inches
5. Land Slope, $S_{st} := 0.40$	$S_{st} = 0.4$	
6. Sheet Flow Time, $T_{st} := \frac{0.007 \cdot (n_{st} \cdot L_{st})^{0.8}}{P_2^{0.5} \cdot S_{st}^{0.4}}$	$T_{st} = 0.056$	hours
SHALLOW CONCENTRATED FLOW	Flowpath: NA	
7. Surface description (paved or unpaved)	____	
8. Flow length, L_{sc}	$L_{sc} := 0$	feet
9. Watercourse Slope, $S_{sc} := 0$	$S_{sc} = 0$	
10. Average Velocity, $V_{sc} := 16.1345 \cdot S_{sc}^{0.5}$	$V_{sc} = 0$	fps
11. Shallow Conc. Flow time, $T_{sc} := \left(\frac{L_{sc}}{3600 \cdot V_{sc}}\right)$	$T_{sc} = 0$	hour
CHANNEL FLOW	Flowpath: b-c	
12. Bottom width, b	$b := 0$	feet
13. Side slopes, z $\quad z := \frac{15 + 2.5}{2}$	$z = 8.75$	
14. Flow depth, d	$d := 1.0$	feet
15. Cross sectional area, $a := (b + z \cdot d) \cdot d$	$a = 8.75$	ft^2
16. Wetted perimeter, $P_w := \left[b + 2 \cdot d \cdot (1 + z^2)^{0.5}\right]$	$P_w = 17.614$	feet
17. Hydraulic radius, $r := \frac{a}{P_w}$	$r = 0.497$	feet
18. Channel Length, L_{ch}	$L_{ch} := 1210$	feet
19. Channel Slope, $S_{ch} := 0.02$	$S_{ch} = 0.02$	
20. Channel lining	Grass	
21. Manning's roughness coeff., n	$n := 0.045$	
22. Velocity, $V_{ch} := \left[\left(\frac{1.49}{n}\right) \cdot \left[r^{\left(\frac{2}{3}\right)}\right] \cdot S_{ch}^{\left(\frac{1}{2}\right)}\right]$	$V_{ch} = 2.937$	fps
22. Channel Flow time, $T_{ch} := \left(\frac{L_{ch}}{3600 \cdot V_{ch}}\right)$	$T_{ch} = 0.114$	hour

Total Watershed Time-of-Concentration, $T_c := T_{st} + T_{sc} + T_{ch}$ $\quad T_c = 0.171$ hour

SUBJECT: Penelec - Keystone West Valley
Phase II Permitting - Ultimate Conditions
BY: SER DATE: 4/9/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KMB DATE: 6/10/96 SHEET NO. 21 OF 45

Time of Concentration Worksheet - SCS Methods Reference: "Urban Hydrology for Small Watersheds", TR-55, Soil Conservation Service, June 1986
Watershed or Basin SE3
Postdevelopment Conditions

SHEET FLOW	Flowpath: a-b	units
1. Surface description (table 3-1)	Dense Grass	
2. Manning's roughness coeff., n_{st} (table 3-1)	$n_{st} := 0.24$	
3. Flow length, L_{st} (total $L_{st} \le 150$ feet)	$L_{st} := 30$	feet
4. Two-year, 24-hour rainfall, P_2	$P_2 := 2.6$	inches
5. Land Slope, $S_{st} := 0.50$	$S_{st} = 0.5$	
6. Sheet Flow Time, $T_{st} := \dfrac{0.007\cdot(n_{st}\cdot L_{st})^{0.8}}{P_2^{0.5}\cdot S_{st}^{0.4}}$	$T_{st} = 0.028$	hours
SHALLOW CONCENTRATED FLOW	Flowpath: NA	
7. Surface description (paved or unpaved)		
8. Flow length, L_{sc}	$L_{sc} := 0$	feet
9. Watercourse Slope, $S_{sc} := 0$	$S_{sc} = 0$	
10. Average Velocity, $V_{sc} := 16.1345\cdot S_{sc}^{0.5}$	$V_{sc} = 0$	fps
11. Shallow Conc. Flow time, $T_{sc} := \left(\dfrac{L_{sc}}{3600\cdot V_{sc}}\right)$	$T_{sc} = 0$	hour
CHANNEL FLOW	Flowpath: b-c	
12. Bottom width, b	$b := 0$	feet
13. Side slopes, $z := \dfrac{15+2}{2}$	$z = 8.5$	
14. Flow depth, d	$d := 1$	feet
15. Cross sectional area, $a := (b + z\cdot d)\cdot d$	$a = 8.5$	ft^2
16. Wetted perimeter, $P_w := \left[b + 2\cdot d\cdot\left(1+z^2\right)^{0.5}\right]$	$P_w = 17.117$	feet
17. Hydraulic radius, $r := \dfrac{a}{P_w}$	$r = 0.497$	feet
18. Channel Length, L_{ch}	$L_{ch} = 1070$	feet
19. Channel Slope, $S_{ch} := \dfrac{1284 - 1265}{L_{ch}}$	$S_{ch} = 0.018$	
20. Channel lining	Grass	
21. Manning's roughness coeff., n	$n := 0.045$	
22. Velocity, $V_{ch} := \left[\left(\dfrac{1.49}{n}\right)\cdot\left[r^{\left(\frac{2}{3}\right)}\right]\cdot S_{ch}^{\left(\frac{1}{2}\right)}\right]$	$V_{ch} = 2.767$	fps
22. Channel Flow time, $T_{ch} := \left(\dfrac{L_{ch}}{3600\cdot V_{ch}}\right)$	$T_{ch} = 0.107$	hour

SUBJECT: Penelec - Keystone West Valley
Phase II Permitting - Ultimate Conditions
BY: SER DATE: 4/9/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KMB DATE: 6/10/96 SHEET NO. 22 OF 45

Time of Concentration Worksheet - SCS Methods
Watershed or Basin SE3 Continued
Postdevelopment Conditions

CHANNEL FLOW — Flowpath: c-d

12. Bottom width, b $b_1 := 3$
13. Side slopes, $z_1 := 2$
14. Flow depth, d $d_1 := 2$
15. Cross sectional area, $a_1 := (b_1 + z_1 \cdot d_1) \cdot d_1$ $a_1 = 14$
16. Wetted perimeter, $P_{w1} := \left[b_1 + 2 \cdot d_1 \cdot \left(1 + z_1^2\right)^{0.5}\right]$ $P_{w1} = 11.944$
17. Hydraulic radius, $r_1 := \frac{a_1}{P_{w1}}$ $r_1 = 1.172$
18. Channel Length, L_{ch} $L_{ch1} := 1010$
19. Channel Slope, $S_{ch1} := \frac{1265 - 1144}{L_{ch1}}$ $S_{ch1} = 0.12$
20. Channel lining Grouted Rock
21. Manning's roughness coeff., n $n_1 := 0.025$
22. Velocity, $V_{ch1} := \left[\left(\frac{1.49}{n_1}\right) \cdot \left[r_1^{\left(\frac{2}{3}\right)}\right] \cdot S_{ch1}^{\left(\frac{1}{2}\right)}\right]$ $V_{ch1} = 22.933$
22. Channel Flow time, $T_{ch1} := \left(\frac{L_{ch1}}{3600 \cdot V_{ch1}}\right)$ $T_{ch1} = 0.012$

CHANNEL FLOW — Flowpath: d-e

12. Bottom width, b $b_2 := 5$
13. Side slopes, $z_2 := 2$
14. Flow depth, d $d_2 := 1$
15. Cross sectional area, $a_2 := (b_2 + z_2 \cdot d_2) \cdot d_2$ $a_2 = 7$
16. Wetted perimeter, $P_{w2} := \left[b_2 + 2 \cdot d_2 \cdot \left(1 + z_2^2\right)^{0.5}\right]$ $P_{w2} = 9.472$
17. Hydraulic radius, $r_2 := \frac{a_2}{P_{w2}}$ $r_2 = 0.739$
18. Channel Length, L_{ch} $L_{ch2} := 250$
19. Channel Slope, $S_{ch2} := \frac{1144 - 1135}{L_{ch2}}$ $S_{ch2} = 0.036$
20. Channel lining Grouted Rock
21. Manning's roughness coeff., n $n_2 := 0.025$
22. Velocity, $V_{ch2} := \left[\left(\frac{1.49}{n_2}\right) \cdot \left[r_2^{\left(\frac{2}{3}\right)}\right] \cdot S_{ch2}^{\left(\frac{1}{2}\right)}\right]$ $V_{ch2} = 9.243$
22. Channel Flow time, $T_{ch2} := \left(\frac{L_{ch2}}{3600 \cdot V_{ch2}}\right)$ $T_{ch2} = 0.008$

Total Watershed Time-of-Concentration, $T_c := T_{st} - T_{sc} + T_{ch} + T_{ch1} - T_{ch2}$ $T_c = 0.155$ hour

SUBJECT: Penelec - Keystone West Valley
Phase II Permitting - Ultimate Conditions
BY: SER DATE: 4/9/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KMB DATE: 6/10/96 SHEET NO. 23 OF 45

Time of Concentration Worksheet - SCS Methods Reference: "Urban Hydrology for Small Watersheds",
Watershed or Basin SE4 TR-55, Soil Conservation Service, June 1986
Postdevelopment Conditions

SHEET FLOW Flowpath: a-b units

1. Surface description (table 3-1) Dense Grass
2. Manning's roughness coeff., n_{st} (table 3-1) $n_{st} := 0.24$
3. Flow length, L_{st} (total $L_{st} \le 150$ feet) $L_{st} := 65$ feet
4. Two-year, 24-hour rainfall, P_2 $P_2 := 2.6$ inches
5. Land Slope, $S_{st} := 0.40$ $S_{st} = 0.4$
6. Sheet Flow Time, $T_{st} := \dfrac{0.007\cdot(n_{st}\cdot L_{st})^{0.8}}{P_2^{\,0.5}\cdot S_{st}^{\,0.4}}$ $T_{st} = 0.056$ hours

SHALLOW CONCENTRATED FLOW Flowpath: NA

7. Surface description (paved or unpaved)
8. Flow length, L_{sc} $L_{sc} := 0$ feet
9. Watercourse Slope, $S_{sc} := 0$ $S_{sc} = 0$
10. Average Velocity, $V_{sc} := 16.1345\cdot S_{sc}^{\,0.5}$ $V_{sc} = 0$ fps
11. Shallow Conc. Flow time, $T_{sc} := \left(\dfrac{L_{sc}}{3600\cdot V_{sc}}\right)$ $T_{sc} = 0$ hour

CHANNEL FLOW Flowpath: b-c Flowpath: c-d Time for flowpath d-e is negligible, assume t=0

12. Bottom width, b $b := 0$ feet $b_1 := 2$
13. Side slopes, $z := \dfrac{15 + 2.5}{2}$ $z = 8.75$ $z_1 := 2$
14. Flow depth, d $d := 1$ feet $d_1 := 2.5$
15. Cross sectional area, $a := (b + z\cdot d)\cdot d$ $a = 8.75$ ft^2 $a_1 := (b_1 - z_1\cdot d_1)\cdot d_1$ $a_1 = 17.5$
16. Wetted perimeter, $P_w := \left[b + 2\cdot d\cdot\left(1 + z^2\right)^{0.5}\right]$ $P_w = 17.614$ feet $P_{w1} := \left[b_1 + 2\cdot d_1\cdot\left(1 + z_1^{\,2}\right)^{0.5}\right]$ $P_{w1} = 13.18$
17. Hydraulic radius, $r := \dfrac{a}{P_w}$ $r = 0.497$ feet $r_1 := \dfrac{a_1}{P_{w1}}$ $r_1 = 1.328$
18. Channel Length, L_{ch} $L_{ch} := 1360$ feet $L_{ch1} := 1750$
19. Channel Slope, $S_{ch} := .02$ $S_{ch} = 0.02$ $S_{ch1} := \dfrac{1395 - 1220}{L_{ch1}}$ $S_{ch1} = 0.1$
20. Channel lining Grass Grouted Rock
21. Manning's roughness coeff., n $n := 0.045$ $n_1 := 0.025$
22. Velocity, $V_{ch} := \left[\left(\dfrac{1.49}{n}\right)\cdot\left[r^{\left(\frac{2}{3}\right)}\right]\cdot S_{ch}^{\left(\frac{1}{2}\right)}\right]$ $V_{ch} = 2.937$ fps $V_{ch1} := \left[\left(\dfrac{1.49}{n_1}\right)\cdot\left[r_1^{\left(\frac{2}{3}\right)}\right]\cdot S_{ch1}^{\left(\frac{1}{2}\right)}\right]$ $V_{ch1} = 22.768$
22. Channel Flow time, $T_{ch} := \left(\dfrac{L_{ch}}{3600\cdot V_{ch}}\right)$ $T_{ch} = 0.129$ hour $T_{ch1} := \left(\dfrac{L_{ch1}}{3600\cdot V_{ch1}}\right)$ $T_{ch1} = 0.021$

Total Watershed Time-of-Concentration, $T_c := T_{st} + T_{sc} + T_{ch} + T_{ch1}$ $T_c = 0.206$ hour

SHEET 24/45

BY SER 6/14/96
✓ KMB 6/14/96

```
JOB TR-20                   FULLPRINT          SUMMARY   NOPLOTS
TITLE 111   KEYSTONE WEST VALLEY - ULT. COND. DITCH DESIGN - 92-220-73-7
  RUNOFF 1 001      1 0.0192    78.        0.31       1           W1
6 RUNOFF 1 001      1 0.0036    78.        0.15       1           N1
6 RUNOFF 1 001      2 0.0072    79.        0.19       1           N2
6 ADDHYD 4 001   1 2 3                                1           N DIT
6 RUNOFF 1 001      4 0.0400    75.        0.32       1           N3
6 ADDHYD 4 001   3 4 5                                1           EV EP
6 RUNOFF 1 001      6 0.0363    78.        0.24       1           S1
6 RUNOFF 1 001      7 0.0163    79.        0.17       1           S2
6 ADDHYD 4 001   6 7 1                                1           SW D
6 RUNOFF 1 001      2 0.013     78.        0.22       1           S3
6 ADDHYD 4 001   1 2 3                                1           S TR
6 RUNOFF 1 001      1 0.0659    77.        0.39       1           S4
6 RUNOFF 1 001      1 0.0027    80.        0.24       1           S6
6 RUNOFF 1 001      1 0.0448    78.        0.28       1           SE1
6 RUNOFF 1 001      2 0.0061    78.        0.17       1           SE2
6 ADDHYD 4 001   1 2 3                                1
6 RUNOFF 1 001      4 0.0166    78.        0.16       1           SE3
6 ADDHYD 4 001   3 4 5                                1           EVWSCC
6 RUNOFF 1 001      6 0.0275    80.        0.21       1           SE4
6 ADDHYD 4 001   5 6 7                                1           EVWSCC
6 RUNOFF 1 001      1 0.0044    80.        0.10       1           LOCAL
6 ADDHYD 4    01 7 1 2                                1           EVWSCC
  ENDATA
7 LIST
7 INCREM 6            0.1
  COMPUT 7 001     01 0.           4.4        1.         2 2      25 YR
  ENDCMP 1
  ENDJOB 2
```

ABOVE ACCESS ROAD CULVERTS

AT ACCESS ROAD CULVERTS' INLETS

LOCAL DRAINAGE BELOW CULVERTS SEE SHEET 45

BELOW ACCESS ROAD CULVERTS

SHEET 25/45

SUMMARY TABLE 1 - SELECTED RESULTS OF STANDARD AND EXECUTIVE CONTROL INSTRUCTIONS IN THE ORDER PERFORMED
(A STAR(*) AFTER THE PEAK DISCHARGE TIME AND RATE (CFS) VALUES INDICATES A FLAT TOP HYDROGRAPH
A QUESTION MARK(?) INDICATES A HYDROGRAPH WITH PEAK AS LAST POINT.)

ON/ CTURE ID		STANDARD CONTROL OPERATION	DRAINAGE AREA (SQ MI)	RAIN TABLE #	ANTEC MOIST COND	MAIN TIME INCREM (HR)	PRECIPITATION BEGIN (HR)	AMOUNT (IN)	DURATION (HR)	RUNOFF AMOUNT (IN)	PEAK DISCHARGE ELEVATION (FT)	TIME (HR)	RATE (CFS)	RATE (CSM)
ALTERNATE 0 STORM 0														
+														
XSECTION	1	RUNOFF	.02	2	2	.10	.0	4.40	24.00	2.21	---	12.09	28.90	1505.1
XSECTION	1	RUNOFF	.00	2	2	.10	.0	4.40	24.00	2.21	---	11.99	6.84	1898.8
XSECTION	1	RUNOFF	.01	2	2	.10	.0	4.40	24.00	2.30	---	12.02	13.15	1826.5
XSECTION	1	ADDHYD	.01	2	2	.10	.0	4.40	24.00	2.27	---	12.01	19.92	1844.4
XSECTION	1	RUNOFF	.04	2	2	.10	.0	4.40	24.00	1.97	---	12.10	52.70	1317.5
XSECTION	1	ADDHYD	.05	2	2	.10	.0	4.40	24.00	2.03	---	12.07	69.33	1364.7
XSECTION	1	RUNOFF	.04	2	2	.10	.0	4.40	24.00	2.22	---	12.05	60.10	1655.7
XSECTION	1	RUNOFF	.02	2	2	.10	.0	4.40	24.00	2.30	---	12.00	30.78	1888.2
XSECTION	1	ADDHYD	.05	2	2	.10	.0	4.40	24.00	2.24	---	12.03	89.28	1697.3
XSECTION	1	RUNOFF	.01	2	2	.10	.0	4.40	24.00	2.21	---	12.04	21.89	1683.9
XSECTION	1	ADDHYD	.07	2	2	.10	.0	4.40	24.00	2.24	---	12.03	111.16	1694.5
XSECTION	1	RUNOFF	.07	2	2	.10	.0	4.40	24.00	2.13	---	12.13	84.99	1289.7
XSECTION	1	RUNOFF	.00	2	2	.10	.0	4.40	24.00	2.38	---	12.05	4.80	1776.4
XSECTION	1	RUNOFF	.04	2	2	.10	.0	4.40	24.00	2.21	---	12.08	70.36	1570.5
XSECTION	1	RUNOFF	.01	2	2	.10	.0	4.40	24.00	2.22	---	12.00	11.11	1821.8
ION	1	ADDHYD	.05	2	2	.10	.0	4.40	24.00	2.21	---	12.07	79.92	1570.2
CTION	1	RUNOFF	.02	2	2	.10	.0	4.40	24.00	2.21	---	12.00	30.81	1855.9
XSECTION	1	ADDHYD	.07	2	2	.10	.0	4.40	24.00	2.21	---	12.04	107.33	1590.1
XSECTION	1	RUNOFF	.03	2	2	.10	.0	4.40	24.00	2.38	---	12.03	50.38	1831.9
XSECTION	1	ADDHYD	.09	2	2	.10	.0	4.40	24.00	2.26	---	12.04	157.61	1659.1
XSECTION	1	RUNOFF	.00	2	2	.10	.0	4.40	24.00	2.35	---	11.97	9.88	2244.6
STRUCTURE	1	ADDHYD	.10	2	2	.10	.0	4.40	24.00	2.27	---	12.03	165.99	1669.9

1

SUMMARY TABLE 3 - DISCHARGE (CFS) AT XSECTIONS AND STRUCTURES FOR ALL STORMS AND ALTERNATES

XSECTION/ STRUCTURE ID		DRAINAGE AREA (SQ MI)	STORM NUMBERS.......... 0
0 STRUCTURE	1	.10	
+ ALTERNATE	0		165.99
XSECTION	1	.00	
+ ALTERNATE	0		9.88

1END OF 1 JOBS IN THIS RUN

GAI CONSULTANTS, INC.
Engineers Geologists Planners
Environmental Specialists

Hydraulics

The design flow, lining, bottom width (b), side slope (z), and maximum and minimum slope for each drainage structure is summarized below.

Drainage Structure	Design Flow (cfs)	Maximum Slope	Minimum Slope	Lining	Bottom Width	Side Slopes, z
West Ditch	29	$\frac{5}{18}=0.278$	$\frac{5}{110}=0.045$	Grouted Rock	2	2
North Ditch -						
Part 1	7	$\frac{5}{70}=0.071$	$\frac{5}{85}=0.059$	Grass	2	2
Part 2	20	$\frac{5}{25}=0.2$	$\frac{5}{50}=0.1$	Grouted Rock	2	2
Part 3	69	$\frac{15}{250}=0.06$	$\frac{15}{250}=0.06$	Grouted Rock	2	2
Northwest Ditch	13	$\frac{5}{270}=0.019$	$\frac{5}{270}=0.019$	Grass	2	2
Southwest Ditch -						
Part 1	90	0.01	0.01	Grass	2	2
Part 2	90	$\frac{5}{15}=0.333$	$\frac{5}{35}=0.143$	Grouted Rock	2	2
Southeast Ditch - Part 1*	22	$\frac{5}{32}=0.156$	$\frac{5}{150}=0.033$	Grouted Rock	2	2
Haul Road Clean Water Ditch	5	0.1	0.1	Grouted Rock	2	2
North Top of Pile Swale	53	$\frac{25}{415}=0.06$	$\frac{5}{135}=0.037$	Grass	0	3
South Top of Pile Swale	85	$\frac{5}{110}=0.045$	$\frac{5}{330}=0.015$	Grass	0	3
Southeast Slope Drain	71	0.4	0.05	Concrete Revetment Uniform Section Mat	2	2
West Slope Drain	60	0.4	0.05	Concrete Revetment Uniform Section Mat	2	2
Existing East Valley West Side Collection Channel -						
Part 1	108	$\frac{45}{255}=0.176$	$\frac{5}{160}=0.031$	Grouted Rock	3	2
Existing East Valley Haul Road Ditch	51	$\frac{25}{250}=0.1$	$\frac{25}{250}=0.1$	Grouted Rock	2	2

* The Southeast Ditch - Part 1 is the Southeast Ditch above the proposed haul road and is designed within this calc. set. The Southeast Ditch - Part 2 is the Southeast Ditch below the proposed haul road and is designed in another calc. set.

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This calc set is in a Mathcad file. The first 4 pages are documentation of the method used and will only be presented for the first channel, the West Ditch. Subsequent channel designs will only present the last page, page 5.

The design herein is based on Manning's Equation (English Units).

INPUT SECTION

Design Flow, $Q_d := 29 \cdot \frac{ft^3}{sec}$

Bottom Width, $b := 2 \cdot ft$

Side Slopes, left side $z_L := 2$, right side $z_R = 2$, average $z := \frac{z_L + z_R}{2}$ or $z = 2$

Channel Lining is Grouted Rock
Manning's roughness coefficient, $n := 0.025$

Channel Maximum Slope, $S_{max} := \frac{5 \cdot ft}{18 \cdot ft}$ or $S_{max} = 0.278 \cdot \frac{ft}{ft}$

Channel Minimum Slope, $S_{min} := \frac{5 \cdot ft}{110 \cdot ft}$ or $S_{min} = 0.045 \cdot \frac{ft}{ft}$

Manning's Equation, $Q(n, A, P, S) := \frac{1.49}{n} \cdot A^{\frac{5}{3}} \cdot P^{-\frac{2}{3}} \cdot S^{\frac{1}{2}}$

Flow Area, $A(d) := (b + z \cdot d) \cdot d$

Wetted Perimeter, $P(d) := \left[b + d \left[\left(1 - z_L^2\right)^{\frac{1}{2}} + \left(1 + z_R^2\right)^{\frac{1}{2}} \right] \right]$

CALCULATION SECTION

Definition of Channel Capacity, considering Maximum Slope and Minimum Depth

$$Q_1(d_{min}) := \left[\frac{1.49 \cdot \frac{ft^{\frac{1}{3}}}{sec}}{n} \right] \left[(b + z \cdot d_{min}) \cdot d_{min} \right]^{\frac{5}{3}} \left[b + d_{min} \left[\left(1 + z_L^2\right)^{\frac{1}{2}} + \left(1 + z_R^2\right)^{\frac{1}{2}} \right] \right]^{-\frac{2}{3}} S_{max}^{\frac{1}{2}}$$

Define function to be solved for, $f(d_{min})$ with d_{min} = minimum depth and $f(d_{min}) := Q_d - Q_1(d_{min})$

Make an initial guess at the minimum depth, $d_{min} := 3 \cdot ft$

Define the solution as the root of the function f, $solution := root(f(d_{min}), d_{min})$ or $solution = 0.558 \cdot ft$

Therefore the Minimum depth is $d_{min} := solution$ or $d_{min} = 0.558 \cdot ft$

and the area of flow at d_{min} is $a_{min} := (b + z \cdot d_{min}) \cdot d_{min}$ or $a_{min} = 1.739 \cdot ft^2$

and the Maximum velocity is $V_{max} := \frac{Q_d}{a_{min}}$ or $V_{max} = 16.676 \cdot ft \cdot sec^{-1}$

The Top Width at d_{min}, $T_{min} := (b + 2 \cdot z \cdot d_{min})$ or $T_{min} = 4.232 \cdot ft$

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Definition of Channel Capacity, considering Minimum Slope and Maximum Depth

$$Q_2(d_{max}) := \left[\frac{1.49 \cdot \frac{ft^{\frac{1}{3}}}{sec}}{n}\right] \cdot \left[(b + z \cdot d_{max}) \cdot d_{max}\right]^{\frac{5}{3}} \cdot \left[b + d_{max} \cdot \left[\left(1 + z_L^{\,2}\right)^{\frac{1}{2}} + \left(1 + z_R^{\,2}\right)^{\frac{1}{2}}\right]\right]^{-\frac{2}{3}} \cdot S_{min}^{\;\frac{1}{2}}$$

Define function to be solved for, $g(d_{max})$ with d_{max} = maximum depth and $g(d_{max}) := Q_d - Q_2(d_{max})$

Make an initial guess at the maximum depth, $d_{max} := 3 \cdot ft$

Define the solution as the root of the function g , $solution := root(g(d_{max}), d_{max})$ or $solution = 0.888 \cdot ft$

Therefore the Maximum depth is $d_{max} := solution$ or $d_{max} = 0.888 \cdot ft$

and the area of flow at d_{max} is $a_{max} := (b + z \cdot d_{max}) \cdot d_{max}$ or $a_{max} = 3.353 \cdot ft^2$

and the Minimum velocity is $V_{min} := \frac{Q_d}{a_{max}}$ or $V_{min} = 8.649 \cdot ft \cdot sec^{-1}$

The Top Width at d_{max}, $T_{max} := (b + 2 \cdot z \cdot d_{max})$ or $T_{max} = 5.552 \cdot ft$

Freeboard
Method as per the PaDER "Erosion and Sediment Pollution Control Program Manual", April 1990 (ESPCPM)

Area of flow at d_{max}, $a_{max} = 3.353 \cdot ft^2$

Wetted Perimeter at d_{max} $P_{max} := \left[b + d_{max} \cdot \left[\left(1 + z_L^{\,2}\right)^{\frac{1}{2}} + \left(1 + z_R^{\,2}\right)^{\frac{1}{2}}\right]\right]$ or $P_{max} = 5.971 \cdot ft$

Mean depth for d_{max}, $D_{max} := \left(\frac{a_{max}}{T_{max}}\right)$ or $D_{max} = 0.604 \cdot ft$

Hydraulic radius for d_{max}, $R_{max} := \frac{a_{max}}{P_{max}}$ or $R_{max} = 0.562 \cdot ft$

Critical slope considering d_{max}, $S_{cmax} := 14.56 \cdot ft^{\frac{1}{3}} \cdot n^2 \cdot \left[\frac{D_{max}}{R_{max}^{\left(\frac{4}{3}\right)}}\right]$ or $S_{cmax} = 0.012$

The ESPCPM defines flow as unstable if channel slope S_o is greater than $0.7 \cdot S_c$ and less than $1.3 \cdot S_c$.
S_o is equal to S_{min} for this calc. $S_o := S_{min}$

$0.7 \cdot S_{cmax} = 0.008$ $S_o = 0.045$ $1.3 \cdot S_{cmax} = 0.015$

SUBJECT: Keystone Station
Phase II Permitting - Ultimate Conditions
BY: SER DATE: 4/12/96 PROJ. NO.: 92-220-73-07
CHKD. BY: MRL DATE: 5/28/96 SHEET NO. 29 OF 45

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Test for stability. Let positive test value = 1 and negative test value = 0

Define a variable called unstable as

$$unstable := if(S_o > 0.7 \cdot S_{cmax}, 1, 0) - if(S_o < 1.3 \cdot S_{cmax}, 1, 0)$$

$unstable = 1$ if unstable = 2, then the flow is unstable
if unstable = 1 or 0, then flow is stable

Define Freeboard for unstable flow, $F_u := max\left(\begin{pmatrix} 0.5 \cdot ft \\ 0.025 \cdot \frac{sec}{ft} \cdot V_{min} \cdot 3 \cdot d_{max} \end{pmatrix}\right)$

Define Freeboard for stable flow, $F_s := max\left(\begin{pmatrix} 0.5 \cdot ft \\ 0.25 \cdot d_{max} \end{pmatrix}\right)$

Define required freeboard for d_{max} as $F_{bmax} := if(unstable > 1, F_u, F_s)$ or $F_{bmax} = 0.5 \cdot ft$

Next check conditions for d_{min}

Area of flow at d_{min}, $a_{min} = 1.739 \cdot ft^2$

Wetted Perimeter at d_{min}, $P_{min} := \left[b - d_{min} \cdot \left[\left(1 + z_L^{\,2}\right)^{\frac{1}{2}} + \left(1 + z_R^{\,2}\right)^{\frac{1}{2}}\right]\right]$ or $P_{min} = 4.496 \cdot ft$

Mean depth for d_{min}, $D_{min} := \left(\frac{a_{min}}{T_{min}}\right)$ or $D_{min} = 0.411 \cdot ft$

Hydraulic radius for d_{min}, $R_{min} := \frac{a_{min}}{P_{min}}$ or $R_{min} = 0.387 \cdot ft$

Critical slope considering d_{min}, $S_{cmin} := 14.56 \cdot ft^{\frac{1}{3}} \cdot n^2 \cdot \left[\frac{D_{min}}{R_{min}^{\left(\frac{4}{3}\right)}}\right]$ or $S_{cmin} = 0.013$

The ESPCPM defines flow as unstable if channel slope S_o is greater than $0.7 \cdot S_c$ and less than $1.3 \cdot S_c$.
S_o is equal to S_{max} for this calc. $S_o := S_{max}$

$0.7 \cdot S_{cmin} = 0.009$ $S_o = 0.278$ $1.3 \cdot S_{cmin} = 0.017$

Test for stability. Let positive test value = 1 and negative test value = 0

Define a variable called unstable as

$$unstable := if(S_o > 0.7 \cdot S_{cmin}, 1, 0) + if(S_o < 1.3 \cdot S_{cmin}, 1, 0)$$

$unstable = 1$ if unstable = 2, then the flow is unstable
if unstable = 1 or 0, then flow is stable

Define Freeboard for unstable flow, $F_u := max\left(\begin{pmatrix} 0.5 \cdot ft \\ 0.025 \cdot \frac{sec}{ft} \cdot V_{max} \cdot 3 \cdot d_{min} \end{pmatrix}\right)$

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Define Freeboard for stable flow, $F_s := \max\left(\begin{pmatrix} 0.5\cdot ft \\ 0.25\cdot d_{min} \end{pmatrix}\right)$

Define required freeboard for d_{min} as $F_{bmin} := if(unstable>1, F_u, F_s)$ or $F_{bmin} = 0.5\cdot ft$

The required Total Depth, D is the maximum of $d_{max} + F_{bmax}$ and $d_{min} + F_{bmin}$

$D := \max\left(\begin{pmatrix} d_{max} + F_{bmax} \\ d_{min} + F_{bmin} \end{pmatrix}\right)$ $d_{max} = 0.888\cdot ft$ $F_{bmax} = 0.5\cdot ft$ $d_{max} + F_{bmax} = 1.388\cdot ft$
$d_{min} = 0.558\cdot ft$ $F_{bmin} = 0.5\cdot ft$ $d_{min} + F_{bmin} = 1.058\cdot ft$

Total depth, $D = 1.388\cdot ft$

Next round the Total Depth to the next highest 0.5 foot if roundup is less than or = 0.4 foot, otherwise round down to the nearest 0.5 foot. The function floor(x) returns the integer of x.

$round := if((D - floor(D) \le 0.1\cdot ft), floor(D), if((D - floor(D) \ge 0.6\cdot ft), floor(D) + 1.0\cdot ft, floor(D) + 0.5\cdot ft))$

$round = 1.5\cdot ft$

$D := round$

Actual Freeboard $F_b := D - d_{max}$ or $F_b = 0.612\cdot ft$

Top Width considering Total Depth, $T_D := (b + 2\cdot z\cdot D)$ or $T_D = 8\cdot ft$

Calculate Capacity of channel considering Total depth and minimum slope

$$Q_{tmin} := \left[\frac{1.49\cdot\frac{ft^{\frac{1}{3}}}{sec}}{n}\right]\cdot((b + z\cdot D)\cdot D)^{\frac{5}{3}}\cdot\left[b + D\cdot\left[\left(1 + z_L^{\,2}\right)^{\frac{1}{2}} + \left(1 + z_R^{\,2}\right)^{\frac{1}{2}}\right]\right]^{-\frac{2}{3}}\cdot S_{min}^{\frac{1}{2}} \quad \text{or} \quad Q_{tmin} = 86.268\cdot ft^3\cdot sec^{-1}$$

Calculate Capacity of channel considering Total depth and maximum slope

$$Q_{tmax} := \left[\frac{1.49\cdot\frac{ft^{\frac{1}{3}}}{sec}}{n}\right]\cdot((b + z\cdot D)\cdot D)^{\frac{5}{3}}\cdot\left[b + D\cdot\left[\left(1 + z_L^{\,2}\right)^{\frac{1}{2}} + \left(1 + z_R^{\,2}\right)^{\frac{1}{2}}\right]\right]^{-\frac{2}{3}}\cdot S_{max}^{\frac{1}{2}} \quad \text{or} \quad Q_{tmax} = 213.261\cdot ft^3\cdot sec^{-1}$$

Dummy Variables for presentation Purposes on the following sheet

$a := 1$ $r := 1$ $s := 1$

SUBJECT: Keystone Station
Phase II Permitting - Ultimate Conditions
BY: SER DATE: 4/12/96 PROJ. NO.: 92-220-73-07
CHKD. BY: MRC DATE: 5/20/96 SHEET NO. 31 OF 45

GAI CONSULTANTS, INC.
Engineers Geologists Planners
Environmental Specialists

Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right) \cdot a \cdot r^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right) \cdot (r)^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$

West Ditch

Design Flow, $Q_d = 29 \cdot ft^3 \cdot sec^{-1}$ from sheet 25 of 45

Bottom Width, $b = 2 \cdot ft$ ✓

Side Slopes, $z = 2$ ✓

Channel Lining is Grouted Rock with Manning's roughness coefficient, $n = 0.025$ ✓

Channel Minimum Slope, $S_{min} := \frac{5 \cdot ft}{110 \cdot ft}$ (from Sheet 26) or $S_{min} = 0.045 \cdot \frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 0.888 \cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 3.4 \cdot ft^2$

Minimum Velocity, $V_{min} = 8.6 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 5.6 \cdot ft$

Freeboard, $F_b = 0.6$ ft by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990

Total depth, $D = 1.5 \cdot ft$ ✓

Top Width at Total Depth, $T_D = 8 \cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 86 \cdot ft^3 \cdot sec^{-1}$

Channel Maximum Slope, $S_{max} := \frac{5 \cdot ft}{18 \cdot ft}$ (from Sheet 26) or $S_{max} = 0.278 \cdot \frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 0.558 \cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 1.7 \cdot ft^2$

Maximum Velocity, $V_{max} = 16.7 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 4.2 \cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 213 \cdot ft^3 \cdot sec^{-1}$

TYPE C-1 CHANNEL
USE TYPE C-2 CHANNEL
SEE STAGE 3 CONDITIONS CALC.

SUBJECT: Keystone Station
Phase II Permitting - Ultimate Conditions
BY: SER DATE: 4/12/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KMB DATE: 6/10/96 SHEET NO. 32 OF 45

GAI CONSULTANTS, INC.
Engineers Geologists Planners
Environmental Specialists

Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right)\cdot a \cdot r^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right)\cdot (r)^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$

North Ditch - Part 1

Design Flow, $Q_d = 7\cdot ft^3 \cdot sec^{-1}$ from sheet 25 of 45

Bottom Width, $b = 2\cdot ft$

Side Slopes, $z = 2$

Channel Lining is Grass with Manning's roughness coefficient, $n = 0.045$

Channel Minimum Slope, $S_{min} := \frac{5\cdot ft}{85\cdot ft}$ (from Sheet 26) or $S_{min} = 0.059 \cdot \frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 0.541\cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 1.7\cdot ft^2$

Minimum Velocity, $V_{min} = 4.2\cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 4.2\cdot ft$

Freeboard, $F_b = 0.5\cdot ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990

Total depth, $D = 1\cdot ft$

Top Width at Total Depth, $T_D = 6\cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 23\cdot ft^3 \cdot sec^{-1}$

Channel Maximum Slope, $S_{max} := \frac{5\cdot ft}{70\cdot ft}$ (from Sheet 26) or $S_{max} = 0.071 \cdot \frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 0.514\cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 1.6\cdot ft^2$

Maximum Velocity, $V_{max} = 4.5\cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 4.1\cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 26\cdot ft^3 \cdot sec^{-1}$

TYPE A-1 CHANNEL

SUBJECT: Keystone Station
Phase II Permitting - Ultimate Conditions
BY: SER DATE: 4/12/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KMB DATE: 6/10/96 SHEET NO. 33 OF 45

CONSULTANTS, INC.
Engineers Geologists Planners
Environmental Specialists

Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right) \cdot a \cdot r^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right) \cdot (r)^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$

North Ditch - Part 2

Design Flow, $Q_d = 20 \cdot ft^3 \cdot sec^{-1}$ from sheet 25 of 45

Bottom Width, $b = 2 \cdot ft$

Side Slopes, $z = 2$

Channel Lining is Grouted Rock with Manning's roughness coefficient, $n = 0.025$

Channel Minimum Slope, $S_{min} := \frac{5 \cdot ft}{50 \cdot ft}$ (from Sheet 26) or $S_{min} = 0.1 \cdot \frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 0.6 \cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 1.9 \cdot ft^2$

Minimum Velocity, $V_{min} = 10.4 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 4.4 \cdot ft$

Freeboard, $F_b = 0.9 \cdot ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990

Total depth, $D = 1.5 \cdot ft$

Top Width at Total Depth, $T_D = 8 \cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 128 \cdot ft^3 \cdot sec^{-1}$

Channel Maximum Slope, $S_{max} := \frac{5 \cdot ft}{25 \cdot ft}$ (from Sheet 26) or $S_{max} = 0.2 \cdot \frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 0.5 \cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 1.5 \cdot ft^2$

Maximum Velocity, $V_{max} = 13.3 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 4 \cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 181 \cdot ft^3 \cdot sec^{-1}$

TYPE C-1 CHANNEL

SUBJECT: Keystone Station
Phase II Permitting - Ultimate Conditions
BY: SER DATE: 4/12/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KMB DATE: 7/24/96 SHEET NO. 34 OF 45

CONSULTANTS, INC.
Engineers Geologists Planners Environmental Specialists

Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right) \cdot a \cdot r^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right) \cdot (r)^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$

North Ditch - Part 3

Design Flow, $Q_d = 69 \cdot ft^3 \cdot sec^{-1}$ from sheet 25 of 45

Bottom Width, $b = 2 \cdot ft$

Side Slopes, $z = 2$

Channel Lining is Grouted Rock with Manning's roughness coefficient, $n = 0.025$

Channel Minimum Slope, $S_{min} := \frac{15 \cdot ft}{250 \cdot ft}$ (from Sheet 26) or $S_{min} = 0.06 \cdot \frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 1.265 \cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 5.7 \cdot ft^2$

Minimum Velocity, $V_{min} = 12 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 7.1 \cdot ft$

Freeboard, $F_b = 0.7 \cdot ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990

Total depth, $D = 2 \cdot ft$

Top Width at Total Depth, $T_D = 10 \cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 186 \cdot ft^3 \cdot sec^{-1}$

Channel Maximum Slope, $S_{max} := \frac{15 \cdot ft}{250 \cdot ft}$ (from Sheet 26) or $S_{max} = 0.06 \cdot \frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 1.265 \cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 5.7 \cdot ft^2$

Maximum Velocity, $V_{max} = 12 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 7.1 \cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 186 \cdot ft^3 \cdot sec^{-1}$

TYPE C-2 CHANNEL

SUBJECT: Keystone Station
Phase II Permitting - Ultimate Conditions
BY: SER DATE: 4/12/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KMA DATE: 6/10/96 SHEET NO. 35 OF 45

GAI CONSULTANTS, INC.
Engineers Geologists Planners
Environmental Specialists

Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right) \cdot a \cdot r^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right) \cdot (r)^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$

Northwest Ditch

Design Flow, $Q_d = 13 \cdot ft^3 \cdot sec^{-1}$ from sheet 25 of 45

Bottom Width, $b = 2 \cdot ft$

Side Slopes, $z = 2$

Channel Lining is Grass with Manning's roughness coefficient, $n = 0.045$

Channel Minimum Slope, $S_{min} := \frac{5 \cdot ft}{270 \cdot ft}$ (from Sheet 26) or $S_{min} = 0.019 \cdot \frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 0.997 \cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 4 \cdot ft^2$

Minimum Velocity, $V_{min} = 3.3 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 6 \cdot ft$

Freeboard, $F_b = 0.5 \cdot ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990

Total depth, $D = 1.5 \cdot ft$

Top Width at Total Depth, $T_D = 8 \cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 31 \cdot ft^3 \cdot sec^{-1}$

Channel Maximum Slope, $S_{max} := \frac{5 \cdot ft}{270 \cdot ft}$ (from Sheet 26) or $S_{max} = 0.019 \cdot \frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 0.997 \cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 4 \cdot ft^2$

Maximum Velocity, $V_{max} = 3.3 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 6 \cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 31 \cdot ft^3 \cdot sec^{-1}$

TYPE A-2 CHANNEL

SUBJECT: Keystone Station
Phase II Permitting - Ultimate Conditions
BY: SER DATE: 4/12/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KMB DATE: 7/26/96 SHEET NO. 36 OF 45

GAI CONSULTANTS INC.
Engineers Geologists Planners
Environmental Specialists

Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right)\cdot a\cdot r^{\left(\frac{2}{3}\right)}\cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right)\cdot(r)^{\left(\frac{2}{3}\right)}\cdot s^{\left(\frac{1}{2}\right)}$

Southwest Ditch - Part 1

Design Flow, $Q_d = 90\cdot ft^3\cdot sec^{-1}$ from sheet 25 of 45

Bottom Width, $b = 2\cdot ft$

Side Slopes, $z = 2$

Channel Lining is Grass with Manning's roughness coefficient, $n = 0.045$

Channel Minimum Slope, $S_{min} := \frac{1\cdot ft}{100\cdot ft}$ (from Sheet 26 or $S_{min} = 0.01\cdot\frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 2.788\cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 21.1\cdot ft^2$

Minimum Velocity, $V_{min} = 4.3\cdot ft\cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 13.2\cdot ft$

Freeboard, $F_b = 1.2\cdot ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990. SER added 0.5 feet.

Total depth, $D = 4\cdot ft$

Top Width at Total Depth, $T_D = 18\cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 211\cdot ft^3\cdot sec^{-1}$

Channel Maximum Slope, $S_{max} := \frac{1\cdot ft}{100\cdot ft}$ (from Sheet 26) or $S_{max} = 0.01\cdot\frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 2.788\cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 21.1\cdot ft^2$

Maximum Velocity, $V_{max} = 4.3\cdot ft\cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 13.2\cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 211\cdot ft^3\cdot sec^{-1}$

TYPE A-3 CHANNEL

SUBJECT: Keystone Station
Phase II Permitting - Ultimate Conditions
BY: SER DATE: 4/12/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KAAB DATE: 6/14/96 SHEET NO. 37 OF 45

GAI CONSULTANTS, INC.
Engineers Geologists Planners
Environmental Specialists

Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right) \cdot a \cdot r^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right) \cdot (r)^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$

Southwest Ditch - Part 2

Design Flow, $Q_d = 90 \cdot ft^3 \cdot sec^{-1}$ from sheet 35 of 45

Bottom Width, $b = 2 \cdot ft$

Side Slopes, $z = 2$

Channel Lining is Grouted Rock with Manning's roughness coefficient, $n = 0.025$

Channel Minimum Slope, $S_{min} := \frac{5 \cdot ft}{35 \cdot ft}$ (from Sheet 26) or $S_{min} = 0.143 \cdot \frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 1.168 \cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 5.1 \cdot ft^2$

Minimum Velocity, $V_{min} = 17.8 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 6.7 \cdot ft$

Freeboard, $F_b = 0.8 \cdot ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990

Total depth, $D = 2 \cdot ft$

Top Width at Total Depth, $T_D = 10 \cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 287 \cdot ft^3 \cdot sec^{-1}$

Channel Maximum Slope, $S_{max} := \frac{5 \cdot ft}{15 \cdot ft}$ (from Sheet 26) or $S_{max} = 0.333 \cdot \frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 0.95 \cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 3.7 \cdot ft^2$

Maximum Velocity, $V_{max} = 24.3 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 5.8 \cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 439 \cdot ft^3 \cdot sec^{-1}$

TYPE C-2 CHANNEL

SUBJECT: Keystone Station
Phase II Permitting - Ultimate Conditions
BY: SER DATE: 4/12/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KMB DATE: 6/10/96 SHEET NO. 38 OF 45

GAI CONSULTANTS, INC.
Engineers Geologists Planners
Environmental Specialists

Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right)\cdot a\cdot r^{\left(\frac{2}{3}\right)}\cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right)\cdot (r)^{\left(\frac{2}{3}\right)}\cdot s^{\left(\frac{1}{2}\right)}$

Southeast Ditch — PART 1

Design Flow, $Q_d = 22\cdot ft^3\cdot sec^{-1}$ from sheet 25 of 45

Bottom Width, $b = 2\cdot ft$

Side Slopes, $z = 2$

Channel Lining is Grouted Rock with Manning's roughness coefficient, $n = 0.025$

Channel Minimum Slope, $S_{min} := \frac{5\cdot ft}{150\cdot ft}$ (from Sheet 26) or $S_{min} = 0.033\cdot\frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 0.836\cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 3.1\cdot ft^2$

Minimum Velocity, $V_{min} = 7.2\cdot ft\cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 5.3\cdot ft$

Freeboard, $F_b = 0.7\cdot ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990

Total depth, $D = 1.5\cdot ft$

Top Width at Total Depth, $T_D = 8\cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 74\cdot ft^3\cdot sec^{-1}$

Channel Maximum Slope, $S_{max} := \frac{5\cdot ft}{32\cdot ft}$ (from Sheet 26) or $S_{max} = 0.156\cdot\frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 0.561\cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 1.8\cdot ft^2$

Maximum Velocity, $V_{max} = 12.5\cdot ft\cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 4.2\cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 160\cdot ft^3\cdot sec^{-1}$

TYPE C-1 CHANNEL

SUBJECT: Keystone Station
Phase II Permitting - Ultimate Conditions
BY: SER DATE: 4/12/96 PROJ. NO.: 92-220-73-07
CHKD. BY: HMB DATE: 6/10/96 SHEET NO. 39 OF 45

CONSULTANTS, INC.
Engineers Geologists Planners
Environmental Specialists

Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right) \cdot a \cdot r^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right) \cdot (r)^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$

Haul Road Clean Water Ditch

Design Flow, $Q_d = 5 \cdot ft^3 \cdot sec^{-1}$ from sheet 25 of 45

Bottom Width, $b = 2 \cdot ft$ ✓

Side Slopes, $z = 2$ ✓

Channel Lining is Grouted Rock with Manning's roughness coefficient, $n = 0.025$

Channel Minimum Slope, $S_{min} := \frac{10 \cdot ft}{100 \cdot ft}$ (from Sheet 26) or $S_{min} = 0.1 \cdot \frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 0.282 \cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 0.7 \cdot ft^2$

Minimum Velocity, $V_{min} = 6.9 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 3.1 \cdot ft$

Freeboard, $F_b = 0.7 \cdot ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990

Total depth, $D = 1 \cdot ft$ ✓

Top Width at Total Depth, $T_D = 6 \cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 55 \cdot ft^3 \cdot sec^{-1}$

Channel Maximum Slope, $S_{max} := \frac{10 \cdot ft}{100 \cdot ft}$ (from Sheet 26) or $S_{max} = 0.1 \cdot \frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 0.282 \cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 0.7 \cdot ft^2$

Maximum Velocity, $V_{max} = 6.9 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 3.1 \cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 55 \cdot ft^3 \cdot sec^{-1}$

TYPE C-3 CHANNEL
SEE DESIGN IN STAGE 3
CONDITIONS CALC.

SUBJECT: Keystone Station
Phase II Permitting - Ultimate Conditions
BY: SER DATE: 4/12/96 PROJ. NO.: 92-220-73-07
CHKD. BY: [illegible] DATE: 6/10/96 SHEET NO. 40 OF 45

GAI CONSULTANTS, INC.
Engineers Geologists Planners
Environmental Specialists

Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right) \cdot a \cdot r^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right) \cdot (r)^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$

North Top of Pile Swale

Design Flow, $Q_d = 53 \cdot ft^3 \cdot sec^{-1}$ from sheet 25 of 45

Bottom Width, $b = 0 \cdot ft$

Side Slopes, $z = 3$

Channel Lining is Grass with Manning's roughness coefficient, $n = 0.045$

Channel Minimum Slope, $S_{min} := \frac{5 \cdot ft}{135 \cdot ft}$ (from Sheet 26) or $S_{min} = 0.037 \cdot \frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 1.766 \cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 9.4 \cdot ft^2$

Minimum Velocity, $V_{min} = 5.7 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 10.6 \cdot ft$

Freeboard, $F_b = 1.2 \cdot ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990.

Total depth, $D = 3 \cdot ft$ All Top of Pile Swales will be 3 feet deep.

Top Width at Total Depth, $T_D = 18 \cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 218 \cdot ft^3 \cdot sec^{-1}$

Channel Maximum Slope, $S_{max} := \frac{25 \cdot ft}{415 \cdot ft}$ (from Sheet 26) or $S_{max} = 0.06 \cdot \frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 1.612 \cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 7.8 \cdot ft^2$

Maximum Velocity, $V_{max} = 6.8 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 9.7 \cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 278 \cdot ft^3 \cdot sec^{-1}$

TYPE A-H CHANNEL

SUBJECT: Keystone Station
Phase II Permitting - Ultimate Conditions
BY: SER DATE: 4/12/96 PROJ. NO.: 92-220-73-07
CHKD. BY: MWB DATE: 6/14/96 SHEET NO. 41 OF 45

GAI CONSULTANTS, INC.
Engineers Geologists Planners Environmental Specialists

Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right)\cdot a\cdot r^{\left(\frac{2}{3}\right)}\cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right)\cdot (r)^{\left(\frac{2}{3}\right)}\cdot s^{\left(\frac{1}{2}\right)}$

South Top of Pile Swale

Design Flow, $Q_d = 85\cdot ft^3\cdot sec^{-1}$ from sheet 25 of 45

Bottom Width, $b = 0\cdot ft$

Side Slopes, $z = 3$

Channel Lining is Grass (with nylon erosion control matting) with Manning's roughness coefficient, $n = 0.045$

Channel Minimum Slope, $S_{min} := \frac{5\cdot ft}{330\cdot ft}$ (from Sheet 26) or $S_{min} = 0.015\cdot\frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 2.493\cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 18.6\cdot ft^2$

Minimum Velocity, $V_{min} = 4.6\cdot ft\cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 15\cdot ft$

Freeboard, $F_b = 0.5\cdot ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990. Edited by SER to be 0.5 feet.

Total depth, $D = 3\cdot ft$

Top Width at Total Depth, $T_D = 18\cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 139\cdot ft^3\cdot sec^{-1}$

Channel Maximum Slope, $S_{max} := \frac{5\cdot ft}{110\cdot ft}$ (from Sheet 26) or $S_{max} = 0.045\cdot\frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 2.029\cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 12.4\cdot ft^2$

Maximum Velocity, $V_{max} = 6.9\cdot ft\cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 12.2\cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 241\cdot ft^3\cdot sec^{-1}$

TYPE B-1 CHANNEL

SUBJECT: Keystone Station
Phase II Permitting - Ultimate Conditions
BY: SER DATE: 4/12/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KMB DATE: 7/26/96 SHEET NO. 42 OF 45

CONSULTANTS, INC.
Engineers Geologists Planners
Environmental Specialists

Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right) \cdot a \cdot r^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right) \cdot (r)^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$

Slope Drains

Design Flow, $Q_d = 71 \cdot ft^3 \cdot sec^{-1}$ from sheet 25 of 45

Bottom Width, $b = 2 \cdot ft$

Side Slopes, $z = 2$

Channel Lining is Concrete Revetment, Uniform Section Mat with Manning's roughness coefficient, $n = 0.015$

Channel Minimum Slope, $S_{min} := \frac{5 \cdot ft}{100 \cdot ft}$ (from Sheet 26) or $S_{min} = 0.05 \cdot \frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 1.048 \cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 4.3 \cdot ft^2$

Minimum Velocity, $V_{min} = 16.5 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 6.2 \cdot ft$

Freeboard, $F_b = 1 \cdot ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April. 1990. SER added 0.5 feet.

Total depth, $D = 2 \cdot ft$

Top Width at Total Depth, $T_D = 10 \cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 283 \cdot ft^3 \cdot sec^{-1}$

Channel Maximum Slope, $S_{max} := \frac{1 \cdot ft}{2.5 \cdot ft}$ (from Sheet 26) or $S_{max} = 0.4 \cdot \frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 0.621 \cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 2 \cdot ft^2$

Maximum Velocity, $V_{max} = 35.3 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 4.5 \cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 802 \cdot ft^3 \cdot sec^{-1}$

SLOPE DRAIN

GAI CONSULTANTS, INC.
Engineers Geologists Planners
Environmental Specialists

Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right) \cdot a \cdot r^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right) \cdot (r)^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$

Existing East Valley West Side Collection Channel - Part 1

Design Flow, $Q_d = 108 \cdot ft^3 \cdot sec^{-1}$ from sheet 25 of 45

Bottom Width, $b = 3 \cdot ft$

Side Slopes, $z = 2$

Channel Lining is Grouted Rock with Manning's roughness coefficient, $n = 0.025$

Channel Minimum Slope, $S_{min} := \frac{5 \cdot ft}{160 \cdot ft}$ (from Sheet 26) or $S_{min} = 0.031 \cdot \frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 1.638 \cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 10.3 \cdot ft^2$

Minimum Velocity, $V_{min} = 10.5 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 9.6 \cdot ft$

Freeboard, $F_b = 0.9 \cdot ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990

Total depth, $D = 2.5 \cdot ft$ ACTUAL DEPTH OF EXISTING CHANNEL

Top Width at Total Depth, $T_D = 13 \cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 265 \cdot ft^3 \cdot sec^{-1}$

Channel Maximum Slope, $S_{max} := \frac{45 \cdot ft}{255 \cdot ft}$ (from Sheet 26) or $S_{max} = 0.176 \cdot \frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 1.064 \cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 5.5 \cdot ft^2$

Maximum Velocity, $V_{max} = 19.8 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 7.3 \cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 630 \cdot ft^3 \cdot sec^{-1}$

TYPE C-6 CHANNEL

SUBJECT: Keystone Station
Phase II Permitting - Ultimate Conditions
BY: SER DATE: 4/12/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KMB DATE: 7/30/96 SHEET NO. 44 OF 45

CONSULTANTS, INC.
Engineers Geologists Planners
Environmental Specialists

Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right) \cdot a \cdot r^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right) \cdot (r)^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$

Existing East Valley Haul Road Ditch

Design Flow, $Q_d = 51 \cdot ft^3 \cdot sec^{-1}$ from sheet 25 of 45

Bottom Width, $b = 2 \cdot ft$

Side Slopes, $z = 2$

Channel Lining is Grouted Rock with Manning's roughness coefficient, $n = 0.025$

Channel Minimum Slope, $S_{min} := \frac{25 \cdot ft}{250 \cdot ft}$ (from Sheet 26) or $S_{min} = 0.1 \cdot \frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 0.966 \cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 3.8 \cdot ft^2$

Minimum Velocity, $V_{min} = 13.4 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 5.9 \cdot ft$

Freeboard, $F_b = 1 \cdot ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990

Total depth, $D = 2 \cdot ft$ Actual depth of existing channel

Top Width at Total Depth, $T_D = 10 \cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 240 \cdot ft^3 \cdot sec^{-1}$

Channel Maximum Slope, $S_{max} := \frac{25 \cdot ft}{250 \cdot ft}$ (from Sheet 26) or $S_{max} = 0.1 \cdot \frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 0.966 \cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 3.8 \cdot ft^2$

Maximum Velocity, $V_{max} = 13.4 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 5.9 \cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 240 \cdot ft^3 \cdot sec^{-1}$

TYPE C-2 CHANNEL

SUBJECT KEYSTONE STATION

BY SER DATE 7/9/96 PROJ. NO. 92-220-73-07

CHKD BY KMB DATE 7/26/96 SHEET NO. 45 OF 45

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LOCAL DRAINAGE CONTRIBUTING TO THE
EAST VALLEY WEST SIDE COLLECTION CHANNEL
BELOW THE ACCESS ROAD CULVERTS

AREA = 2.8 ACRES = 0.0044 MI^2
FROM WORKSHEET FOR CALC BY EHK 2/7/85
TITLED "HYDROLOGIC PARAMATERS FOR CHANNEL DESIGN"

USE CN = 80, OFF SITE PASTURE

ASSUME t_c = 0.1 HOUR

SUBJECT KEYSTONE - WEST VALLEY
PHASE II PERMITTING

BY SER DATE 4/25/96 PROJ. NO. 92-220-73-07
CHKD. BY KMB DATE 5/31/96 SHEET NO. 1 OF 34

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STAGE 3 - DRAINAGE FACILITIES

HYDROLOGY

PURPOSE: ESTIMATE THE DESIGN FLOWS FOR THE STAGE 3 DRAINAGE FACILITIES

DESIGN STORMS: REFERENCE CALC BY SER 3/19/96 92-220-73-07 "ULTIMATE CONDITIONS - DRAINAGE FACILITIES"

METHODOLOGY: SAME REF

SEE SHEET 21 FOR HYDRAULICS DESIGN

SUBJECT KEYSTONE - WEST VALLEY
PHASE II PERMITTING
BY SER DATE 4/25/96 PROJ. NO. 92-220-73-07
CHKD. BY KMB DATE 5/31/96 SHEET NO. 2 OF 34

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A SKETCH OF "STAGE 3 DRAINAGE" IS SHOWN ON SHEET 3 AND A SCHEMATIC OF STAGE 3 DRAINAGE FACILITIES AND WATERSHEDS IS SHOWN ON SHEET 4.

THE ATTACHED WORKSHEET LABELLED "STAGE 3 WORKSHEET" 92-220-73-07-SER4 SHOWS THE DITCHES, SLOPE DRAINS, AND WATERSHEDS IN GREATER DETAIL.

NOTES:

1) FACILITIES WHICH ARE DESIGNED SOLELY FOR ULTIMATE CONDITIONS ARE NOT ADDRESSED HEREIN INCLUDING

2) FACILITIES WHICH MAY AFFECT ULTIMATE CONDITIONS DESIGNS HAVE DESIGN FLOWS DETERMINED HEREIN AND HYDRAULIC DESIGNS DOCUMENTED IN THE ULTIMATE CONDITIONS - DRAINAGE FACILITIES CALC., SEE SHEET 1. THESE FACILITIES ARE THE WEST CLEAN STORMWATER MANAGEMENT POND, AND THE WEST DITCH.

SUBJECT Keystone - West Valley
Phase II Permitting

BY SER DATE 4/25/96 PROJ. NO. 92-220-13-07

CHKD. BY KMB DATE 5/31/96 SHEET NO. 3 OF 34

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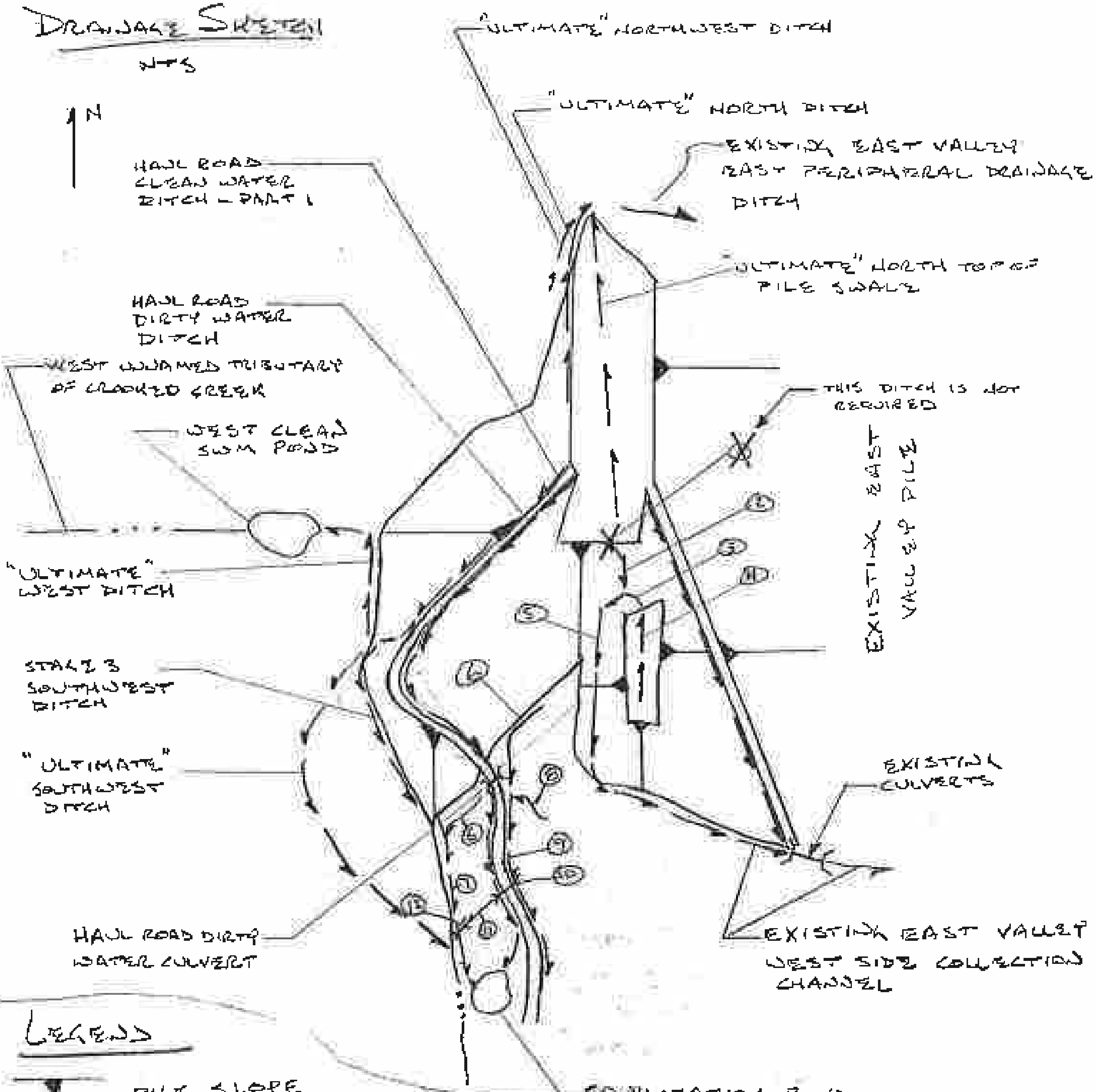
Stage 3 Drainage Sketch

NTS



Legend

PILE SLOPE

DRAINAGE DITCH OR SLOPE DRAIN

~~① NORTHEAST SLOPE DRAIN~~

② EAST SLOPE DRAIN

③ NORTHEAST DITCH

④ SOUTHEAST TOP OF PILE SWALE

⑤ EAST DITCH

⑥ NORTH DIRTY WATER DITCH

⑦ WEST DIRTY WATER DITCH

⑧ SOUTHEAST DITCH

⑨ SOUTH DITCH

⑩ ULTIMATE SOUTHEAST DITCH CULVERT 1

⑪ ULTIMATE SOUTHEAST DITCH

⑫ ULTIMATE SOUTHEAST DITCH CULVERT 2

SUBJECT KEYSTONE - WEST VALLEY

BY SER DATE 4/25/96 PROJ. NO. 92-220-73-07

CHKD. BY KMB DATE 5/31/96 SHEET NO. 4 OF 34

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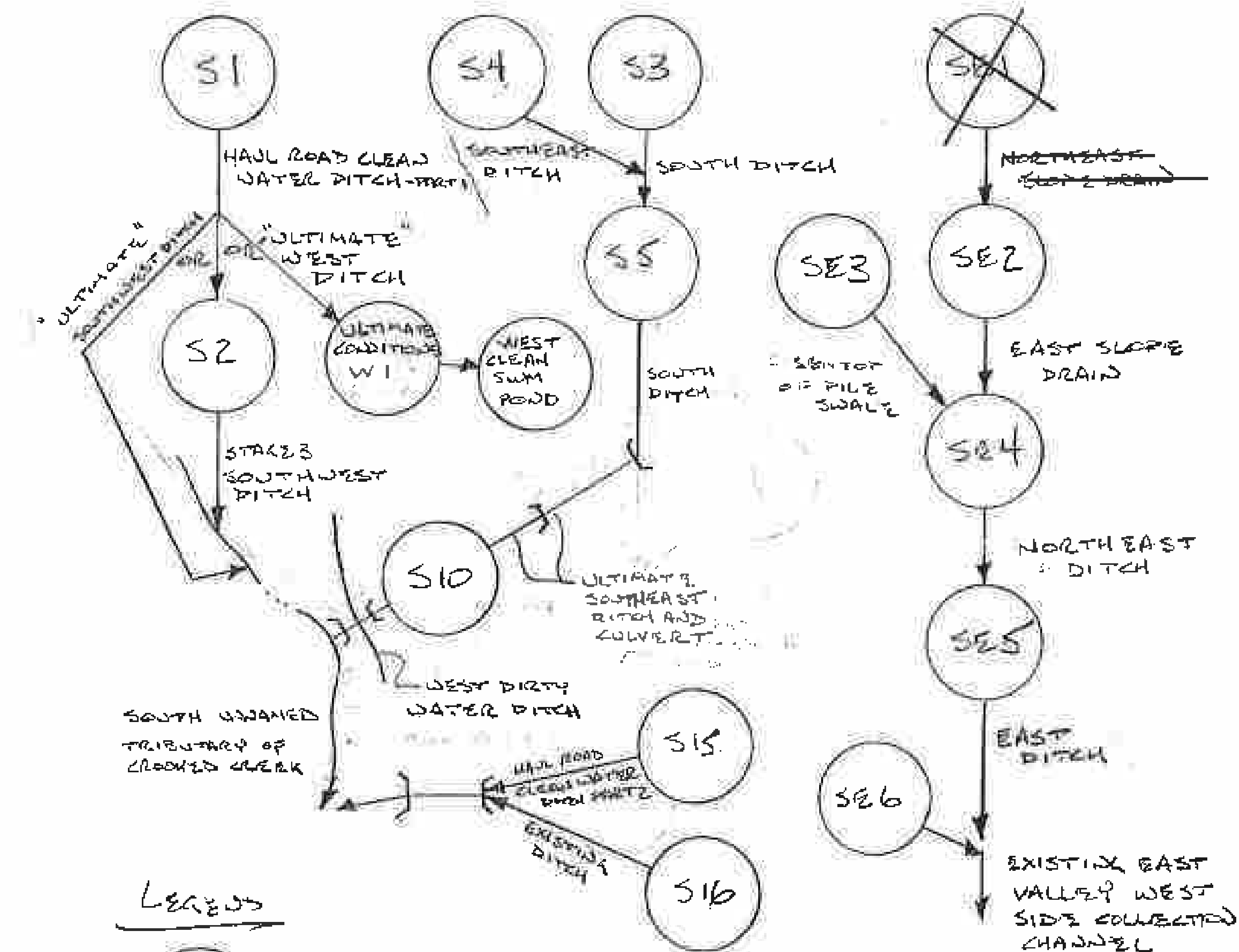
STAGE 3 DRAINAGE

SCHEMATIC



LEGEND

S1 WATERSHED S1

→ DRAINAGE DITCH OR SLOPE DRAIN

DRAINAGE AREAS S6, S7, S8, S9, S11, S12, S13, & S14 ARE REFERENCED IN OTHER CALCS.

Keystone West Valley
Phase II Permitting

By : SER Date: 4/25/96
Chkd By: KMB Date: 5/21/96

Project No. 92-220-73-7
Sheet No. 5 of 34

Stage 3 Conditions

Area and Curve Number Summary

Watershed	Total Area (Acres)	Total Area (SQ. MILES)	Composite CN		Areas of Individual Land Covers (Acres) Revegetated Pile Top	Revegetated Pile Bench Face	Active Area or Bottom Ash Haul Road	Paved Haul Road	Ponds	Pasture Offsite
				CN =	75	78	85	98	100	80
S1	8.6	0.0134	78		0.0	8.6	0.0	0.0	0.0	0.0
S2	6.4	0.0100	78		1.1	3.6	0.0	0.0	0.0	1.7
S3	4.1	0.0064	79		0.0	3.0	0.0	0.0	0.0	1.1
S4	3.5	0.0055	79		0.0	2.3	0.0	0.0	0.0	1.2
S5	9.7	0.0152	80		0.0	0.0	0.0	0.0	0.0	9.7
S10	0.5	0.0008	80		0.0	0.0	0.0	0.0	0.0	0.5
S15	3.0	0.0047	80		0.0	0.0	0.0	0.0	0.0	3.0
S16	19.4	0.0303	80		0.0	0.0	0.0	0.0	0.0	19.4
SE1	1.1	0.0017	75		1.1	0.0	0.0	0.0	0.0	0.0
SE2	8.8	0.0138	78		0.0	8.8	0.0	0.0	0.0	0.0
SE3	4.8	0.0075	75		4.8	0.0	0.0	0.0	0.0	0.0
SE4	11.2	0.0175	78		0.7	10.5	0.0	0.0	0.0	0.0
SE5	1.3	0.0020	78		0.0	1.3	0.0	0.0	0.0	0.0
SE6	33.5	0.0523	79		0.0	29.5	4.0	0.0	0.0	0.0

Note: Drainage area SE1 will flow to the "ultimate conditions" North Top of Pile Swale. This swale has previously been designed to accept flow from an area which drainage area SE1. Drainage area SE1 will not be removed from the hydrology model for flows to the southeast since it is very small and will not significantly incre the southeast.

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SUBJECT: Penelec - Keystone West Valley
Phase II Permitting - Stage 3
BY: SER DATE: 4/25/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KMB DATE: 5/31/96 SHEET NO. 6 OF 34

Time of Concentration Worksheet - SCS Methods Reference: "Urban Hydrology for Small Watersheds", TR-55, Soil Conservation Service, June 1986
Watershed or Basin S1
Postdevelopment Conditions

SHEET FLOW	Flowpath: a-b	units
1. Surface description (table 3-1)	Dense Grass	
2. Manning's roughness coeff., n_{st} (table 3-1)	$n_{st} := 0.24$	
3. Flow length, L_{st} (total $L_{st} \le 150$ feet)	$L_{st} := 65$	feet
4. Two-year, 24-hour rainfall, P_2	$P_2 := 2.6$	inches
5. Land Slope, $S_{st} := 0.40$	$S_{st} = 0.4$	
6. Sheet Flow Time, $T_{st} := \frac{0.007 \cdot (n_{st} \cdot L_{st})^{0.8}}{P_2^{0.5} \cdot S_{st}^{0.4}}$	$T_{st} = 0.056$	hours

Flowpath a-b is on a slope above a proposed bench. The proposed bench layout is shown on sheet 7 of the Ultimate Conditions Drainage Facilities calc by SER 3/19/96.

Flowpath b-c is channel flow on a proposed bench.

SHALLOW CONCENTRATED FLOW	Flowpath: NA	
7. Surface description (paved or unpaved)	———	
8. Flow length, L_{sc}	$L_{sc} := 0$	feet
9. Watercourse Slope, $S_{sc} := 0$	$S_{sc} = 0$	
10. Average Velocity, $V_{sc} := 16.1345 \cdot S_{sc}^{0.5}$	$V_{sc} = 0$	fps
11. Shallow Conc. Flow time, $T_{sc} := \left(\frac{L_{sc}}{3600 \cdot V_{sc}}\right)$	$T_{sc} = 0$	hour

Note that the coefficient used in the formula for V_{sc} is only appropriate for unpaved shallow concentrated flow.

CHANNEL FLOW	Flowpath: b-c		Flowpath: c-d
12. Bottom width, b	$b := 0$	feet	$b_1 := 2$
13. Side slopes, z $\quad z := \frac{15 + 2.5}{2}$	$z = 8.75$		$z_1 := 2$
14. Flow depth, d	$d := 1$	feet	$d_1 := 1.5$
15. Cross sectional area, $a := (b + z \cdot d) \cdot d$	$a = 8.75$	ft^2	$a_1 := (b_1 + z_1 \cdot d_1) \cdot d_1$ $\quad a_1 = 7.5$
16. Wetted perimeter, $P_w := \left[b + 2 \cdot d \cdot (1 + z^2)^{0.5}\right]$	$P_w = 17.614$	feet	$P_{w1} := \left[b_1 + 2 \cdot d_1 \cdot (1 + z_1^2)^{0.5}\right]$ $\quad P_{w1} = 8.708$
17. Hydraulic radius, $r := \frac{a}{P_w}$	$r = 0.497$	feet	$r_1 := \frac{a_1}{P_{w1}}$ $\quad r_1 = 0.861$
18. Channel Length, L_{ch}	$L_{ch} := 1600$	feet	$L_{ch1} := 390$
19. Channel Slope, $S_{ch} := 0.02$	$S_{ch} = 0.02$		$S_{ch1} := \frac{1288 - 1265}{L_{ch1}}$ $\quad S_{ch1} = 0.059$
20. Channel lining	GRASS		Grouted Rock
21. Manning's roughness coeff., n	$n := 0.045$		$n_1 := 0.025$
22. Velocity, $V_{ch} := \left[\left(\frac{1.49}{n}\right) \cdot \left[r^{\left(\frac{2}{3}\right)}\right] \cdot S_{ch}^{\left(\frac{1}{2}\right)}\right]$	$V_{ch} = 2.937$	fps	$V_{ch1} := \left[\left(\frac{1.49}{n_1}\right) \cdot \left[r_1^{\left(\frac{2}{3}\right)}\right] \cdot S_{ch1}^{\left(\frac{1}{2}\right)}\right]$ $\quad V_{ch1} = 13.102$
22. Channel Flow time, $T_{ch} := \left(\frac{L_{ch}}{3600 \cdot V_{ch}}\right)$	$T_{ch} = 0.151$	hour	$T_{ch1} := \left(\frac{L_{ch1}}{3600 \cdot V_{ch1}}\right)$ $\quad T_{ch1} = 0.008$

Total Watershed Time-of-Concentration, $T_c := T_{st} + T_{sc} + T_{ch} + T_{ch1}$ $\quad T_c = 0.216$ hour

SUBJECT: Penelec - Keystone West Valley
Phase II Permitting - Stage 3
BY: SER DATE: 4/25/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KMB DATE: 5/31/96 SHEET NO. 7 OF 34

Time of Concentration Worksheet - SCS Methods Reference: "Urban Hydrology for Small Watersheds",
Watershed or Basin S2 TR-55, Soil Conservation Service, June 1986
Postdevelopment Conditions

SHEET FLOW	Flowpath: a-b	units
1. Surface description (table 3-1)	Dense Grass	
2. Manning's roughness coeff., n_{st} (table 3-1)	$n_{st} := 0.24$	
3. Flow length, L_{st} (total $L_{st} \le 150$ feet)	$L_{st} := 65$	feet
4. Two-year, 24-hour rainfall, P_2	$P_2 := 2.6$	inches
5. Land Slope, $S_{st} := 0.40$	$S_{st} = 0.4$	
6. Sheet Flow Time, $T_{st} := \dfrac{0.007 \cdot (n_{st} \cdot L_{st})^{0.8}}{P_2^{0.5} \cdot S_{st}^{0.4}}$	$T_{st} = 0.056$	hours

SHALLOW CONCENTRATED FLOW	Flowpath: NA	
7. Surface description (paved or unpaved)	______	
8. Flow length, L_{sc}	$L_{sc} := 0$	feet
9. Watercourse Slope, $S_{sc} := 0$	$S_{sc} = 0$	
10. Average Velocity, $V_{sc} := 16.1345 \cdot S_{sc}^{0.5}$	$V_{sc} = 0$	fps
11. Shallow Conc. Flow time, $T_{sc} := \left(\dfrac{L_{sc}}{3600 \cdot V_{sc}}\right)$	$T_{sc} = 0$	hour

CHANNEL FLOW	Flowpath: b-c		
12. Bottom width, b	$b := 0$	feet	The time for flowpath c-d is negligible. Assume t=0.
13. Side slopes, z $\quad z := \dfrac{15 + 2.5}{2}$	$z = 8.75$		
14. Flow depth, d	$d := 1.0$	feet	
15. Cross sectional area, $a := (b + z \cdot d) \cdot d$	$a = 8.75$	ft^2	
16. Wetted perimeter, $P_w := \left[b + 2 \cdot d \cdot \left(1 + z^2\right)^{0.5}\right]$	$P_w = 17.614$	feet	
17. Hydraulic radius, $r := \dfrac{a}{P_w}$	$r = 0.497$	feet	
18. Channel Length, L_{ch}	$L_{ch} := 420$	feet	
19. Channel Slope, $S_{ch} := 0.02$	$S_{ch} = 0.02$		
20. Channel lining	Grass		
21. Manning's roughness coeff., n	$n := 0.045$		
22. Velocity, $V_{ch} := \left[\left(\dfrac{1.49}{n}\right) \cdot \left[r^{\left(\frac{2}{3}\right)}\right] \cdot S_{ch}^{\left(\frac{1}{2}\right)}\right]$	$V_{ch} = 2.937$	fps	
22. Channel Flow time, $T_{ch} := \left(\dfrac{L_{ch}}{3600 \cdot V_{ch}}\right)$	$T_{ch} = 0.04$	hour	

Total Watershed Time-of-Concentration, $T_c := T_{st} + T_{sc} + T_{ch}$ $\quad T_c = 0.096$ hour

SUBJECT: Penelec - Keystone West Valley
Phase II Permitting - Stage 3
BY: SER DATE: 4/25/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KWB DATE: 5/31/96 SHEET NO. 8 OF 34

Time of Concentration Worksheet - SCS Methods Reference: "Urban Hydrology for Small Watersheds", TR-55, Soil Conservation Service, June 1986
Watershed or Basin S3
Postdevelopment Conditions

	Flowpath: a-b	units
SHEET FLOW		
1. Surface description (table 3-1)	Dense Grass	
2. Manning's roughness coeff., n_{st} (table 3-1)	$n_{st} := 0.24$	
3. Flow length, L_{st} (total $L_{st} \le 150$ feet)	$L_{st} := 65$	feet
4. Two-year, 24-hour rainfall, P_2	$P_2 := 2.6$	inches
5. Land Slope, $S_{st} := 0.4$	$S_{st} = 0.4$	
6. Sheet Flow Time, $T_{st} := \frac{0.007 \cdot (n_{st} \cdot L_{st})^{0.8}}{P_2^{0.5} \cdot S_{st}^{0.4}}$	$T_{st} = 0.056$	hours

	Flowpath: NA	
SHALLOW CONCENTRATED FLOW		
7. Surface description (paved or unpaved)		
8. Flow length, L_{sc}	$L_{sc} := 0$	feet
9. Watercourse Slope, $S_{sc} := 0$	$S_{sc} = 0$	
10. Average Velocity, $V_{sc} := 16.1345 \cdot S_{sc}^{0.5}$	$V_{sc} = 0$	fps
11. Shallow Conc. Flow time, $T_{sc} := \left(\frac{L_{sc}}{3600 \cdot V_{sc}}\right)$	$T_{sc} = 0$	hour

CHANNEL FLOW	Flowpath: b-c		Flowpath: c-d
12. Bottom width, b	$b := 0$	feet	$b_1 := 2$
13. Side slopes, $z := \frac{15 + 2.5}{2}$	$z = 8.75$		$z_1 = 2$
14. Flow depth, d	$d := 1$	feet	$d_1 := 1.5$
15. Cross sectional area, $a := (b + z \cdot d) \cdot d$	$a = 8.75$	ft^2	$a_1 := (b_1 + z_1 \cdot d_1) \cdot d_1$ $a_1 = 7.5$
16. Wetted perimeter, $P_w := \left[b + 2 \cdot d \cdot (1 + z^2)^{0.5}\right]$	$P_w = 17.614$	feet	$P_{w1} := \left[b_1 + 2 \cdot d_1 \cdot (1 + z_1^2)^{0.5}\right]$ $P_{w1} = 8.708$
17. Hydraulic radius, $r := \frac{a}{P_w}$	$r = 0.497$	feet	$r_1 := \frac{a_1}{(P_{w1})}$ $r_1 = 0.861$
18. Channel Length, L_{ch}	$L_{ch} := 1040$	feet	$L_{ch1} := 210$
19. Channel Slope, $S_{ch} := 0.02$	$S_{ch} = 0.02$		$S_{ch1} := \frac{1261 - 1239}{L_{ch1}}$ $S_{ch1} = 0.105$
20. Channel lining	Grass		Grouted Rock
21. Manning's roughness coeff., n	$n := 0.045$		$n_1 := 0.025$
22. Velocity, $V_{ch} := \left[\left(\frac{1.49}{n}\right)\left[r^{\left(\frac{2}{3}\right)}\right] S_{ch}^{\left(\frac{1}{2}\right)}\right]$	$V_{ch} = 2.937$	fps	$V_{ch1} := \left[\left(\frac{1.49}{n_1}\right)\left[r_1^{\left(\frac{2}{3}\right)}\right] S_{ch1}^{\left(\frac{1}{2}\right)}\right]$ $V_{ch1} = 17.462$
22. Channel Flow time, $T_{ch} := \left(\frac{L_{ch}}{3600 \cdot V_{ch}}\right)$	$T_{ch} = 0.098$	hour	$T_{ch1} := \left(\frac{L_{ch1}}{3600 \cdot V_{ch1}}\right)$ $T_{ch1} = 0.003$

Watershed or Basin S3 (Continued)
Postdevelopment Conditions

CHANNEL FLOW — Flowpath: d-e

The time for flowpath e-f is negligible. Assume t=0.

12. Bottom width, b	$b_2 := 2$	feet
13. Side slopes, $z_2 := 2$	$z_2 = 2$	
14. Flow depth, d	$d_2 := 1.5$	feet
15. Cross sectional area, $a_2 := (b_2 + z_2 \cdot d_2) \cdot d_2$	$a_2 = 7.5$	ft^2
16. Wetted perimeter, $P_{w2} := \left[b_2 + 2 \cdot d_2 \cdot \left(1 + z_2^{\,2}\right)^{0.5}\right]$	$P_{w2} = 8.708$	feet
17. Hydraulic radius, $r_2 := \dfrac{a_2}{P_{w2}}$	$r_2 = 0.861$	feet
18. Channel Length, L_{ch}	$L_{ch2} := 280$	feet
19. Channel Slope, $S_{ch2} := \dfrac{1239 - 1235}{L_{ch2}}$	$S_{ch2} = 0.014$	
20. Channel lining	Grouted Rock	
21. Manning's roughness coeff., n	$n_2 := 0.025$	
22. Velocity, $V_{ch2} := \left[\left(\dfrac{1.49}{n_2}\right) \cdot \left[r_2^{\left(\frac{2}{3}\right)}\right] \cdot S_{ch2}^{\left(\frac{1}{2}\right)}\right]$	$V_{ch2} = 6.448$	fps
22. Channel Flow time, $T_{ch2} := \left(\dfrac{L_{ch2}}{3600 \cdot V_{ch2}}\right)$	$T_{ch2} = 0.012$	hour

Total Watershed Time-of-Concentration, $T_c := T_{st} + T_{sc} + T_{ch} + T_{ch1} + T_{ch2}$ $\quad T_c = 0.17$ hour

SUBJECT: Penelec - Keystone West Valley
Phase II Permitting - Stage 3
BY: SER DATE: 4/25/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KMB DATE: 5/31/96 SHEET NO. 10 OF 34

Time of Concentration Worksheet - SCS Methods Reference: "Urban Hydrology for Small Watersheds",
Watershed or Basin S4 TR-55, Soil Conservation Service, June 1986
Postdevelopment Conditions

Flowpath a-b is on a slope above an existing bench. The existing bench layout is shown on sheet 7 of the Ultimate Conditions Drainage Facilities calc by SER 3/19/96.

Flowpath b-c is channel flow on an existing bench.

SHEET FLOW — Flowpath: a-b — units

1. Surface description (table 3-1) — Dense Grass
2. Manning's roughness coeff., n_{st} (table 3-1) — $n_{st} := 0.24$
3. Flow length, L_{st} (total $L_{st} \le 150$ feet) — $L_{st} := 30$ feet
4. Two-year, 24-hour rainfall, P_2 — $P_2 := 2.6$ inches
5. Land Slope, $S_{st} := 0.5$ — $S_{st} = 0.5$
6. Sheet Flow Time, $T_{st} := \dfrac{0.007\cdot(n_{st}\cdot L_{st})^{0.8}}{P_2^{0.5}\cdot S_{st}^{0.4}}$ — $T_{st} = 0.028$ hours

SHALLOW CONCENTRATED FLOW — Flowpath: NA

7. Surface description (paved or unpaved)
8. Flow length, L_{sc} — $L_{sc} := 0$ feet
9. Watercourse Slope, $S_{sc} := 0$ — $S_{sc} = 0$
10. Average Velocity, $V_{sc} := 16.1345\cdot S_{sc}^{0.5}$ — $V_{sc} = 0$ fps
11. Shallow Conc. Flow time, $T_{sc} := \left(\dfrac{L_{sc}}{3600\cdot V_{sc}}\right)$ — $T_{sc} = 0$ hour

CHANNEL FLOW	Flowpath: b-c		Flowpath: c-f
12. Bottom width, b	$b := 13.5$	feet	$b_1 := 2$
13. Side slopes, $z := 2$	$z = 2$		$z_1 = 2$
14. Flow depth, d	$d := 1.0$	feet	$d_1 := 1.5$
15. Cross sectional area, $a := (b + z\cdot d)\cdot d$	$a = 15.5$	ft^2	$a_1 := (b_1 - z_1\cdot d_1)\cdot d_1$ $a_1 = 7.5$
16. Wetted perimeter, $P_w := \left[b + 2\cdot d\cdot(1+z^2)^{0.5}\right]$	$P_w = 17.972$	feet	$P_{w1} := \left[b_1 + 2\cdot d_1\cdot(1+z_1^2)^{0.5}\right]$ $P_{w1} = 8.708$
17. Hydraulic radius, $r := \dfrac{a}{P_w}$	$r = 0.862$	feet	$r_1 := \dfrac{a_1}{(P_{w1})}$ $r_1 = 0.861$
18. Channel Length, L_{ch}	$L_{ch} := 820$	feet	$L_{ch1} := 450$
19. Channel Slope, $S_{ch} := 0.01$	$S_{ch} = 0.01$		$S_{ch1} := \dfrac{1264 - 1220}{L_{ch1}}$ $S_{ch1} = 0.098$
20. Channel lining	GRASS		Grouted Rock
21. Manning's roughness coeff., n	$n := 0.045$		$n_1 = 0.025$
22. Velocity, $V_{ch} := \left[\left(\dfrac{1.49}{n}\right)\cdot\left[r^{\left(\frac{2}{3}\right)}\right]\cdot S_{ch}^{\left(\frac{1}{2}\right)}\right]$	$V_{ch} = 3$	fps	$V_{ch1} := \left[\left(\dfrac{1.49}{n_1}\right)\cdot\left[r_1^{\left(\frac{2}{3}\right)}\right]\cdot S_{ch1}^{\left(\frac{1}{2}\right)}\right]$ $V_{ch1} = 16.87$
22. Channel Flow time, $T_{ch} := \left(\dfrac{L_{ch}}{3600\cdot V_{ch}}\right)$	$T_{ch} = 0.076$	hour	$T_{ch1} := \left(\dfrac{L_{ch1}}{3600\cdot V_{ch1}}\right)$ $T_{ch1} = 0.007$

Total Watershed Time-of-Concentration, $T_c := T_{st} + T_{sc} + T_{ch} + T_{ch1}$ — $T_c = 0.111$ hour

SUBJECT: Penetec - Keystone West Valley
Phase II Permitting - Stage 3
BY: SER DATE: 4/25/96 PROJ. NO.: 92-220-73-07
CHKD. BY: [handwritten initials] DATE: 5/31/96 SHEET NO. 11 OF 34

Time of Concentration Worksheet - SCS Methods Reference: "Urban Hydrology for Small Watersheds", TR-55, Soil Conservation Service, June 1986
Watershed or Basin S5
Postdevelopment Conditions

SHEET FLOW	Flowpath: a-b	units
1. Surface description (table 3-1)	Dense Grass	
2. Manning's roughness coeff., n_{st} (table 3-1)	$n_{st} := 0.24$	
3. Flow length, L_{st} (total $L_{st} \le 150$ feet)	$L_{st} := 150$	feet
4. Two-year, 24-hour rainfall, P_2	$P_2 := 2.6$	inches
5. Land Slope, $S_{st} := \frac{1265-1230}{320}$	$S_{st} = 0.109$	
6. Sheet Flow Time, $T_{st} := \frac{0.007\cdot(n_{st}\cdot L_{st})^{0.8}}{P_2^{0.5}\cdot S_{st}^{0.4}}$	$T_{st} = 0.185$	hours

SHALLOW CONCENTRATED FLOW	Flowpath: a-b		
7. Surface description (paved or unpaved)	unpaved		Flowpath b-c is steep and short . Assume t = 0
8. Flow length, L_{sc}	$L_{sc} := 170$	feet	
9. Watercourse Slope, $S_{sc} := S_{st}$	$S_{sc} = 0.109$		
10. Average Velocity, $V_{sc} := 16.1345\cdot S_{sc}^{0.5}$	$V_{sc} = 5.336$	fps	
11. Shallow Conc. Flow time, $T_{sc} := \left(\frac{L_{sc}}{3600\cdot V_{sc}}\right)$	$T_{sc} = 0.009$	hour	

CHANNEL FLOW	Flowpath: c-d		Flowpath: d-e	
12. Bottom width, b	$b := 2$	feet	$b_1 := 2$	
13. Side slopes, z $\quad z := 2$	$z = 2$		$z_1 = 2$	
14. Flow depth, d	$d := 1.5$	feet	$d_1 := 1.5$	
15. Cross sectional area, $a := (b + z\cdot d)\cdot d$	$a = 4$	ft^2	$a_1 := (b_1 + z_1\cdot d_1)\cdot d_1$	$a_1 = 7.5$
16. Wetted perimeter, $P_w := \left[b + 2\cdot d\cdot\left(1+z^2\right)^{0.5}\right]$	$P_w = 6.472$	feet	$P_{w1} := \left[b_1 + 2\cdot d_1\cdot\left(1+z_1^2\right)^{0.5}\right]$	$P_{w1} = 8.708$
17. Hydraulic radius, $r := \frac{a}{P_w}$	$r = 0.618$	feet	$r_1 := \frac{a_1}{(P_{w1})}$	$r_1 = 0.861$
18. Channel Length, L_{ch}	$L_{ch} := 200$	feet	$L_{ch1} := 880$	
19. Channel Slope, $S_{ch} := \frac{5}{350}$	$S_{ch} = 0.014$		$S_{ch1} := \frac{30}{500}$	$S_{ch1} = 0.06$
20. Channel lining	Grouted Rock		Grouted Rock	
21. Manning's roughness coeff., n	$n := 0.025$		$n_1 := 0.025$	
22. Velocity, $V_{ch} := \left[\left(\frac{1.49}{n}\right)\cdot\left[r^{\left(\frac{2}{3}\right)}\right]\cdot S_{ch}^{\left(\frac{1}{2}\right)}\right]$	$V_{ch} = 5.169$	fps	$V_{ch1} := \left[\left(\frac{1.49}{n_1}\right)\cdot\left[r_1^{\left(\frac{2}{3}\right)}\right]\cdot S_{ch1}^{\left(\frac{1}{2}\right)}\right]$	$V_{ch1} = 13.215$
22. Channel Flow time, $T_{ch} := \left(\frac{L_{ch}}{3600\cdot V_{ch}}\right)$	$T_{ch} = 0.011$	hour	$T_{ch1} := \left(\frac{L_{ch1}}{3600\cdot V_{ch1}}\right)$	$T_{ch1} = 0.018$

Total Watershed Time-of-Concentration, $T_c := T_{st} + T_{sc} + T_{ch} + T_{ch1}$ $\quad T_c = 0.223$ hour

SUBJECT: Penelec - Keystone West Valley
Phase II Permitting - Stage 3
BY: SER DATE: 4/25/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KMB DATE: 5/31/96 SHEET NO. 12 OF 34

Time of Concentration Worksheet - SCS Methods Reference: "Urban Hydrology for Small Watersheds", TR-55, Soil Conservation Service, June 1986
Watershed or Basin SE1
Postdevelopment Conditions

SHEET FLOW — Flowpath: a-b — units

1. Surface description (table 3-1) — Dense Grass
2. Manning's roughness coeff., n_{st} (table 3-1) — $n_{st} := 0.24$
3. Flow length, L_{st} (total $L_{st} \le 150$ feet) — $L_{st} := 150$ feet
4. Two-year, 24-hour rainfall, P_2 — $P_2 := 2.6$ inches
5. Land Slope, $S_{st} := \frac{1445 - 1442}{80}$ — $S_{st} = 0.037$
6. Sheet Flow Time, $T_{st} := \frac{0.007 \cdot (n_{st} \cdot L_{st})^{0.8}}{P_2^{0.5} \cdot S_{st}^{0.4}}$ — $T_{st} = 0.284$ hours

SHALLOW CONCENTRATED FLOW — Flowpath: NA

7. Surface description (paved or unpaved)
8. Flow length, L_{sc} — $L_{sc} := 0$ feet
9. Watercourse Slope, $S_{sc} := 0$ — $S_{sc} = 0$
10. Average Velocity, $V_{sc} := 16.1345 \cdot S_{sc}^{0.5}$ — $V_{sc} = 0$ fps
11. Shallow Conc. Flow time, $T_{sc} := \left(\frac{L_{sc}}{3600 \cdot V_{sc}}\right)$ — $T_{sc} = 0$ hour

CHANNEL FLOW — Flowpath: b-c

12. Bottom width, b — $b := 0$ feet
13. Side slopes, z — $z := \frac{15 + 2.5}{2}$ — $z = 8.75$
14. Flow depth, d — $d := 1$ feet
15. Cross sectional area, $a := (b + z \cdot d) \cdot d$ — $a = 8.75$ ft^2
16. Wetted perimeter, $P_w := \left[b + 2 \cdot d \cdot (1 + z^2)^{0.5}\right]$ — $P_w = 17.614$ feet
17. Hydraulic radius, $r := \frac{a}{P_w}$ — $r = 0.497$ feet
18. Channel Length, L_{ch} — $L_{ch} := 490$ feet
19. Channel Slope, $S_{ch} := 0.02$ — $S_{ch} = 0.02$
20. Channel lining — Grass
21. Manning's roughness coeff., n — $n := 0.045$
22. Velocity, $V_{ch} := \left[\left(\frac{1.49}{n}\right) \cdot r^{\left(\frac{2}{3}\right)} \cdot S_{ch}^{\left(\frac{1}{2}\right)}\right]$ — $V_{ch} = 2.937$ fps
22. Channel Flow time, $T_{ch} := \left(\frac{L_{ch}}{3600 \cdot V_{ch}}\right)$ — $T_{ch} = 0.046$ hour

Total Watershed Time-of-Concentration, $T_c := T_{st} + T_{sc} + T_{ch}$ — $T_c = 0.33$ hour

THIS DITCH/ SLOPE DRAIN IS NOT REQUIRED. DRAINAGE AREA WILL FLOW TO "ULTIMATE CONDITIONS" NORTH TOP OF PILE SWALE.

SUBJECT: Penelec - Keystone West Valley
Phase II Permitting - Stage 3
BY: SER DATE: 4/25/96 PROJ. NO.: 92-220-73-07
CHKD. BY: [illegible] DATE: 5/31/96 SHEET NO. 13 OF 34

Time of Concentration Worksheet - SCS Methods Reference: "Urban Hydrology for Small Watersheds", TR-55, Soil Conservation Service, June 1986
Watershed or Basin SE2
Postdevelopment Conditions

SHEET FLOW	Flowpath: a-b	units	
1. Surface description (table 3-1)	Dense Grass		
2. Manning's roughness coeff., n_{st} (table 3-1)	$n_{st} := 0.24$		
3. Flow length, L_{st} (total $L_{st} \le 150$ feet)	$L_{st} := 65$	feet	
4. Two-year, 24-hour rainfall, P_2	$P_2 := 2.6$	inches	
5. Land Slope, $S_{st} := 0.40$	$S_{st} = 0.4$		
6. Sheet Flow Time, $T_{st} := \dfrac{0.007 \cdot (n_{st} \cdot L_{st})^{0.8}}{P_2^{0.5} \cdot S_{st}^{0.4}}$	$T_{st} = 0.056$	hours	
SHALLOW CONCENTRATED FLOW	Flowpath: NA		
7. Surface description (paved or unpaved)			
8. Flow length, L_{sc}	$L_{sc} := 0$	feet	
9. Watercourse Slope, $S_{sc} := 0$	$S_{sc} = 0$		
10. Average Velocity, $V_{sc} := 16.1345 \cdot S_{sc}^{0.5}$	$V_{sc} = 0$	fps	
11. Shallow Conc. Flow time, $T_{sc} := \left(\dfrac{L_{sc}}{3600 \cdot V_{sc}}\right)$	$T_{sc} = 0$	hour	
CHANNEL FLOW	Flowpath: b-c		
12. Bottom width, b	$b := 0$	feet	
13. Side slopes, z $\quad z := \dfrac{15 + 2.5}{2}$	$z = 8.75$		
14. Flow depth, d	$d := 1$	feet	
15. Cross sectional area, $a := (b + z \cdot d) \cdot d$	$a = 8.75$	ft^2	
16. Wetted perimeter, $P_w := \left[b + 2 \cdot d \cdot (1 + z^2)^{0.5}\right]$	$P_w = 17.614$	feet	
17. Hydraulic radius, $r := \dfrac{a}{P_w}$	$r = 0.497$	feet	
18. Channel Length, L_{ch}	$L_{ch} := 1620$	feet	
19. Channel Slope, $S_{ch} := 0.02$	$S_{ch} = 0.02$		
20. Channel lining	GRASS		
21. Manning's roughness coeff., n	$n = 0.045$		
22. Velocity, $V_{ch} := \left[\left(\dfrac{1.49}{n}\right) \cdot \left[r^{\left(\frac{2}{3}\right)}\right] \cdot S_{ch}^{\left(\frac{1}{2}\right)}\right]$	$V_{ch} = 2.937$	fps	
22. Channel Flow time, $T_{ch} := \left(\dfrac{L_{ch}}{3600 \cdot V_{ch}}\right)$	$T_{ch} = 0.153$	hour	

Total Watershed Time-of-Concentration, $T_c := T_{st} + T_{sc} + T_{ch}$ $\qquad T_c = 0.21$ hour

Time of Concentration Worksheet - SCS Methods. Reference: "Urban Hydrology for Small Watersheds",
Watershed or Basin SE3 TR-55, Soil Conservation Service, June 1986
Postdevelopment Conditions

SHEET FLOW Flowpath: a-b units

1. Surface description (table 3-1) Dense Grass
2. Manning's roughness coeff., n_{st} (table 3-1) $n_{st} := 0.24$
3. Flow length, L_{st} (total $L_{st} \le 150$ feet) $L_{st} := 150$ feet
4. Two-year, 24-hour rainfall, P_2 $P_2 := 2.6$ inches
5. Land Slope, $S_{st} := \frac{5}{130}$ $S_{st} = 0.038$
6. Sheet Flow Time, $T_{st} := \frac{0.007 \cdot (n_{st} \cdot L_{st})^{0.8}}{P_2^{0.5} \cdot S_{st}^{0.4}}$ $T_{st} = 0.281$ hours

SHALLOW CONCENTRATED FLOW Flowpath: NA

7. Surface description (paved or unpaved)
8. Flow length, L_{sc} $L_{sc} := 0$ feet
9. Watercourse Slope, $S_{sc} := 0$ $S_{sc} = 0$
10. Average Velocity, $V_{sc} := 16.1345 \cdot S_{sc}^{0.5}$ $V_{sc} = 0$ fps
11. Shallow Conc. Flow time, $T_{sc} := \left(\frac{L_{sc}}{3600 \cdot V_{sc}}\right)$ $T_{sc} = 0$ hour

CHANNEL FLOW Flowpath: b-c

12. Bottom width, b $b := 2$ feet
13. Side slopes, z $z := 2$
14. Flow depth, d $d := 1.5$ feet
15. Cross sectional area, $a := (b + z \cdot d) \cdot d$ $a = 7.5$ ft^2
16. Wetted perimeter, $P_w := \left[b + 2 \cdot d \cdot (1 + z^2)^{0.5}\right]$ $P_w = 8.708$ feet
17. Hydraulic radius, $r := \frac{a}{P_w}$ $r = 0.861$ feet
18. Channel Length, L_{ch} $L_{ch} := 635$ feet
19. Channel Slope, $S_{ch} := \frac{1365 - 1344}{L_{ch}}$ $S_{ch} = 0.033$
20. Channel lining Grouted Rock
21. Manning's roughness coeff., n $n := 0.025$
22. Velocity, $V_{ch} := \left[\left(\frac{1.49}{n}\right) \cdot \left[r^{\left(\frac{2}{3}\right)}\right] \cdot S_{ch}^{\left(\frac{1}{2}\right)}\right]$ $V_{ch} = 9.811$ fps
22. Channel Flow time, $T_{ch} := \left(\frac{L_{ch}}{3600 \cdot V_{ch}}\right)$ $T_{ch} = 0.018$ hour

Total Watershed Time-of-Concentration, $T_c := T_{st} + T_{sc} + T_{ch}$ $T_c = 0.299$ hour

Time of Concentration Worksheet - SCS Methods Reference: "Urban Hydrology for Small Watersheds", TR-55, Soil Conservation Service, June 1986
Watershed or Basin SE4
Postdevelopment Conditions

SHEET FLOW	Flowpath: a-b	units
1. Surface description (table 3-1)	Dense Grass	
2. Manning's roughness coeff., n_{st} (table 3-1)	$n_{st} := 0.24$	
3. Flow length, L_{st} (total $L_{st} \le 150$ feet)	$L_{st} := 65$	feet
4. Two-year, 24-hour rainfall, P_2	$P_2 := 2.6$	inches
5. Land Slope, $S_{st} := 0.4$	$S_{st} = 0.4$	
6. Sheet Flow Time, $T_{st} := \frac{0.007\cdot(n_{st}\cdot L_{st})^{0.8}}{P_2^{0.5}\cdot S_{st}^{0.4}}$	$T_{st} = 0.056$	hours

SHALLOW CONCENTRATED FLOW	Flowpath: NA	
7. Surface description (paved or unpaved)		
8. Flow length, L_{sc}	$L_{sc} := 0$	feet
9. Watercourse Slope, $S_{sc} := 0$	$S_{sc} = 0$	
10. Average Velocity, $V_{sc} := 16.1345\cdot S_{sc}^{0.5}$	$V_{sc} = 0$	fps
11. Shallow Conc. Flow time, $T_{sc} := \left(\frac{L_{sc}}{3600\cdot V_{sc}}\right)$	$T_{sc} = 0$	hour

CHANNEL FLOW	Flowpath: b-c		
12. Bottom width, b	$b := 0$	feet	Flowpath c-d is a steep channel. Assume t = 0
13. Side slopes, $z := \frac{15+2.5}{2}$	$z = 8.75$		
14. Flow depth, d	$d := 1$	feet	
15. Cross sectional area, $a := (b + z\cdot d)\cdot d$	$a = 8.75$	ft^2	
16. Wetted perimeter, $P_w := \left[b + 2\cdot d\cdot(1+z^2)^{0.5}\right]$	$P_w = 17.614$	feet	
17. Hydraulic radius, $r := \frac{a}{P_w}$	$r = 0.497$	feet	
18. Channel Length, L_{ch}	$L_{ch} := 1650$	feet	
19. Channel Slope, $S_{ch} := 0.02$	$S_{ch} = 0.02$		
20. Channel lining	Grass		
21. Manning's roughness coeff., n	$n := 0.045$		
22. Velocity, $V_{ch} := \left[\left(\frac{1.49}{n}\right)\cdot\left[r^{\left(\frac{2}{3}\right)}\right]\cdot S_{ch}^{\left(\frac{1}{2}\right)}\right]$	$V_{ch} = 2.937$	fps	
22. Channel Flow time, $T_{ch} := \left(\frac{L_{ch}}{3600\cdot V_{ch}}\right)$	$T_{ch} = 0.156$	hour	

Total Watershed Time-of-Concentration, $T_c := T_{st} + T_{sc} + T_{ch}$ $T_c = 0.212$ hour

SUBJECT: Penelec - Keystone West Valley
Phase II Permitting - Stage 3
BY: SER DATE: 4/25/96 PROJ. NO.: 92-220-73-07
CHKD. BY: LWB DATE: 5/31/96 SHEET NO. 16 OF 34

Time of Concentration Worksheet - SCS Methods Reference: "Urban Hydrology for Small Watersheds", TR-55, Soil Conservation Service, June 1986
Watershed or Basin SE5
Postdevelopment Conditions

SHEET FLOW — Flowpath: a-b — units

1. Surface description (table 3-1) — Dense Grass
2. Manning's roughness coeff., n_{st} (table 3-1) — $n_{st} := 0.24$
3. Flow length, L_{st} (total $L_{st} \le 150$ feet) — $L_{st} := 30$ feet
4. Two-year, 24-hour rainfall, P_2 — $P_2 := 2.6$ inches
5. Land Slope, $S_{st} := 0.5$ — $S_{st} = 0.5$
6. Sheet Flow Time, $T_{st} := \dfrac{0.007 \cdot (n_{st} \cdot L_{st})^{0.8}}{P_2^{0.5} \cdot S_{st}^{0.4}}$ — $T_{st} = 0.028$ hours

SHALLOW CONCENTRATED FLOW — Flowpath: NA

7. Surface description (paved or unpaved)
8. Flow length, L_{sc} — $L_{sc} := 0$ feet
9. Watercourse Slope, $S_{sc} := 0$ — $S_{sc} = 0$
10. Average Velocity, $V_{sc} := 16.1345 \cdot S_{sc}^{0.5}$ — $V_{sc} = 0$ fps
11. Shallow Conc. Flow time, $T_{sc} := \left(\dfrac{L_{sc}}{3600 \cdot V_{sc}}\right)$ — $T_{sc} = 0$ hour

CHANNEL FLOW — Flowpath: b-c

12. Bottom width, b — $b := 13.5$ feet
13. Side slopes, z — $z := 2$
14. Flow depth, d — $d := 1.0$ feet
15. Cross sectional area, $a := (b + z \cdot d) \cdot d$ — $a = 15.5$ ft^2
16. Wetted perimeter, $P_w := \left[b + 2 \cdot d \cdot \left(1 + z^2\right)^{0.5}\right]$ — $P_w = 17.972$ feet
17. Hydraulic radius, $r := \dfrac{a}{P_w}$ — $r = 0.862$ feet
18. Channel Length, L_{ch} — $L_{ch} := 950$ feet
19. Channel Slope, $S_{ch} := 0.01$ — $S_{ch} = 0.01$
20. Channel lining — Grass with Enkamat
21. Manning's roughness coeff., n — $n := 0.045$
22. Velocity, $V_{ch} := \left[\left(\dfrac{1.49}{n}\right) \cdot \left[r^{\left(\frac{2}{3}\right)}\right] \cdot S_{ch}^{\left(\frac{1}{2}\right)}\right]$ — $V_{ch} = 3$ fps
22. Channel Flow time, $T_{ch} := \left(\dfrac{L_{ch}}{3600 \cdot V_{ch}}\right)$ — $T_{ch} = 0.088$ hour

Total Watershed Time-of-Concentration, $T_c := T_{st} + T_{sc} + T_{ch}$ — $T_c = 0.116$ hour

SUBJECT: Genco - Kestone West Valley
Phase II Permitting - Stage 3
BY: SER DATE: 4/30/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KMB DATE: 6/21/96 SHEET NO. 17 OF 34

Time of Concentration Worksheet - SCS Methods
Watershed - Area S15
Postdevelopment Conditions

Reference: "Urban Hydrology for Small Watersheds", TR-55, Soil Conservation Service, June 1986

SHEET FLOW	Flowpath: a-b	units
1. Surface description (table 3-1)	Grass	
2. Manning's roughness coeff., n_{st} (table 3-1)	$n_{st} := 0.24$	
3. Flow length, L_{st} (total $L_{st} \le 150$ feet)	$L_{st} := 150$	feet
4. Two-year, 24-hour rainfall, P_2	$P_2 := 2.6$	inches
5. Land Slope, $S_{st} := \dfrac{1208 - 1180}{L_{st}}$	$S_{st} = 0.187$	
6. Sheet Flow Time, $T_{st} := \dfrac{0.007 \cdot (n_{st} \cdot L_{st})^{0.8}}{P_2^{\,0.5} \cdot S_{st}^{\,0.4}}$	$T_{st} = 0.149$	hours

SHALLOW CONCENTRATED FLOW	Flowpath: a-b	
7. Surface description (paved or unpaved)	unpaved	
8. Flow length, L_{sc}	$L_{sc} = 250$	feet
9. Watercourse Slope, $S_{sc} := \dfrac{1180 - 1127}{L_{sc}}$	$S_{sc} = 0.212$	
10. Average Velocity, $V_{sc} := 16.1345 \cdot S_{sc}^{\,0.5}$	$V_{sc} = 7.429$	fps
11. Shallow Conc. Flow time, $T_{sc} := \left(\dfrac{L_{sc}}{3600 \cdot V_{sc}}\right)$	$T_{sc} = 0.0093$	hour

CHANNEL FLOW	Flowpath: b-c	
12. Bottom width, b	$b := 2$	feet
13. Side slopes, z	$z := 2$	
14. Flow depth, d	$d := 1.5$	feet
15. Cross sectional area, $a := (b + z \cdot d) \cdot d$	$a = 7.5$	ft^2
16. Wetted perimeter, $P_w := \left[b + 2 \cdot d \cdot (1 + z^2)^{0.5}\right]$	$P_w = 8.708$	feet
17. Hydraulic radius, $r := \dfrac{a}{P_w}$	$r = 0.861$	feet
18. Channel Length, L_{ch}	$L_{ch} := 380$	feet
19. Channel Slope, $S_{ch} := \dfrac{1127 - 1097}{L_{ch}}$	$S_{ch} = 0.079$	
20. Channel lining	Grouted Rock	
21. Manning's roughness coeff., n	$n := 0.025$	
22. Velocity, $V_{ch} := \left[\left(\dfrac{1.49}{n}\right) \cdot \left[r^{\left(\frac{2}{3}\right)}\right] \cdot S_{ch}^{\left(\frac{1}{2}\right)}\right]$	$V_{ch} = 15.159$	fps
22. Channel Flow time, $T_{ch} := \left(\dfrac{L_{ch}}{3600 \cdot V_{ch}}\right)$	$T_{ch} = 0.007$	hour

Total Watershed Time-of-Concentration, $T_c := T_{st} + T_{sc} + T_{ch}$ $T_c = 0.17$ hour

SUBJECT: Genco - Kestone West Valley
Phase II Permitting - Stage 3
BY: SER DATE: 4/30/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KMR DATE: 6/11/96 SHEET NO. 18 OF 34

Time of Concentration Worksheet - SCS Methods
Watershed - Area S16
Postdevelopment Conditions

Reference: "Urban Hydrology for Small Watersheds", TR-55, Soil Conservation Service, June 1986

SHEET FLOW	Flowpath: a-b	units	
1. Surface description (table 3-1)	Grass		
2. Manning's roughness coeff., n_{st} (table 3-1)	$n_{st} := 0.24$		
3. Flow length, L_{st} (total $L_{st} \le 150$ feet)	$L_{st} := 150$	feet	
4. Two-year, 24-hour rainfall, P_2	$P_2 := 2.6$	inches	
5. Land Slope, $S_{st} := \dfrac{1216 - 1210}{L_{st}}$	$S_{st} = 0.04$		
6. Sheet Flow Time, $T_{st} := \dfrac{0.007 \cdot (n_{st} \cdot L_{st})^{0.8}}{P_2^{0.5} \cdot S_{st}^{0.4}}$	$T_{st} = 0.277$	hours	
SHALLOW CONCENTRATED FLOW	Flowpath: a-b		
7. Surface description (paved or unpaved)	unpaved		
8. Flow length, L_{sc}	$L_{sc} := 690$	feet	
9. Watercourse Slope, $S_{sc} := \dfrac{1210 - 1117}{L_{sc}}$	$S_{sc} = 0.135$		
10. Average Velocity, $V_{sc} := 16.1345 \cdot S_{sc}^{0.5}$	$V_{sc} = 5.923$	fps	
11. Shallow Conc. Flow time, $T_{sc} := \left(\dfrac{L_{sc}}{3600 \cdot V_{sc}}\right)$	$T_{sc} = 0.0324$	hour	
CHANNEL FLOW	Flowpath: b-c		
12. Bottom width, b	$b := 2$	feet	assumed channel dimensions
13. Side slopes, z	$z := 2$		
14. Flow depth, d	$d := 1.5$	feet	
15. Cross sectional area, $a := (b + z \cdot d) \cdot d$	$a = 7.5$	ft^2	
16. Wetted perimeter, $P_w := \left[b + 2 \cdot d \cdot (1 + z^2)^{0.5}\right]$	$P_w = 8.708$	feet	
17. Hydraulic radius, $r := \dfrac{a}{P_w}$	$r = 0.861$	feet	
18. Channel Length, L_{ch}	$L_{ch} := 330$	feet	
19. Channel Slope, $S_{ch} := \dfrac{1117 - 1097}{L_{ch}}$	$S_{ch} = 0.061$		
20. Channel lining	Grouted Rock		
21. Manning's roughness coeff., n	$n := 0.025$		
22. Velocity, $V_{ch} := \left[\left(\dfrac{1.49}{n}\right) \cdot \left[r^{\left(\frac{2}{3}\right)}\right] \cdot S_{ch}^{\left(\frac{1}{2}\right)}\right]$	$V_{ch} = 13.282$	fps	
22. Channel Flow time, $T_{ch} := \left(\dfrac{L_{ch}}{3600 \cdot V_{ch}}\right)$	$T_{ch} = 0.0069$	hour	

Total Watershed Time-of-Concentration, $T_c := T_{st} + T_{sc} + T_{ch}$ $T_c = 0.32$ hour

GAI CONSULTANTS, INC.
Engineers Geologists Planners
Environmental Specialists

Notes:

1) The time-of-concentration for Stage 3 drainage area SE6 is equal to the time-of-concentration of the ultimate conditions drainage area SE4 of 0.21 hours. Reference "Ultimate Conditions - Drainage Facilities" calc by SER 3/19 /96.

2) The data for ultimate conditions drainage area W1 is from "Ultimate Conditions - Drainage Facilities" calc by SER 3/19 /96.

3) The ultimate conditions Southeast Ditch - Part 2, below the proposed haul road is designed in the "Dirty Water Ditches and Related Facilities" calc by SER 5/24/96.

SHEET 19/34
BY SER 6/20/96
✓ KMB 6/20/96

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JOB TR-20                      FULLPRINT          SUMMARY   NOPLOTS
TITLE 111   KEYSTONE WEST VALLEY - STAGE 3 DITCH DESIGN - 92-220-73-7
 6 RUNOFF 1 001        1 0.0134      78.          0.22        1            S1
 6 RUNOFF 1 001        2 0.0100      78.          0.10        1            S2
 6 ADDHYD 4 001    1 2 3                                      1            SW DIT
 6 RUNOFF 1 001        4 0.0064      79.          0.17        1            S3
 6 RUNOFF 1 001        5 0.0055      79.          0.11        1            S4-SE DIT
 6 ADDHYD 4 001    4 5 6                                      1            S DIT
 6 RUNOFF 1 001        7 0.0154      80.          0.22        1            S5
 6 ADDHYD 4 001    6 7 1                                      1            S DIT
 6 RUNOFF 1 001        4 0.0017      75.          0.33        1            SE1
 6 RUNOFF 1 001        5 0.0138      78.          0.21        1            SE2
 6 ADDHYD 4 001    4 5 6                                      1            E SD
 6 RUNOFF 1 001        7 0.0075      75.          0.30        1            SE3
 6 ADDHYD 4 001    6 7 1                                      1            NE DIT
 6 RUNOFF 1 001        2 0.0175      78.          0.21        1            SE4
 6 ADDHYD 4 001    1 2 3                                      1            NE DIT
 6 RUNOFF 1 001        4 0.0020      78.          0.12        1            SE5
 6 ADDHYD 4 001    3 4 5                                      1            E DIT
 6 RUNOFF 1 001        6 0.0523      79.          0.21        1            SE6
 6 ADDHYD 4 001    5 6 7                                      1            EVWSCC
 6 RUNOFF 1 001        1 0.0134      78.          0.22        1            S1
 6 RUNOFF 1 001        2 0.0192      78.          0.31        1            UW1
 6 ADDHYD 4 001    1 2 3                                      1            U W DIT
 6 RUNOFF 1 001        4 0.0047      80.          0.17        1            WRCWDP2
 6 RUNOFF 1 001        5 0.0303      80.          0.32        1            EX DIT
 6 ADDHYD 4      01 4 5 6                                     1            WR CULV
   ENDATA
 7 LIST
   INCREM 6             0.1
   COMPUT 7 001       01 0.            4.4          1.          2 2        25 YR
   ENDCMP 1
   ENDJOB 2
```

SHEET 20/34
BYSTR 6/20/96

SUMMARY TABLE 1 - SELECTED RESULTS OF STANDARD AND EXECUTIVE CONTROL INSTRUCTIONS IN THE ORDER PERFORMED
(A STAR(*) AFTER THE PEAK DISCHARGE TIME AND RATE (CFS) VALUES INDICATES A FLAT TOP HYDROGRAPH
A QUESTION MARK(?) INDICATES A HYDROGRAPH WITH PEAK AS LAST POINT.)

:TION/ STRUCTURE ID	STANDARD CONTROL OPERATION	DRAINAGE AREA (SQ MI)	RAIN TABLE #	ANTEC MOIST COND	MAIN TIME INCREM (HR)	PRECIPITATION BEGIN (HR)	AMOUNT (IN)	DURATION (HR)	RUNOFF AMOUNT (IN)	PEAK DISCHARGE ELEVATION (FT)	TIME (HR)	RATE (CFS)	RATE (CSM)
ALTERNATE 0 STORM 0													
XSECTION 1	RUNOFF	.01	2	2	.10	.0	4.40	24.00	2.21	---	12.04	22.56	1683.9
XSECTION 1	RUNOFF	.01	2	2	.10	.0	4.40	24.00	2.18	---	11.97	21.01	2100.5
XSECTION 1	ADDHYD	.02	2	2	.10	.0	4.40	24.00	2.20	---	11.99	42.30	1807.7
XSECTION 1	RUNOFF	.01	2	2	.10	.0	4.40	24.00	2.30	---	12.00	12.08	1888.2
XSECTION 1	RUNOFF	.01	2	2	.10	.0	4.40	24.00	2.27	---	11.97	11.70	2126.6
XSECTION 1	ADDHYD	.01	2	2	.10	.0	4.40	24.00	2.29	---	11.99	23.62	1984.7
XSECTION 1	RUNOFF	.02	2	2	.10	.0	4.40	24.00	2.38	---	12.04	27.86	1809.3
XSECTION 1	ADDHYD	.03	2	2	.10	.0	4.40	24.00	2.34	---	12.01	50.73	1858.3
XSECTION 1	RUNOFF	.00	2	2	.10	.0	4.40	24.00	1.97	---	12.10	2.23	1313.8
XSECTION 1	RUNOFF	.01	2	2	.10	.0	4.40	24.00	2.22	---	12.03	23.53	1705.3
XSECTION 1	ADDHYD	.02	2	2	.10	.0	4.40	24.00	2.19	---	12.04	25.50	1645.2
XSECTION 1	RUNOFF	.01	2	2	.10	.0	4.40	24.00	1.97	---	12.09	10.23	1364.2
XSECTION 1	ADDHYD	.02	2	2	.10	.0	4.40	24.00	2.12	---	12.05	35.31	1535.1
XSECTION 1	RUNOFF	.02	2	2	.10	.0	4.40	24.00	2.22	---	12.03	29.84	1705.3
XSECTION 1	ADDHYD	.04	2	2	.10	.0	4.40	24.00	2.16	---	12.04	64.82	1600.4
:CTION 1	RUNOFF	.00	2	2	.10	.0	4.40	24.00	2.20	---	11.98	4.03	2017.0
XSECTION 1	ADDHYD	.04	2	2	.10	.0	4.40	24.00	2.16	---	12.04	68.35	1608.3
XSECTION 1	RUNOFF	.05	2	2	.10	.0	4.40	24.00	2.30	---	12.03	92.49	1768.5
XSECTION 1	ADDHYD	.09	2	2	.10	.0	4.40	24.00	2.24	---	12.03	160.82	1696.4
XSECTION 1	RUNOFF	.01	2	2	.10	.0	4.40	24.00	2.21	---	12.04	22.56	1683.9
XSECTION 1	RUNOFF	.02	2	2	.10	.0	4.40	24.00	2.21	---	12.09	28.90	1505.1
XSECTION 1	ADDHYD	.03	2	2	.10	.0	4.40	24.00	2.21	---	12.07	50.76	1557.2
XSECTION 1	RUNOFF	.00	2	2	.10	.0	4.40	24.00	2.38	---	12.00	9.19	1954.9
XSECTION 1	RUNOFF	.03	2	2	.10	.0	4.40	24.00	2.37	---	12.09	48.29	1593.6
STRUCTURE 1	ADDHYD	.04	2	2	.10	.0	4.40	24.00	2.38	---	12.08	55.75	1592.8

1

SUMMARY TABLE 3 - DISCHARGE (CFS) AT XSECTIONS AND STRUCTURES FOR ALL STORMS AND ALTERNATES

XSECTION/ STRUCTURE ID	DRAINAGE AREA (SQ MI)	STORM NUMBERS.......... 0
0 STRUCTURE 1	.04	
ALTERNATE 0		55.75
0 XSECTION 1	.03	

SUBJECT KEYSTONE - WEST VALLEY
PHASE II PERMITTING

BY SER DATE 5/4/96 PROJ. NO. 92-220-73-07

CHKD. BY KMB DATE 7/26/96 SHEET NO. 21 OF 34

GAI CONSULTANTS, INC.
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STAGE 3 - DRAINAGE FACILITIES

HYDRAULICS

PURPOSE: DESIGN THE STAGE 3 DRAINAGE FACILITIES

METHODOLOGY: MATHCAD DITCH DESIGN WORKSHEET, SEE SHEETS 27 TO 31 OF CALC BY SER "ULTIMATE CONDITIONS - DRAINAGE FACILITIES", 3/19/96, 92-220-73-07

A SUMMARY OF DESIGN FLOWS MAXIMUM AND MINIMUM SLOPES, LINING, BOTTOM WIDTH AND SIDE SLOPES IS SHOWN ON SHEET 22

DESIGNS ARE SHOWN ON SHEETS 23-34

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Hydraulics

The design flow, lining, bottom width (b), side slope (z), and maximum and minimum slope for each drainage structure is summarized below.

Drainage Structure	Design Flow (cfs)	Maximum Slope	Minimum Slope	Lining	Bottom Width	Side Slopes, z
Ultimate Conditions West Ditch	51	$\frac{5}{18}=0.278$	$\frac{5}{110}=0.045$	Grouted Rock	2	2
Stage 3 Conditions						
Northeast Ditch	65	$\frac{5}{17}=0.294$	$\frac{5}{70}=0.071$	Grouted Rock	2	2
East Ditch	69	$\frac{5}{500}=0.01$	$\frac{5}{500}=0.01$	Grass	13.5	2
South Ditch	51	$\frac{5}{80}=0.063$	$\frac{5}{350}=0.014$	Grouted Rock	2	2
Southeast Ditch	12	$\frac{5}{40}=0.125$	$\frac{5}{90}=0.056$	Grass with nylon erosion control mat	2	2
Haul Road Clean Water Ditch - Part 1	23	$\frac{5}{50}=0.1$	$\frac{5}{85}=0.059$	Grouted Rock	2	2
Haul Road Clean Water Ditch - Part 2	10	$\frac{5}{50}=0.1$	$\frac{5}{100}=0.05$	Grouted Rock	2	2
Southwest Ditch	42	$\frac{5}{15}=0.333$	$\frac{5}{80}=0.063$	Grouted Rock	2	2
Southeast Top of Pile Swale	11	$\frac{20}{640}=0.031$	$\frac{20}{640}=0.031$	Grass	0	3
East Slope Drain	26	$\frac{1}{2.5}=0.4$	$\frac{5}{100}=0.05$	Concrete Revetment Uniform Section Mat	2	2
Existing East Valley West Side Collection Channel	161	$\frac{45}{255}=0.176$	$\frac{5}{160}=0.031$	Grouted Rock	3	2

SUBJECT: Keystone Station
Phase II Permitting - Ultimate Conditions
BY: SER DATE: 4/12/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KMB DATE: 6/11/96 SHEET NO. 23 OF 34

GAI CONSULTANTS, INC.
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Purpose: Ditch Design

Methodology: Manning's Equation, $Q = \left(\frac{1.49}{n}\right) \cdot a \cdot r^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$ or $V = \left(\frac{1.49}{n}\right) \cdot (r)^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$

Ultimate West Ditch under Stage 3 Conditions

Design Flow, $Q_d = 51 \cdot ft^3 \cdot sec^{-1}$ from sheet 20 of 34

Bottom Width, $b = 2 \cdot ft$

Side Slopes, $z = 2$

Channel Lining is Grouted Rock with Manning's roughness coefficient, $n = 0.025$

Channel Minimum Slope, $S_{min} := \frac{5 \cdot ft}{110 \cdot ft}$ (from Sheet 22) or $S_{min} = 0.045 \cdot \frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 1.17 \cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 5.1 \cdot ft^2$

Minimum Velocity, $V_{min} = 10 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 6.7 \cdot ft$

Freeboard, $F_b = 0.8 \cdot ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990

Total depth, $D = 2 \cdot ft$

Top Width at Total Depth, $T_D = 10 \cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 162 \cdot ft^3 \cdot sec^{-1}$

Channel Maximum Slope, $S_{max} := \frac{5 \cdot ft}{18 \cdot ft}$ (from Sheet 22) or $S_{max} = 0.278 \cdot \frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 0.748 \cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 2.6 \cdot ft^2$

Maximum Velocity, $V_{max} = 19.5 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 5 \cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 401 \cdot ft^3 \cdot sec^{-1}$

TYPE C-2 CHANNEL

SUBJECT: Keystone Station
Phase II Permitting
BY: SER DATE: 4/12/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KMB DATE: 6/21/96 SHEET NO. 25 OF 34
Sheet no. 24 has been omitted.

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Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right) \cdot a \cdot r^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right) \cdot (r)^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$

Stage 3 Conditions Northeast Ditch

Design Flow, $Q_d = 65 \cdot ft^3 \cdot sec^{-1}$ from sheet 20 of 34

Bottom Width, $b = 2 \cdot ft$

Side Slopes, $z = 2$

Channel Lining is Grouted Rock with Manning's roughness coefficient, $n = 0.025$

Channel Minimum Slope, $S_{min} := \frac{5 \cdot ft}{70 \cdot ft}$ (from Sheet 20) or $S_{min} = 0.071 \cdot \frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 1.18 \cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 5.1 \cdot ft^2$

Minimum Velocity, $V_{min} = 12.6 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 6.7 \cdot ft$

Freeboard, $F_b = 0.8 \cdot ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990

Total depth, $D = 2 \cdot ft$

Top Width at Total Depth, $T_D = 10 \cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 203 \cdot ft^3 \cdot sec^{-1}$

Channel Maximum Slope, $S_{max} := \frac{5 \cdot ft}{17 \cdot ft}$ (from Sheet 20) or $S_{max} = 0.294 \cdot \frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 0.834 \cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 3.1 \cdot ft^2$

Maximum Velocity, $V_{max} = 21.3 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 5.3 \cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 412 \cdot ft^3 \cdot sec^{-1}$

TYPE C-2 CHANNEL

SUBJECT: Keystone Station
Phase II Permitting
BY: SER DATE: 4/12/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KMB DATE: 6/12/96 SHEET NO. 26 OF 34

CONSULTANTS, INC.
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Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right) \cdot a \cdot r^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right) \cdot (r)^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$

Stage 3 Conditions East Ditch

Design Flow, $Q_d = 69 \cdot ft^3 \cdot sec^{-1}$ from sheet 20 of 34

Bottom Width, $b = 13.5 \cdot ft$ ✓

Side Slopes, $z = 2$ ✓

Channel Lining is Grass ✓ with Manning's roughness coefficient, $n = 0.045$

Channel Minimum Slope, $S_{min} := \frac{5 \cdot ft}{500 \cdot ft}$ (from Sheet 22) or $S_{min} = 0.01 \cdot \frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 1.257 \cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 20.1 \cdot ft^2$

Minimum Velocity, $V_{min} = 3.4 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 18.5 \cdot ft$

Freeboard, $F_b = 0.7 \cdot ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990

Total depth, $D = 2 \cdot ft$ ✓

Top Width at Total Depth, $T_D = 21.5 \cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 156 \cdot ft^3 \cdot sec^{-1}$

Channel Maximum Slope, $S_{max} := \frac{5 \cdot ft}{500 \cdot ft}$ (from Sheet 22) or $S_{max} = 0.01 \cdot \frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 1.257 \cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 20.1 \cdot ft^2$

Maximum Velocity, $V_{max} = 3.4 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 18.5 \cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 156 \cdot ft^3 \cdot sec^{-1}$

TYPE A-5 CHANNEL

SUBJECT: Keystone Station
Phase II Permitting
BY: SER DATE: 4/12/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KMB DATE: 6/21/96 SHEET NO. 27 OF 34

GAI CONSULTANTS, INC.
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Environmental Specialists

Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right) \cdot a \cdot r^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right) \cdot (r)^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$

Stage 3 Conditions South Ditch

Design Flow, $Q_d = 51 \cdot ft^3 \cdot sec^{-1}$ from sheet 20 of 34

Bottom Width, $b = 2 \cdot ft$ ✓

Side Slopes, $z = 2$ ✓

Channel Lining is Grouted Rock with Manning's roughness coefficient, $n = 0.025$

Channel Minimum Slope, $S_{min} = \frac{5 \cdot ft}{350 \cdot ft}$ (from Sheet 27 or $S_{min} = 0.014 \cdot \frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 1.537 \cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 7.8 \cdot ft^2$

Minimum Velocity, $V_{min} = 6.5 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 8.1 \cdot ft$

Freeboard, $F_b = 0.5 \cdot ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990

Total depth, $D = 2 \cdot ft$ ✓

Top Width at Total Depth, $T_D = 10 \cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 91 \cdot ft^3 \cdot sec^{-1}$

Channel Maximum Slope, $S_{max} = \frac{5 \cdot ft}{80 \cdot ft}$ (from Sheet 27 or $S_{max} = 0.063 \cdot \frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 1.084 \cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 4.5 \cdot ft^2$

Maximum Velocity, $V_{max} = 11.3 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 6.3 \cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 190 \cdot ft^3 \cdot sec^{-1}$

TYPE C-2 CHANNEL

SUBJECT: Keystone Station
Phase II Permitting
BY: SER DATE: 4/12/96 PROJ. NO.: 92-220-73-07
CHKD. BY: WMB DATE 6/21/96 SHEET NO. 28 OF 34

CONSULTANTS, INC.
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Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right) \cdot a \cdot r^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right) \cdot (r)^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$

Stage 3 Conditions Southeast Ditch

Design Flow, $Q_d = 12 \cdot ft^3 \cdot sec^{-1}$ from sheet 20 of 34

Bottom Width, $b = 2 \cdot ft$

Side Slopes, $z = 2$

Channel Lining is Grass with nylon erosion control mat with Manning's roughness coefficient, $n = 0.045$

Channel Minimum Slope, $S_{min} := \frac{5 \cdot ft}{90 \cdot ft}$ (from Sheet 22) or $S_{min} = 0.056 \cdot \frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 0.728 \cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 2.5 \cdot ft^2$

Minimum Velocity, $V_{min} = 4.8 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 4.9 \cdot ft$

Freeboard, $F_b = 0.8 \cdot ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990

Total depth, $D = 1.5 \cdot ft$

Top Width at Total Depth, $T_D = 8 \cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 53 \cdot ft^3 \cdot sec^{-1}$

Channel Maximum Slope, $S_{max} := \frac{5 \cdot ft}{40 \cdot ft}$ (from Sheet 22) or $S_{max} = 0.125 \cdot \frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 0.59 \cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 1.9 \cdot ft^2$

Maximum Velocity, $V_{max} = 6.4 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 4.4 \cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 79 \cdot ft^3 \cdot sec^{-1}$

TYPE B-2 CHANNEL

SUBJECT: Keystone Station
Phase II Permitting
BY: SER DATE: 4/12/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KWB DATE: 6/21/96 SHEET NO. 29 OF 34

CONSULTANTS, INC.
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Environmental Specialists

Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right) \cdot a \cdot r^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right) \cdot (r)^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$

Stage 3 Conditions Haul Road Clean Water Ditch - Part 1

Design Flow, $Q_d = 23 \cdot ft^3 \cdot sec^{-1}$ from sheet 20 of 34

Bottom Width, $b = 2 \cdot ft$

Side Slopes, $z = 2$

Channel Lining is Grouted Rock with Manning's roughness coefficient, $n = 0.025$

Channel Minimum Slope, $S_{min} := \frac{5 \cdot ft}{85 \cdot ft}$ (from Sheet 22) or $S_{min} = 0.059 \cdot \frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 0.74 \cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 2.6 \cdot ft^2$

Minimum Velocity, $V_{min} = 8.9 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 5 \cdot ft$

Freeboard, $F_b = 0.8 \cdot ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990

Total depth, $D = 1.5 \cdot ft$

Top Width at Total Depth, $T_D = 8 \cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 98 \cdot ft^3 \cdot sec^{-1}$

Channel Maximum Slope, $S_{max} := \frac{5 \cdot ft}{50 \cdot ft}$ (from Sheet 22) or $S_{max} = 0.1 \cdot \frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 0.646 \cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 2.1 \cdot ft^2$

Maximum Velocity, $V_{max} = 10.8 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 4.6 \cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 128 \cdot ft^3 \cdot sec^{-1}$

TYPE C-1 CHANNEL

SUBJECT: Keystone Station
Phase II Permitting
BY: SER DATE: 4/12/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KWB DATE: 6/3/96 SHEET NO. 32 OF 34

GAI CONSULTANTS, INC.
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Environmental Specialists

Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right) \cdot a \cdot r^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right) \cdot (r)^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$

Stage 3 Conditions Haul Road Clean Water Ditch - Part 2

Design Flow, $Q_d = 10 \cdot ft^3 \cdot sec^{-1}$ from sheet 20 of 34

Bottom Width, $b = 2 \cdot ft$

Side Slopes, $z = 2$

Channel Lining is Grouted Rock with Manning's roughness coefficient, $n = 0.025$

Channel Minimum Slope, $S_{min} := \frac{5 \cdot ft}{100 \cdot ft}$ (from Sheet 22) or $S_{min} = 0.05 \cdot \frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 0.5 \cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 1.5 \cdot ft^2$

Minimum Velocity, $V_{min} = 6.7 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 4 \cdot ft$

Freeboard, $F_b = 0.5 \cdot ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990

Total depth, $D = 1 \cdot ft$

Top Width at Total Depth, $T_D = 6 \cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 39 \cdot ft^3 \cdot sec^{-1}$

Channel Maximum Slope, $S_{max} := \frac{5 \cdot ft}{50 \cdot ft}$ (from Sheet 22) or $S_{max} = 0.1 \cdot \frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 0.415 \cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 1.2 \cdot ft^2$

Maximum Velocity, $V_{max} = 8.5 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 3.7 \cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 55 \cdot ft^3 \cdot sec^{-1}$

TYPE C-3 CHANNEL

SUBJECT: Keystone Station
Phase II Permitting
BY: SER DATE: 4/12/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KAB DATE: 7/24/96 SHEET NO. 31 OF 34

GAI CONSULTANTS, INC.
Engineers Geologists Planners
Environmental Specialists

Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right) \cdot a \cdot r^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right) \cdot (r)^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$

Stage 3 Conditions Southwest Ditch

Design Flow, $Q_d = 42 \cdot ft^3 \cdot sec^{-1}$ from sheet 20 of 34

Bottom Width, $b = 2 \cdot ft$

Side Slopes, $z = 2$

Channel Lining is Grouted Rock with Manning's roughness coefficient, $n = 0.025$

Channel Minimum Slope, $S_{min} := \frac{5 \cdot ft}{80 \cdot ft}$ (from Sheet 22) or $S_{min} = 0.063 \cdot \frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 0.986 \cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 3.9 \cdot ft^2$

Minimum Velocity, $V_{min} = 10.7 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 5.9 \cdot ft$

Freeboard, $F_b = 1 \cdot ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990. SER added 0.5 feet.

Total depth, $D = 2 \cdot ft$

Top Width at Total Depth, $T_D = 10 \cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 190 \cdot ft^3 \cdot sec^{-1}$

Channel Maximum Slope, $S_{max} := \frac{5 \cdot ft}{15 \cdot ft}$ (from Sheet 22) or $S_{max} = 0.333 \cdot \frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 0.646 \cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 2.1 \cdot ft^2$

Maximum Velocity, $V_{max} = 19.8 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 4.6 \cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 439 \cdot ft^3 \cdot sec^{-1}$

TYPE C-2 CHANNEL

SUBJECT: Keystone Station
Phase II Permitting
BY: SER DATE: 4/12/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KRR DATE: 6/12/96 SHEET NO. 32 OF 34

GAI CONSULTANTS, INC.
Engineers Geologists Planners
Environmental Specialists

Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right) \cdot a \cdot r^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right) \cdot (r)^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$

Stage 3 Conditions Southeast Top of Pile Swale

Design Flow, $Q_d = 11 \cdot ft^3 \cdot sec^{-1}$ from sheet 20 of 34

Bottom Width, $b = 0 \cdot ft$

Side Slopes, $z = 3$

Channel Lining is Grass with Manning's roughness coefficient, $n = 0.045$

Channel Minimum Slope, $S_{min} := \frac{20 \cdot ft}{640 \cdot ft}$ (from Sheet 22) or $S_{min} = 0.031 \cdot \frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 1.011 \cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 3.1 \cdot ft^2$

Minimum Velocity, $V_{min} = 3.6 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 6.1 \cdot ft$

Freeboard, $F_b = 2 \cdot ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990. SER made total depth = 3 feet.

Total depth, $D = 3 \cdot ft$

Top Width at Total Depth, $T_D = 18 \cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 200 \cdot ft^3 \cdot sec^{-1}$

Channel Maximum Slope, $S_{max} = \frac{20 \cdot ft}{640 \cdot ft}$ (from Sheet 22) or $S_{max} = 0.031 \cdot \frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 1.011 \cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 3.1 \cdot ft^2$

Maximum Velocity, $V_{max} = 3.6 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 6.1 \cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 200 \cdot ft^3 \cdot sec^{-1}$

TYPE A-4 CHANNEL

SUBJECT: Keystone Station
Phase II Permitting
BY: SER DATE: 4/12/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KMB DATE: 7/30/96 SHEET NO. 33 OF 34

GAI CONSULTANTS, INC.
Engineers Geologists Planners
Environmental Specialists

Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right) \cdot a \cdot r^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right) \cdot (r)^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$

Stage 3 Conditions Slope Drains

Design Flow, $Q_d = 26 \cdot ft^3 \cdot sec^{-1}$ from sheet 20 of 34

Bottom Width, $b = 2 \cdot ft$

Side Slopes, $z = 2$

Channel Lining is Concrete Revetment, Uniform Section Mat with Manning's roughness coefficient, $n = 0.025$

Channel Minimum Slope, $S_{min} := \frac{5 \cdot ft}{100 \cdot ft}$ (from Sheet 22) or $S_{min} = 0.05 \frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 0.821 \cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 3 \cdot ft^2$

Minimum Velocity, $V_{min} = 8.7 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 5.3 \cdot ft$

Freeboard, $F_b = 1.2 \cdot ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990

Total depth, $D = 2 \cdot ft$

Top Width at Total Depth, $T_D = 10 \cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 170 \cdot ft^3 \cdot sec^{-1}$

Channel Maximum Slope, $S_{max} := \frac{1 \cdot ft}{2.5 \cdot ft}$ (from Sheet 22) or $S_{max} = 0.4 \frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 0.542 \cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 1.7 \cdot ft^2$

Maximum Velocity, $V_{max} = 15.6 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 4.2 \cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 380 \cdot ft^3 \cdot sec^{-1}$

SUBJECT: Keystone Station
Phase II Permitting - Ultimate Conditions
BY: SER DATE: 4/12/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KMB DATE: 6/11/96 SHEET NO. 34 OF 34

GAI CONSULTANTS, INC.
Engineers Geologists Planners
Environmental Specialists

Purpose: Ditch Design

Methodology: Manning's Equation, $Q = \left(\frac{1.49}{n}\right) \cdot a \cdot r^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right) \cdot (r)^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$

Existing East Valley West Side Collection Channel - Part 1 under Stage 3 Conditions

Design Flow, $Q_d = 161 \cdot ft^3 \cdot sec^{-1}$ from sheet 20 of 34

Bottom Width, $b = 3 \cdot ft$

Side Slopes, $z = 2$

Channel Lining is Grouted Rock with Manning's roughness coefficient, $n = 0.025$

Channel Minimum Slope, $S_{min} = \frac{5 \cdot ft}{160 \cdot ft}$ (from Sheet 22) or $S_{min} = 0.031 \cdot \frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 1.983 \cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 13.8 \cdot ft^2$

Minimum Velocity, $V_{min} = 11.7 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 10.9 \cdot ft$

Freeboard, $F_b = 0.5 \cdot ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990

Total depth, $D = 2.5 \cdot ft$

Top Width at Total Depth, $T_D = 13 \cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 265 \cdot ft^3 \cdot sec^{-1}$

Channel Maximum Slope, $S_{max} := \frac{45 \cdot ft}{255 \cdot ft}$ (from Sheet 22) or $S_{max} = 0.176 \cdot \frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 1.302 \cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 7.3 \cdot ft^2$

Maximum Velocity, $V_{max} = 22.1 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 8.2 \cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 630 \cdot ft^3 \cdot sec^{-1}$

TYPE C-6 CHANNEL

SUBJECT: Keystone Station
Phase II Permitting
BY: SER DATE: 5/24/96 PROJ. NO.: 92-220-73-07
CHKD. BY: WMB DATE: 6/14/96 SHEET NO. 1 OF 26

GAI CONSULTANTS, INC.
Engineers Geologists Planners
Environmental Specialists

DIRTY WATER DITCHES AND RELATED FACILITIES

Purpose: Design the Haul Road Dirty Water Ditch and Culverts, the West Dirty Water Ditch, North Dirty Water Ditch, South Dirty Water Ditch and South Temporary Diversion Ditch, East Temporary Diversion Ditch, Pond Diversion Ditch and Culvert, and the "Ultimate Conditions" Southeast Ditch Part 2 Ditch and Culvert.

Overview: Runoff from active surfaces of the pile, dirty water, will be carried by the dirty water ditches (collection ditches) to the West Valley Equalization Pond. The drainage patterns are shown on Worksheets 92-220-73-7-SER2 and SER4.

During initial construction of Stage 3, the dirty water will be collected by the North Dirty Water Ditch and will then discharge to the West Dirty Water Ditch which in turn will discharge to the West Valley Equalization Pond. The pile development will reach a point where the Stage 3 Haul Road Dirty Water Ditch - Part 1 will drain the dirty water. The water will be passed under the haul road near the edge of the liner in a culvert which will discharge to the North Dirty Water Ditch.

During initial construction of Stage 4, the dirty water will be collected by the South Dirty Water Ditch and will then discharge to the West Dirty Water Ditch. The pile development will reach a point where the Stage 4 Haul Road Dirty Water Ditch - Part 1 will drain the dirty water. The water will be passed under the haul road near the edge of the liner in a culvert which will discharge to the South Dirty Water Ditch.

The haul road downslope of the dirty water culverts mentioned above will be drained by the Haul Road Dirty Water Ditch - Part 2 which will discharge to the West Valley Equalization Pond.

The West Dirty Water Ditch will interrupt flows from a perennial spring during Stage 3. The water from this spring will be passed under the West Dirty Water Ditch to the stream in a culvert. Three temporary diversion ditches will divert the maximum area possible from the West Dirty Water Ditch. The North Temporary Diversion Ditch will be passed under the West Dirty Water Ditch in the same culvert which will carry flows from the perennial spring. This culvert will be referred to as the North Temporary Diversion Culvert. During Stage 4 the flow from the perennial spring will be carried by the underdrain system of the liner. The two other temporary diversion ditches, the East and Pond Diversion Ditches will meet with the "Ultimate Conditions" Southeast Ditch - Part 2 and the total flow will be passed under the West Dirty Water Ditch in the "Ultimate Conditions" Southeast Ditch - Part 2 Culvert.

See sheets 2 and 3 for drainage schematics.

Design Storm: All drainage facilities are to be designed to pass the runoff from the 25-year 24-hour storm as required in Chapter 288.151 and 288.242 of the PaDEP regulations.

SUBJECT: Keystone Station
Phase II Permitting
BY: SER DATE: 5/24/96 PROJ. NO.: 92-220-73-07
CHKD. BY: WMB DATE: 6/14/96 SHEET NO. 2 OF 26

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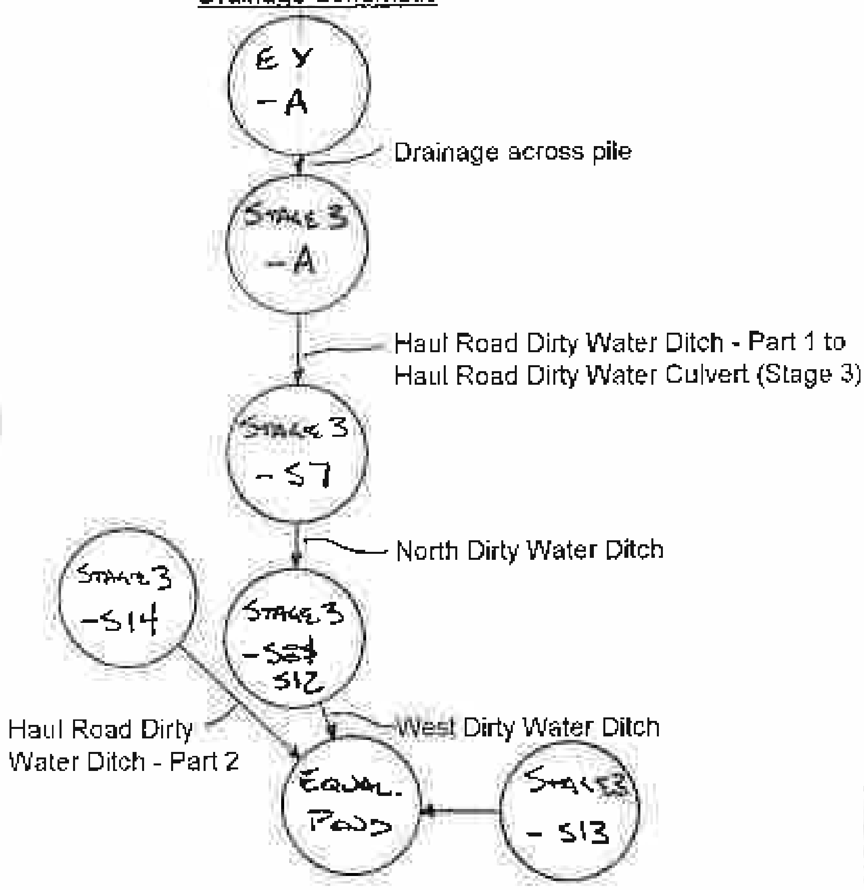
Stage 3 Worst Case

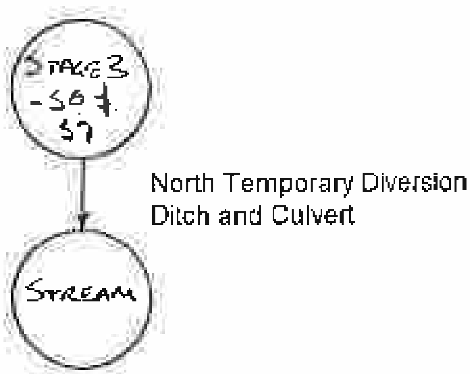
The worst case for stage 3 is assumed to be when all liner has been installed and ash has reached the level of the haul road as it comes onto the pile. This is shown on Worksheet 92-220-73-7-SER2. Note that two East Valley slope drains will be isolated at this time and will flow across the active surface of the pile to the Haul Road Dirty Water Ditch.

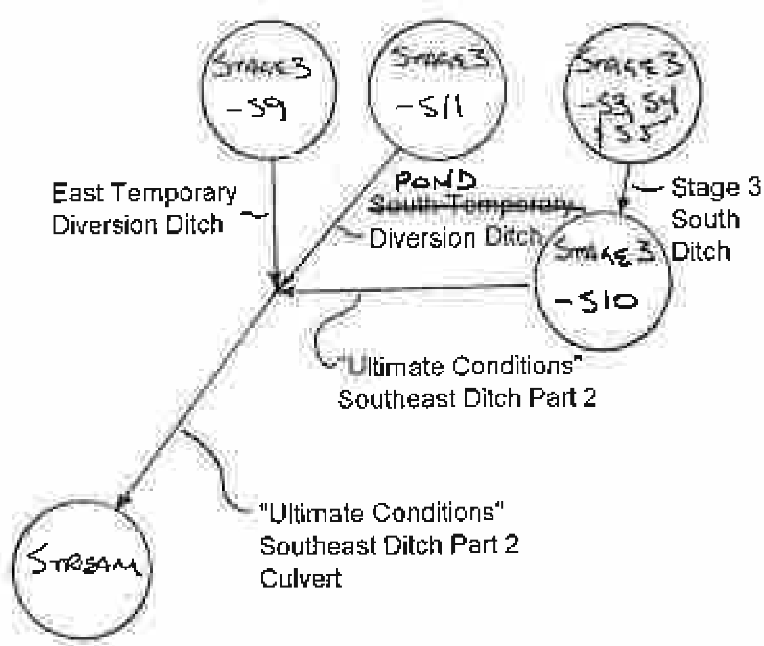
Drainage Schematic



Areas EV-A and Stage 3-A are shown on Worksheet 92-220-73-7-SER2 and Areas Stage 3-S6, -S7, -S8, -S9, -S10, -S11, -S12, -S13 and -S14 are shown on Worksheet 92-220-73-7-SER4. The area Stage 4 - S3 is shown on Worksheet 92-220-73-7-SER1.

Model the Haul Road Dirty Water Ditch as one composite area and the North and West Dirty Water Ditches as separate composite areas. Model the drainage to the "Ultimate Conditions" Southeast Ditch - Part 2 Culvert as one composite area.
The t_c calcs are shown on sheets 4 to 10, and the CN and area calcs are shown on sheet 12. TR-20 input and output summaries are shown on sheets 13 and 14. Reference "Stage 3 - Drainage Facilities" calc. by SER 4/25/96 for Stage 3 South Ditch t_c, and area and CN for areas Stage 3 - S3, -S4, and -S5.





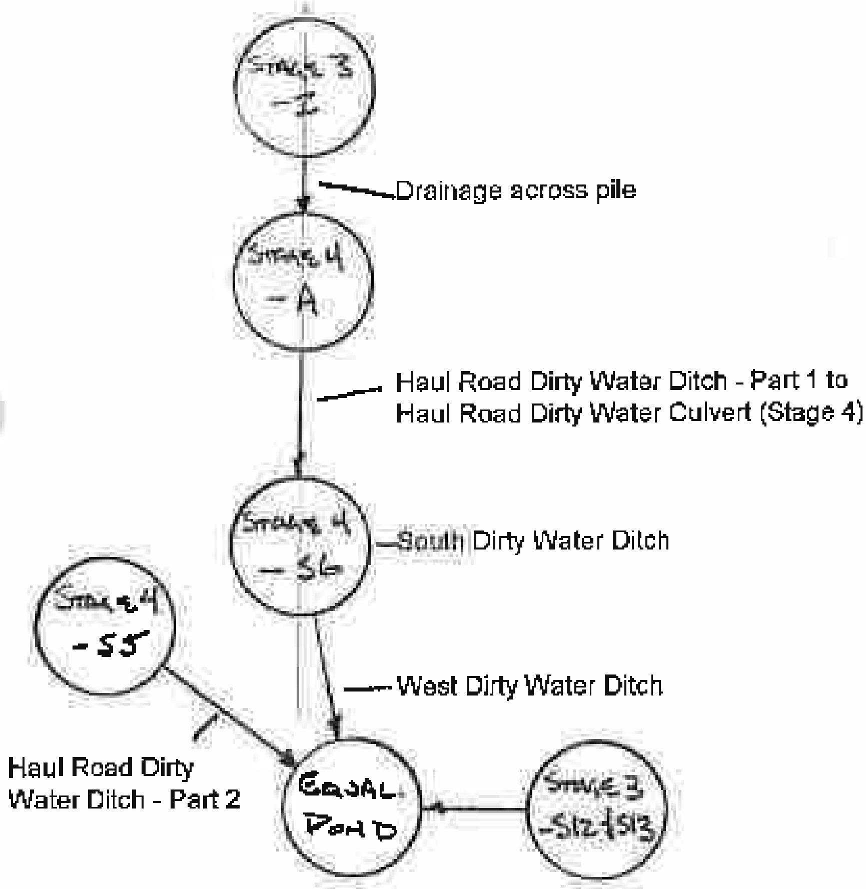
SUBJECT: Keystone Station
Phase II Permitting
BY: SER DATE: 6/24/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KMB DATE: 6/14/96 SHEET NO. 3 OF 26

CONSULTANTS, INC.
Engineers Geologists Planners
Environmental Specialists

Stage 4 Worst Case

The worst case for stage 4 is assumed to be when all liner has been installed and ash has reached the level of the haul road as it comes onto the pile. This is shown on Worksheet 92-220-73-7-SER5. Note that portions of Stage 3 will be isolated at this time and will flow across the active surface of the pile to the Haul Road Dirty Water Ditch.

Drainage Schematic



Areas Stage 4-A and Stage 3-I are shown on Worksheet 92-220-73-7-SER5 and Area Stage 4-S5 is shown on Worksheet 92-220-73-7-SER1.

Model the Haul Road Dirty Water Ditch as one composite area and the South and West Dirty Water Ditches as separate composite areas. The t_c calcs are shown on sheets 11 ~~and ___~~, and the CN and area calcs are shown on sheet 12, TR-20 input and output summaries are shown on sheets 13 and 14.

SUBJECT: Genco - Keystone West Valley
Phase II Permitting
BY: SER DATE: 5/23/96 PROJ. NO.: 92-220-73-07
CHKD. BY:KMB DATE: 6/14/96 SHEET NO. 4 OF 26
REVD. BY: JMJ DATE: 12/15/99
REV. CHKD. BY: SRR DATE: 12/16/99

Time of Concentration Worksheet - SCS Methods
Watershed - Stage 3 West Dirty Water Ditch
Postdevelopment Conditions

Reference: "Urban Hydrology for Small Watersheds", TR-55, Soil Conservation Service, June 1986

SHEET FLOW — Flowpath: a-b — units

1. Surface description — Packed Ash
2. Manning's roughness coeff., n_{st} — $n_{st} := 0.1$
3. Flow length, L_{st} (total $L_{st} < 150$ feet) — $L_{st} := 150$ feet
4. Two-year, 24-hour rainfall, P_2 — $P_2 := 2.6$ inches
5. Land Slope, $S_{st} := 0.001$ — $S_{st} = 0.001$
6. Sheet Flow Time, $T_{st} = \dfrac{0.007 \cdot (n_{st} \cdot L_{st})^{0.8}}{P_2^{0.5} \cdot S_{st}^{0.4}}$ — $T_{st} = 0.6$ hours

See Worksheet 92-220-73-7-SER2 for location of flowpaths a-b

Assume active ash area has a sheet flow n value = 0.1 which is the value for bare packed soil (PA E&S Manual p4.10) Assume active ash area slope = 0.1% at head of flowpath and on working surface. Assume sheet flow length can be maximum of 150 feet on active ash surface.

SHALLOW CONCENTRATED FLOW — Flowpath: b-c — Flowpath: c-d

7. Surface description (paved or unpaved) — unpaved — unpaved
8. Flow length, L_{sc} — $L_{sc} := 210$ feet — $L_{sc1} := 1300$
9. Watercourse Slope, $S_{sc} := \dfrac{1250 - 1216}{190}$ — $S_{sc} = 0.179$ — $S_{sc1} := 0.001$
10. Average Velocity, $V_{sc} := 16.1345 \cdot S_{sc}^{0.5}$ — $V_{sc} = 6.825$ fps — $V_{sc1} := 16.1345 \cdot S_{sc1}^{0.5}$ — $V_{sc1} = 0.51$
11. Shallow Conc. Flow time, $T_{sc} := \left(\dfrac{L_{sc}}{3600 \cdot V_{sc}}\right)$ — $T_{sc} = 0.0085$ hour — $T_{sc1} := \left(\dfrac{L_{sc1}}{3600 \cdot V_{sc1}}\right)$ — $T_{sc1} = 0.708$

Calculations continued on next sheet.

Point d on worksheet 92-220-73-7-SER2 is equivalent to point e on worksheet 92-220-73-7-SER4.

See Worksheet 92-220-73-7-SER4 for location of flowpath e-j.

Flowpath e-f is a short pipe, assume $t_c = 0$.

SUBJECT: Genco - Keystone West Valley
Phase II Permitting
BY: SER DATE: 5/23/96 PROJ. NO.: 92-220-73-07
CHKD. BY: [illegible] DATE: 6/14/96 SHEET NO. 4 OF 26

Time of Concentration Worksheet - SCS Methods
Watershed - Stage 3 West Dirty Water Ditch
Postdevelopment Conditions

Reference: "Urban Hydrology for Small Watersheds", TR-55, Soil Conservation Service, June 1986

SHEET FLOW — Flowpath: a-b — units

See Worksheet 92-220-73-7-SER2 for location of flowpaths a-b

1. Surface description (table 3-1) — Fallow
2. Manning's roughness coeff., n_{st} (table 3-1) — $n_{st} := 0.05$
3. Flow length, L_{st} (total $L_{st} \le 150$ feet) — $L_{st} := 300$ feet
4. Two-year, 24-hour rainfall, P_2 — $P_2 := 2.6$ inches
5. Land Slope, $S_{st} := 0.001$ — $S_{st} = 1 \cdot 10^{-3}$
6. Sheet Flow Time, $T_{st} := \dfrac{0.007 \cdot (n_{st} \cdot L_{st})^{0.8}}{P_2^{0.5} \cdot S_{st}^{0.4}}$ — $T_{st} = 0.6$ hours

.13 fps

Assume active ash area has a sheet flow n value = 0.05 which is the value for fallow ground.
Assume active ash area slope = 0.1% at head of flowpath and on working surface.
Assume sheet flow length can be maximum of 300 feet on active ash surface.

SHALLOW CONCENTRATED FLOW — Flowpath: b-c — Flowpath: c-d

7. Surface description (paved or unpaved) — unpaved — unpaved
8. Flow length, L_{sc} — $L_{sc} := 210$ feet — $L_{sc1} := 1300$
9. Watercourse Slope, $S_{sc} := \dfrac{1250 - 1216}{190}$ — $S_{sc} = 0.179$ — $S_{sc1} := 0.001$
10. Average Velocity, $V_{sc} = 16.1345 \cdot S_{sc}^{0.5}$ — $V_{sc} = 6.825$ fps — $V_{sc1} := 16.1345 \cdot S_{sc1}^{0.5}$ — $V_{sc1} = 0.51$
11. Shallow Conc. Flow time, $T_{sc} := \left(\dfrac{L_{sc}}{3600 \cdot V_{sc}}\right)$ — $T_{sc} = 0.0085$ hour — $T_{sc1} := \left(\dfrac{L_{sc1}}{3600 \cdot V_{sc1}}\right)$ — $T_{sc1} = 0.708$

Calculations continued on next sheet.

Point d on worksheet 92-220-73-7-SER2 is equivalent to point e on worksheet 92-220-73-7-SER4.

See Worksheet 92-220-73-7-SER4 for location of flowpath e-j.

Flowpath e-f is a short pipe, assume $t_c = 0$.

SUBJECT: Genco - Keystone West Valley
Phase II Permitting
BY: SER DATE: 5/23/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KMB DATE: 6/14/96 SHEET NO. 5 OF 26

Time of Concentration Worksheet - SCS Methods
Watershed - Stage 3 West Dirty Water Ditch
Postdevelopment Conditions (cont.)

CHANNEL FLOW	Flowpath: f-g			Flowpath: g-h	
12. Bottom width, b		$b_2 := 2$	feet	$b_3 := 2$	
13. Side slopes, z		$z_2 := 2$		$z_3 := 2$	
14. Flow depth, d		$d_2 := 1.5$	feet	$d_3 := 1.5$	
15. Cross sectional area, $a_2 := (b_2 + z_2 \cdot d_2) \cdot d_2$		$a_2 = 7.5$	ft^2	$a_3 := (b_3 + z_3 \cdot d_3) \cdot d_3$	$a_3 = 7.5$
16. Wetted perimeter, $P_{w2} := \left[b_2 + 2 \cdot d_2 \cdot \left(1 + z_2^{2}\right)^{0.5}\right]$		$P_{w2} = 8.708$	feet	$P_{w3} := \left[b_3 + 2 \cdot d_3 \cdot \left(1 + z_3^{2}\right)^{0.5}\right]$	$P_{w3} = 8.708$
17. Hydraulic radius, $r_2 := \frac{a_2}{P_{w2}}$		$r_2 = 0.861$	feet	$r_3 := \frac{a_3}{P_{w3}}$	$r_3 = 0.861$
18. Channel Length, L_{ch}		$L_{ch2} := 540$	feet	$L_{ch3} := 900$	
19. Channel Slope, $S_{ch2} := \frac{1212 - 1151}{540}$		$S_{ch2} = 0.113$		$S_{ch3} := 0.01$	
20. Channel lining	Grouted Rock			Uniform Section Mat	
21. Manning's roughness coeff., n		$n_2 := 0.025$		$n_3 := 0.015$	
22. Velocity, $V_{ch2} := \left[\left(\frac{1.49}{n_2}\right) \cdot \left[r_2^{\left(\frac{2}{3}\right)}\right] \cdot S_{ch2}^{\left(\frac{1}{2}\right)}\right]$		$V_{ch2} = 18.133$	fps	$V_{ch3} := \left[\left(\frac{1.49}{n_3}\right) \cdot \left[r_3^{\left(\frac{2}{3}\right)}\right] \cdot S_{ch3}^{\left(\frac{1}{2}\right)}\right]$	$V_{ch3} = 8.992$
22. Channel Flow time, $T_{ch2} := \left(\frac{L_{ch2}}{3600 \cdot V_{ch2}}\right)$		$T_{ch2} = 0.008$	hour	$T_{ch3} := \left(\frac{L_{ch3}}{3600 \cdot V_{ch3}}\right)$	$T_{ch3} = 0.028$

Flow beyond point h is a travel time and will be neglected.

Total Watershed Time-of-Concentration, $T_c := T_{st} + T_{sc} + T_{sc1} + T_{ch2} - T_{ch3}$

$T_c = 1.35$ hour for the West Dirty Water Ditch

The t_c to point d (=e) is the t_c for the Haul Road Dirty Water Ditch - part 1

$T_{cb} := T_{st} + T_{sc} + T_{sc1}$ $T_{cb} = 1.32$ hour

The t_c to point g is the t_c for the North Dirty Water Ditch

$T_{cg} := T_{st} + T_{sc} + T_{sc1} + T_{ch2}$ $T_{cg} = 1.33$ hour

SUBJECT: Genco - Keystone West Valley
Phase II Permitting
BY: SER DATE: 5/15/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KWB DATE: 6/14/96 SHEET NO. 6 OF 26

Time of Concentration Worksheet - SCS Methods
Watershed - Stage 3 Haul Road Dirty Water Ditch - Part 2
Postdevelopment Conditions

Reference: "Urban Hydrology for Small Watersheds", TR-55, Soil Conservation Service, June 1986

SHEET FLOW	Flowpath: a-b	units	
1. Surface description (table 3-1)	paved		Flowpath a-b is flow cross the haul road pavement.
2. Manning's roughness coeff., n_{st} (table 3-1)	$n_{st} := 0.011$		
3. Flow length, L_{st} (total $L_{st} \le 150$ feet)	$L_{st} := 52$	feet	
4. Two-year, 24-hour rainfall, P_2	$P_2 := 2.6$	inches	
5. Land Slope, $S_{st} := 0.039$	$S_{st} = 0.039$		
6. Sheet Flow Time, $T_{st} := \dfrac{0.007 \cdot (n_{st} \cdot L_{st})^{0.8}}{P_2^{0.5} \cdot S_{st}^{0.4}}$	$T_{st} = 0.01$	hours	

SHALLOW CONCENTRATED FLOW	Flowpath: NA		
7. Surface description (paved or unpaved)			
8. Flow length, L_{sc}	$L_{sc} := 0$	feet	
9. Watercourse Slope, $S_{sc} = 0$	$S_{sc} = 0$		Flowpath b-d is in the haul road dirty water ditch part 2.
10. Average Velocity, $V_{sc} = 16.1345 \cdot S_{sc}^{0.5}$	$V_{sc} = 0$	fps	
11. Shallow Conc. Flow time, $T_{sc} := \left(\dfrac{L_{sc}}{3600 \cdot V_{sc}}\right)$	$T_{sc} = 0$	hour	

CHANNEL FLOW	Flowpath:b-c		Flowpath:c-d	
12. Bottom width, b	$b := 2$	feet	$b := 2$	
13. Side slopes, z	$z := 2$		$z := 2$	
14. Flow depth, d	$d := 1.5$	feet	$d := 1.5$	
15. Cross sectional area, $a := (b + z \cdot d) \cdot d$	$a = 7.5$	ft^2	$a = 7.5$	
16. Wetted perimeter, $P_w := \left[b + 2 \cdot d \cdot (1 + z^2)^{0.5}\right]$	$P_w = 8.708$	feet	$P_w = 8.708$	
17. Hydraulic radius, $r := \dfrac{a}{P_w}$	$r = 0.861$	feet	$r = 0.861$	
18. Channel Length, L_{ch}	$L_{ch} := 810$	feet	$L_{ch1} := 1580$	
19. Channel Slope, $S_{ch} := \dfrac{1220 - 1212}{2716 - 1952}$	$S_{ch} = 0.01$		$S_{ch1} := \dfrac{1212 - 1093}{1952 - 374}$	$S_{ch1} = 0.075$
20. Channel lining	Uniform Section Mat		Uniform Section Mat	
21. Manning's roughness coeff., n	$n := 0.015$		$n := 0.015$	
22. Velocity, $V_{ch} := \left[\left(\dfrac{1.49}{n}\right) \cdot \left[r^{\left(\frac{2}{3}\right)}\right] \cdot S_{ch}^{\left(\frac{1}{2}\right)}\right]$	$V_{ch} = 9.201$ fps		$V_{ch1} := \left[\left(\dfrac{1.49}{n}\right) \cdot \left[r^{\left(\frac{2}{3}\right)}\right] \cdot S_{ch1}^{\left(\frac{1}{2}\right)}\right]$	$V_{ch1} = 24.693$
22. Channel Flow time, $T_{ch} := \left(\dfrac{L_{ch}}{3600 \cdot V_{ch}}\right)$	$T_{ch} = 0.024$	hour	$T_{ch1} := \left(\dfrac{L_{ch1}}{3600 \cdot V_{ch1}}\right)$	$T_{ch1} = 0.018$

Total Watershed Time-of-Concentration, $T_c := T_{st} + T_{sc} + T_{ch} + T_{ch1}$ $T_c = 0.05$ hour

TCS3HRDW.MCD, 6/12/96, 4:23 PM, 1

SUBJECT: Genco - Keystone West Valley
Phase II Permitting - Stage 3
BY: SER DATE: 5/15/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KMB DATE: 6/14/96 SHEET NO. 7 OF 26

Time of Concentration Worksheet - SCS Methods
Watershed - Stage 3 North Temporary Diversion - Path1
Postdevelopment Conditions

Reference: "Urban Hydrology for Small Watersheds", TR-55, Soil Conservation Service, June 1986

SHEET FLOW	Flowpath: a-b	units
1. Surface description (table 3-1)	Grass	
2. Manning's roughness coeff., n_{st} (table 3-1)	$n_{st} := 0.24$	
3. Flow length, L_{st} (total $L_{st} \le 150$ feet)	$L_{st} := 150$	feet
4. Two-year, 24-hour rainfall, P_2	$P_2 := 2.6$	inches
5. Land Slope, $S_{st} := \frac{24}{160}$	$S_{st} = 0.15$	
6. Sheet Flow Time, $T_{st} := \frac{0.007 \cdot (n_{st} \cdot L_{st})^{0.8}}{P_2^{0.5} \cdot S_{st}^{0.4}}$	$T_{st} = 0.163$	hours
SHALLOW CONCENTRATED FLOW	Flowpath: b-c	
7. Surface description (paved or unpaved)	unpaved	
8. Flow length, L_{sc}	$L_{sc} := 260$	feet
9. Watercourse Slope, $S_{sc} := \frac{40}{240}$	$S_{sc} = 0.167$	
10. Average Velocity, $V_{sc} := 16.1345 \cdot S_{sc}^{0.5}$	$V_{sc} = 6.587$	fps
11. Shallow Conc. Flow time, $T_{sc} := \left(\frac{L_{sc}}{3600 \cdot V_{sc}}\right)$	$T_{sc} = 0.011$	hour
CHANNEL FLOW	Flowpath: c-d	
12. Bottom width, b	$b := 0$	feet
13. Side slopes, z	$z := 0$	
14. Flow depth, d	$d := 0$	feet
15. Cross sectional area, $a := (b + z \cdot d) \cdot d$	$a = 0$	ft^2
16. Wetted perimeter, $P_w := \left[b + 2 \cdot d \cdot (1 + z^2)^{0.5}\right]$	$P_w = 0$	feet
17. Hydraulic radius, $r := \frac{a}{P_w}$	$r = 0$	feet
18. Channel Length, L_{ch}	$L_{ch} := 0$	feet
19. Channel Slope, $S_{ch} := 0$	$S_{ch} = 0$	
20. Channel lining	Grass	
21. Manning's roughness coeff., n	$n := 0.045$	
22. Velocity, $V_{ch} := \left[\left(\frac{1.49}{n}\right) \cdot \left[r^{\left(\frac{2}{3}\right)}\right] \cdot S_{ch}^{\left(\frac{1}{2}\right)}\right]$	$V_{ch} = 0$	fps
22. Channel Flow time, $T_{ch} := \left(\frac{L_{ch}}{3600 \cdot V_{ch}}\right)$	$T_{ch} = 0$	hour

Flowpath c-d is very short assume $t_c = 0$

Total Watershed Time-of-Concentration, $T_c := T_{st} + T_{sc} + T_{ch}$ $T_c = 0.17$ hour

TCS3NTD.MCD, 6/12/96, 4:27 PM, 1

SUBJECT: Genco - Keystone West Valley
Phase II Permitting - Stage 3
BY: SER DATE: 5/15/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KMS DATE: 6/14/96 SHEET NO. 8 OF 26

Time of Concentration Worksheet - SCS Methods
Watershed - Stage 3 North Temporary Diversion - Path2
Postdevelopment Conditions

Reference: "Urban Hydrology for Small Watersheds", TR-55, Soil Conservation Service, June 1986

SHEET FLOW	Flowpath: a'-b'	units
1. Surface description (table 3-1)	Grass	
2. Manning's roughness coeff., n_{st} (table 3-1)	$n_{st} := 0.24$	
3. Flow length, L_{st} (total $L_{st} \le 150$ feet)	$L_{st} := 150$	feet
4. Two-year, 24-hour rainfall, P_2	$P_2 := 2.6$	inches
5. Land Slope, $S_{st} := \frac{40}{150}$	$S_{st} = 0.267$	
6. Sheet Flow Time, $T_{st} := \frac{0.007 \cdot (n_{st} \cdot L_{st})^{0.8}}{P_2^{0.5} \cdot S_{st}^{0.4}}$	$T_{st} = 0.129$	hours
SHALLOW CONCENTRATED FLOW	Flowpath: NA	
7. Surface description (paved or unpaved)	unpaved	
8. Flow length, L_{sc}	$L_{sc} := 0$	feet
9. Watercourse Slope, $S_{sc} := 0$	$S_{sc} = 0$	
10. Average Velocity, $V_{sc} := 16.1345 \cdot S_{sc}^{0.5}$	$V_{sc} = 0$	fps
11. Shallow Conc. Flow time, $T_{sc} := \left(\frac{L_{sc}}{3600 \cdot V_{sc}}\right)$	$T_{sc} = 0$	hour
CHANNEL FLOW	Flowpath: b'-d	
12. Bottom width, b	$b := 2$	feet
13. Side slopes, z	$z := 2$	
14. Flow depth, d	$d := 1$	feet
15. Cross sectional area, $a := (b + z \cdot d) \cdot d$	$a = 4$	ft^2
16. Wetted perimeter, $P_w := \left[b + 2 \cdot d \cdot (1 + z^2)^{0.5}\right]$	$P_w = 6.472$	feet
17. Hydraulic radius, $r := \frac{a}{P_w}$	$r = 0.618$	feet
18. Channel Length, L_{ch}	$L_{ch} := 690$	feet
19. Channel Slope, $S_{ch} := 0.01$	$S_{ch} = 0.01$	
20. Channel lining	Grass	
21. Manning's roughness coeff., n	$n := 0.045$	
22. Velocity, $V_{ch} := \left[\left(\frac{1.49}{n}\right) \cdot \left[r^{\left(\frac{2}{3}\right)}\right] \cdot S_{ch}^{\left(\frac{1}{2}\right)}\right]$	$V_{ch} = 2.402$	fps
22. Channel Flow time, $T_{ch} := \left(\frac{L_{ch}}{3600 \cdot V_{ch}}\right)$	$T_{ch} = 0.08$	hour

Total Watershed Time-of-Concentration, $T_c := T_{st} + T_{sc} + T_{ch}$ $T_c = 0.21$ hour

TCS3NTD.MCD, 6/12/96, 4:27 PM, 2

SUBJECT: Penelec - Keystone West Valley
Phase II Permitting - Stage 3
BY: SER DATE: 06/13/96 PROJ. NO.: 92-220-73-07
CHKD. BY: [illegible] DATE: [illegible] SHEET NO. 9 OF 26

Time of Concentration Worksheet - SCS Methods Reference: "Urban Hydrology for Small Watersheds", TR-55, Soil Conservation Service, June 1986
East Temporary Diversion Ditch
Postdevelopment Conditions

SHEET FLOW — Flowpath: a-b — units

1. Surface description (table 3-1) — Dense Grass
2. Manning's roughness coeff., n_{st} (table 3-1) — $n_{st} := 0.24$
3. Flow length, L_{st} (total $L_{st} \le 150$ feet) — $L_{st} := 150$ feet
4. Two-year, 24-hour rainfall, P_2 — $P_2 := 2.6$ inches
5. Land Slope, $S_{st} := \dfrac{1155 - 1145}{105}$ — $S_{st} = 0.095$
6. Sheet Flow Time, $T_{st} := \dfrac{0.007 \cdot (n_{st} \cdot L_{st})^{0.8}}{P_2^{0.5} \cdot S_{st}^{0.4}}$ — $T_{st} = 0.195$ hours

SHALLOW CONCENTRATED FLOW — Flowpath: a-b

7. Surface description (paved or unpaved) — unpaved
8. Flow length, L_{sc} — $L_{sc} := 35$ feet
9. Watercourse Slope, $S_{sc} := S_{st}$ — $S_{sc} = 0.095$
10. Average Velocity, $V_{sc} := 16.1345 \cdot S_{sc}^{0.5}$ — $V_{sc} = 4.979$ fps
11. Shallow Conc. Flow time, $T_{sc} := \left(\dfrac{L_{sc}}{3600 \cdot V_{sc}}\right)$ — $T_{sc} = 0.002$ hour

CHANNEL FLOW — Flowpath: b-c

12. Bottom width, b — $b := 2$ feet
13. Side slopes, z $z := 2$ — $z = 2$
14. Flow depth, d — $d := 1.5$ feet
15. Cross sectional area, $a := (b + z \cdot d) \cdot d$ — $a = 7.5$ ft^2
16. Wetted perimeter, $P_w := \left[b + 2 \cdot d \cdot (1 + z^2)^{0.5}\right]$ — $P_w = 8.708$ feet
17. Hydraulic radius, $r := \dfrac{a}{P_w}$ — $r = 0.861$ feet
18. Channel Length, L_{ch} — $L_{ch} := 640$ feet
19. Channel Slope, $S_{ch} := \dfrac{1140 - 1095}{640}$ — $S_{ch} = 0.07$
20. Channel lining — GRASS
21. Manning's roughness coeff., n — $n := 0.045$
22. Velocity, $V_{ch} := \left[\left(\dfrac{1.49}{n}\right) \cdot \left[r^{\left(\frac{2}{3}\right)}\right] \cdot S_{ch}^{\left(\frac{1}{2}\right)}\right]$ — $V_{ch} = 7.948$ fps
22. Channel Flow time, $T_{ch} := \left(\dfrac{L_{ch}}{3600 \cdot V_{ch}}\right)$ — $T_{ch} = 0.022$ hour

Total Watershed Time-of-Concentration, $T_c := (T_{st} + T_{sc} + T_{ch})$ — $T_c = 0.22$ hour

TCETDD.MCD, 6/14/96, page 1

SUBJECT: Penelec - Keystone West Valley
Phase II Permitting - Stage 3
BY: SER DATE: 06/13/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KDB DATE: 6/14/96 SHEET NO. 10 OF 26

Time of Concentration Worksheet - SCS Methods Reference: "Urban Hydrology for Small Watersheds", TR-55, Soil Conservation Service, June 1986
Pond Diversion Ditch
Postdevelopment Conditions

SHEET FLOW — Flowpath: a-b — units

1. Surface description (table 3-1) — Dense Grass
2. Manning's roughness coeff., n_{st} (table 3-1) — $n_{st} := 0.24$
3. Flow length, L_{st} (total $L_{st} \le 150$ feet) — $L_{st} := 150$ — feet
4. Two-year, 24-hour rainfall, P_2 — $P_2 := 2.6$ — inches
5. Land Slope, $S_{st} := \frac{1140 - 1120}{140}$ — $S_{st} = 0.143$
6. Sheet Flow Time, $T_{st} := \frac{0.007 \cdot (n_{st} \cdot L_{st})^{0.8}}{P_2^{0.5} \cdot S_{st}^{0.4}}$ — $T_{st} = 0.166$ — hours

SHALLOW CONCENTRATED FLOW — Flowpath: NA

7. Surface description (paved or unpaved) — ——
8. Flow length, L_{sc} — $L_{sc} := 0$ — feet
9. Watercourse Slope, $S_{sc} := 0$ — $S_{sc} = 0$
10. Average Velocity, $V_{sc} := 16.1345 \cdot S_{sc}^{0.5}$ — $V_{sc} = 0$ — fps
11. Shallow Conc. Flow time, $T_{sc} := \left(\frac{L_{sc}}{3600 \cdot V_{sc}}\right)$ — $T_{sc} = 0$ — hour

CHANNEL FLOW — Flowpath: b-c

12. Bottom width, b — $b := 2$ — feet
13. Side slopes, z $z := 2$ — $z = 2$
14. Flow depth, d — $d := 1.5$ — feet
15. Cross sectional area, $a := (b + z \cdot d) \cdot d$ — $a = 7.5$ — ft^2
16. Wetted perimeter, $P_w := \left[b + 2 \cdot d \cdot (1 + z^2)^{0.5}\right]$ — $P_w = 8.708$ — feet
17. Hydraulic radius, $r := \frac{a}{P_w}$ — $r = 0.861$ — feet
18. Channel Length, L_{ch} — $L_{ch} := 550$ — feet
19. Channel Slope, $S_{ch} := \frac{1120 - 1100}{550}$ — $S_{ch} = 0.036$
20. Channel lining — GRASS
21. Manning's roughness coeff., n — $n := 0.045$
22. Velocity, $V_{ch} := \left[\left(\frac{1.49}{n}\right) \cdot \left[r^{\left(\frac{2}{3}\right)}\right] \cdot S_{ch}^{\left(\frac{1}{2}\right)}\right]$ — $V_{ch} = 5.716$ — fps
22. Channel Flow time, $T_{ch} := \left(\frac{L_{ch}}{3600 \cdot V_{ch}}\right)$ — $T_{ch} = 0.027$ — hour

Total Watershed Time-of-Concentration, $T_c := (T_{st} + T_{sc} + T_{ch})$ — $T_c = 0.193$ — hour

SUBJECT: Genco - Keystone West Valley
Phase II Permitting
BY: SER DATE: 5/23/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KMB DATE: 6/14/96 SHEET NO. 11 OF 26
REVD BY: JMJ DATE:12/15/99
REV. CHKD. BY: SSR DATE: 12/16/99

Time of Concentration Worksheet - SCS Methods
Watershed - Stage 4 West Dirty Water Ditch
Postdevelopment Conditions

Reference: "Urban Hydrology for Small Watersheds", TR-55, Soil Conservation Service, June 1986

SHEET FLOW	Flowpath: a-b	units
1. Surface description	Fallow	
2. Manning's roughness coeff., n_{st}	$n_{st} := 0.1$	
3. Flow length, L_{st} (total $L_{st} \le 150$ feet)	$L_{st} := 150$	feet
4. Two-year, 24-hour rainfall, P_2	$P_2 := 2.6$	inches
5. Land Slope, $S_{st} := 0.001$	$S_{st} = 0.001$	
6. Sheet Flow Time, $T_{st} := \frac{0.007 \cdot (n_{st} \cdot L_{st})^{0.8}}{P_2^{0.5} \cdot S_{st}^{0.4}}$	$T_{st} = 0.6$	hours

Assume active ash area has a sheet flow n value = 0.1 which is the value for bare packed soil (Pa E&S Manual p. 4.10)
Assume active ash area slope = 0.1%.
Assume sheet flow can be maximum of 150 feet on active ash surface.

Flowpath a-b is shown on worksheet 92-220-73-7-SER5 and flowpath b-e is shown on worksheet 92-220-73-7-SER1.

SHALLOW CONCENTRATED FLOW	Flowpath: a-b	
7. Surface description (paved or unpaved)	unpaved	
8. Flow length, L_{sc}	$L_{sc} := 1200$	feet
9. Watercourse Slope, $S_{sc} := 0.001$	$S_{sc} = 0.001$	
10. Average Velocity, $V_{sc} := 16.1345 \cdot S_{sc}^{0.5}$	$V_{sc} = 0.51$	fps
11. Shallow Conc. Flow time, $T_{sc} := \left(\frac{L_{sc}}{3600 \cdot V_{sc}}\right)$	$T_{sc} = 0.653$	hour

CHANNEL FLOW	Flowpath: c-d	
12. Bottom width, b	$b := 2$	feet
13. Side slopes, z	$z := 2$	
14. Flow depth, d	$d := 1.5$	feet
15. Cross sectional area, $a := (b + z \cdot d) \cdot d$	$a = 7.5$	ft^2
16. Wetted perimeter, $P_w := \left[b + 2 \cdot d \cdot (1 + z^2)^{0.5}\right]$	$P_w = 8.708$	feet
17. Hydraulic radius, $r := \frac{a}{P_w}$	$r = 0.861$	feet
18. Channel Length, L_{ch}	$L_{ch} := 640$	feet
19. Channel Slope, $S_{ch} := \frac{1150 - 1092}{L_{ch}}$	$S_{ch} = 0.091$	
20. Channel lining	Uniform Section Mat	
21. Manning's roughness coeff., n	$n := 0.015$	
22. Velocity, $V_{ch} := \left[\left(\frac{1.49}{n}\right) \cdot \left[r^{\left(\frac{2}{3}\right)}\right] \cdot S_{ch}^{\left(\frac{1}{2}\right)}\right]$	$V_{ch} = 27.069$	fps
22. Channel Flow time, $T_{ch} := \left(\frac{L_{ch}}{3600 \cdot V_{ch}}\right)$	$T_{ch} = 0.007$	hour

Flowpath b-c is a short pipe, assume $t_c = 0$.

Flowpath d-e is a short ditch, assume $t_c = 0$.

Total Watershed Time-of-Concentration, $T_c := T_{st} + T_{sc} + T_{ch}$ $T_c = 1.26$ hour

SUBJECT: Genco - Keystone West Valley
Phase II Permitting
BY: SER DATE: 5/23/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KMB DATE: 6/14/96 SHEET NO. 11 OF 26

Time of Concentration Worksheet - SCS Methods
Watershed - Stage 4 West Dirty Water Ditch
Postdevelopment Conditions

Reference: "Urban Hydrology for Small Watersheds", TR-55, Soil Conservation Service, June 1986

SHEET FLOW — Flowpath: a-b — units

1. Surface description (table 3-1) — Fallow
2. Manning's roughness coeff., n_{st} (table 3-1) — $n_{st} := 0.05$
3. Flow length, L_{st} (total $L_{st} \le 150$ feet) — $L_{st} := 300$ feet
4. Two-year, 24-hour rainfall, P_2 — $P_2 := 2.6$ inches
5. Land Slope, $S_{st} := 0.001$ — $S_{st} = 0.001$
6. Sheet Flow Time, $T_{st} := \dfrac{0.007\cdot(n_{st}\cdot L_{st})^{0.8}}{P_2^{0.5}\cdot S_{st}^{0.4}}$ — $T_{st} = 0.6$ hours

Assume active ash area has a sheet flow n value = 0.05 which is the value for fallow ground.
Assume active ash area slope = 0.1%.
Assume sheet flow can be maximum of 300 feet on active ash surface.

Flowpath a-b is shown on worksheet 92-220-73-7-SER5 and flowpath b-e is shown on worksheet 92-220-73-7-SER1.

SHALLOW CONCENTRATED FLOW — Flowpath: a-b

7. Surface description (paved or unpaved) — unpaved
8. Flow length, L_{sc} — $L_{sc} = 1200$ feet
9. Watercourse Slope, $S_{sc} := 0.001$ — $S_{sc} = 1\cdot 10^{-3}$
10. Average Velocity, $V_{sc} := 16.1345\cdot S_{sc}^{0.5}$ — $V_{sc} = 0.51$ fps
11. Shallow Conc. Flow time, $T_{sc} := \left(\dfrac{L_{sc}}{3600\cdot V_{sc}}\right)$ — $T_{sc} = 0.653$ hour

CHANNEL FLOW — Flowpath: c-d

Flowpath b-c is a short pipe, assume $t_c = 0$.

Flowpath d-e is a short ditch, assume $t_c = 0$.

12. Bottom width, b — $b := 2$ feet
13. Side slopes, z — $z := 2$
14. Flow depth, d — $d := 1.5$ feet
15. Cross sectional area, $a := (b + z\cdot d)\cdot d$ — $a = 7.5$ ft^2
16. Wetted perimeter, $P_w := \left[b + 2\cdot d\cdot(1 + z^2)^{0.5}\right]$ — $P_w = 8.708$ feet
17. Hydraulic radius, $r := \dfrac{a}{P_w}$ — $r = 0.861$ feet
18. Channel Length, L_{ch} — $L_{ch} := 640$ feet
19. Channel Slope, $S_{ch} := \dfrac{1150 - 1092}{L_{ch}}$ — $S_{ch} = 0.091$
20. Channel lining — Uniform Section Mat
21. Manning's roughness coeff., n — $n := 0.015$
22. Velocity, $V_{ch} := \left[\left(\dfrac{1.49}{n}\right)\cdot\left[r^{\left(\frac{2}{3}\right)}\right]\cdot S_{ch}^{\left(\frac{1}{2}\right)}\right]$ — $V_{ch} = 27.069$ fps
22. Channel Flow time, $T_{ch} := \left(\dfrac{L_{ch}}{3600\cdot V_{ch}}\right)$ — $T_{ch} = 0.007$ hour

Total Watershed Time-of-Concentration, $T_c := T_{st} + T_{sc} + T_{ch}$ — $T_c = 1.26$ hour

TCS4WDWA.MCD, 6/14/96, 2:19 PM, 1

Keystone West Valley
Phase II Permitting

By : SER Date: 4/30/96
Chkd By: JMB Date: 6/14/96

Project No. 92-220-73-7
Sheet No. 17 of 26

Haul Road Dirty Water Ditches and Culverts, West and North Dirty Water Ditches, and South Temporary Diversion Ditch and Culvert

Area and Curve Number Summary

Watershed	Total Area (Acres)	Total Area (SQ. MILES)	Composite CN	Areas of Individual Land Covers (Acres): Revegetated Pile Top	Revegetated Pile Bench Face	Active Area or Bottom Ash Haul Road	Paved Haul Road	Ponds	Pasture Offsite
CN =				75	78	85	98	100	80
Stage 3 Worst Case									
East Valley – A	14.4	0.0225	78	0.0	14.4	0.0	0.0	0.0	0.0
Stage 3 – A	48.1	0.0752	85	0.0	0.0	48.1	0.0	0.0	0.0
Stage 3 – S7	2.1	0.0033	80	0.0	0.0	0.0	0.0	0.0	2.1
Stage 3 – S8	1.4	0.0022	80	0.0	0.0	0.0	0.0	0.0	1.4
Stage 3 – S12	0.6	0.0009	80	0.0	0.0	0.0	0.0	0.0	0.6
Stage 3 – S13	4.4	0.0069	92	0.0	0.0	0.0	0.0	2.6	1.8
Stage 3 – S14	4.5	0.0070	98	0.0	0.0	0.0	4.5	0.0	0.0
Equal. Pond Composite	75.5	0.118	85						
West DWD Composite	66.6	0.104	83						
North DWD Composite	66.0	0.103	83						
HR DWD Part 1 Composite	62.5	0.098	83						
HR DWD Part 2 Composite	4.5	0.0070	98						
Stage 4 Worst Case									
Stage 3 – I	18.4	0.0288	79	0.0	15.6	2.8	0.0	0.0	0.0
Stage 4 – A	55.8	0.0872	85	0.0	0.0	55.8	0.0	0.0	0.0
Stage 4 – S6	0.5	0.0008	80	0.0	0.0	0.0	0.0	0.0	0.5
Stage 3 – S12	0.6	0.0009	80	0.0	0.0	0.0	0.0	0.0	0.6
Stage 3 – S13	4.4	0.0069	92	0.0	0.0	0.0	0.0	2.6	1.8
Stage 4 – S5	0.7	0.0011	98	0.0	0.0	0.0	0.7	0.0	0.0
Equal. Pond Composite	80.4	0.126	84						
West/ South DWD's Comp.	74.7	0.117	83						
HR DWD Part 1 Composite	74.2	0.116	84						
HR DWD Part 2 Composite	0.7	0.0011	98						
North Temporary Diversion Ditch									
Stage 3 – S6	4.5	0.0070	80	0	0	0	0	0	4.5
Stage 3 – S7	2.1	0.0033	80	0	0	0	0	0	2.1
Composite	6.6	0.010	80						
East Temporary Diversion Ditch									
Stage 3 – S9	3.5	0.0055	80	0	0	0	0	0	3.5
South Temporary Diversion Ditch									
Stage 3 – S11	3.0	0.0047	80	0	0	0	0	0	3
"Ultimate Conditions" Southeast Ditch – Part 2 Culvert									
Stage 3 – S3	4.1	0.0064	79	0.0	3.0	0.0	0.0	0.0	1.1
Stage 3 – S4	3.5	0.0055	79	0.0	2.3	0.0	0.0	0.0	1.2
Stage 3 – S5	9.7	0.0152	80	0.0	0.0	0.0	0.0	0.0	9.7
Stage 3 – S10	0.4	0.001	80	0.0	0.0	0.0	0.0	0.0	0.4
Stage 3 – S9	3.5	0.0055	80	0	0	0	0	0	3.5
Stage 3 – S11	3.0	0.0047	80	0	0	0	0	0	3
"Ultimate Conditions" Southeast Ditch – Part 2 Culvert Composite	24.2	0.0378	80						
"Ultimate Conditions" Southeast Ditch – Part 2 Composite	17.7	0.0277	80						

Notes: 1) Area Stage 3 – S7 is accounted for twice, once for the North Temporary Diversion Ditch and once for Stage 3 Worst Case for Eq. Pond, West DWD, and HR DWD.
2) The time-of-concentration is 0.22 hour for the Stage 3 South Ditch from "Stage 3 – Drainage Facilities" calc. by SER 4/25/96.

d:\penelec\keystone\phase2\ksph2acn.wk3

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JOB TR-20                   FULLPRINT          SUMMARY   NOPLOTS
TITLE 111 KEYSTONE EQ. POND, DIRTY WATER DITCHES AND RELATED FACS.-92-220-73-7
 6 RUNOFF 1 001       1 0.118      85.        1.35        1          EQPS3
   RUNOFF 1 001       1 0.104      83.        1.35        1          WDWDS3
   RUNOFF 1 001       1 0.103      83.        1.33        1          NDWDS3
 6 RUNOFF 1 001       1 0.098      83.        1.32        1          HRDWDP1S3
 6 RUNOFF 1 001       1 0.0070     98.        0.05        1          HRDWDP2S3
 6 RUNOFF 1 001       1 0.010      80.        0.21        1          NTDD
 6 RUNOFF 1 001       1 0.0055     80.        0.22        1          ETDD
 6 RUNOFF 1 001       1 0.0047     80.        0.19        1          PONDDD
 6 RUNOFF 1 001       1 0.028      80.        0.22        1          ULTSEDP2
 6 RUNOFF 1 001       1 0.038      80.        0.22        1          ULTSEDP2CULV
 6 RUNOFF 1 001       1 0.126      84.        1.26        1          EQPS4
 6 RUNOFF 1 001       1 0.117      83.        1.26        1          W&SDWDS4
 6 RUNOFF 1 001       1 0.116      84.        1.26        1          HRDWDP1S4
 6 RUNOFF 1    01     1 0.0011     98.        0.05        1          HRDWDP2S4
   ENDATA
 7 LIST
 7 INCREM 6             0.1
 7 COMPUT 7 001      01 0.          4.4        1.          2 2        25 YR
   ENDCMP 1
   ENDJOB 2
```

SHEET 13/26
BY SER 6/14/96
✓KRB 6/14/96

SUMMARY TABLE 1 - SELECTED RESULTS OF STANDARD AND EXECUTIVE CONTROL INSTRUCTIONS IN THE ORDER PERFORMED
(A STAR(*) AFTER THE PEAK DISCHARGE TIME AND RATE (CFS) VALUES INDICATES A FLAT TOP HYDROGRAPH
A QUESTION MARK(?) INDICATES A HYDROGRAPH WITH PEAK AS LAST POINT.)

SHEET 14 / 26

XSECTION/ STRUCTURE ID		STANDARD CONTROL OPERATION	DRAINAGE AREA (SQ MI)	RAIN TABLE #	ANTEC MOIST COND	MAIN TIME INCREM (HR)	PRECIPITATION BEGIN (HR)	AMOUNT (IN)	DURATION (HR)	RUNOFF AMOUNT (IN)	PEAK DISCHARGE ELEVATION (FT)	TIME (HR)	RATE (CFS)	RATE (CSM)
ALTERNATE 0		STORM 0												
XSECTION	1	RUNOFF	.12	2	2	.10	.0	4.40	24.00	2.82	---	12.75	90.83	769.7
XSECTION	1	RUNOFF	.10	2	2	.10	.0	4.40	24.00	2.64	---	12.75	74.77	718.9
XSECTION	1	RUNOFF	.10	2	2	.10	.0	4.40	24.00	2.64	---	12.74	74.77	725.9
XSECTION	1	RUNOFF	.10	2	2	.10	.0	4.40	24.00	2.64	---	12.74	71.51	729.7
XSECTION	1	RUNOFF	.01	2	2	.10	.0	4.40	24.00	4.12	---	11.95	24.03	3432.2
XSECTION	1	RUNOFF	.01	2	2	.10	.0	4.40	24.00	2.38	---	12.03	18.32	1831.9
XSECTION	1	RUNOFF	.01	2	2	.10	.0	4.40	24.00	2.38	---	12.04	9.95	1809.3
XSECTION	1	RUNOFF	.00	2	2	.10	.0	4.40	24.00	2.38	---	12.02	8.89	1891.5
XSECTION	1	RUNOFF	.03	2	2	.10	.0	4.40	24.00	2.38	---	12.04	50.66	1809.3
XSECTION	1	RUNOFF	.04	2	2	.10	.0	4.40	24.00	2.38	---	12.04	68.75	1809.3
XSECTION	1	RUNOFF	.13	2	2	.10	.0	4.40	24.00	2.73	---	12.69	98.29	780.1
XSECTION	1	RUNOFF	.12	2	2	.10	.0	4.40	24.00	2.64	---	12.69	88.21	753.9
XSECTION	1	RUNOFF	.12	2	2	.10	.0	4.40	24.00	2.73	---	12.69	90.49	780.1
STRUCTURE	1	RUNOFF	.00	2	2	.10	.0	4.40	24.00	4.12	---	11.95	3.78	3432.2

1

SUMMARY TABLE 3 - DISCHARGE (CFS) AT XSECTIONS AND STRUCTURES FOR ALL STORMS AND ALTERNATES

XSECTION/ STRUCTURE ID		DRAINAGE AREA (SQ MI)	STORM NUMBERS.......... 0
0 STRUCTURE	1	.00	
ALTERNATE	0		3.78
0 XSECTION	1	.12	
ALTERNATE	0		90.49

1END OF 1 JOBS IN THIS RUN

GAI CONSULTANTS INC.
Engineers Geologists Planners Environmental Specialists

Hydraulics

The design flow, lining, bottom width (b), side slope (z), and maximum and minimum slope for each drainage structure is summarized below.

Drainage Structure	Design Flow (cfs)	Maximum Slope	Minimum Slope	Lining	Bottom Width	Side Slopes, z
Haul Road Dirty Water Ditch						
Stage 3 - Part 1	72	$\frac{5}{50}=0.1$	$\frac{5}{70}=0.071$	Grouted Rock	4	3 and 2.5
Stage 4 - Part 1	91	$\frac{5}{50}=0.1$	$\frac{5}{70}=0.071$	Grouted Rock	2	3 and 2.5 Left and Right
Part 2	24	$\frac{5}{50}=0.1$	$\frac{5}{500}=0.01$	Uniform Section Mat Concrete Revetment	2	2.5
North Dirty Water Ditch	75	$\frac{5}{20}=0.25$	$\frac{5}{70}=0.071$	Grouted Rock	2	2
West Dirty Water Ditch	91	$\frac{5}{20}=0.25$	$\frac{5}{500}=0.01$	Uniform Section Mat Concrete Revetment	2	2
South Dirty Water Ditch	91	$\frac{5}{20}=0.25$	$\frac{5}{80}=0.063$	Grouted Rock	2	2
North Temporary Diversion Ditch	19	$\frac{5}{500}=0.01$	$\frac{5}{500}=0.01$	Grass	2	2
East Temporary Diversion Ditch	10	$\frac{5}{20}=0.25$	$\frac{5}{500}=0.01$	Grouted Rock	2	2
Pond Diversion Ditch						
Part 1	9	$\frac{5}{500}=0.01$	$\frac{5}{500}=0.01$	Grass	2	2
Part 2	9	$\frac{5}{50}=0.1$	$\frac{5}{500}=0.01$	Grouted Rock	2	2
"Ultimate Conditions" Southeast Ditch - Part 2	51	$\frac{5}{20}=0.25$	$\frac{5}{80}=0.063$	Grouted Rock	2	2

GAI CONSULTANTS, INC.
Engineers Geologists Planners
Environmental Specialists

Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right) \cdot a \cdot r^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right) \cdot (r)^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$

Haul Road Dirty Water Ditch - Part 1 - Stage 4

Design Flow, $Q_d = 91 \cdot ft^3 \cdot sec^{-1}$ from sheet 14 of 26

Bottom Width, $b = 2 \cdot ft$

Side Slopes left and right, $z_L = 3$ and $z_R = 2.5$

Channel Lining is Grouted Rock with Manning's roughness coefficient, $n = 0.025$

Channel Minimum Slope, $S_{min} := \frac{5 \cdot ft}{70 \cdot ft}$ (from Sheet 15) or $S_{min} = 0.071 \cdot \frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 1.271 \cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 7 \cdot ft^2$

Minimum Velocity, $V_{min} = 13 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 9 \cdot ft$

Freeboard, $F_b = 0.7 \cdot ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990.

Total depth, $D = 2 \cdot ft$

Top Width at Total Depth, $T_D = 13 \cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 254 \cdot ft^3 \cdot sec^{-1}$

Channel Maximum Slope, $S_{max} := \frac{5 \cdot ft}{50 \cdot ft}$ (from Sheet 15) or $S_{max} = 0.1 \cdot \frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 1.177 \cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 6.2 \cdot ft^2$

Maximum Velocity, $V_{max} = 14.8 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 8.5 \cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 300 \cdot ft^3 \cdot sec^{-1}$

TYPE C-4 CHANNEL

SUBJECT: Keystone Station
Phase II Permitting
BY: SER DATE: 4/12/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KMB DATE: 6/21/96 SHEET NO. 16A OF 26

GAI CONSULTANTS, INC.
Engineers Geologists Planners Environmental Specialists

Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right) \cdot a \cdot r^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right) \cdot (r)^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$

Haul Road Dirty Water Ditch - Part 1 - Stage 3

Design Flow, $Q_d = 72 \cdot ft^3 \cdot sec^{-1}$ from sheet 14 of 26

Bottom Width, $b = 4 \cdot ft$

Side Slopes left and right, $z_L = 3$ and $z_R = 2.5$

Channel Lining is Grouted Rock with Manning's roughness coefficient, $n = 0.025$

Channel Minimum Slope, $S_{min} := \frac{5 \cdot ft}{70 \cdot ft}$ (from Sheet 15) or $S_{min} = 0.071 \cdot \frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 0.926 \cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 6.1 \cdot ft^2$

Minimum Velocity, $V_{min} = 11.9 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 9.1 \cdot ft$

Freeboard, $F_b = 0.6 \cdot ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990.

Total depth, $D = 1.5 \cdot ft$

Top Width at Total Depth, $T_D = 12.3 \cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 188 \cdot ft^3 \cdot sec^{-1}$

Channel Maximum Slope, $S_{max} := \frac{5 \cdot ft}{50 \cdot ft}$ (from Sheet 15) or $S_{max} = 0.1 \cdot \frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 0.849 \cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 5.4 \cdot ft^2$

Maximum Velocity, $V_{max} = 13.4 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 8.7 \cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 223 \cdot ft^3 \cdot sec^{-1}$

TYPE C-5 CHANNEL

SUBJECT: Keystone Station
Phase II Permitting - Ultimate Conditions
BY: SER DATE: 4/12/96 PROJ. NO.: 92-220-73-07
CHKD. BY: [illegible] DATE: 6/14/96 SHEET NO. 17 OF 26

GAI CONSULTANTS, INC.
Engineers Geologists Planners
Environmental Specialists

Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right)\cdot a\cdot r^{\left(\frac{2}{3}\right)}\cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right)\cdot(r)^{\left(\frac{2}{3}\right)}\cdot s^{\left(\frac{1}{2}\right)}$

Haul Road Dirty Water Ditch - Part 2

Design Flow, $Q_d = 24\cdot ft^3\cdot sec^{-1}$ from sheet 14 of 26

Bottom Width, $b = 2\cdot ft$

Side Slopes, $z = 2.5$

Channel Lining is Uniform Section Mat Concrete Revetment with Manning's roughness coefficient, $n = 0.015$

Channel Minimum Slope, $S_{min} := \frac{5\cdot ft}{500\cdot ft}$ (from Sheet 15) or $S_{min} = 0.01\cdot\frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 0.87\cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 3.6\cdot ft^2$

Minimum Velocity, $V_{min} = 6.6\cdot ft\cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 6.3\cdot ft$

Freeboard, $F_b = 1.1\cdot ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990. SER added 0.5 feet.

Total depth, $D = 2\cdot ft$

Top Width at Total Depth, $T_D = 12\cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 148\cdot ft^3\cdot sec^{-1}$

Channel Maximum Slope, $S_{max} := \frac{5\cdot ft}{50\cdot ft}$ (from Sheet 15) or $S_{max} = 0.1\cdot\frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 0.488\cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 1.6\cdot ft^2$

Maximum Velocity, $V_{max} = 15.3\cdot ft\cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 4.4\cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 468\cdot ft^3\cdot sec^{-1}$

TYPE D-4 CHANNEL

SUBJECT: Keystone Station
Phase II Permitting - Ultimate Conditions
BY: SER DATE: 4/12/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KJB DATE: 6/14/96 SHEET NO. 18 OF 26

CONSULTANTS, INC.
Engineers Geologists Planners
Environmental Specialists

Purpose: Ditch Design

Methodology: Manning's Equation, $Q = \left(\frac{1.49}{n}\right)\cdot a\cdot r^{\left(\frac{2}{3}\right)}\cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right)\cdot(r)^{\left(\frac{2}{3}\right)}\cdot s^{\left(\frac{1}{2}\right)}$

North Dirty Water Ditch

Design Flow, $Q_d = 75\cdot ft^3\cdot sec^{-1}$ from sheet 14 of 26

Bottom Width, $b = 2\cdot ft$

Side Slopes, $z = 2$

Channel Lining is Grouted Rock with Manning's roughness coefficient, $n = 0.025$

Channel Minimum Slope, $S_{min} = \frac{5\cdot ft}{70\cdot ft}$ (from Sheet 15) or $S_{min} = 0.071\cdot\frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 1.263\cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 5.7\cdot ft^2$

Minimum Velocity, $V_{min} = 13.1\cdot ft\cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 7.1\cdot ft$

Freeboard, $F_b = 0.7\cdot ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990

Total depth, $D = 2\cdot ft$

Top Width at Total Depth, $T_D = 10\cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 203\cdot ft^3\cdot sec^{-1}$

Channel Maximum Slope, $S_{max} = \frac{5\cdot ft}{20\cdot ft}$ (from Sheet 15) or $S_{max} = 0.25\cdot\frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 0.932\cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 3.6\cdot ft^2$

Maximum Velocity, $V_{max} = 20.8\cdot ft\cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 5.7\cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 380\cdot ft^3\cdot sec^{-1}$

TYPE C-2 CHANNEL

SUBJECT: Keystone Station
Phase II Permitting - Ultimate Conditions
BY: SER DATE: 4/12/96 PROJ. NO.: 92-220-73-07
CHKD. BY: [illegible] DATE: 6/14/96 SHEET NO. 19 OF 26

GAI CONSULTANTS, INC.
Engineers Geologists Planners
Environmental Specialists

Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right) \cdot a \cdot r^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right) \cdot (r)^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$

West Dirty Water Ditch

Design Flow, $Q_d = 91 \cdot ft^3 \cdot sec^{-1}$ from sheet 14 of 26

Bottom Width, $b = 2 \cdot ft$

Side Slopes, $z = 2$

Channel Lining is Uniform Section Mat Concrete Revetment with Manning's roughness coefficient, $n = 0.015$

Channel Minimum Slope, $S_{min} := \frac{5 \cdot ft}{500 \cdot ft}$ (from Sheet 15) or $S_{min} = 0.01 \cdot \frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 1.722 \cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 9.4 \cdot ft^2$

Minimum Velocity, $V_{min} = 9.7 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 8.9 \cdot ft$

Freeboard, $F_b = 0.8 \cdot ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990

Total depth, $D = 2.5 \cdot ft$

Top Width at Total Depth, $T_D = 12 \cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 210 \cdot ft^3 \cdot sec^{-1}$

Channel Maximum Slope, $S_{max} := \frac{5 \cdot ft}{20 \cdot ft}$ (from Sheet 15) or $S_{max} = 0.25 \cdot \frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 0.795 \cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 2.9 \cdot ft^2$

Maximum Velocity, $V_{max} = 31.9 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 5.2 \cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 1 \cdot 10^3 \cdot ft^3 \cdot sec^{-1}$

TYPE D-2 CHANNEL

SUBJECT: Keystone Station
Phase II Permitting - Ultimate Conditions
BY: SER DATE: 4/12/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KWB DATE: 6/14/96 SHEET NO. 20 OF 26

GAI CONSULTANTS, INC.
Engineers Geologists Planners
Environmental Specialists

Purpose: Ditch Design

Methodology: Manning's Equation, $Q = \left(\frac{1.49}{n}\right) \cdot a \cdot r^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$ or $V = \left(\frac{1.49}{n}\right) \cdot (r)^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$

South Dirty Water Ditch

Design Flow, $Q_d = 91 \cdot ft^3 \cdot sec^{-1}$ from sheet 14 of 26

Bottom Width, $b = 2 \cdot ft$

Side Slopes, $z = 2$

Channel Lining is Grouted Rock with Manning's roughness coefficient, $n = 0.025$

Channel Minimum Slope, $S_{min} = \frac{5 \cdot ft}{80 \cdot ft}$ (from Sheet 15) or $S_{min} = 0.063 \cdot \frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 1.428 \cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 6.9 \cdot ft^2$

Minimum Velocity, $V_{min} = 13.1 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 7.7 \cdot ft$

Freeboard, $F_b = 0.6$ ft by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990

Total depth, $D = 2 \cdot ft$

Top Width at Total Depth, $T_D = 10 \cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 190 \cdot ft^3 \cdot sec^{-1}$

Channel Maximum Slope, $S_{max} = \frac{5 \cdot ft}{20 \cdot ft}$ (from Sheet 15) or $S_{max} = 0.25 \cdot \frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 1.025 \cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 4.2 \cdot ft^2$

Maximum Velocity, $V_{max} = 21.9 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 6.1 \cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 380 \cdot ft^3 \cdot sec^{-1}$

TYPE C-2 CHANNEL

SUBJECT: Keystone Station
Phase II Permitting - Ultimate Conditions
BY: SER DATE: 4/12/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KMB DATE: 6/14/96 SHEET NO. 21 OF 26

GAI CONSULTANTS, INC.
Engineers Geologists Planners
Environmental Specialists

Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right) \cdot a \cdot r^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right) \cdot (r)^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$

North Temporary Diversion Ditch

Design Flow, $Q_d = 19 \cdot ft^3 \cdot sec^{-1}$ from sheet 14 of 26

Bottom Width, $b = 2 \cdot ft$ ✓

Side Slopes, $z = 2$ ✓

Channel Lining is Grass with Manning's roughness coefficient, $n = 0.045$

Channel Minimum Slope, $S_{min} = \frac{5 \cdot ft}{500 \cdot ft}$ (from Sheet 15) or $S_{min} = 0.01 \cdot \frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 1.387 \cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 6.6 \cdot ft^2$

Minimum Velocity, $V_{min} = 2.9 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 7.5 \cdot ft$

Freeboard, $F_b = 0.6 \cdot ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990

Total depth, $D = 2 \cdot ft$ ✓

Top Width at Total Depth, $T_D = 10 \cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 42 \cdot ft^3 \cdot sec^{-1}$

Channel Maximum Slope, $S_{max} = \frac{5 \cdot ft}{500 \cdot ft}$ (from Sheet 15) or $S_{max} = 0.01 \cdot \frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 1.387 \cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 6.6 \cdot ft^2$

Maximum Velocity, $V_{max} = 2.9 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 7.5 \cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 42 \cdot ft^3 \cdot sec^{-1}$

TYPE A-6 CHANNEL

SUBJECT: Keystone Station
Phase II Permitting - Ultimate Conditions
BY: SER DATE: 4/12/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KMR DATE: 7/26/96 SHEET NO. 22 OF 26

GAI CONSULTANTS, INC.
Engineers Geologists Planners
Environmental Specialists

Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right) \cdot a \cdot r^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right) \cdot (r)^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$

East Temporary Diversion Ditch

Design Flow, $Q_d = 10 \cdot ft^3 \cdot sec^{-1}$ from sheet 14 of 26

Bottom Width, $b = 2 \cdot ft$

Side Slopes, $z = 2$

Channel Lining is Grouted Rock with Manning's roughness coefficient, $n = 0.025$

Channel Minimum Slope, $S_{min} := \frac{5 \cdot ft}{500 \cdot ft}$ (from Sheet 15) or $S_{min} = 0.01 \cdot \frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 0.761 \cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 2.7 \cdot ft^2$

Minimum Velocity, $V_{min} = 3.7 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 5 \cdot ft$

Freeboard, $F_b = 0.7 \cdot ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990.

Total depth, $D = 1.5 \cdot ft$

Top Width at Total Depth, $T_D = 8 \cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 40 \cdot ft^3 \cdot sec^{-1}$

Channel Maximum Slope, $S_{max} := \frac{5 \cdot ft}{20 \cdot ft}$ (from Sheet 15) or $S_{max} = 0.25 \cdot \frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 0.322 \cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 0.9 \cdot ft^2$

Maximum Velocity, $V_{max} = 11.7 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 3.3 \cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 202 \cdot ft^3 \cdot sec^{-1}$

TYPE C-1 CHANNEL

SUBJECT: Keystone Station
Phase II Permitting - Ultimate Conditions
BY: SER DATE: 4/12/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KMB DATE: 6/19/96 SHEET NO. 23 OF 26

GAI CONSULTANTS, INC.
Engineers Geologists Planners
Environmental Specialists

Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right) \cdot a \cdot r^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right) \cdot (r)^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$

Pond Diversion Ditch - Part 1

Design Flow, $Q_d = 9 \cdot ft^3 \cdot sec^{-1}$ from sheet 14 of 26

Bottom Width, $b = 2 \cdot ft$

Side Slopes, $z = 2$

Channel Lining is Grass with Manning's roughness coefficient, $n = 0.045$

Channel Minimum Slope, $S_{min} := \frac{5 \cdot ft}{500 \cdot ft}$ (from Sheet 15) or $S_{min} = 0.01 \cdot \frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 0.968$ ft from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 3.8 \cdot ft^2$

Minimum Velocity, $V_{min} = 2.4 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 5.9 \cdot ft$

Freeboard, $F_b = 0.5$ ft by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990

Total depth, $D = 1.5 \cdot ft$

Top Width at Total Depth, $T_D = 8 \cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 22 \cdot ft^3 \cdot sec^{-1}$

Channel Maximum Slope, $S_{max} := \frac{5 \cdot ft}{500 \cdot ft}$ (from Sheet 15) or $S_{max} = 0.01 \cdot \frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 0.968$ ft from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 3.8 \cdot ft^2$

Maximum Velocity, $V_{max} = 2.4 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 5.9 \cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 22 \cdot ft^3 \cdot sec^{-1}$

TYPE A-2 CHANNEL

SUBJECT: Keystone Station
Phase II Permitting - Ultimate Conditions
BY: SER DATE: 4/12/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KMB DATE: 6/14/96 SHEET NO. 24 OF 26

GAI CONSULTANTS INC
Engineers Geologists Planners
Environmental Specialists

Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right) \cdot a \cdot r^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right) \cdot (r)^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$

Pond Diversion Ditch - Part 2

Design Flow, $Q_d = 9 \cdot ft^3 \cdot sec^{-1}$ from sheet 14 of 26

Bottom Width, $b = 2 \cdot ft$

Side Slopes, $z = 2$

Channel Lining is Grouted Rock with Manning's roughness coefficient, $n = 0.025$

Channel Minimum Slope, $S_{min} := \frac{5 \cdot ft}{500 \cdot ft}$ (from Sheet 15) or $S_{min} = 0.01 \cdot \frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 0.721 \cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 2.5 \cdot ft^2$

Minimum Velocity, $V_{min} = 3.6 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 4.9 \cdot ft$

Freeboard, $F_b = 1.3 \cdot ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990. SER added 0.5 feet.

Total depth, $D = 2 \cdot ft$

Top Width at Total Depth, $T_D = 10 \cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 76 \cdot ft^3 \cdot sec^{-1}$

Channel Maximum Slope, $S_{max} := \frac{5 \cdot ft}{50 \cdot ft}$ (from Sheet 15) or $S_{max} = 0.1 \cdot \frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 0.391 \cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 1.1 \cdot ft^2$

Maximum Velocity, $V_{max} = 8.3 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 3.6 \cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 240 \cdot ft^3 \cdot sec^{-1}$

TYPE C-2 CHANNEL

GAI CONSULTANTS, INC.
Engineers Geologists Planners
Environmental Specialists

Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right) \cdot a \cdot r^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right) \cdot (r)^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$

Ultimate Conditions Southeast Ditch - Part 2

Design Flow, $Q_d = 51 \cdot ft^3 \cdot sec^{-1}$ from sheet 14 of 26

Bottom Width, $b = 2 \cdot ft$

Side Slopes, $z = 2$

Channel Lining is Grouted Rock with Manning's roughness coefficient, $n = 0.025$

Channel Minimum Slope, $S_{min} := \frac{5 \cdot ft}{80 \cdot ft}$ (from Sheet 15) or $S_{min} = 0.063 \cdot \frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 1.084 \cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 4.5 \cdot ft^2$

Minimum Velocity, $V_{min} = 11.3 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 6.3 \cdot ft$

Freeboard, $F_b = 0.4 \cdot ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990

Total depth, $D = 1.5 \cdot ft$

Top Width at Total Depth, $T_D = 8 \cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 101 \cdot ft^3 \cdot sec^{-1}$

Channel Maximum Slope, $S_{max} := \frac{5 \cdot ft}{20 \cdot ft}$ (from Sheet 15) or $S_{max} = 0.25 \cdot \frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 0.768 \cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 2.7 \cdot ft^2$

Maximum Velocity, $V_{max} = 18.8 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 5.1 \cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 202 \cdot ft^3 \cdot sec^{-1}$

TYPE C-1 CHANNEL

SUBJECT: Keystone Station
Phase II Permitting
BY: SER DATE: 5/24/96 PROJ. NO.: 92-220-73-07
CHKD. BY WMS DATE: 6/14/96 SHEET NO. 26 OF 26

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Notes:

1.) The Design flows into the West Valley Equilization Pond and the Ultimate Conditions Southeast Ditch - Part 2 Culvert are calculated within this calc set and will be used in other calcs.

2.) The Ultimate Conditions Southeast Ditch - Part 2 cross section and lining has changed from that calculated for Part 1 in "Ultimate Conditions - Drainage Facilities" calc. by SER 3/19/96.

SUBJECT KEYSTONE			
BY SER	DATE 6/30/96	PROJ. NO. 92-220-73-07	
CHKD. BY MRL	DATE 7/24/96	SHEET NO. 1 OF 53	

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CULVERTS

PURPOSE: DESIGN THE FOLLOWING CULVERTS

1) THE HAUL ROAD DIRTY WATER DITCH CULVERT - STAGE 3
2) THE HAUL ROAD DIRTY WATER DITCH CULVERT - STAGE 4
3) ULTIMATE CONDITIONS SOUTHEAST DITCH CULVERTS - 1 BENEATH THE HAUL ROAD AND 1 BENEATH THE WEST DIRTY WATER DITCH Outfall 016
4) STAGE 3 NORTH TEMPORARY DIVERSION CULVERT Outfall 018
5) CULVERT AT INTERSECTION OF EAST AND WEST VALLEY HAUL ROAD Outfall 015
6) ULTIMATE CONDITIONS SOUTHWEST DITCH CULVERT

SEE SHEET 28 FOR PIPE STRENGTH DESIGN

SUBJECT: Keystone Station
Phase II Permitting
BY: SER DATE: 6/17/96 PROJ. NO.: 92-220-73-07
CHKD. BY: MRL DATE: 7/24/96 SHEET NO. 2 OF 53

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CULVERT DESIGN - HAUL ROAD DIRTY WATER DITCH STAGE 3

Purpose: Design the culvert which will carry dirty water beneath the haul road during stage 3.

Methodology: "Hydraulic Design of Highway Culverts", HDS No. 5, Federal Highway Administration, September 1985

Data Input Section

Design Flow, $Q := 72 \cdot \frac{ft^3}{sec}$ 25-year, 24-hour peak flow for Haul Road Dirty Water Ditch - Stage 3 from "Dirty Water Ditches and Related Facilities" calc by SER 5/24/96

Inlet invert elevation, $EL_i := 1213.0 \cdot ft$

Outlet invert elevation, $EL_o := 1210.0 \cdot ft$

Limiting headwater elevation, $EL_l := 1221.0 \cdot ft$

Pipe Length, $L := 100 \cdot ft$

Pipe Slope, $S := \frac{EL_i - EL_o}{L}$ $S = 0.03$

Pipe diameter, $D := \frac{42 \cdot in}{12 \cdot \frac{in}{ft}}$ $D = 3.5 \cdot ft$

Pipe material is HDPE (Spirolite) with headwall at entrance.

Flow Area, $A := \frac{D^2 \cdot \pi}{4}$ $A = 9.621 \cdot ft^2$

Flow Velocity, $V := \frac{Q}{A}$ $V = 7.484 \cdot ft \cdot sec^{-1}$

Hydraulic Radius, $R := \frac{D}{4}$ $R = 0.875 \cdot ft$

Entrance Loss Coefficient, $k_e := 0.5$ from HDS No. 5 for concrete pipe with square edged headwall. Use this for best match with proposed pipe configuration.

Manning's loss Coefficient $n := 0.011$

Critical Depth, $d_c := 2.7 \cdot ft$ from chart in HDS-5

Parameters for use in Equation 28 of HDS No. 5, for Submerged Conditions Inlet Control

$c := 0.0398 \cdot \frac{sec^2}{ft}$ from HDS No. 5 for concrete pipe with square edged headwall, units by dimensional analysis of Equation (28) below.

$Y := 0.67$ from HDS No. 5 for given pipe material and entrance type

Use these values for best match with proposed pipe configuration.

SUBJECT: Keystone Station
Phase II Permitting
BY: SER DATE: 6/17/96 PROJ. NO.: 92-220-73-07
CHKD. BY: MRL DATE: 7/24/96 SHEET NO. 3 OF 53

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Inlet Control Calculation Section

Submerged Equation (28) from HDS No. 5,

$$HW_i := D\cdot\left[c\cdot\left(\left(\frac{Q}{A\cdot D^{0.5}}\right)\right)^2 + Y - 0.5\cdot S\right] \qquad HW_i = 4.5\cdot ft$$

Inlet Control Headwater Elevation,

$$EL_{hi} := EL_i + HW_i \qquad EL_{hi} = 1217.5\cdot ft$$

Outlet Control Calculation Section

Pipe Head Loss Equation from HDS No. 5,

$$H := \left(1 + k_e + \frac{29\cdot n^2\cdot L}{R^{1.33}}\cdot ft^{0.33}\right)\cdot\frac{V^2}{2\cdot g} \qquad H = 1.7\cdot ft$$

$$h_0 := \frac{D + d_c}{2}$$

$$h_0 = 3.1\cdot ft$$

Outlet Control Headwater Elevation,

$$EL_{ho} := EL_o + H + h_0 \qquad EL_{ho} = 1214.8\cdot ft$$

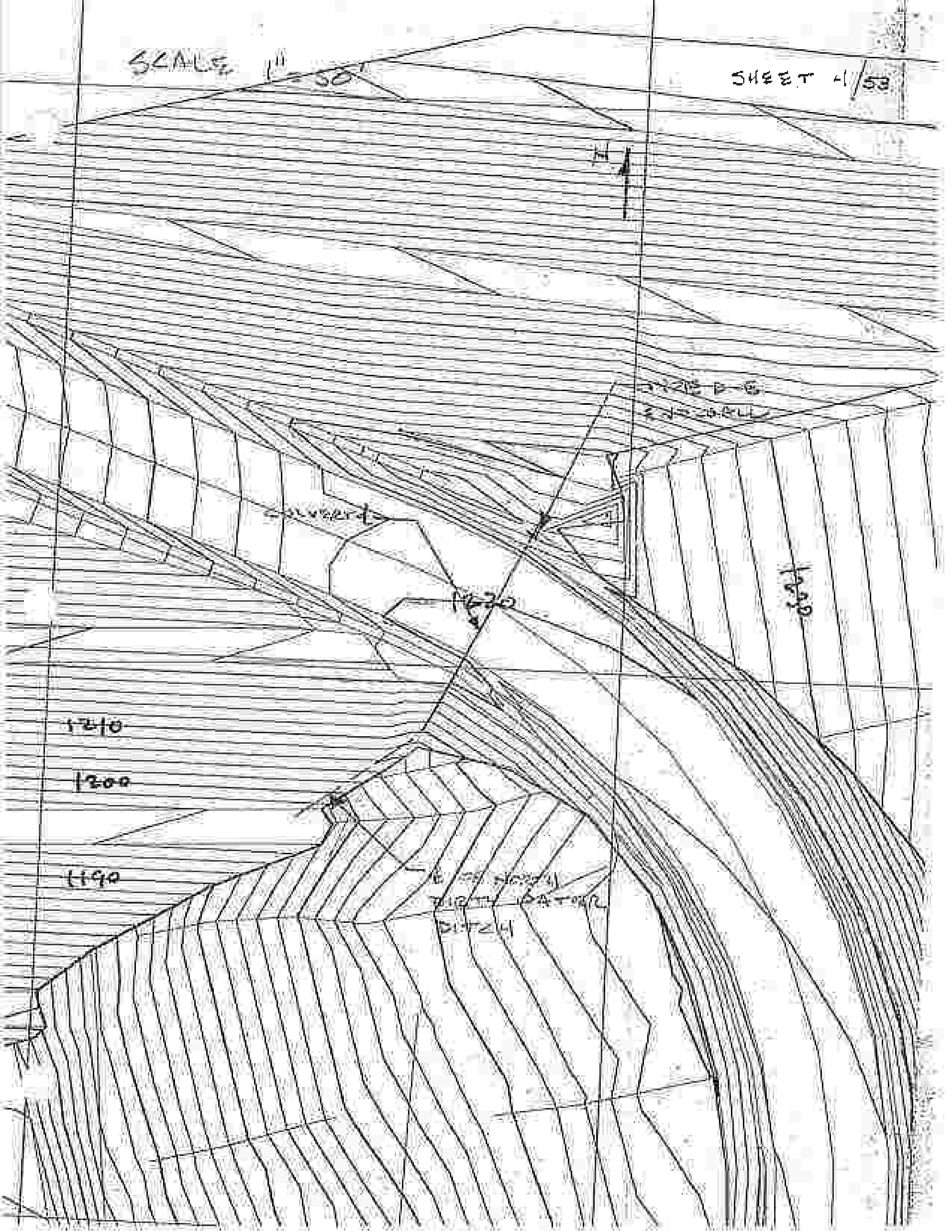
Controlling Headwater Elevation

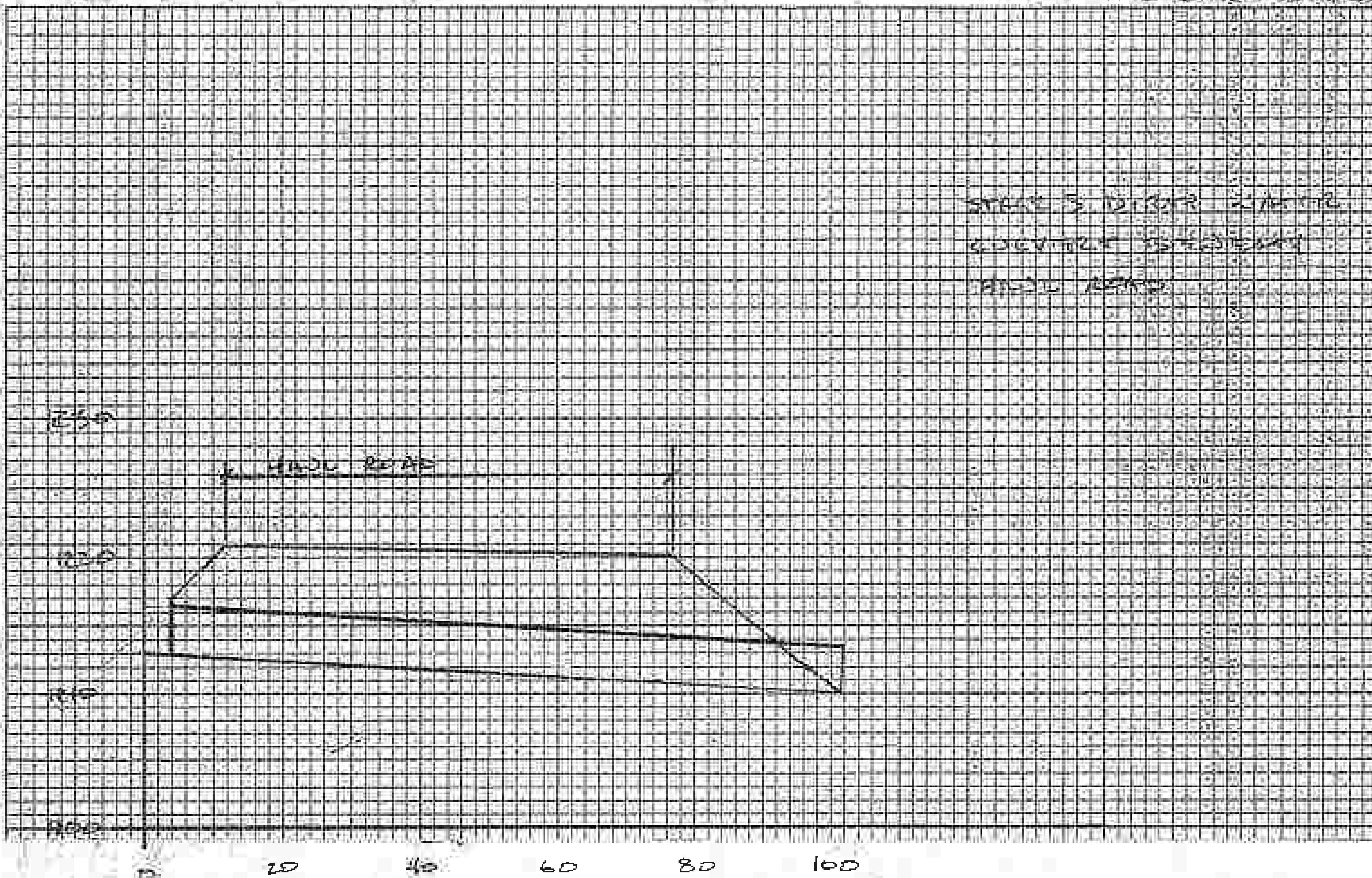
$$EL_{hc} := \max\left(\left(\begin{matrix} EL_{hi} \\ EL_{ho} \end{matrix}\right)\right) \qquad EL_{hc} = 1217.5\cdot ft$$

Compare to the limiting headwater elevation,

$$EL_l = 1221.0\cdot ft$$

$EL_{hc} < EL_l$, Therefore Pipe design is OK


SCALE 1" = 50'
SHEET 4/53
N
CULVERT
1220
1210
1200
1190
DITCH



Reviewed by WRL 7/24/96.

SUBJECT: Keystone Station
Phase II Permitting
BY: SER DATE: 6/17/96 PROJ. NO.: 92-220-73-07
CHKD. BY: MRL DATE: 7/24/96 SHEET NO. 6 OF 53

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CULVERT DESIGN - HAUL ROAD DIRTY WATER DITCH STAGE 4

Purpose: Design the culvert which will carry dirty water beneath the haul road during stage 4.

Methodology: "Hydraulic Design of Highway Culverts", HDS No. 5, Federal Highway Administration, September 1985

Data Input Section

Design Flow, $Q := 91 \cdot \frac{ft^3}{sec}$ — 25-year, 24-hour peak flow for Haul Road Dirty Water Ditch - Stage 4 from "Dirty Water Ditches and Related Facilities" calc by SER 5/24/96

Inlet invert elevation, $EL_i := 1152.5 \cdot ft$

Outlet invert elevation, $EL_o := 1150.0 \cdot ft$

Limiting headwater elevation, $EL_1 := 1160.0 \cdot ft$

Pipe Length, $L := 90 \cdot ft$

Pipe Slope, $S := \frac{EL_i - EL_o}{L}$ $\quad S = 0.028$

Pipe diameter, $D := \frac{42 \cdot in}{12 \cdot \frac{in}{ft}}$ $\quad D = 3.5 \cdot ft$

Pipe material is HDPE (Spirolite) with headwall at entrance.

Flow Area, $A := \frac{D^2 \cdot \pi}{4}$ $\quad A = 9.621 \cdot ft^2$

Flow Velocity, $V := \frac{Q}{A}$ $\quad V = 9.458 \cdot ft \cdot sec^{-1}$

Hydraulic Radius, $R := \frac{D}{4}$ $\quad R = 0.875 \cdot ft$

Entrance Loss Coefficient, $k_e := 0.5$ — from HDS No. 5 for concrete pipe with square edged headwall. Use this for best match with proposed pipe configuration.

Manning's loss Coefficient $n := 0.011$

Critical Depth, $d_c := 2.9 \cdot ft$ — from chart in HDS-5

Parameters for use in Equation 28 of HDS No. 5, for Submerged Conditions Inlet Control

$c := 0.0398 \cdot \frac{sec^2}{ft}$ — from HDS No. 5 for concrete pipe with square edged headwall, units by dimensional analysis of Equation (28) below.

$Y := 0.67$ — from HDS No. 5 for given pipe material and entrance type

Use these values for best match with proposed pipe configuration.

SUBJECT: Keystone Station
Phase II Permitting
BY: SER DATE: 6/17/96 PROJ. NO.: 92-220-73-07
CHKD. BY: MCL DATE: 7/24/96 SHEET NO. 2 OF 53

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Inlet Control Calculation Section

Submerged Equation (28) from HDS No. 5,

$$HW_i := D\cdot\left[c\cdot\left(\left(\frac{Q}{A\cdot D^{0.5}}\right)\right)^2 + Y - 0.5\cdot S\right] \qquad HW_i = 5.9\cdot ft$$

Inlet Control Headwater Elevation,

$$EL_{hi} := EL_i + HW_i \qquad EL_{hi} = 1158.4\cdot ft$$

Outlet Control Calculation Section

Pipe Head Loss Equation from HDS No. 5,

$$H := \left(1 + k_e + \frac{29\cdot n^2\cdot L}{R^{1.33}}\cdot ft^{0.33}\right)\cdot\frac{V^2}{2\cdot g} \qquad H = 2.6\cdot ft$$

$$h_0 := \frac{D + d_c}{2}$$

$$h_0 = 3.2\cdot ft$$

Outlet Control Headwater Elevation,

$$EL_{ho} := EL_o + H + h_0 \qquad EL_{ho} = 1155.8\cdot ft$$

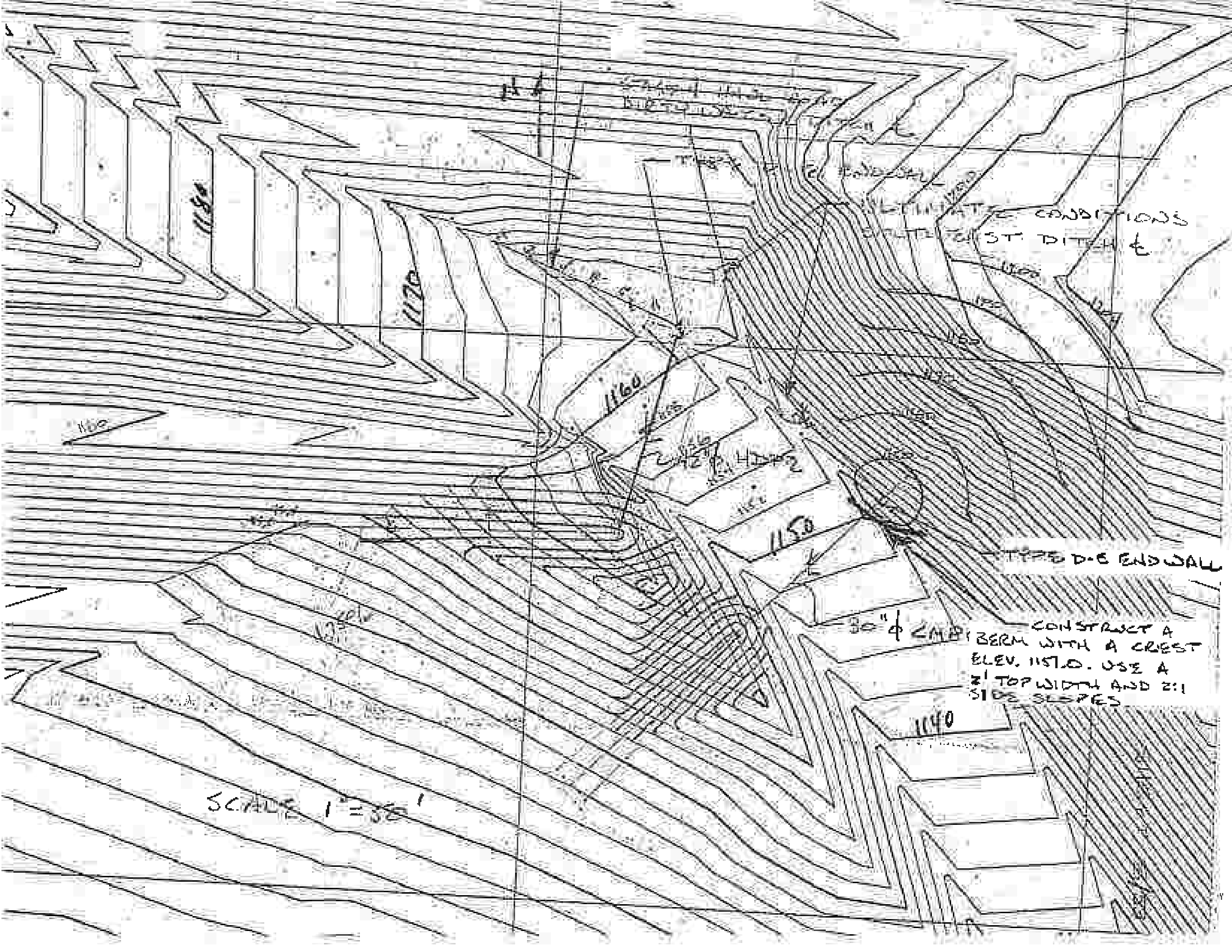
Controlling Headwater Elevation

$$EL_{hc} := \max\left(\begin{pmatrix} EL_{hi} \\ EL_{ho} \end{pmatrix}\right) \qquad EL_{hc} = 1158.4\cdot ft$$

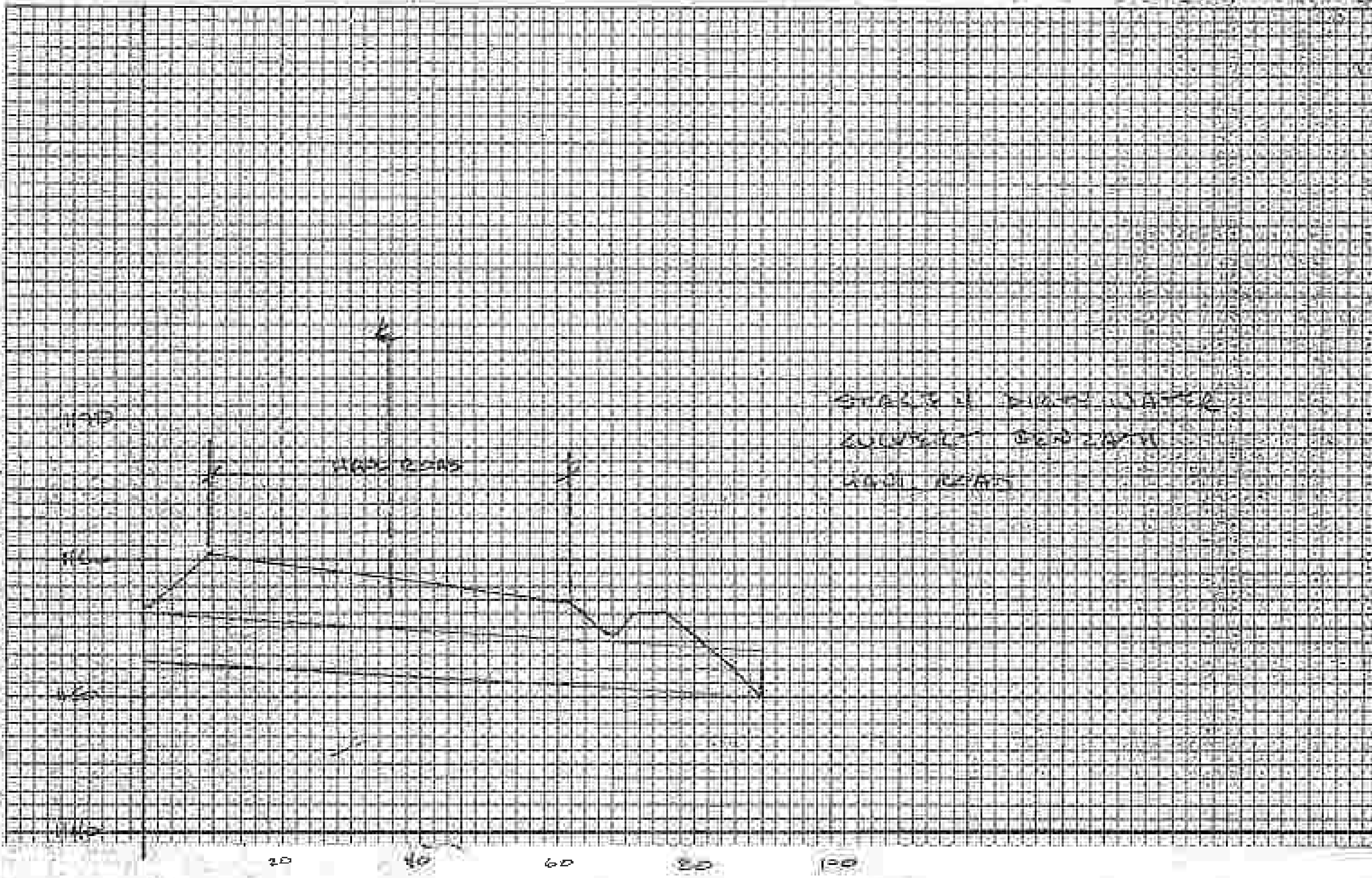
Compare to the limiting headwater elevation,

$$EL_1 = 1160.0\cdot ft$$

$EL_{hc} < EL_1$, **Therefore Pipe design is OK**



ULTIMATE CONDITIONS
SOUTHEAST DITCH ℄
TYPE D-E ENDWALL
30" ϕ CMP
CONSTRUCT A BERM WITH A CREST ELEV. 1157.0. USE A 2' TOP WIDTH AND 2:1 SIDE SLOPES
42" ϕ HDPE
1160
1150
1140
1170
1180
SCALE 1" = 30'



SHEET 9/53

Reviewed by MEL 7/24/96

SUBJECT: Keystone Station
Phase II Permitting
BY: SER DATE: 6/17/96 PROJ. NO.: 92-220-73-07
CHKD. BY: MRL DATE: 7/24/96 SHEET NO. 10 OF 53

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CULVERT DESIGN - ULTIMATE CONDITIONS SOUTHEAST DITCH CULVERTS

Purpose: Design the culvert which will carry the ultimate conditions southeast ditch beneath the haul road and the culvert which will carry the ultimate conditions southeast ditch beneath the West Dirty Water Ditch.

Methodology: "Hydraulic Design of Highway Culverts", HDS No. 5, Federal Highway Administration, September 1985

First design the culvert beneath the haul road

Data Input Section

Design Flow, $Q := 51 \cdot \frac{ft^3}{sec}$ 25-year, 24-hour peak flow for the ultimate conditions southeast ditch from "Dirty Water Ditches and Related Facilities" calc by SER 5/24/96

Inlet invert elevation, $EL_i := 1143.0 \cdot ft$

Outlet invert elevation, $EL_o := 1142.0 \cdot ft$ See sheet 8 for plan view sketch.

Limiting headwater elevation, $EL_l := 1151.0 \cdot ft$

Pipe Length, $L := 89 \cdot ft$

Pipe Slope, $S := \frac{EL_i - EL_o}{L}$ $S = 0.011$

Pipe diameter, $D := \frac{30 \cdot in}{12 \cdot \frac{in}{ft}}$ $D = 2.5 \cdot ft$

Pipe material is BCCMP with headwall.

Flow Area, $A := \frac{D^2 \cdot \pi}{4}$ $A = 4.909 \cdot ft^2$

Flow Velocity, $V := \frac{Q}{A}$ $V = 10.39 \cdot ft \cdot sec^{-1}$

Hydraulic Radius, $R := \frac{D}{4}$ $R = 0.625 \cdot ft$

Entrance Loss Coefficient, $k_e := 0.5$ from HDS No. 5 for CMP with square edged headwall.

Manning's loss Coefficient $n := 0.022$

Critical Depth, $d_c := 2.3 \cdot ft$ from chart in HDS-5

Parameters for use in Equation 28 of HDS No. 5, for Submerged Conditions Inlet Control

$c := 0.0379 \cdot \frac{sec^2}{ft}$ from HDS No. 5 for CMP pipe with square edged headwall, units by dimensional analysis of Equation (28) below.

$Y := 0.69$ from HDS No. 5 for given pipe material and entrance type

SUBJECT: Keystone Station
Phase II Permitting
BY: SER DATE: 6/17/06 PROJ. NO.: 92-220-73-07
CHKD. BY: MRL DATE: 7/24/06 SHEET NO. 11 OF 53

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Inlet Control Calculation Section

Submerged Equation (28) from HDS No. 5,

$$HW_i := D\cdot\left[c\cdot\left(\left(\frac{Q}{A\cdot D^{0.5}}\right)\right)^2 + Y - 0.5\cdot S\right] \qquad HW_i = 5.8\cdot ft$$

Inlet Control Headwater Elevation,

$$EL_{hi} := EL_i + HW_i \qquad EL_{hi} = 1148.8\cdot ft$$

Outlet Control Calculation Section

Pipe Head Loss Equation from HDS No. 5,

$$H := \left(1 + k_e + \frac{29\cdot n^2\cdot L}{R^{1.33}}\cdot ft^{0.33}\right)\cdot\frac{V^2}{2\cdot g} \qquad H = 6.4\cdot ft$$

$$h_0 := \frac{D + d_c}{2}$$

$$h_0 = 2.4\cdot ft$$

Outlet Control Headwater Elevation,

$$EL_{ho} := EL_o + H + h_0 \qquad EL_{ho} = 1150.8\cdot ft$$

Controlling Headwater Elevation

$$EL_{hc} := \max\left(\begin{pmatrix} EL_{hi} \\ EL_{ho} \end{pmatrix}\right) \qquad EL_{hc} = 1150.8\cdot ft$$

Compare to the limiting headwater elevation,

$$EL_l = 1151.0\cdot ft$$

$EL_{hc} < EL_l$, Therefore Pipe design is OK

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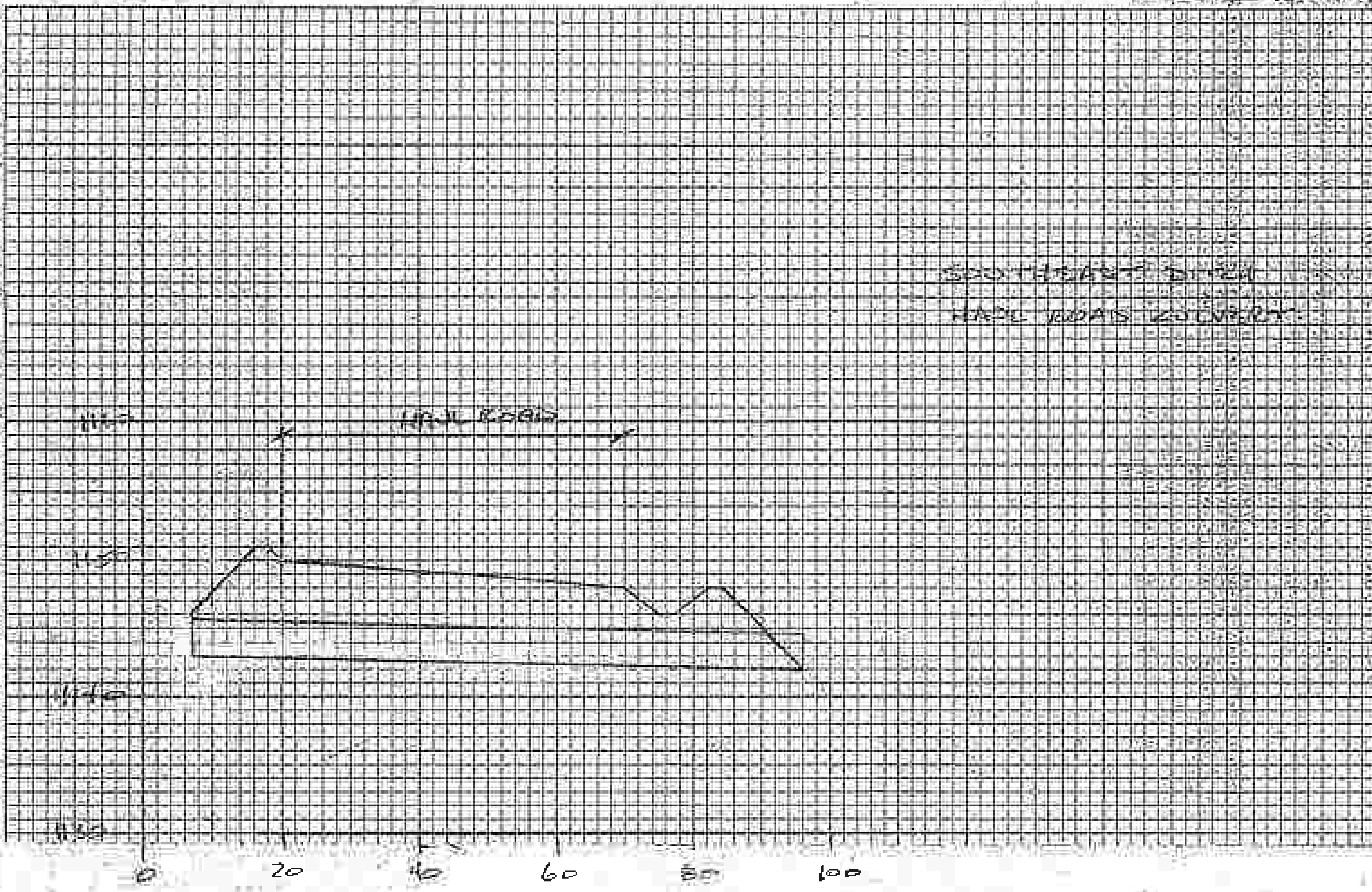
47 0782



Reviewed by MRL 7/24/96

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Next design the culvert beneath the West Dirty Water Ditch

Data Input Section

Design Flow, $Q := 69 \cdot \frac{ft^3}{sec}$ — 25-year, 24-hour peak flow for the ultimate conditions southeast ditch from "Dirty Water Ditches and Related Facilities" calc by SER 5/24/96, see sheets 2,13, and 14 for design flow.

Inlet invert elevation, $EL_i := 1095.5 \cdot ft$

Outlet invert elevation, $EL_o := 1094.87 \cdot ft$

Limiting headwater elevation, $EL_l := 1102.7 \cdot ft$

Pipe Length, $L := 63 \cdot ft$

Pipe Slope, $S := \frac{EL_i - EL_o}{L}$ $\quad S = 0.01$

Pipe diameter, $D := \frac{36 \cdot in}{12 \cdot \frac{in}{ft}}$ $\quad D = 3 \cdot ft$

Pipe material is BCCMP projecting from fill.

Flow Area, $A := \frac{D^2 \cdot \pi}{4}$ $\quad A = 7.069 \cdot ft^2$

Flow Velocity, $V := \frac{Q}{A}$ $\quad V = 9.762 \cdot ft \cdot sec^{-1}$

Hydraulic Radius, $R := \frac{D}{4}$ $\quad R = 0.75 \cdot ft$

Entrance Loss Coefficient, $k_e := 0.9$ — from HDS No. 5 for CMP projecting from fill.

Manning's loss Coefficient $n := 0.022$

Critical Depth, $d_c := 2.7 \cdot ft$ — from chart in HDS-5

Parameters for use in Equation 28 of HDS No. 5, for Submerged Conditions Inlet Control

$c := 0.0553 \cdot \frac{sec^2}{ft}$ — from HDS No. 5 for CMP pipe projecting from fill, units by dimensional analysis of Equation (28) below.

$Y := 0.54$ — from HDS No. 5 for given pipe material and entrance type

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Inlet Control Calculation Section

Submerged Equation (28) from HDS No. 5,

$$HW_i := D \cdot \left[c \cdot \left(\left(\frac{Q}{A \cdot D^{0.5}} \right) \right)^2 + Y - 0.5 \cdot S \right] \qquad HW_i = 6.9 \cdot ft$$

Inlet Control Headwater Elevation,

$$EL_{hi} := EL_i + HW_i \qquad EL_{hi} = 1102.4 \cdot ft$$

Outlet Control Calculation Section

Pipe Head Loss Equation from HDS No. 5,

$$H := \left(1 + k_e + \frac{29 \cdot n^2 \cdot L}{R^{1.33}} \cdot ft^{0.33} \right) \cdot \frac{V^2}{2 \cdot g} \qquad H = 4.7 \cdot ft$$

$$h_0 := \frac{D + d_c}{2}$$

$$h_0 = 2.9 \cdot ft$$

Outlet Control Headwater Elevation,

$$EL_{ho} := EL_o + H + h_0 \qquad EL_{ho} = 1102.5 \cdot ft$$

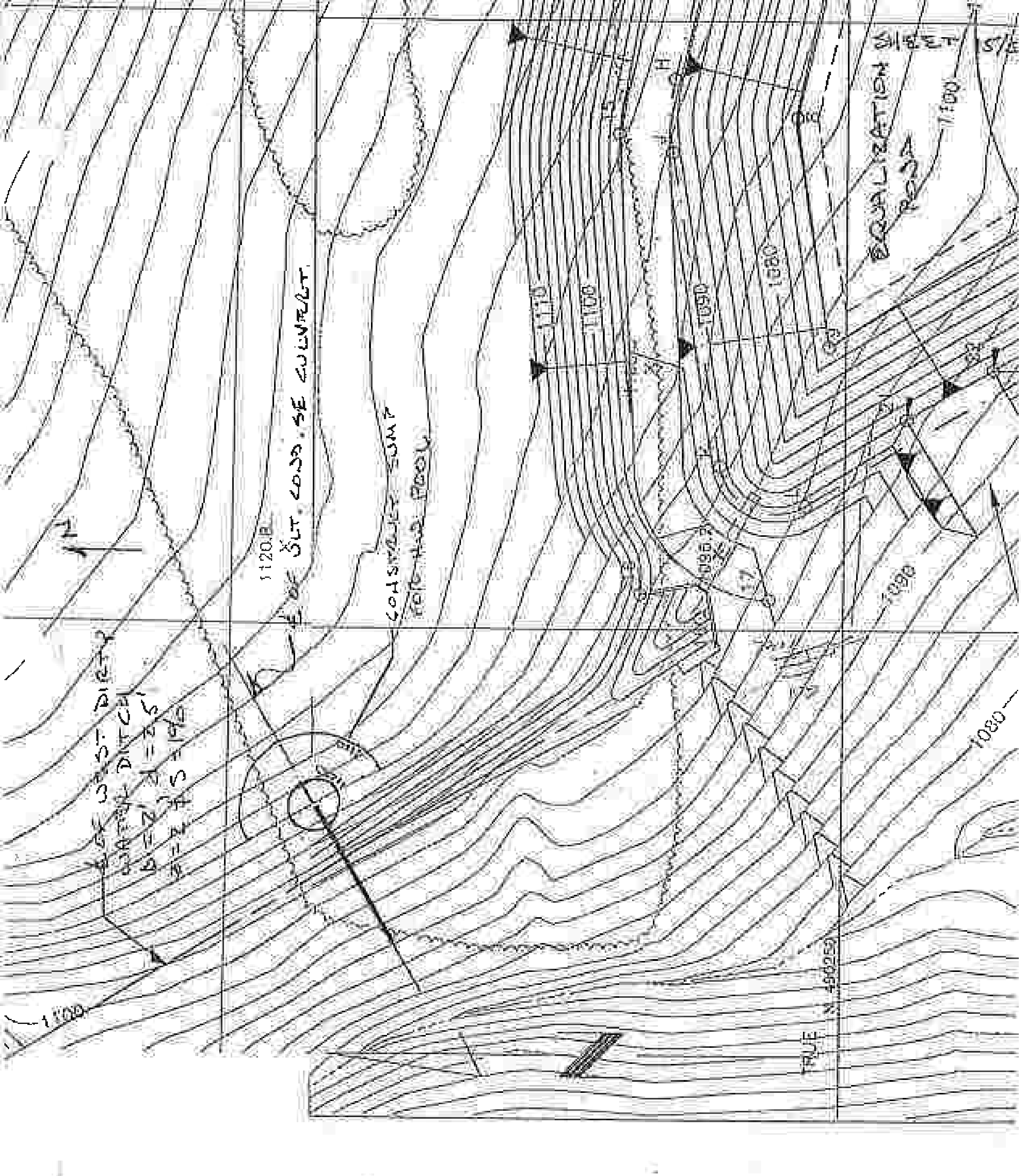
Controlling Headwater Elevation

$$EL_{hc} := \max\left(\begin{pmatrix} EL_{hi} \\ EL_{ho} \end{pmatrix} \right) \qquad EL_{hc} = 1102.5 \cdot ft$$

Compare to the limiting headwater elevation,

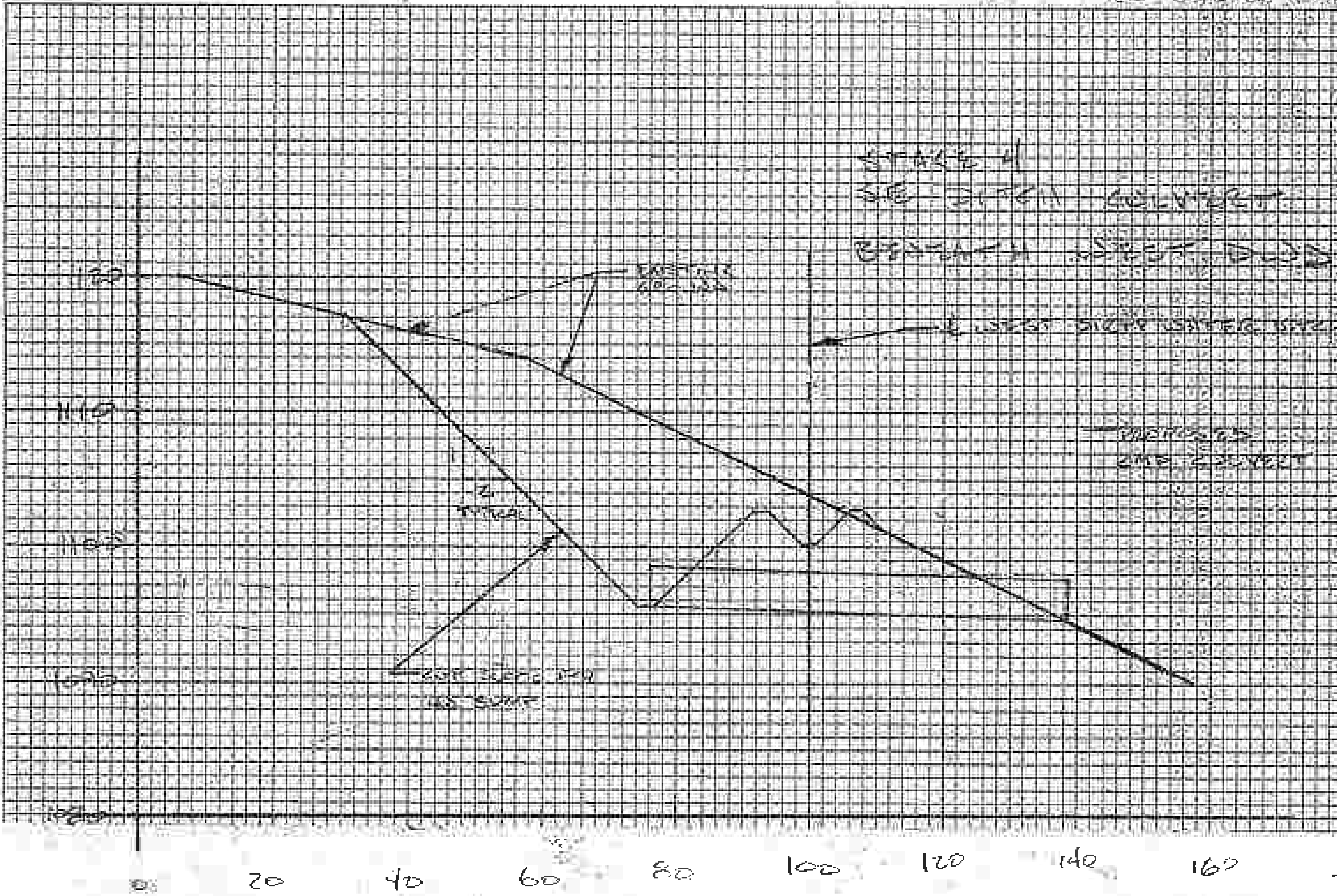
$$EL_l = 1102.7 \cdot ft$$

$EL_{hc} < EL_l$, Therefore Pipe design is OK


SHEET 15/5
EQUALIZATION POND
1100
1080
1090
1110
1120.8
SLT. COND. SE CULVERT
CONSTRUCT SUMP FOR H2O POOL
1096
1080
1100
TRUE N

SCALE 1" = 50'

47 0782



Reviewed by MRL 7/24/96

SUBJECT: Keystone Station
Phase II Permitting
BY: SER DATE: 6/17/96 PROJ. NO.: 92-220-73-07
CHKD. BY: MRL DATE: 7/24/96 SHEET NO. 17 OF 53

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CULVERT DESIGN - STAGE 3 NORTH TEMPORARY DIVERSION CULVERT

Purpose: Design the culvert which will carry flows from the Stage 3 North Temporary Diversion Ditch beneath the West Dirty Water Ditch

Methodology: "Hydraulic Design of Highway Culverts", HDS No. 5, Federal Highway Administration, September 1985

Data Input Section

Design Flow, $Q := 20\cdot\frac{ft^3}{sec}$ 25-year, 24-hour peak flow from "Dirty Water Ditches and Related Facilities" calc by SER 5/24/96.

Inlet invert elevation, $EL_i := 1148.0\cdot ft$

Outlet invert elevation, $EL_o := 1145.0\cdot ft$

Limiting headwater elevation, $EL_1 := 1156.2\cdot ft$

Pipe Length, $L := 56\cdot ft$

Pipe Slope, $S := \frac{EL_i - EL_o}{L}$ $S = 0.0536$

Pipe diameter, $D := \frac{18\cdot in}{12\cdot\frac{in}{ft}}$ $D = 1.5\cdot ft$

Pipe material is BCCMP projecting from fill.

Flow Area, $A := \frac{D^2\cdot\pi}{4}$ $A = 1.767\cdot ft^2$

Flow Velocity, $V := \frac{Q}{A}$ $V = 11.318\cdot ft\cdot sec^{-1}$

Hydraulic Radius, $R := \frac{D}{4}$ $R = 0.375\cdot ft$

Entrance Loss Coefficient, $k_e := 0.9$ from HDS No. 5 for CMP projecting from fill.

Manning's loss Coefficient $n := 0.022$

Critical Depth, $d_c := 1.7\cdot ft$ from chart in HDS-5

Parameters for use in Equation 28 of HDS No. 5, for Submerged Conditions Inlet Control

$c := 0.0553\cdot\frac{sec^2}{ft}$ from HDS No. 5 for CMP pipe projecting from fill, units by dimensional analysis of Equation (28) below.

$Y := 0.54$ from HDS No. 5 for given pipe material and entrance type

SUBJECT: Keystone Station
Phase II Permitting
BY: SER DATE: 6/17/96 PROJ. NO.: 92-220-73-07
CHKD. BY: MRL DATE: 7/24/96 SHEET NO. 18 OF 53

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Inlet Control Calculation Section

Submerged Equation (28) from HDS No. 5,

$$HW_i := D\left[c\cdot\left(\left(\frac{Q}{A\cdot D^{0.5}}\right)\right)^2 + Y - 0.5\cdot S\right] \qquad HW_i = 7.9\cdot ft$$

Inlet Control Headwater Elevation,

$$EL_{hi} := EL_i + HW_i \qquad EL_{hi} = 1155.9\cdot ft$$

Outlet Control Calculation Section

Pipe Head Loss Equation from HDS No. 5,

$$H := \left(1 + k_e + \frac{29\cdot n^2\cdot L}{R^{1.33}}\cdot ft^{0.33}\right)\cdot\frac{V^2}{2\cdot g} \qquad H = 9.5\cdot ft$$

$$h_0 := \frac{D + d_c}{2}$$

$$h_0 = 1.6\cdot ft$$

Outlet Control Headwater Elevation,

$$EL_{ho} := EL_o + H + h_0 \qquad EL_{ho} = 1156.1\cdot ft$$

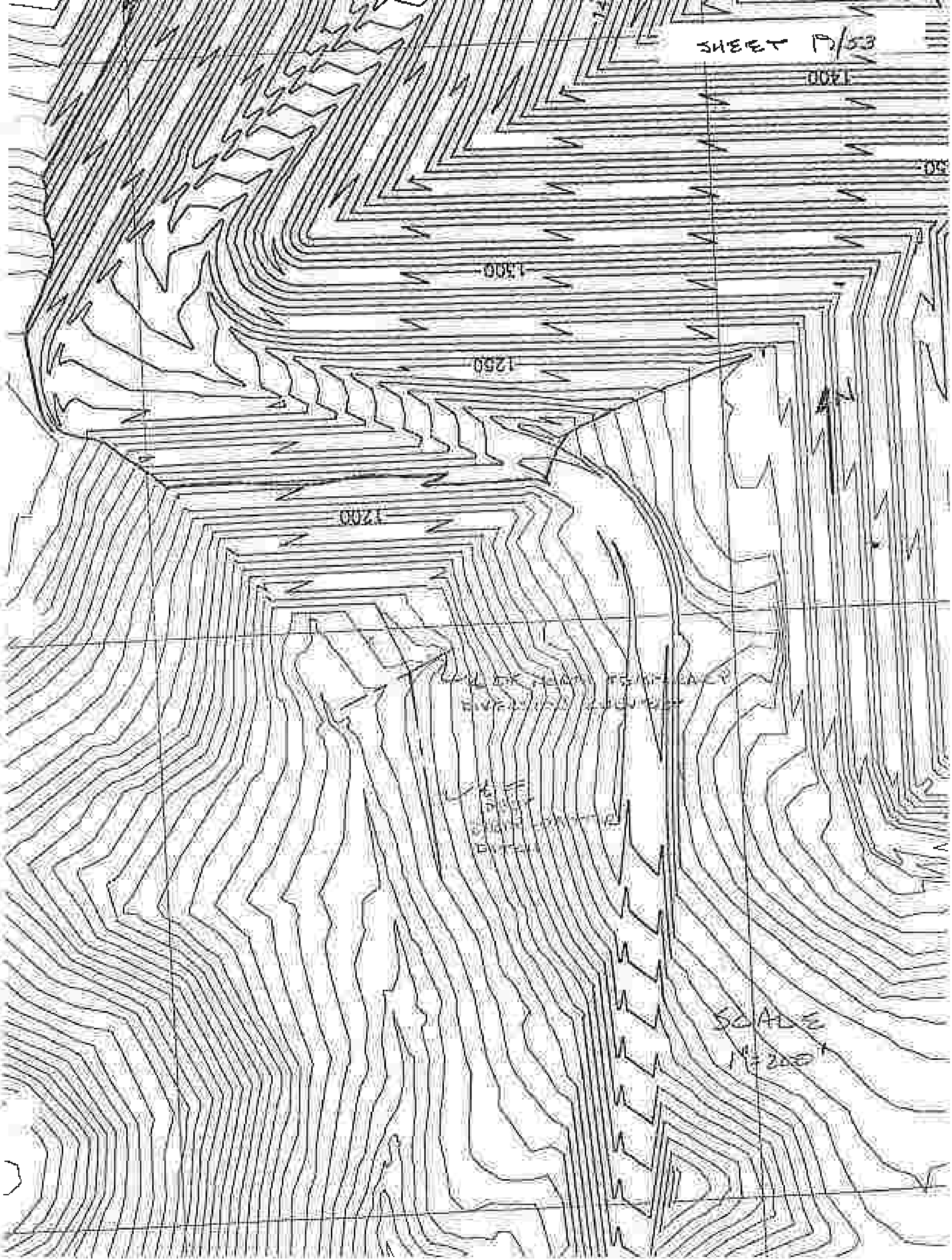
Controlling Headwater Elevation

$$EL_{hc} := \max\left(\begin{pmatrix} EL_{hi} \\ EL_{ho} \end{pmatrix}\right) \qquad EL_{hc} = 1156.1\cdot ft$$

Compare to the limiting headwater elevation,

$$EL_l = 1156.2\cdot ft$$

$EL_{hc} \le EL_l$ Therefore Pipe design is OK



SHEET 19/53
1300
1250
1200
SCALE
1" = 200'

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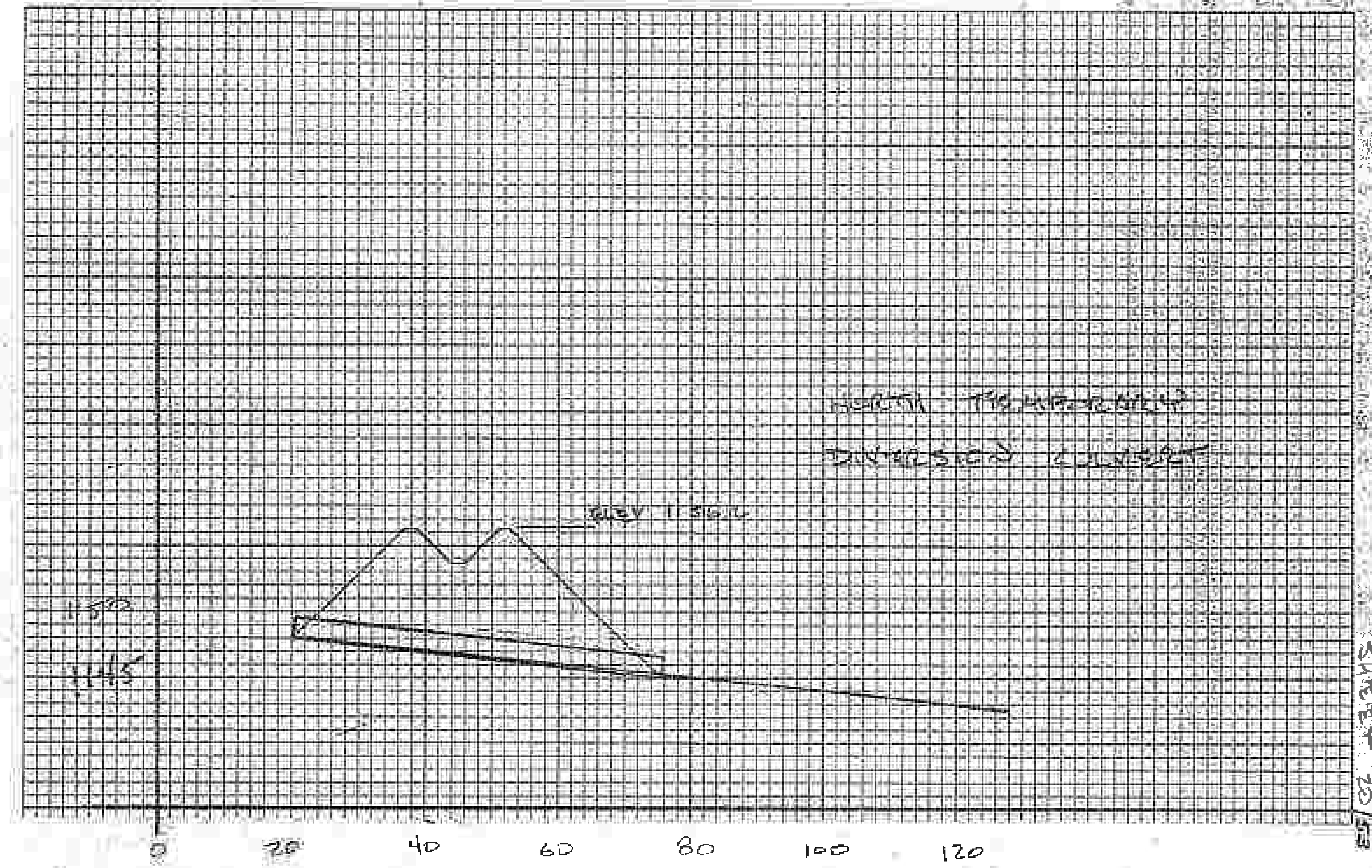

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SUBJECT: Keystone Station
Phase II Permitting
BY: SER DATE: 6/17/96 PROJ. NO.: 92-220-73-07
CHKD. BY: MRL DATE: 7/24/96 SHEET NO. 21 OF 53

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CULVERT DESIGN - CULVERT AT INTERSECTION OF EAST VALLEY AND WEST VALLEY HAUL ROADS

Purpose: Design the culvert which will carry flows from the ditch on the northeast side of the west valley haul road and the ditch on the north side of the east valley haul road, beneath the west valley haul road.

Methodology: "Hydraulic Design of Highway Culverts",
HDS No. 5, Federal Highway Administration, September 1985

Data Input Section

Design Flow, $Q := 56 \cdot \frac{ft^3}{sec}$ 25-year, 24-hour peak flow from "Stage 3 - Drainage Facilities" calc by SER 4/25/96.

Inlet invert elevation, $EL_i := 1079.0 \cdot ft$

Outlet invert elevation, $EL_o := 1078.0 \cdot ft$

Limiting headwater elevation, $EL_l := 1085.0 \cdot ft$

Pipe Length, $L := 130 \cdot ft$

Pipe Slope, $S := \frac{EL_i - EL_o}{L}$ $S = 0.0077$

Pipe diameter, $D := \frac{36 \cdot in}{12 \cdot \frac{in}{ft}}$ $D = 3 \cdot ft$

Pipe material is BCCMP with headwall.

Flow Area, $A := \frac{D^2 \cdot \pi}{4}$ $A = 7.069 \cdot ft^2$

Flow Velocity, $V := \frac{Q}{A}$ $V = 7.922 \cdot ft \cdot sec^{-1}$

Hydraulic Radius, $R := \frac{D}{4}$ $R = 0.75 \cdot ft$

Entrance Loss Coefficient, $k_e := 0.5$ from HDS No. 5 for CMP with square edged headwall.

Manning's loss Coefficient $n := 0.022$

Critical Depth, $d_c := 2.4 \cdot ft$ from chart in HDS-5

Parameters for use in Equation 28 of HDS No. 5, for Submerged Conditions Inlet Control

$c := 0.0379 \cdot \frac{sec^2}{ft}$ from HDS No. 5 for CMP pipe with square edged headwall, units by dimensional analysis of Equation (28) below.

$Y := 0.69$ from HDS No. 5 for given pipe material and entrance type

SUBJECT: Keystone Station
Phase II Permitting
BY: SER DATE: 6/17/96 PROJ. NO.: 92-220-73-07
CHKD. BY: MRL DATE: 7/24/96 SHEET NO. 22 OF 53

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Inlet Control Calculation Section

Submerged Equation (28) from HDS No. 5,

$$HW_i := D\cdot\left[c\cdot\left(\left(\frac{Q}{A\cdot D^{0.5}}\right)\right)^2 + Y - 0.5\cdot S\right] \qquad HW_i = 4.4\cdot ft$$

Inlet Control Headwater Elevation,

$$EL_{hi} := EL_i + HW_i \qquad EL_{hi} = 1083.4\cdot ft$$

Outlet Control Calculation Section

Pipe Head Loss Equation from HDS No. 5,

$$H := \left(1 + k_e + \frac{29\cdot n^2\cdot L}{R^{1.33}}\cdot ft^{0.33}\right)\cdot\frac{V^2}{2\cdot g} \qquad H = 4.1\cdot ft$$

$$h_0 := \frac{D + d_c}{2}$$

$$h_0 = 2.7\cdot ft$$

Outlet Control Headwater Elevation,

$$EL_{ho} := EL_o + H + h_0 \qquad EL_{ho} = 1084.8\cdot ft$$

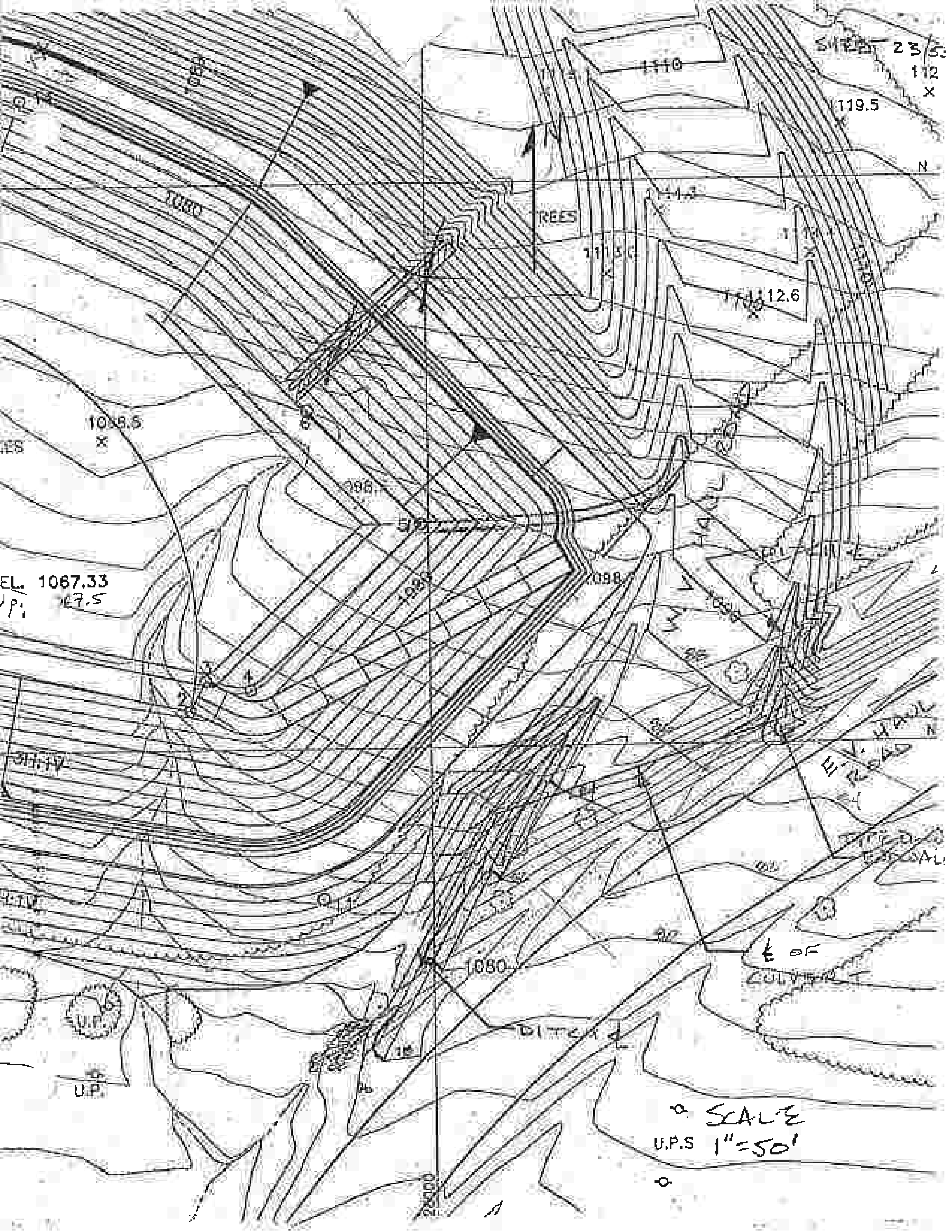
Controlling Headwater Elevation

$$EL_{hc} := \max\left(\begin{pmatrix} EL_{hi} \\ EL_{ho} \end{pmatrix}\right) \qquad EL_{hc} = 1084.8\cdot ft$$

Compare to the limiting headwater elevation,

$$EL_l = 1085.0\cdot ft$$

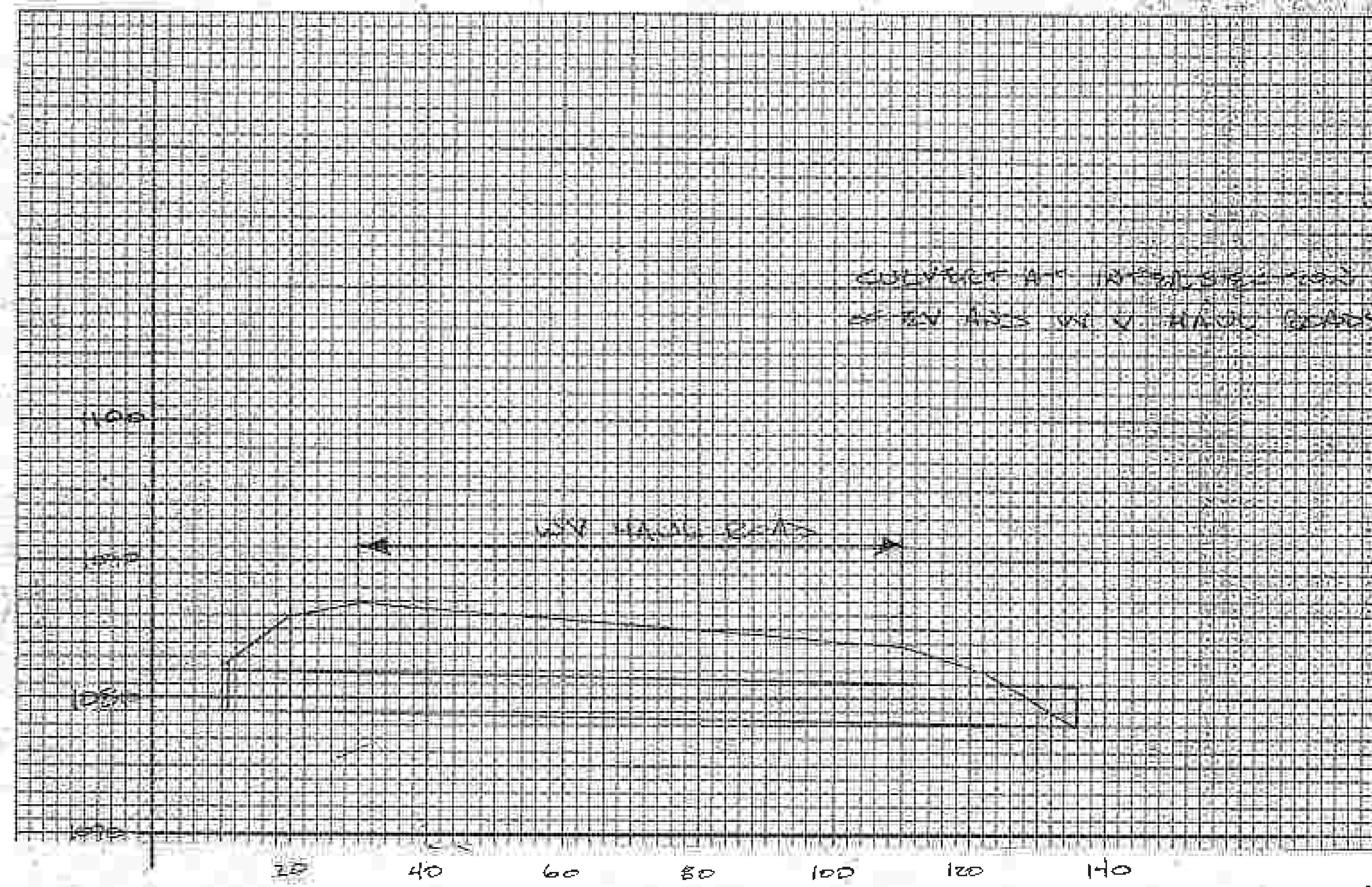
$EL_{hc} \le EL_l$ Therefore Pipe design is OK



SHEET 23/53
1110
119.5
TREES
1080
1113.6
112.6
1086.5
1090
EL. 1067.33
1080
HAUL
HAUL
ROAD
E OF CULVERT
1080
DITCH
U.P.
U.P.
SCALE
U.P.S 1"=50'

K+E 10 X 10 TO THE INCH • 10 X 15 INCHES
KEUFFEL & ESSER CO. MADE IN U.S.A.

47 0782



SHEET - 24/53

Reviewed by MEL 7/24/96

SUBJECT: Keystone Station
Phase II Permitting
BY: SER DATE: 6/17/96 PROJ. NO.: 92-220-73-07
CHKD. BY: MRL DATE: 7/24/96 SHEET NO. 25 OF 53

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CULVERT DESIGN - ULTIMATE CONDITIONS SOUTHWEST DITCH CULVERT

Purpose: Design the culvert which will carry the ultimate conditions southwest ditch beneath an access road which will be located along the portion of the southwest ditch which is at a 1 % slope.

Methodology: "Hydraulic Design of Highway Culverts", HDS No. 5, Federal Highway Administration, September 1985

Data Input Section.

Design Flow, $Q := 90 \cdot \frac{ft^3}{sec}$ — 25-year, 24-hour peak flow for the ultimate conditions southwest ditch from "Ultimate Conditions - Drainage Facilities" calc by SER 3/19/96

Flow per Barrel $Q := \frac{90}{2} \cdot \frac{ft^3}{sec}$ — Use 2 barrels.

Inlet invert elevation, $EL_i := 0.33 \cdot ft$

Outlet invert elevation, $EL_o := 0.0 \cdot ft$ — Elevations are arbitrary.

Limiting headwater elevation, $EL_1 := 4.5 \cdot ft$

Pipe Length, $L := 33 \cdot ft$

Pipe Slope, $S := \frac{EL_i - EL_o}{L}$ $\quad S = 0.01$

Pipe diameter, $D := \frac{36 \cdot in}{12 \cdot \frac{in}{ft}}$ $\quad D = 3 \cdot ft$

Pipe material is BCCMP projecting from fill.

Flow Area, $A := \frac{D^2 \cdot \pi}{4}$ $\quad A = 7.069 \cdot ft^2$

Flow Velocity, $V := \frac{Q}{A}$ $\quad V = 6.366 \cdot ft \cdot sec^{-1}$

Hydraulic Radius, $R := \frac{D}{4}$ $\quad R = 0.75 \cdot ft$

Entrance Loss Coefficient, $k_e := 0.9$ — from HDS No. 5 for CMP projecting from fill.

Manning's loss Coefficient $n := 0.022$

Critical Depth, $d_c := 2.2 \cdot ft$ — from chart in HDS-5

Parameters for use in Equation 28 of HDS No. 5, for Submerged Conditions Inlet Control

$c := 0.0553 \cdot \frac{sec^2}{ft}$ — from HDS No. 5 for CMP pipe projecting from fill, units by dimensional analysis of Equation (28) below.

$Y := 0.54$ — from HDS No. 5 for given pipe material and entrance type

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Inlet Control Calculation Section

Submerged Equation (28) from HDS No. 5,

$$HW_i := D\cdot\left[c\cdot\left(\left(\frac{Q}{A\cdot D^{0.5}}\right)\right)^2 + Y - 0.5\cdot S\right] \qquad HW_i = 3.8\cdot ft$$

Inlet Control Headwater Elevation,

$$EL_{hi} := EL_i + HW_i \qquad EL_{hi} = 4.2\cdot ft$$

Outlet Control Calculation Section

Pipe Head Loss Equation from HDS No. 5,

$$H := \left(1 + k_e + \frac{29\cdot n^2\cdot L}{R^{1.33}}\cdot ft^{0.33}\right)\frac{V^2}{2\cdot g} \qquad H = 1.6\cdot ft$$

$$h_0 := \frac{D + d_c}{2}$$

$$h_0 = 2.6\cdot ft$$

Outlet Control Headwater Elevation,

$$EL_{ho} := EL_o + H + h_0 \qquad EL_{ho} = 4.2\cdot ft$$

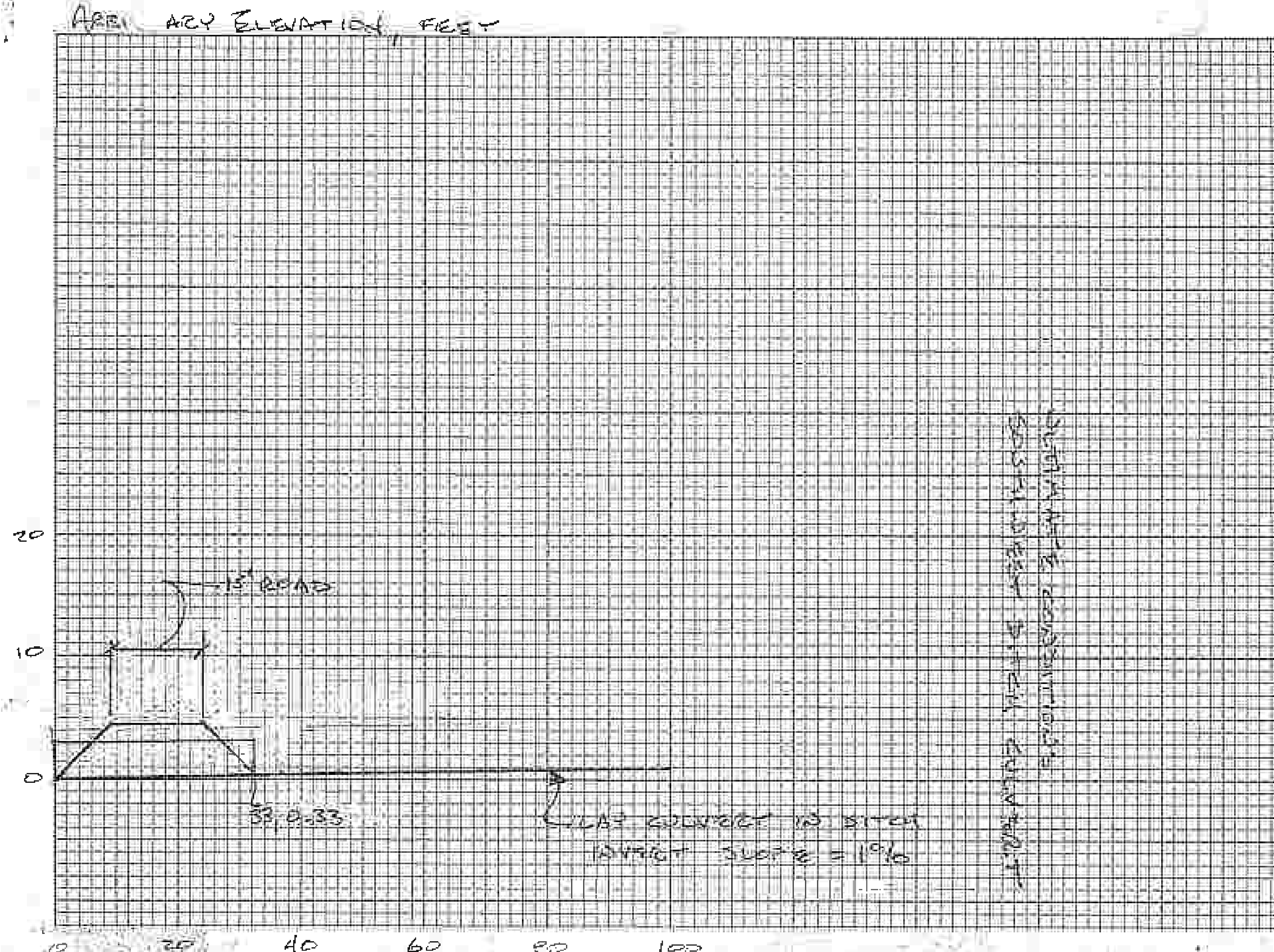
Controlling Headwater Elevation

$$EL_{hc} := \max\left(\begin{pmatrix} EL_{hi} \\ EL_{ho} \end{pmatrix}\right) \qquad EL_{hc} = 4.2\cdot ft$$

Compare to the limiting headwater elevation,

$$EL_l = 4.5\cdot ft$$

$EL_{hc} < EL_l$, Therefore Pipe design is OK



Reviewed by MRL 7/24/96

PIPE STRENGTH DESIGN

PURPOSE: DESIGN THE PROPOSED CULVERTS WITH RESPECT TO STRENGTH. SEE SHEETS 33 TO 39 FOR DESIGN METHODS.

HAUL ROAD DIRTY WATER DITCH CULVERT - STAGE 3

42"ϕ SPIROLITE PIPE

DESIGN VEHICLE:

120 TON EUCLID R50 (LOADED WEIGHT), USE MAX. ANTICIPATED LOAD (FOR SCRUBBER SLUDGE). REFERENCE "KEYSTONE ASH HAUL ROAD PAVEMENT" CALC. BY REL 5/2/95.

ASSUME THAT THE LIVE LOAD FROM THIS VEHICLE IS 6 $\left(\frac{120 \text{ TON}}{20 \text{ TON}}\right)$ TIMES AN H20 LOAD. NOTE THAT THIS

ASSUMPTION IS CONSERVATIVE WITH RESPECT TO SPEED AND IMPACT, SINCE THE DESIGN SPEED ON THE HAUL ROAD IS 15 MPH (AS PER JMJ).

DESIGN LIVE LOAD

∴ DESIGN LIVE LOAD = 6 x H20 LOAD

AT DOWNSTREAM EDGE OF PAVEMENT
DEPTH OF COVER = 6.0 FT - THICKNESS OF PIPE WALL
AT UPSTREAM EDGE OF PAVEMENT
DEPTH OF COVER = 4.7' - THICKNESS OF PIPE WALL

LIVE LOAD

USE THICKNESS OF PIPE WALL = 2.5 IN = 0.2 FT, CLASS 160 42"ϕ PIPE

FOR DEPTH OF COVER = (6.0 - 0.2) FT = 5.8 FT
H20 LOAD = 275 psf FROM FIG. 7, SHEET 32
DESIGN LIVE LOAD = 6·275 psf = 1650 psf
FOR DEPTH OF COVER = (4.7 - 0.2) FT = 4.5 FT
H20 LOAD = 385 psf FROM FIG. 7, SHEET 32
DESIGN LIVE LOAD = 6·385 psf = 2310 psf

SUBJECT KEYSTONE

PHASE II PERMITTING

BY SER DATE 7/16/96 PROJ. NO. 92-220-73-7

CHKD. BY MRL DATE 7/24/96 SHEET NO. 29 OF 53

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NEXT ESTIMATE EARTH LOAD

USE WALL THICKNESS = 0.2 FT CLASS 160 42" ϕ

ASSUME UNIT WEIGHT OF COVER = 120 pcf

FOR DEPTH OF COVER = 5.8 FT
EARTH LOAD = 5.8 FT · 120 pcf ≈ 700 psf
FOR DEPTH OF COVER = 4.5 FT
EARTH LOAD = 4.5 FT · 120 pcf = 540 psf

TOTAL LOAD

FOR DOWNSTREAM EDGE OF PAVEMENT
P_T = 1650 + 700 = 2350 psf
FOR UPSTREAM EDGE OF PAVEMENT
P_T = 2310 + 540 = 2850 psf

USE DESIGN TOTAL LOAD = 2900 psf

COMPRESSIVE STRENGTH

$$S_c = \frac{N D_o P}{288 A}$$ SEE SHEET 37

N = SAFETY FACTOR, USE 2.0
D_o = PIPE O.D., IN
P = TOTAL LOAD = 2900 psf
A = AVERAGE PROFILE AREA IN^2/IN
S_c = LONG TERM COMPRESSIVE STRESS = 1600 psi MAX.

$$S_c = \frac{2 \cdot D_o \cdot 2900\text{ psf}}{288 \cdot A} = 20.138 \frac{D_o}{A}$$

SUBJECT KEYSTONE
PHASE II PERMITTING
BY SER DATE 7/16/96 PROJ. NO. 92-220-73-7
CHKD. BY MRL DATE 7/24/96 SHEET NO. 30 OF 53

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CHART OF A, D_o, & S_c VERSUS CLASS
42" I.D. SPIROLITE FROM SHEETS 34 AND 35

CLASS	D_o (IN.)	A (IN^2/IN)	S_c (CALCULATED) PSI	
40	45.84	0.361	2560	NO GOOD
63	45.96	0.427	2170	NO GOOD
100	46.74	0.504	1870	NO GOOD
160	47.04	0.689	1370	< 1600 ∴ GOOD

USE CLASS 160 (#/FT)

DEFLECTION

$$\frac{Y}{D_i} = \frac{P}{144} \cdot \frac{0.1 \cdot L}{1.24 \cdot (RSC/D_i) + 0.061 E'}$$ SEE SHEET 38

Y = VERTICAL PIPE DEFORMATION, IN.
D_i = INSIDE ϕ = 42 IN.
P = LOAD ON PIPE = 2900 psf
RSC = RING STIFFNESS CONSTANT #/ft = 160 #/FT
L = DEFLECTION LAG FACTOR = 1.5 SEE SHEET 33
E' = MODULUS OF SOIL REACTION = 2000 psi FOR
85-95% STANDARD PROCTOR, COARSE GRAINED SOILS WITH
LITTLE OR NO FINES (SPEC. TO MATCH THIS REQUIREMENT)
SEE SHEET 36

$$\frac{Y}{42} = \frac{2900}{144} \cdot \frac{0.1 \cdot 1.5}{1.24 \cdot (160/42) + 0.061 \cdot 2000}$$

$$\frac{Y}{42} = 0.024$$ < 5.0% ∴ OK
SEE SHEET 33

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WALL BUCKLING

EVALUATE WALL BUCKLING EVEN THOUGH PIPE IS NOT BELOW GROUNDWATER.

I = REQUIRED MOMENT OF INERTIA IN^4/IN

$$I = \frac{P^2 N^2 D_M^3}{(5.65^2)\, R B' E' E}$$

P = LOAD = 2900 psf = 20 psi
N = SAFETY FACTOR = 2.0
D_M = MEAN DIA. = $(D_i + 2Z)$ = 42 + 2·0.74 = 43.48 IN.
R = 1.0 NO REDUCTION FOR GROUNDWATER

$$B' = \frac{1}{1+4e^{-0.065H}} = \frac{1}{1+4e^{-0.065 \cdot 5.8}} = 0.2671$$

H = 5.8 FT
E' = 2000 psi AS BEFORE
E = PIPE MOD. OF ELASTICITY = 42,200 psi SEE SHEET 37

$$I = \frac{20^2 \cdot 2^2 \cdot 43.48^3}{5.65^2 \cdot 1.0 \cdot 0.2671 \cdot 2000 \cdot 42{,}200} = 0.183\ IN^4/IN$$

ACTUAL I FOR 42"ϕ CLASS 160
I = 0.380 IN^4/IN > 0.183 IN^4/IN ∴ CLASS 160 IS OK

SUBJECT Keystone
Phase II Permitting

BY SER DATE 7/16/96 PROJ. NO 92-720-73-07
CHKD. BY MRL DATE 7/24/96 SHEET NO. 32 OF 53

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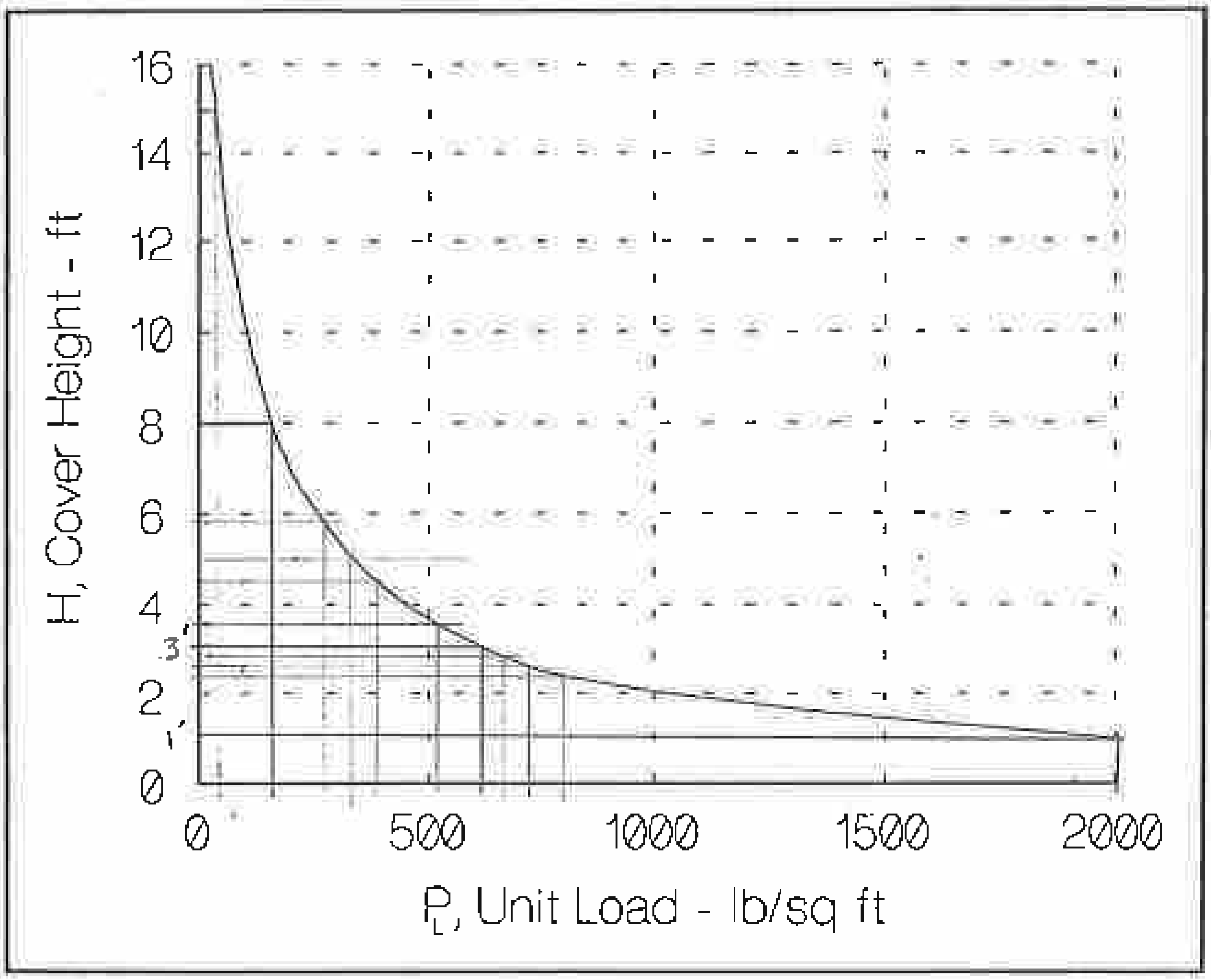
Figure 7 H20 and HS20 Highway Loading



Reference: Plexco/Spirolite Engineering Manual
Part 3, System DESIGN

SHEET 33/53

PIPE SELECTION

Spirolite pipe is manufactured in four standard ring stiffness classes. In preparing a specification, the designer selects a class of pipe appropriate for the application. The following tables may be used to assist the designer in making that selection. It is important that the designer perform all necessary calculations to verify the adequacy of a given class of pipe and be acquainted with all assumptions and installation requirements. Other design methods may be applicable.

The design of HDPE pipe for subsurface applications is typically based on the following performance limits: (1) wall crush strength, (2) constrained buckling resistance, and (3) deflection. Equations for these performance limits are given in the Appendix and were used to produce Table 1 and Table 2. The suitability of a class of pipe for installation at a given depth depends on the installation achieving the design E' and on the pipe being installed in accordance with ASTM D-2321 and the Spirolite Installation Guide. The designer is advised to review the applicability of these equations to each use of Spirolite.

The classes and depths shown in the tables are based on a design soil weight (dry or saturated) of 120 lbs/ft^3 and an applied H-20 live load. (Where live load is present, Spirolite pipe normally requires a minimum depth of cover of one pipe diameter or three feet whichever is greater. Where this condition cannot be met, please consult Plexco/Spirolite.) The earth load for calculating crush resistance was found using the arching coefficients given in Figure 10. The prism load was used for buckling and deflection calculations. Deflection was calculated using 75% of the E' value given at the top of the respective column, a deflection lag factor of 1.5, and a deflection limit of 5 percent. Buckling was calculated using the E' value listed and a long-term pipe modulus value of 28,250 psi. Buckling resistance was considered only for pipe subjected to ground water, as buckling is normally not a controlling factor for dry ground installations in the range of depths given in the tables. A safety factor of two was applied to the crush and buckling values.

BURIAL ABOVE GROUND WATER LEVEL

Table 1 is based on calculations made assuming the ground water level is always below pipe grade elevation. For other sizes, and burial depths or conditions not listed, consult with Plexco/Spirolite.

Table 1: SPIROLITE PIPE CLASS SELECTION FOR BURIAL *ABOVE* THE GROUND WATER LEVEL

Pipe Diameter	18 INCH			21 INCH			24 INCH			27 INCH			30 INCH			33 INCH			36 INCH			42 INCH		
E'	1000	2000	3000	1000	2000	3000	1000	2000	3000	1000	2000	3000	1000	2000	3000	1000	2000	3000	1000	2000	3000	1000	2000	3000
Depth of Cover (ft.) 2	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40
4	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40
6	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40
8	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40
10	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40
12	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40
14	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40
16	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40
18	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40
20	63	40	40	63	40	40	63	40	40	100	40	40	100	40	40	100	63	63	100	63	63	160	63	63
22	160	40	40	160	40	40	160	40	40		40	40		40	40		63	63		63	63		63	63
24		40	40		40	40		40	40		63	63		63	63		63	63		100	100		100	100
26		40	40		40	40		63	63		63	63		100	100		100	100		100	100		100	100
28		40	40		40	40		63	63		63	63		100	100		100	100		100	100		160	160
30		40	40		63	63		100	100		100	100		100	100		100	100		100	100		160	160
32		40	40		100	100		100	100		160	160		160	160		160	160		160	160		160	160
34		40	40		100	100		100	100		160	160		160	160		160	160		160	160		160	160
36		40	40		100	100		100	100		160	160		160	160		160	160		160	160		160	160
38		100	100		100	100		100	100		160	160		160	160									

Note: See text regarding live load.

SHEET 34/53

PIPE PROPERTIES

The following tables provide nominal dimensions and properties for **Spirolite®** pipe. Figure 6 shows a typical cross section of each profile and its derived properties.

NOTE: "Se" is the effective wall thickness required in a solid wall section yielding the same moment of inertia.

FIGURE 6: CROSS SECTION OF SPIROLITE PIPE

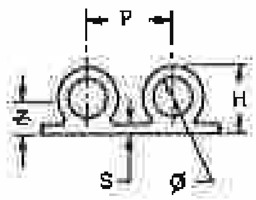


TABLE 3: SPIROLITE PIPE NOMINAL DIMENSIONS AND PROPERTIES *CLASS 40*

I.D. (In.)	Allowable Crush Load (Lb./Ft.²)*	P (Period) (In.)	H (Wall Height) (In.)	S (Wall) (In.)	Ø (Core Dia.) (In.)	I (Wall Moment) (In.⁴/In.)*	Se (Effective Wall) (In.)	A (Average Profile Area) (In²/In.)*	Z (Centroid) (In.)
18	2854	5.50	1.47	0.21	1.18	0.031	0.808	0.260	0.30
21	2498	5.50	1.47	0.21	1.18	0.031	0.808	0.260	0.30
24	2221	5.50	1.47	0.21	1.18	0.031	0.808	0.260	0.30
27	2125	5.00	1.49	0.21	1.18	0.038	0.859	0.277	0.33
30	2032	5.00	1.53	0.21	1.18	0.047	0.916	0.295	0.36
33	1867	5.70	1.85	0.22	1.57	0.077	1.073	0.299	0.42
36	1784	5.70	1.86	0.23	1.57	0.078	1.079	0.309	0.42
42	1810	5.60	1.92	0.27	1.57	0.095	1.143	0.361	0.44
48	1706	5.50	1.96	0.27	1.57	0.119	1.215	0.386	0.49
54	1579	5.60	2.27	0.27	1.96	0.169	1.375	0.403	0.55
60	1554	5.60	2.32	0.30	1.96	0.194	1.432	0.446	0.57
66	1612	5.40	2.37	0.33	1.96	0.227	1.503	0.496	0.60
72	1577	5.00	2.39	0.33	1.96	0.266	1.570	0.527	0.65
84	1737	5.00	2.55	0.43	1.96	0.369	1.745	0.673	0.72
96	1731	4.20	2.59	0.43	1.96	0.474	1.891	0.762	0.81

TABLE 4: SPIROLITE PIPE NOMINAL DIMENSIONS AND PROPERTIES *CLASS 63*

I.D. (In.)	Allowable Crush Load (Lb./Ft.²)*	P (Period) (In.)	H (Wall Height) (In.)	S (Wall) (In.)	Ø (Core Dia.) (In.)	I (Wall Moment) (In.⁴/In.)*	Se (Effective Wall) (In.)	A (Average Profile Area) (In²/In.)*	Z (Centroid) (In.)
18	2854	5.50	1.47	0.21	1.18	0.031	0.808	0.260	0.30
21	2586	5.40	1.49	0.21	1.18	0.035	0.842	0.270	0.32
24	2486	5.10	1.53	0.21	1.18	0.048	0.912	0.293	0.36
27	2455	4.70	1.57	0.21	1.18	0.061	0.985	0.322	0.41
30	2233	5.70	1.88	0.25	1.57	0.081	1.091	0.329	0.42
33	2237	5.70	1.92	0.27	1.57	0.094	1.137	0.359	0.44
36	2155	5.50	1.94	0.27	1.57	0.107	1.182	0.374	0.47
42	2134	4.60	1.98	0.27	1.57	0.146	1.303	0.427	0.55
48	2018	5.08	2.34	0.32	1.96	0.194	1.432	0.460	0.56
54	1950	5.70	2.39	0.33	1.96	0.238	1.519	0.500	0.61
60	1956	4.80	2.41	0.33	1.96	0.294	1.622	0.552	0.68
66	2147	4.70	2.52	0.42	1.96	0.356	1.729	0.664	0.71
72	2138	4.40	2.56	0.42	1.96	0.427	1.828	0.718	0.77
84	2287	4.00	2.70	0.52	1.96	0.577	2.013	0.890	0.86
96	2637	4.00	2.98	0.80	1.98	0.766	2.208	1.170	0.91

SHEET 35/53

PIPE PROPERTIES

TABLE 5: SPIROLITE PIPE NOMINAL DIMENSIONS AND PROPERTIES *CLASS 100*

I.D. (In.)	Allowable Crush Load (Lb./Ft.^{2})*	P (Period) (In.)	H (Wall Height) (In.)	S (Wall) (In.)	Ø (Core Dia.) (In.)	I (Wall Moment) (In.^{4}/In.)*	Se (Effective Wall) (In.)	A (Average Profile Area) (In^2/In.)*	Z (Centroid) (In.)
18	3147	4.90	1.51	0.21	1.18	0.044	0.893	0.288	0.35
21	3089	4.30	1.55	0.21	1.18	0.059	0.980	0.324	0.41
24	3334	3.80	1.61	0.25	1.18	0.077	1.066	0.395	0.44
27	2686	5.60	1.92	0.27	1.57	0.097	1.143	0.361	0.44
30	2666	4.80	1.94	0.27	1.57	0.119	1.224	0.394	0.50
33	2627	4.70	1.98	0.27	1.57	0.144	1.296	0.423	0.54
36	2692	4.40	2.02	0.29	1.57	0.171	1.363	0.470	0.58
42	2472	5.20	2.37	0.33	1.96	0.234	1.518	0.504	0.61
48	2470	4.50	2.41	0.33	1.96	0.305	1.648	0.569	0.70
54	2705	4.20	2.52	0.42	1.96	0.387	1.777	0.696	0.74
60	2712	4.00	2.58	0.42	1.96	0.485	1.905	0.770	0.83
66	2830	4.00	2.69	0.51	1.96	0.571	2.006	0.880	0.86
72	2987	4.00	2.82	0.62	1.96	0.678	2.120	1.010	0.89
84	3385	4.00	3.14	0.94	1.96	0.921	2.342	1.330	0.98
96	3663	4.00	3.45	1.25	1.96	1.210	2.560	1.640	1.08

TABLE 6: SPIROLITE PIPE NOMINAL DIMENSIONS AND PROPERTIES *CLASS 160*

I.D. (In.)	Allowable Crush Load (Lb./Ft.^{2})*	P (Period) (In.)	H (Wall Height) (In.)	S (Wall) (In.)	Ø (Core Dia.) (In.)	I (Wall Moment) (In.^{4}/In.)*	Se (Effective Wall) (In.)	A (Average Profile Area) (In^2/In.)*	Z (Centroid) (In.)
18	3982	4.80	1.63	0.25	1.18	0.071	1.033	0.369	0.42
21	4249	3.80	1.67	0.27	1.18	0.096	1.135	0.440	0.48
24	3257	5.10	1.96	0.27	1.57	0.124	1.238	0.397	0.50
27	3227	4.70	2.00	0.27	1.57	0.157	1.327	0.436	0.56
30	3425	3.70	2.02	0.29	1.57	0.194	1.422	0.508	0.62
33	3034	5.30	2.37	0.33	1.96	0.232	1.510	0.500	0.61
36	3041	4.70	2.39	0.33	1.96	0.276	1.594	0.541	0.66
42	3358	4.30	2.52	0.42	1.96	0.380	1.767	0.689	0.74
48	3363	4.00	2.59	0.43	1.96	0.491	1.913	0.780	0.83
54	3661	4.00	2.76	0.58	1.96	0.616	2.056	0.950	0.87
60	3937	4.00	2.94	0.74	1.96	0.764	2.204	1.130	0.92
66	4223	4.00	3.14	0.94	1.96	0.921	2.342	1.330	0.98
72	4466	4.00	3.34	1.14	1.96	1.100	2.482	1.530	1.04
84	4751	4.00	3.70	1.50	1.96	1.497	2.741	1.890	1.18
96	4946	4.00	4.05	1.85	1.96	1.995	3.006	2.240	1.33

*Properties are based on minimum profile dimensions

DEFLECTION CONTROL

A realistic approach to deflection control in flexible pipe installations involves assessment of the deflection occuring during installation as well as that occuring due to the service loads, i.e. soil and superimposed loading.

The placement and compaction of bedding material tends to deform flexible pipe, at times causing more deflection than the service load. The lateral forces acting on a pipe during the compaction of the embedment material between the pipe's invert and springline tend to produce a slight increase in the pipe's vertical diameter. This is known as "rise." Rise can be an advantageous property, as it will offset service load deflection.

Because a flexible conduit interacts with the surrounding soil, the nature of the pipe embedment material and the quality of its placement are important to the control of deflection. Some conduit deflection is natural, and is essential to the development of necessary soil support. However, the maximum deflection at any point along a pipe must be limited to safeguard its performance capabilities (such as joint tightness) and to protect pipe walls from excessive stressing. Consequently, one of the key objectives in the selection and installation of a flexible pipe is deflection control.

Spirolite® can withstand large amounts of deflection because of its ductility and ability to relieve stress under load. However, common design practice is to limit long term deflection to 7.5%.

The primary contributor to deflection control is the support provided by the pipe embedment material. Support is the result of mobilization of passive resistance in the embedment material during horizontal deflection of the pipe. The amount of support is measured by and directly proportional to a constant known as the modulus of soil reaction (E'). Values of the modulus of soil reaction are given in Figure 8.

The effect on pipe deflection of various levels of side support versus pipe ring stiffness is illustrated in Figure 7.

It should be noted that, with a modulus of soil reaction of 1000 psi at a burial depth of 10 feet, there is virtually no difference in the amount of anticipated deflection regardless of pipe class. Additionally, a Class 100 pipe buried to a depth of 10 feet may, depending on the quality of the pipe's embedment (E'), deflect substantially more than a Class 40 pipe buried to a depth of 16 feet. The greater E' enables the more flexible pipe, under substantially greater load, to see considerably less deflection.

Studies, and extensive field experience, show this to be the case and indicate that the vertical deflection of buried flexible pipes is about equal to the vertical compression (soil strain) of the pipe's sidefill.

FIGURE 7

VERTICAL DEFLECTION (%)*			
	E' = 1000	E' = 2000	E' = 3000
Depth of Cover = 10'	%	%	%
Class 40	2.8	1.4	.9
Class 63	2.8	1.4	.9
Class 100	2.7	1.4	.9
Depth of Cover = 16'	%	%	%
Class 40	4.0	2.0	1.4
Class 63	4.0	2.0	1.3
Class 100	4.0	2.0	1.3

*(1) 36" Pipe *(2) Soil Weight = 120 lb./ft.³ *(3) With H 20 loading

FIGURE 8: VALUES OF E' FOR SPIROLITE PIPE

Class ASTM D-2321	Soil type for pipe bedding material (Unified Classification System*1)	Dumped	Slight <85% Std. Proctor*3 <40% Rel. Den.*4	Moderate 85-95% Std. Proctor 40-70% Rel. Den.	High >95% Std. Proctor >70% Rel. Den.
I	Crushed Rock Manufactured angular, granular material with little or no fine. (¼"-1½")	1,000	3,000	3,000	3,000
II	Coarse-grained Soils with Little or no Fines GW, GP, SW, SP*2 containing less than 12 percent fines (maximum particle size 1½")	NR	1,000	2,000	3,000
III	Coarse-grained Soils with Fines GM, GC, SM, SC*2 containing less than 12 percent fines (maximum particle size 1½")	NR	NR	1,000	2,000
IV(a)	Fine-grained Soil (LL<50) Soils with medium to no plasticity CL, ML, ML-CL, with more than 25 percent coarse-grained particles	NR	NR	1,000*5	2,000*5
IV(b)	Fine-grained Soils (LL>50) Soils with high plasticity CH, MH, CH-MH Fine-grained Soils (LL<50) Soils with medium to no plasticity CL, ML, ML-CL with less than 25 percent coarse-grained particles	NR	NR	NR	NR
	Accuracy in terms of Percentage Deflection	+2	+2	+1	+0.5

*1. ASTM Designation D-2487, USBR Designation E-3.
*2. Or any borderline soil beginning with some of these symbols (i.e., GM, GC, GC-SC).
*3. Percent Proctor based on laboratory maximum dry density from test standards using about 12,500 ft. - lb./ft.³ (598,000 joules/m³)(ASTM D-698, AASHTO-99, USBR Designation E-11).
*4. Relative Density per ASTM D-2049.
*5. Under some circumstances Class IV(a) soils are suitable as primary initial backfill. They are not suitable under heavy dead loads, dynamic loads, or beneath the water table. Compact with moisture content at optimum or slightly dry of optimum. Consult a Geotechnical Engineer before using.

NOTES:
1. Organic soils OL, OM, and PT as well as soils containing frozen earth, debris, and large rocks are not recommended for initial backfill.
2. NR = Use not recommended per ASTM D-2321.
3. LL = Liquid Limit.
4. For shovel-sliced Class I material, E' typically equals 1000.

Figure 8 based on: **Bureau Of Reclamation Values Of E' For Iowa Equation**

SHEET 37/53

APPENDIX

This section provides a detailed approach to selection of the proper class of pipe for a specific subsurface installation. An example of this approach is also included.

The following considerations apply in the selection of **Spirolite®** as well as other flexible pipes: resistance to crush, resistance to buckling, and resistance to deflection due to construction and service loads.

Selection of a class of **Spirolite®** pipe generally depends on the crushing resistance of the pipe wall rather than on the anticipated deflection of the pipe. In cases where the pipe is buried beneath the ground-water table, the constrained buckling resistance of the pipe must also be considered. Pipe class has little influence on long term service load deflection in most installations. Deflection is controlled by the enveloping soil stiffness, as shown in the section "Deflection Control."

The class of **Spirolite®** pipe selected for a given application should have allowable crush and buckling loads in excess of the service load. The service load includes traffic loads, earth load, and surcharge load.

WALL CRUSH STRENGTH

The allowable crushing load for a confined conduit is determined by the compressive strength of its walls. The allowable crushing loads for all **Spirolite®** sizes and classes are listed in Tables 3-6. These values have been calculated using the following equation.

EQUATION 2

$$P_c = \frac{288\ AS_c}{ND_o}$$

Where P_c = allowable crushing load (lbs./ft.²)
S_c = long term compressive stress (psi) — 1600 psi at 73.4°F.
N = safety factor (generally taken as 2)
A = average profile area (in ²/in.)—See Tables 3-6
D_o = pipe outside diameter (in.) = pipe inside diameter +2 times wall height—See Tables 3-6

NOTE: The constant in this equation includes the appropriate units conversion factor.

CONSTRAINED BUCKLING RESISTANCE

Occasionally, when pipe is buried below the groundwater table, wall buckling resistance will govern the class selection of **Spirolite®** pipe. Constrainment of pipe in a trench greatly increases its resistance to wall buckling under hydrostatic load. For a constrained pipe buried to a depth of cover greater than 4 feet, the following equation[1] may be used to determine the allowable buckling pressure.

EQUATION 3

$$P_{wc} = \frac{5.65}{N} \bullet \sqrt{\frac{R\ B'\ E'\ EI}{D^3_m}}$$

$$\text{Where } B' = \frac{1}{1 + 4e^{(-.065H)}}$$

P_{WC} = allowable constrained buckling pressure (psi)
H = height of cover (ft.)
R = buoyancy reduction factor = (1 - .33 H'/H) for H' < H
N = safety factor (generally taken as 2)
E' = modulus of soil reaction (psi)
H' = height of groundwater surface above pipe (ft.)
E = modulus of elasticity of pipe material (psi) (for pipe permanently beneath the water table, E typically equals 28,250 psi. When hydrostatically loaded for less than 3 months out of the year, E may be taken as 42,200 psi.)
I = moment of inertia of wall section (in. ⁴/in.)
D_m = (D_i + 2 Z) mean diameter (in.)

HYDROSTATIC COLLAPSE RESISTANCE

In the special case of underwater installations where the pipe is submerged directly in water or other fluids, the pipe's allowable hydrostatic collapse pressure may be determined by the following equation:

EQUATION 4

$$P_W = \frac{24EI}{(1-\mu^2)\ D^3_m} \bullet \frac{C}{N}$$

Where P_W = critical hydrostatic collapse pressure of unconstrained pipe (psi)
E = modulus of elasticity of pipe material (psi) (Ranges from 113,000 psi for short term loading at 73.4°F. to 25% of that value for continuous long term loading)
I = moment of inertia of wall section (in. ⁴/in.) —See Tables 3-6
μ = Poisson's ratio for pipe material (ranges from about 0.35 for short term loading to 0.48 for long term loading.)
D_m = (D_i + 2 Z) inside pipe diameter (in.)
D_i = inside pipe diameter (in.)
Z = distance from inner pipe surface to the centroid of the wall section (in.)—See Tables 3-6
C = ovality correction factor as follows:

Ovality	C
1%	0.91
2%	0.84
3%	0.76
4%	0.70
5%	0.64

N = safety factor (generally taken as 2.5)

[1] "Recommendations for Elastic Buckling Design Requirements for Buried, Flexible Pipe," Proceedings, Part 1, AWWA 1982 Annual Conference, "Better Water for the Americas."

SHEET 38/53

RING STIFFNESS CONSTANT (RSC)

Pipe's sensitivity to deflection rise during installation is controlled by the pipe's ring stiffness. Ring stiffness is defined in terms of the deflection resulting from the load applied between parallel plates. The Ring Stiffness Constant (RSC) is the value obtained by dividing the parallel plate load in pounds per foot of pipe length by the resulting deflection in percent, at 3% deflection. (As described in ASTM F-894.)

EQUATION 5

$$RSC = \frac{6.44\,EI}{D_m^2}$$

Where RSC = ring stiffness constant (parallel plate load in pounds per foot of pipe which causes a 1% reduction in diameter)
- I = moment of inertia of wall section (in.^{4}/in.)—See Tables 3-6
- E = short term modulus of pipe material (113,000 psi @ 73.4°F.)
- D_m = $(D_i + 2\,Z)$ = mean diameter (in.)
- D_i = vertical inside diameter of pipe prior to loading (in.)
- Z = distance from inner pipe surface to the centroid of the wall section (in.)—See Tables 3-6

The nominal ring stiffness constant of a specific **Spirolite®** pipe can be directly related to the pipe's class designation. That is, a Class 40 pipe has a nominal ring stiffness constant of 40, the RSC of Class 63 is 63, and so forth. The minimum RSC for any diameter of pipe within a class is 90% of the class nominal value.*

The classes are shown in Tables 3-6. All sizes of pipe in the same class will deflect uniformly under parallel plate load, i.e. the same parallel plate load will produce approximately the same percent of deflection in all pipe of a given class. For example, any Class 40 pipe will deflect approximately 2% under an 80 lb./lineal ft. load.

To further illustrate this, consider a Class 40 pipe, which is the most flexible **Spirolite®** pipe. Although the exact force applied to a flexible pipe during compaction is not easily calculated, it is known that, for ordinary levels of compactive effort, Class 40 pipe possesses adequate stiffness to achieve a beneficial amount of rise while not impeding the installation or creating significant stresses in the pipe wall. Field observation indicates a typical rise of one or two percent in the vertical diameter. However, variations in embedment materials, their placement, and in compactive techniques make it difficult to estimate rise prior to the actual installation.

Beyond initial installation, pipe stiffness plays an insignificant role in controlling deflection.

*The minimum value of RSC for Spirolite® pipe is approximately the same as the minimum value for flexible culverts given in the AASHTO Interim Design Specification, 1981.

ESTIMATING DEFLECTION

Total deflection of a flexible pipe includes both the deflection incurred during installation and the deflection due to soil and superimposed loads. Most proposed relationships for estimating deflection deal only with the latter loads. However, sufficient empirical data exists to make reasonable estimates of total deflection.

A well known relationship for calculating the average vertical deflection in a buried flexible pipe resulting from soil loading only is Spangler's Modified Iowa Equation. This equation, as shown below is modified and expressed in terms of RSC values and assumes a bedding constant of K=0.1 (for typical bedding support).

The U.S. Bureau of Reclamation (USBR) and others have investigated the load/deflection relationship of buried flexible pipe. As a result of hundreds of field measurements, and computer analysis, a series of soil reaction (E') values were developed for use with the above Equation. These E' values are useful in estimating the initial deflection resulting from soil loading. They are presented in Figure 8 in terms of the embedment materials.

EQUATION 6

$$\frac{Y}{D_i} = \frac{P}{144} \cdot \frac{(.1)\,L}{(1.24\,(RSC)/D_i) + 0.061\,E'}$$

Where
- Y = vertical pipe deformation (in.)
- D_i = inside pipe diameter (in.)
- P = load on pipe (lbs./linear ft.^{2})
- RSC = ring stiffness constant (lbs./linear ft.)—See Tables 3-6
- E' = modulus of soil reaction (psi)—See Figure 8
- L = deflection lag factor (Typical values range from 1.0 to 1.50)

NOTE: The constant in this equation includes the appropriate units conversion factor.

LIVE AND DEAD LOADS

In the design of buried pipelines, both earth loads and live loads must be considered for the proper selection of pipe classes.

Thus, the total load on a pipe is expressed by the following equation:

EQUATION 7

Total Load = Soil Load + Live Load

SHEET 39/53

EXAMPLE CALCULATION

This example provides a step-by-step approach for determining which class of **Spirolite®** is suitable for a specific installation. The example utilizes the three basic pipe properties of wall crush, constrained buckling resistance and deflection to select the proper class of pipe for this particular installation. For this example we will select a 60" **Spirolite®** pipe for installation with 18 feet of cover. The pipe will be 9 feet beneath the permanent water table. The native soil is clayey with a design unit weight of 120 pcf. The embedment material chosen for the job is coarse graded sand that is classified Class II per ASTM D-2321. The embedment material will be compacted to 90% Standard Proctor Density with an average E' value of 2000 psi (See Figure 8).

SELECT THE CLASS OF PIPE

1. First determine the total load on the pipe. Use the following values for this example:

Unit weight of soil	W = 120 pcf
Height of cover	H = 18 ft.
Live Load	L = 0 psf
Soil Arching Factor	F = .86 (See Figure 10)

Use Equation 8 to calculate the total load on the pipe:

$$P = WHF + L$$
$$= (120)(18)(.86) + 0$$
$$= 1858 \text{ psf}$$

2. Determine the pipe wall compressive strength requirement by evaluating the cross sectional area of the pipe wall. First, rearrange the terms in Equation 2:

$$A = \frac{N D_o P}{288 S_c}$$

Before solving this equation an outside diameter of the pipe must be determined. To compute D_o assume that Class 63 pipe will be used. (A small error in assuming D_o will have minimal effect on pipe selection.)

$$A = \frac{(2)\,[(60 \text{ in.} + (2)(2.41)]\,(1858)}{(288)(1600)}$$

Area Required = 0.523 in.^{2}

Using Tables 3-6 for 60" pipe search for a class of pipe sufficient to provide the required area. 60" Class 63 has an area of 0.552 which is greater than the required area of 0.523. Therefore, Class 63 is chosen to satisfy the wall compressive load.

3. Determine the pipe's constrained wall buckling resistance with Equation 3 by evaluating the required moment of inertia of the pipe wall. If the pipe is above the water table it is not normally required to check for buckling.

Rearrange the terms in Equation 3:

$$I = \frac{P_{WC}^2 N^2 D_m^3}{(5.65^2)\, RB'\, E'\, E}$$

where:

H = 18 ft.
H = 9 ft.
R = (1-.33 (9/18)) = 0.835

$$B' = \frac{1}{1 + 4e^{(-0.065H)}}$$

N = 2
E' = 2000 psi
E = 28250 psi
D_m = 60 + (2) (0.68) = 61.36 in.

P = WHF + L

Note: Use F = 1.0 for this evaluation – prism load

= (120) (18) (1.0) + 0 = 2160 psf (In psi: 2160/144 = 15 psi)

$$I = \frac{(15^2)(2^2)(61.36^3)}{(5.65^2)(0.835)(0.446)(2000)(28250)}$$

Required Moment of Inertia = 0.310 in.^{4}/in.

Again using Tables 3-6, search the 60" Moment of Inertia column (I) for a Moment of Inertia greater than or equal to 0.310 in/in. A pipe of Class 100 (I = 0.485) is required to satisfy the constrained wall buckling resistance equation.

4. The final design evaluation calculates the average initial pipe deflection. Use Spangler's Iowa Equation (Equation 6):

$$\frac{Y}{D_i} = \frac{P}{144} \cdot \frac{0.1L}{(1.24)(RSC)/D + .061\,E'}$$

Where:

P = WHF + Live Load (Note: Use F = 1.0 for this evaluation – prism load)
= (120) (18) (1) + 0 = 2160 psf

RSC = 100 (highest value selected from Steps 1-2)
L = 1.0
D = 60"
E' = 2000 psi
Y = Vertical pipe deformation (in.)

$$\frac{Y}{D_i} = \frac{2160}{144} \cdot \frac{(0.1)(1)}{(1.24)(100/60) + .061\,(2000)}$$

$$\frac{Y}{D} = 1.2\% \text{ Average Deflection}$$

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PIPE STRENGTH DESIGN (CONT.)

HAUL ROAD DIRTY WATER DITCH CULVERT - STAGE 4

42"Φ SPIROLITE PIPE

USE SAME DESIGN VEHICLE AND DESIGN LIVE LOAD AS STAGE 3 SEE SHEET 28

LIVE LOAD

AT DOWNSTREAM EDGE OF PAVEMENT
DEPTH OF COVER = 2.6 FT - WALL THICKNESS
AT UPSTREAM EDGE OF PAVEMENT
DEPTH OF COVER = 4.7 FT - WALL THICKNESS

USE THICKNESS OF PIPE WALL = 0.2 FT, CLASS 160 42"Φ PIPE

AT D.S. EDGE
DEPTH = 2.4 FT
H20 LOAD = 800 psf FROM SHEET 32
DESIGN LIVE LOAD = 6·800 psf = 4800 psf
AT U.S. EDGE
DEPTH = 4.5 FT
H20 LOAD = 385 psf FROM SHEET 32
DESIGN LIVE LOAD = 6 · 385 psf = 2310 psf

EARTH LOAD

USE WALL THICKNESS = 0.2 FT

AT D.S. EDGE
DEPTH = 2.4 FT
EARTH LOAD = 120 pcf · 2.4 FT = 290 psf
AT U.S. EDGE
DEPTH = 4.5 FT
EARTH LOAD = 120 pcf · 4.5 FT = 540 psf

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TOTAL LOAD

AT D.S. EDGE
TOTAL = 4800 + 290 = 5100 psf

AT U.S. EDGE
TOTAL = 2310 + 540 = 2850 psf

USE DESIGN TOTAL LOAD = 5100 psf NOT USED SEE CALCS BELOW.

COMPRESSIVE STRENGTH

$$N = \frac{288 \cdot A \cdot S_c}{P \cdot D_o}$$ USE CLASS 160

N = FACTOR OF SAFETY
D_o = 47.04" AS BEFORE
P = 5100 psf
A = 0.689 IN^2/IN AS BEFORE
S_c = 1600 psi ALLOWABLE COMPRESSIVE STRENGTH

$$N = \frac{288 \cdot 0.689 \cdot 1600}{5100 \cdot 47.04} = 1.3$$ NOT ACCEPTABLE

CLASS 160 WILL NOT WORK FOR GIVEN LOADING

REEVALUATE LOADING CONDITIONS

LOADED TRUCKS, 120 TON, WILL TRAVEL IN THE RIGHT LANE HEADING ONTO THE PILE WHICH IS ON THE UPSTREAM SIDE OF THE PIPE. UNLOADED TRUCKS, 50 TON AS PER JMJ, WILL TRAVEL ON THE OPPOSITE SIDE. EVALUATE CONDITIONS AT THE MEDIAN AND REEVALUATE CONDITIONS AT THE DOWNSTREAM EDGE OF PAVEMENT.

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CONDITIONS AT MEDIAN

DEPTH OF COVER = 3.7 - 0.2 = 3.5 FT

EARTH LOAD = 120 pcf · 3.5 FT = 420 psf

H20 LOAD = 520 psf

LIVE LOAD = 6 · H20 = 6 · 520 psf = 3120 psf

TOTAL LOAD = 3120 + 420 = 3540 psf

CONDITIONS AT D.S. EDGE

DEPTH OF COVER = 2.4 FT

EARTH LOAD = 290 psf

H20 LOAD = 800 psf

LIVE LOAD = 2.5 · H20 USE 2.5 TIMES $\left(\frac{50\text{ TON}}{20\text{ TON}}\right)$ H20

LIVE LOAD = 2000 psf

TOTAL LOAD = 2000 + 290 = 2300 psf

USE 3500 psf FOR TOTAL LOAD

COMPRESSIVE STRENGTH USING CLASS 160

$$N = \frac{288 \cdot 0.689 \cdot 1600}{3500 - 47.04} = 1.9$$

A CLASS 160 PIPE HAS A FACTOR OF SAFETY OF 1.9 FOR COMPRESSIVE STRESS WHICH IS ACCEPTABLE.

SUBJECT KEYSTONE

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DEFLECTION

$$\frac{Y}{D_i} = \frac{P}{144} \cdot \frac{0.1 \cdot L}{1.24(RSC/D_i) + 0.061E'}$$

P = LOAD ON PIPE = 3500psf

ALL OTHER CONDITIONS ARE SAME AS THAT FOR THE STAGE 3 PIPE

$$\frac{Y}{D_i} = \frac{3500}{144} \cdot \frac{0.1 \cdot (1.5)}{1.24\left(\frac{160}{42}\right) + 0.061 \cdot (2000)}$$

$$\frac{Y}{D_i} = 0.029 < 5\% \therefore OK$$

SEE SHEET 33

WALL BUCKLING

EVALUATE WALL BUCKLING EVEN THOUGH PIPE IS NOT BELOW GROUNDWATER

$$I = \frac{P^2 N^2 D_M^3}{5.65^2 R B' E' E}$$ USE CLASS 160

P = 3500psf = 24. psi

N = 2 AS BEFORE

D_M = 43.48" AS BEFORE

R = 1 AS BEFORE

E' = 2000 psi AS BEFORE

E = 42,000 psi AS BEFORE

$$B' = \frac{1}{1 + 4e^{-0.065H}} = 0.239 \quad H = 3.5FT$$

$$I = \frac{24^2 \cdot 2^2 \cdot 43.48^3}{5.65^2 \cdot 1 \cdot 0.239 \cdot 2000 \cdot 42{,}000}$$

$I = 0.294$ IN^4/IN = REQUIRED I

ACTUAL I FOR 42"Ø CLASS 160 IS 0.380 IN^4/IN $\therefore$ OK

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ULTIMATE CONDITIONS SOUTHEAST DITCH CULVERT BENEATH HAUL ROAD

30" ⌀ BCCMP SEE SHEETS 47 TO 53 FOR DESIGN METHOD
USE NO CORRECTION FOR WALL THICKNESS OR CORRUGATION

AT DS EDGE OF PAVEMENT
DEPTH OF COVER = 3.0 FT
EARTH LOAD = 120 pcf · 3.0 FT = 360 psf
H20 LOAD = 620 psf FROM SHEET 32
LIVE LOAD = 6 · 620 psf = 3720 psf
TOTAL LOAD = (3720 + 360) psf = 4080 psf

AT US EDGE OF PAVEMENT
DEPTH OF COVER = 4.5 FT
EARTH LOAD = 120 pcf · 4.5 FT = 540 psf
H20 LOAD = 385 psf FROM SHEET 32
LIVE LOAD = 6 · 385 psf = 2310 psf
TOTAL LOAD = 2310 + 540 = 2900 psf

USE 4100 psf

RING COMPRESSION, C

$$C = P \cdot \frac{S}{2}$$

$S = 30'' = 2.5'$

$C = 4100 \text{ psf} \cdot \frac{2.5'}{2} = 5125 \text{ \#/FT}$

= RING COMPRESSION

ALLOWABLE WALL STRESS, f_c

$f_b = 33{,}000$ psi FROM SHEET 49 FOR 30" ⌀ PIPE AND $2\frac{2}{3}'' \times \frac{1}{2}''$ CORRUGATION

$$f_c = \frac{f_b}{2} = 16{,}500 \text{ psi}$$

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WALL CROSS-SECTIONAL AREA, A

$$A = \frac{C}{f_c} = \frac{5125\ \#/FT}{16{,}500\ psi} = 0.311\ IN^2/FT \text{ REQUIRED}$$

FROM TABLE 4.3 SHEET 50
ANY CORRUGATION OR WALL THICKNESS WILL WORK
USE SAME DESIGN AS FOR CULVERT AT INTERSECTION OF EAST VALLEY AND WEST VALLEY HAUL ROADS.

CULVERT AT INTERSECTION OF EAST VALLEY AND WEST VALLEY HAUL ROADS

36"Φ BCCMP
USE NO CORRECTION FOR WALL THICKNESS OR CORRUGATION AS BEFORE

AT DS EDGE OF PAVEMENT
DEPTH OF COVER = 2.8 FT
EARTH LOAD = 120 pcf · 2.8 FT = 340 psf
H20 LOAD = 670 psf
LIVE LOAD = 6 · 670 psf = 4020 psf
TOTAL LOAD = 4020 + 340 = 4400 psf

AT US EDGE OF PAVEMENT
DEPTH OF COVER = 5.0 FT
EARTH LOAD = 120 pcf · 5.0 FT = 600 psf
H20 LOAD = 330 psf
LIVE LOAD = 6 · 330 psf = 1980 psf
TOTAL LOAD = 1980 + 600 = 2600 psf

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RING COMPRESSION $C = P \cdot \frac{S}{2} = 4400 psf \cdot \frac{3'}{2} = 6600$ #/FT

$f_b = 33{,}000 psi$ FOR 36" ϕ SEE SHEET 49

$f_c = \frac{f_b}{2} = 16{,}500 psi$ = ALL. WALL STRESS

WALL CROSS-SECT. AREA, $A = \frac{C}{f_c} = \frac{6600 \text{ \#/FT}}{16500 psi} = 0.40$ IN²/FT

= GAGE 16

USE A THICKNESS OF 0.064 IN OR THICKER AND
USE 2⅔" x ½" CORRUGATIONS
SEE SHEET 50, TABLE 4.3
A = 0.775 IN²/FT ∴ OK

ALL OTHER CULVERTS

EACH OF THE OTHER CULVERTS WILL BE SUBJECT TO AT WORST CASE AN H20 (ACTUALLY MUCH LESS) LOADING.
LIST MIN AND MAX COVER FOR EACH CULVERT.

36" ϕ SE DITCH BENEATH WEST DIRTY WATER DITCH
MIN COVER = 2 FT
MAX COVER = 4.8 FT
18" ϕ NORTH TEMPORARY DIVERSION
MIN COVER = 5.4 FT
MAX COVER = 8.3 FT
36" ϕ SW DITCH
MIN COVER = 1.5 FT
MAX COVER = 1.5 FT

= GAGE 16

THE MIN AND MAX DEPTH OF COVER FOR EACH PIPE IS WITHIN THAT RECOMMENDED IN TABLE HC-1 SEE SHEET 53
USE MIN THICKNESS 0.064 IN AND 2⅔" x ½" CORRUGATIONS

SHEETS 47 TO 53 FROM "HANDBOOK OF STEEL DRAINAGE & HIGHWAY CONSTRUCTION PRODUCTS", AISI, 4th ED, 1993

SHEET 47 / 53

AIRPORT LOADS

The significance of aircraft loads is principally in the area of required minimum cover. Some modern airport design involves very heavy wheel loads of planes not yet designed. Projected wheel configurations and weights for airplanes weighing up to 1½ to 2 million pounds have been used to develop minimum cover tables for the Federal Aviation Administration. See Tables II C-23, -24, -25, and -26, pages 273 to 275.

Table 4.1 Highway and Railway Live Loads (LL)*

Highway Loading**			Railway E 80 Loading**	
Depth of Cover, Feet	Load, psf H 20	Load, psf H 25	Depth of Cover, Feet	Load, psf
1	1800	2280	2	3800
2	800	1150	5	2400
3	600	720	8	1600
4	400	470	10	1100
5	250	330	12	800
6	200	240	15	600
7	175	180	20	300
8	100	140	30	100
*	—	110	—	—

* See ASTM A 796

** Neglect live load when less than 100 psf; use dead load only.

DEAD LOADS

The dead load is considered to be the soil prism over the pipe. The unit pressure of this prism acting on the horizontal plane at the top of pipe is equal to

$$DL = w \times H \quad \ldots\ldots\ldots\ldots (1)$$

where w = Unit weight of soil, lb/ft³

H = Height of fill over pipe, ft

DL = Dead load pressure, lb/ft²

STRUCTURAL DESIGN OF BURIED STRUCTURES

The structural design process consists of the following:

1. Select the backfill soil density to be required or expected.
2. Calculate the design pressure.
3. Compute the compression in the pipe wall.
4. Select the allowable compressive stress.
5. Determine the thickness required.
6. Check minimum handling stiffness.
7. Check seam requirements (when applicable).
8. Check pipe-arches and arches.

1. Backfill Density

Select a percent compaction of pipe backfill for *design*. The value chosen should reflect the importance and size of the structure, and the quality that reasonably can be expected. The recommended value for routine use is 85%. This value easily will apply to ordinary installations in which most specifications will call for compaction to 90%. However, for more important structures in higher fill situations, select higher quality backfill and require the same in construction. This will extend the allowable fill height or save on thickness.

2. Design Pressure

When the height of cover is equal to or greater than the span or diameter of the structure, enter the load factor chart, Fig. 4.5, to determine the percentage of the total load acting on the steel. For routine use the 85% soil value will provide a factor of 0.86. The load factor, K, is applied to the total load to obtain the design pressure, P_v, acting on the steel. *If the height of cover is less than one pipe diameter, the total load is assumed to act on the pipe.*

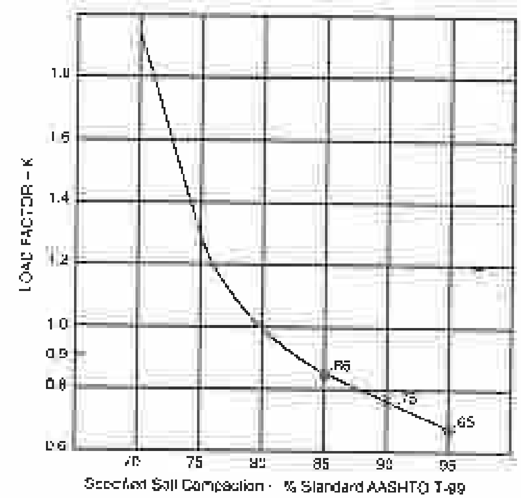


Figure 4.5 Load factors for corrugated steel pipe for backfill compacted to AASHTO T-99 density.

Total load on pipe becomes:

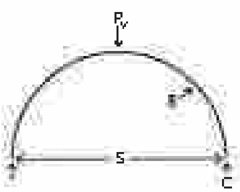
$$P_v = K \times (DL + LL), \text{ when } H \geq S \quad (2a)$$

$$P_v = (DL + LL), \text{ when } H < S \quad (2b)$$

where: P_v = Design pressure, in psf
K = Load factor
DL = Dead load, in psf
LL = Live load, in psf
H = Height of cover
S = Span

3. Ring Compression

The compressive thrust in the conduit wall is equal to the radial pressure acting on the wall multiplied by the wall radius or: $C = P \times R$. This thrust, called ring compression, is the force carried by the steel. The ring compression force acts tangentially to the conduit wall. For conventional structures in which the top arc approaches a semicircle, it is convenient to substitute half the span for the wall radius.



Then: $$C = P_v \times \frac{S}{2} \quad (3)$$

where: C = Ring compression, lb/ft
P_v = Design pressure, lb/ft^2
S = Span, in ft

4. Allowable Wall Stress

The ultimate compressive stresses, f_b, for corrugated steel structures with backfill compacted to 85% standard AASHTO density and a minimum yield point of 33,000 psi, are shown in Fig. 4.7. The ultimate compression in the pipe wall is expressed by the following equations: (4), (5), (6). The first is the specified minimum yield point of the steel which represents the *zone of wall crushing* or yielding. The second represents the *interaction zone of yielding* and ring buckling. And third, the *ring buckling zone*.

$$f_b = f_y = 33{,}000 \text{ psi, when } \frac{D}{r} < 294 \quad (4)$$

$$f_b = 40{,}000 - .081\left(\frac{D}{r}\right)^2, \text{ when } \frac{D}{r} > 294 \text{ and } < 500 \quad (5)$$

$$f_b = \frac{4.93\times10^9}{\left(\frac{D}{r}\right)^2}, \text{ when } \frac{D}{r} > 500 \quad (6)$$

where: D = Diam. or span, in.
r = Radius of gyration, in.

A factor of safety of 2 is applied to the ultimate wall stress to obtain the design stress, f_c.

$$f_c = \frac{f_b}{2} \quad (7)$$

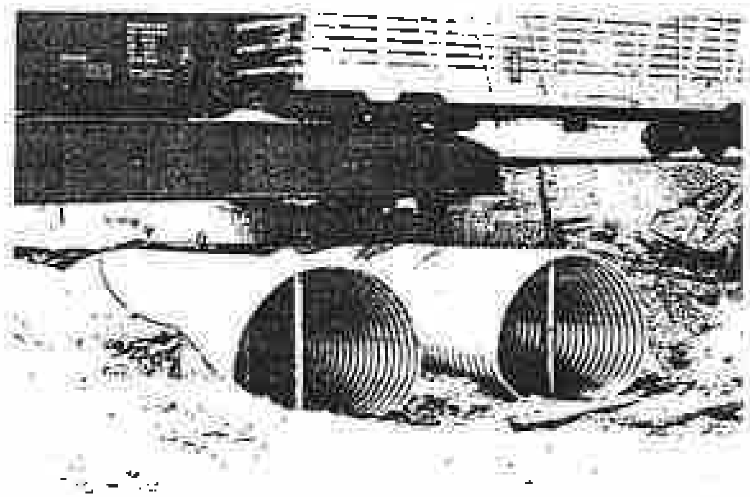


Figure 4.6 Twin corrugated steel pipe-arches are erected in place under an existing short span structure. This technique minimizes track down time and eliminates the need for a detour.

SHEET 48/53

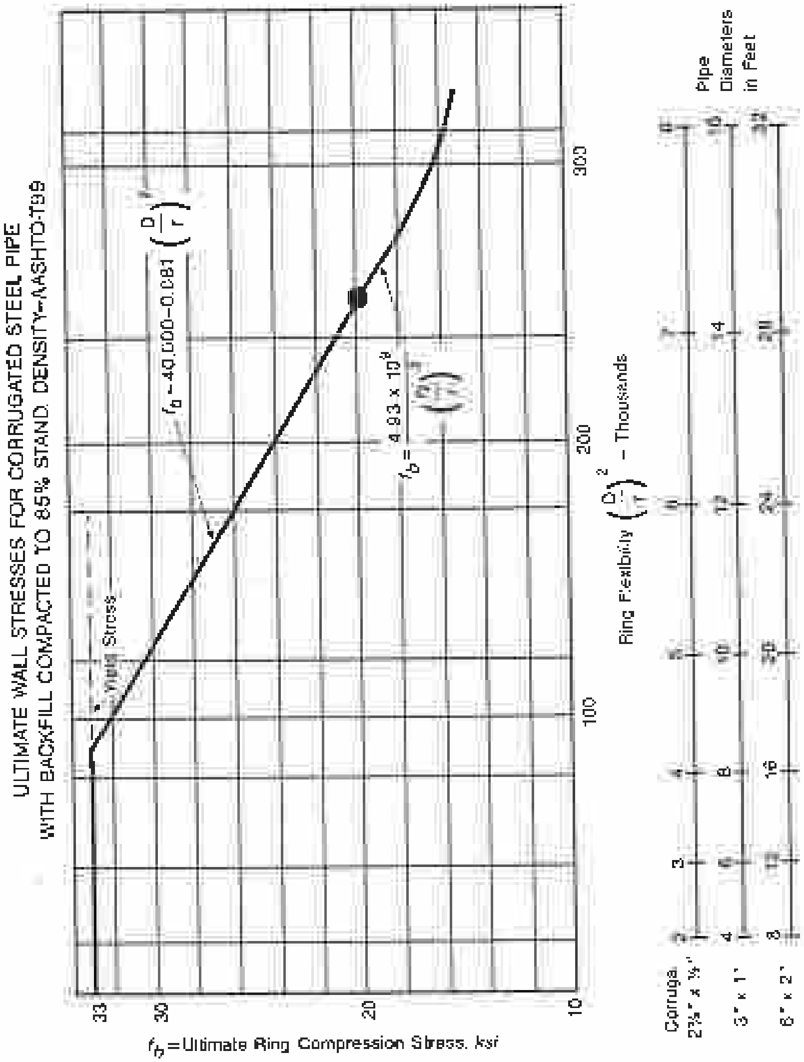


Figure 4.7 Ultimate wall or buckling stresses for corrugated steel pipe of various diameters and corrugations. The allowable stress is taken as one-half the ultimate.

5. Wall Thickness

Required wall area, A, is computed from calculated compression in the pipe wall, C, and the allowable stress, f_c.

$$A = \frac{C}{f_c} \quad \text{.......} (8)$$

From Table 4.3 select the wall thickness providing the required area in the same corrugation used to select the allowable stress.

6. Check Handling Stiffness

Minimum pipe stiffness requirements for practical handling and installation without undue care or bracing have been established through experience and formulated. The resultant flexibility factor, FF, limits the size of each combination of corrugation and metal thickness.

$$FF = \frac{D^2}{EI} \quad \text{.......} (9)$$

where: E = Modulus of elasticity = 30×10^6 psi
D = Diameter or span, in.
I = Moment of inertia of wall, in.^{4}/in.

Recommended maximum values of **FF** for ordinary installations:

FF = 0.0433 for factory-made pipe with riveted, welded, or helical seams

FF = 0.0200 for field-assembled pipe with bolted seams

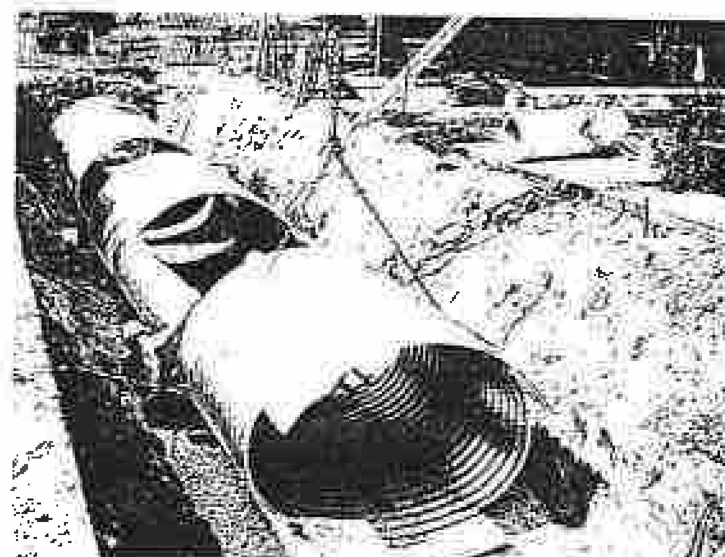


Figure 4.8 Preassembling structural plate pipe speeds installation.

SHEET 49/53

Increase the maximum values of FF for pipe-arch, arch and underpass shapes as follows:

Pipe-Arch FF = 1.5 × FF shown for round pipe
Arch FF = 1.5 × FF shown for round pipe
Underpass FF = same as shown for round pipe

Higher values can be used with special care or where experience has so proved. Trench condition, as in sewer design, is one example. Aluminum pipe experiences are another. For example, the flexibility factor permitted for aluminum pipe in some national specifications is more than twice that recommended above for steel. This has come about because aluminum has only one-third the stiffness of steel, the modulus for aluminum being approximately 10×10^6 psi vs 30×10^6 for steel. Where this degree of flexibility is acceptable in aluminum, it will be equally acceptable in steel.

For spiral rib pipe, a somewhat different approach is used. To obtain better control, the flexibility factors are varied with corrugation profile, sheet thickness, and type of installation, as shown in Table 4.2. The details of the installation requirements are given subsequently with the allowable fill heights in Table HC-2.

Table 4.2 Flexibility Factors for Spiral Rib Pipe

Installation Type	Flexibility Factor, in./lb						
	¾ × 1 × 11½ Corr.			¾ × ¾ × 7½ Corr.			
	Specified Thickness, Inches			Specified Thickness, Inches			
	0.064	0.079	0.109	0.064	0.079	0.109	0.138
I	0.022	0.025	0.028	0.022	0.025	0.029	0.0308
II	0.027	0.030	0.033	0.028	0.032	0.038	0.0426
III	0.033	0.040	0.044	0.038	0.044	0.050	0.0584

Table 4.3 Moment of Inertia (I) and Cross-Sectional Area (A) of Corrugated Steel Pipe for Underground Conduits

Corrugation Pitch × Depth, inches	Specified Thickness, Inches*									
	0.052	0.064	0.079	0.109 0.111	0.138 0.140	0.168 0.170	0.188	0.218	0.249	0.280
	Moment of Inertia, I, inches^4 per Foot of Width									
1½ × ¼	.0041	.0053	.0068	.0103	.0145	0.0196				
2 × ½	.0184	.0233	.0295	.0425	[illegible]	0.0719				
2⅔ × ½	.0180	.0227	.0287	.0411	.0544	0.0687				
3 × 1	.0327	.1039	.1306	.1855	.2421	0.3010				
5 × 1		.1052	.1331	.1873	.2430	0.3011				
6 × 2				.725	.938	1.154	1.296	1.523	1.754	1.990
¾ × ¾ × 7½**		.0431	.0569	.0858	0.1157					
¾ × 1 × 11½**		.0550	.0730	.1111						
	Cross-Sectional Wall Area, inches^2 per Foot of Width									
1½ × ¼	.608	.761	.950	1.331	1.712	2.090				
2 × ½	.652	.815	1.019	1.428	1.838	2.249				
2⅔ × ½	.619	.775	.968	1.356	1.744	2.133				
3 × 1	.711	.890	1.113	1.560	2.008	2.458				
5 × 1		.794	.992	1.390	1.788	2.196				
6 × 2				1.556	2.003	2.449	2.739	3.199	3.658	4.119
¾ × ¾ × 7½**		.511	.715	1.192	1.729					
¾ × 1 × 11½**		.374	.524	.883						

* Where two thicknesses are shown top is corrugated steel pipe and bottom is structural plate.
** Ribbed pipe. Properties are effective values.

7. Check Longitudinal Seams

Most pipe seams develop the full yield strength of the pipe wall. However, there are exceptions in standard pipe manufacture and these are identified here. Shown below are those standard riveted and bolted seams which do not develop a strength equivalent to fy = 33,000 psi. To maintain a consistent factor of safety of 2.0 for these pipes, it is necessary to reduce the maximum ring compression to one half the indicated seam strength. Nonstandard, or new longitudinal seam details should be checked for this same possible condition. Since helical lockseam and continuously-welded-seam pipe have no longitudinal seams, there is no seam strength check for these types of pipe.

Table 4.4 Ultimate Longitudinal Seam* Strengths (lb/ft)

Thickness, in.		6 x 2 in. 4 Bolts Per Ft	3 x 1 in.	2⅔ x ½ in. Rivet Seams		
Corrugated Steel Pipe	Structural Plate			5/16 in. Single Rivet	⅜ in. Single Rivet	⅜ in. Double Rivet
0.064			28,700[1]	16,700		
0.079			35,700[1]	18,200		
0.109	0.111	42,000			23,400	
0.138	0.140	62,000	53,700[2]		24,500	49,000
0.168			70,700[2]		25,600	51,200

* See Chapter 2 for seam details.
Standard seams not shown develop full yield strength of pipe wall.
[1] Double ⅜-in. rivets.
[2] Double 7/16-in. rivets.

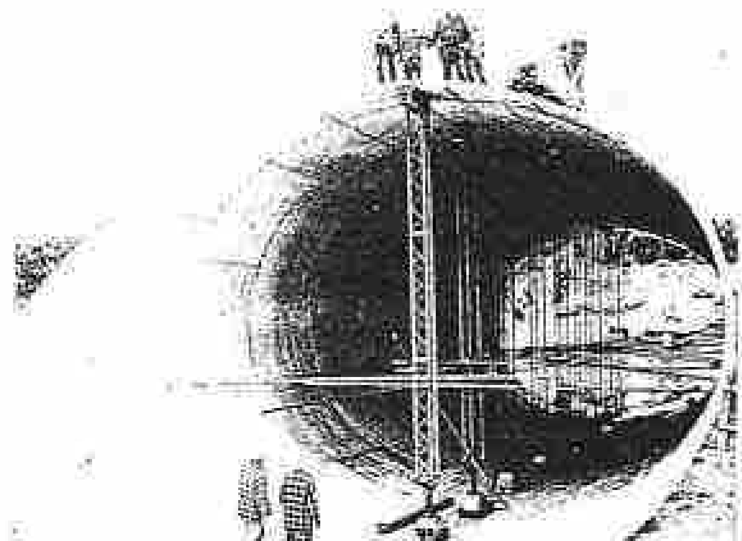


Figure 4.9 Tightening bolts on a long span structural plate culvert. Research underpass is fully instrumented prior to back filling.

SHEET 50/53

12.7.5 Multiple Structures

Care must be exercised on the design of multiple, closely spaced structures to control unbalanced loading. Fills should be kept level over the series of structures when possible. Significant roadway grades across a series of structures require checking of the stability of the flexible structures under the resultant unbalanced loading.

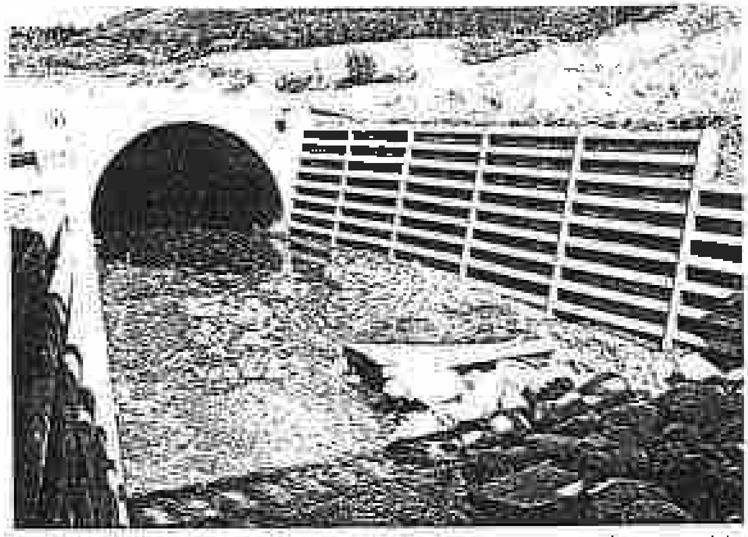
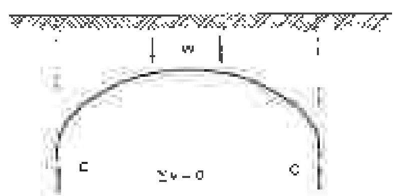


Figure 4.12 Soil-steel stream enclosure utilizing cellular steel or bin-type retaining walls to control erosive flows.

ALTERNATE METHOD OF CALCULATING RING COMPRESSION

In lieu of using twice the top arc radius times the design pressure (P_v) to calculate ring compression, a more accurate method is to calculate the vertical reaction at the horizontal springline. The ring compression C is then equal to this reaction. Since vertical forces must sum to zero ($\Sigma v = 0$), the ring compression in the pipe wall at the horizontal springline must be equal to one half the total weight over the span at that point. See Fig. 4.13 below.



$$C = \frac{W}{2} \qquad (12)$$

W = Total Weight of Soil and Live Loads over Structure

Figure 4.13 Alternate calculation of ring compression.

CORRUGATED STEEL BOX CULVERTS

The preceding design criteria does not apply to steel box culverts. The extreme geometry and shallow cover used with these structures require a different design method. Finite element computer programs have been employed to solve the indeterminate structural problems presented by the heavily stiffened box shapes. The results have been used to develop a special AASHTO design procedure as detailed in Reference 16. Height-of-cover limits are typically 1.4 to 5.0 feet and live loads are limited to H20 or H25. Individual manufacturers of these structures may also be contacted for details regarding design of this product.

ASTM STANDARD PRACTICES

A procedure for the structural design of pipe is provided by ASTM A 796, "Standard Practice for Structural Design of Corrugated Steel Pipe, Pipe-Arches, and Arches for Storm and Sanitary Sewers and Other Buried Applications." The practice applies to structures installed in accordance with A 798, "Standard Practice for Installing Factory-Made Corrugated Steel Pipe for Sewers and Other Applications," or A 807, "Standard Practice for Installing Corrugated Steel Structural Plate Pipe for Sewers and Other Applications." These practices are frequently referenced in project specifications.

The design procedure in A 796 is similar to that described in this chapter but differs in several respects. First, for the dead load, ASTM uses the weight of the entire prism of soil over the pipe and does not recognize the load reduction factor. It uses a more conservative form of the buckling equation. It provides flexibility factors for both trench and embankment conditions, some of which are more conservative than those listed here. It includes more specific information on acceptable soil types. In spite of all these differences, the resulting designs for typical projects will usually not differ greatly from those provided in this chapter.

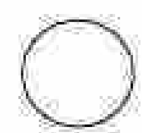
DESIGN EXAMPLES

EXAMPLES

The following examples illustrate the application of design procedures developed in the preceding pages. They include: (1) 48-in. diameter pipe under a 60-ft fill; (2) 120-in. diameter pipe under a 63-ft fill; (3) a 20-ft × 13-ft pipe-arch under 6 ft of cover; and (4) a 23-ft arch.

Example 1

Given: Pipe diameter required = 48 in.
Height of cover, H = 60 ft
Live load, LL = H 20
Weight of soil, (unit) w = 120 lb/ft^3



Find: Wall thickness and type of corrugation

SHEET 51/53

SOLUTION:

1. **Backfill Soil Density (compaction) Required:**
   90% Standard AASHTO specified. Assume a minimum of 85% for design. Height of cover is greater than span. Therefore, K = 0.86.

2. **Design Pressure:**
   $P_v = k\,(DL + LL)$
   where: $DL = H \times w = 60 \times 120 = 7200$ psf
   From Table 4.1, LL = negligible for cover greater than 8 ft
   Then $P_v = 0.86\ (7200 + 0) = 6190$ psf

3. **Ring Compression:**
   $C = P_v \times \dfrac{S}{2}$
   where: S = span, ft
   Then $C = 6190 \times \dfrac{4}{2} = 12{,}380$ lb/ft

4. **Allowable Wall Stress:**
   From Fig. 4.7, $f_b = 33{,}000$ psi for $2\frac{2}{3} \times \frac{1}{2}$ in. corrugation
   Then $f_c = f_b/2 = 16{,}500$ psi

5. **Wall Cross-Sectional Area:**
   $A = \dfrac{C}{f_c} = \dfrac{12{,}380}{16{,}500} = 0.750$ in.^{2}/ft required
   From Table 4.3 a specified *thickness* of 0.064 in. provides an uncoated wall *area* of 0.775 in.^{2}/ft

6. **Handling Stiffness:**
   $FF = \dfrac{D^2}{EI}$ = flexibility factor = 0.043 max.
   where: D = diameter = 48 in.
   E = modulus of elasticity = 30×10^6 psi
   I = moment of inertia, in.^{4}/in.
   From Table 4.3 for 0.064 in. specified thickness.
   I = 0.00189 in.^{4}/in.*

Then $FF = \dfrac{48^2}{30 \times 10^6 \times 0.00189} = 0.0406$

$0.0406 < 0.0433$; therefore $2\frac{2}{3} \times \frac{1}{2}$ in. corrugation is OK.

*Values in Table 4.3 are per ft. Divide by 12.

ALTERNATE SOLUTION—Using 5×1 in. Corrugated Pipe

4A. *Allowable Wall Stress:*
(using same computations) 16,500 psi

5A. *Wall Cross-Sectional Area:*
From Table 4.3 a specified *thickness* of 0.064 in. provides an uncoated wall *area* of 0.794 in.^{2}/ft

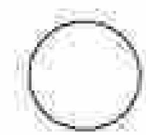
6A. *Handling Stiffness:*
From Table 4.3 for 0.064-in. specified thickness, $I = 0.00885$ in.^{4}/in.

Then $FF = \dfrac{48^2}{30 \times 10^6 \times 0.00885} = 0.0087$

$0.0087 < 0.0433$; therefore, 5×1 in. corrugation is OK.

ANSWER: A specified wall thickness of 0.064 in. is adequate for corrugated steel pipe of either $2\frac{2}{3} \times \frac{1}{2}$ in. or 5×1 in. corrugations.

Example 2



Given: Pipe diameter required = 120 in.
Height of cover, H = 65 ft
Live load, $LL = E\ 80$
Weight of soil, (unit) $w = 120$ lb/ft^3

Find: Wall thickness and type of corrugation (Try 5×1 in. and 6×2 in. corrugation)

SOLUTION:

1. **Backfill Soil Density Required:**
   90% Standard AASHTO specified. Assume a minimum of 85% for design. Height of cover is greater than span. Therefore, K = 0.86.

2. **Design Pressure:**
   $DL = H \times w = 65 \times 120 = 7800$ psf (*LL*, negligible)
   $P_v = 0.86\ (7800 + 0) = 6710$ psf

3. **Ring Compression:**
   $C = P_v \times \dfrac{S}{2} = 6710 \times \dfrac{10}{2} = 33{,}500$ lb/ft

4. **Allowable Wall Stress:**
   From Fig. 4.7 or Eq. 5, $f_b = 31{,}300$ psi for 5×1 in. corrugation

SHEET 52/53

Table HC-1 Height-of-Cover Limits for Corrugated Steel Pipe H 20 or H25 LIVE LOAD 2⅔ x ½ In. Corrugations

Diameter or Span, In.	Minimum* Cover, in.	Maximum Cover, ft				
		Specified Thickness, in.				
		0.064	0.079	0.109	0.138	0.168
12	12	248	310			
15		199	248			
18		166	207			
21		142	178	249		
24		124	155	218		
30		99	124	174		
36		83	103	145	166	
42		71	88	124	160	195
48		62	77	109	140	171
54			60	93	122	150
60				79	104	128
66				68	88	109
72					75	93
78						79
84	12					66

*Minimum covers are for H20 and H25 loads. See table 4.6 (page 283) for heavy construction load requirements. Minimum covers are measured from top of pipe to bottom of flexible pavement or top of pipe to top of rigid pavement. Minimum cover must be maintained in unpaved traffic areas.

Table HC-2 Height-of-Cover Limits for Spiral Rib Pipe H20 or H25 LIVE LOAD

Diameter or Span, in.	Minimum Cover, in.	Maximum Depth of Cover Above Top of Pipe (Feet)						
		¾ x 1 x 11½ in. Corrugation			¾ x ¾ x 7½ in. Corrugation			
		0.064	0.079	0.109	0.064	0.079	0.109	0.138
24	12	51	72	121	51	72	121	
30	12	41	58	97	41	58	97	
36	12	34	48	81	34	48	81	
42	12	29	41	69	29	41	60	
48	12	26	36	61	26	36	61	
54	18	23	32	54	[23]	32	54	
60	18	[21]	29	49	[21]	29	49	73
66	18	[19]	26	44		(26)	44	65
72	18		(24)	40		[24]	40	[illegible]
78	24		[22]	37		[22]	(37)	55
84	24		[21]	(35)			(35)	52
90	24			(32)			[32]	47
96	24			[30]			[30]	44
102	30			[29]			(29)	43
108	30			[27]				[41]

Notes: 1. Minimum covers are for H20 and H25 loads. See table 4.6 (page 283) for heavy construction load requirements. Minimum covers are measured from top of pipe to bottom of flexible pavement or top of pipe to top of rigid pavement. Minimum cover must be maintained in unpaved traffic areas.
2. TYPE 1 installations are allowed unless otherwise shown.
3. ( ) Requires TYPE II installation.
4. [ ] Requires TYPE III installation.

INSTALLATION AND BACKFILL OF SPIRAL RIB PIPE

Satisfactory backfill material, proper placement, and compaction are key factors in obtaining satisfactory performance.

Minimum pipe metal thickness (gage) is dependent upon minimum & maximum cover and installation TYPE I, II, or III, as noted in the fill height table. Backfill in the pipe envelope shall be granular materials with little or no plasticity; free from rocks, frozen lumps, and foreign matter that could cause hard spots or that could decompose and create voids; compacted to a minimum 90% standard density per ASTM D698 (AASHTO T99).

Installation types are:

Type I Installations can be in an embankment or fill condition. Installations shall meet ASTM A798 requirements. ML and CL materials are typically not recommended. Compaction equipment or methods that cause excessive deflection, distortion, or damage shall not be used.

Type II Installations require trench-like conditions where compaction is obtained by hand, or walk behind equipment, or by saturation and vibration. Backfill materials are the same as for TYPE I installations. Special attention should be paid to proper lift thicknesses. Controlled moisture content and uniform gradation of the backfill may be required to limit the compaction effort while maintaining pipe shape.

Type III Installations have the same requirements as TYPE II installations except that backfill materials are limited to clean, non-plastic materials that require little or no compaction effort (GP, SP), or to well graded granular materials classified as GW, SW, GM, SM, GC, or SC with a maximum plastic index (PI) of 10. Maximum loose lift thickness shall be 8". Special attention to moisture content to limit compaction effort may be required. Soil cement or cement slurries may be used in lieu of the selected granular materials.

Note: Simple shape monitoring — measuring the rise and span at several points in the run — is recommended as good practice with all types of installation. It provides a good check on proper backfill placement and compaction methods. Use soil placement and compaction methods which will insure that the vertical pipe dimension (rise) does not increase in excess of 5% of the nominal diameter. Use methods which will insure that the horizontal pipe dimension (span) does not increase in excess of 3% of the nominal diameter. These guidelines will help insure that the final deflections are within normal limits.

SHEET 53 / 53

SUBJECT PENELEC - KEYSTONE WEST VALLEY
PHASE II PERMITTING

BY KMB DATE 6/11/96 PROJ. NO. 92-220-73-07
CHKD. BY MRL DATE 6/17/96 SHEET NO. 1 OF 20

GAI CONSULTANTS, INC.
Engineers • Geologists • Planners
Environmental Specialists

STAGE 3 AND STAGE 4 TEMPORARY DIVERSIONS

AT PENELEC'S KEYSTONE DISPOSAL SITE, LANDFILL DEVELOPMENT WILL OCCUR IN THE SITE'S WEST VALLEY. THIS CALCULATION SECTION WILL DESIGN DIVERSION DITCHES THAT WILL DIRECT RUNOFF AROUND AREAS THAT WILL BE LINED DURING VARIOUS CONSTRUCTION STAGES.

THE DITCHES WILL BE SIZED TO CONVEY THE PEAK FLOW THAT RESULTS FROM RUNOFF FROM THE 25-YEAR 24-HOUR STORM. THE PRECIPITATION FROM THAT STORM IS 4.4 INCHES. (SEE NEXT PAGE)

TWO CONSTRUCTION STAGES (STAGE 3 AND STAGE 4) WILL NEED TO HAVE DIVERSIONS DESIGNED. DRAWINGS SHOWING THE DIVERSION CHANNEL VICINITY ARE ATTACHED.

2/20

TABLE 4.1 Pennsylvania Rainfall by Counties

COUNTY	24 HOUR RAINFALL FOR VARIOUS FREQUENCIES 1yr	2yr	5yr	10yr	25yr	50yr	100yr
ADAMS	2.5	3.0	3.9	4.8	5.3	6.0	6.7
ALLEGHENY	2.3	2.6	3.3	3.9	4.4	4.9	5.2
ARMSTRONG	2.3	2.6	3.3	3.9	4.4	4.9	5.2
BEAVER	2.3	2.6	3.2	3.8	4.3	4.7	4.9
BEDFORD	2.4	2.8	3.6	4.5	4.9	5.5	6.0
BERKS	2.5	3.1	4.1	4.9	5.5	6.1	6.9
BLAIR	2.4	2.8	3.6	4.3	4.8	5.3	5.8
BRADFORD	2.3	2.8	3.6	4.2	4.9	5.4	5.8
BUCKS	2.5	3.3	4.2	5.0	5.8	6.4	7.2
BUTLER	2.3	2.6	3.3	3.8	4.3	4.8	5.0
CAMBRIA	2.4	2.8	3.4	4.2	4.8	5.2	5.7
CAMERON	2.3	2.7	3.4	4.0	4.5	5.0	5.4
CARBON	2.5	3.0	4.0	4.8	5.3	6.0	6.7
CENTRE	2.3	2.8	3.6	4.3	4.8	5.4	5.8
CHESTER	2.6	3.2	4.2	5.0	5.6	6.3	7.1
CLARION	2.2	2.6	3.3	3.7	4.4	4.8	5.1
CLEARFIELD	2.3	2.7	3.5	4.0	4.6	5.1	5.5
CLINTON	2.3	2.8	3.6	4.2	4.8	5.3	5.7
COLUMBIA	2.4	2.9	3.7	4.6	5.1	5.7	6.2
CRAWFORD	2.2	2.5	3.1	3.6	4.2	4.7	4.8
CUMBERLAND	2.4	2.9	3.8	4.7	5.1	5.8	6.4
DAUPHIN	2.5	2.9	3.9	4.8	5.2	5.9	6.5
DELAWARE	2.6	3.3	4.2	5.0	5.7	6.4	7.3
ELK	2.3	2.7	3.4	3.9	4.5	4.9	5.3
ERIE	2.1	2.5	3.1	3.6	4.1	4.6	4.7
FAYETTE	2.4	2.7	3.4	4.1	4.6	5.1	5.6
FOREST	2.2	2.6	3.3	3.8	4.3	4.8	5.1
FRANKLIN	2.4	2.9	3.8	4.8	5.1	5.9	6.4
FULTON	2.4	2.8	3.7	4.6	4.9	5.6	6.2
GREENE	2.3	2.6	3.4	3.9	4.4	4.9	5.2
HUNTINGDON	2.4	2.8	3.7	4.6	4.9	5.5	5.9
INDIANA	2.3	2.7	3.4	4.0	4.6	5.0	5.4
JEFFERSON	2.3	2.6	3.4	3.9	4.5	4.9	5.3
JUNIATA	2.4	2.9	3.7	4.5	4.9	5.6	6.1
LACKAWANNA	2.4	2.9	3.9	4.7	5.2	5.8	6.5
LANCASTER	2.5	3.1	4.1	5.0	5.5	6.2	6.9
LAWRENCE	2.2	2.5	3.2	3.7	4.2	4.7	4.8
LEBANON	2.5	3.0	4.0	4.8	5.3	6.0	6.7
LEHIGH	2.5	3.1	4.1	4.9	5.5	6.1	6.9
LUZERNE	2.4	2.9	3.9	4.7	5.2	5.8	6.4
LYCOMING	2.4	2.8	3.6	4.3	4.9	5.5	5.9
MCKEAN	2.2	2.6	3.3	3.9	4.4	4.8	5.2
MERCER	2.2	2.5	3.2	3.7	4.2	4.7	4.8
MIFFLIN	2.4	2.8	3.6	4.4	4.8	5.5	6.0
MONROE	2.5	3.0	4.0	4.8	5.4	6.1	6.8
MONTGOMERY	2.6	3.2	4.2	5.0	5.7	6.4	7.1
MONTOUR	2.4	2.9	3.7	4.6	5.0	5.6	6.2
NORTHAMPTON	2.5	3.1	4.1	4.9	5.5	6.1	6.9
NORTHUMBERLAND	2.4	2.9	3.8	4.6	5.0	5.7	6.3
PERRY	2.4	2.9	3.8	4.6	5.0	5.7	6.3
PHILADELPHIA	2.6	3.3	4.3	5.0	5.7	6.4	7.3
PIKE	2.6	3.0	4.0	4.9	5.4	6.1	7.0
POTTER	2.3	2.7	3.4	4.0	4.6	5.0	5.4
SCHUYLKILL	2.5	3.0	3.9	4.7	5.3	5.9	6.5
SNYDER	2.4	2.9	3.7	4.5	5.0	5.6	6.1
SOMERSET	2.4	2.6	3.5	4.3	4.8	5.3	5.8
SULLIVAN	2.4	2.8	3.7	4.4	4.9	5.5	6.0
SUSQUEHANNA	2.4	2.9	3.8	4.5	5.0	5.7	6.2
TIOGA	2.3	2.7	3.5	4.2	4.7	5.1	5.6
UNION	2.4	2.8	3.7	4.4	4.9	5.5	6.0
VENANGO	2.2	2.5	3.3	3.7	4.2	4.7	4.9
WARREN	2.2	2.5	3.2	3.8	4.3	4.8	4.9
WASHINGTON	2.3	2.6	3.3	3.9	4.4	4.9	5.2
WAYNE	2.4	2.9	3.9	4.7	5.2	6.0	6.7
WESTMORELAND	2.3	2.7	3.4	4.0	4.6	5.0	5.4
WYOMING	2.4	2.9	3.8	4.5	5.0	5.6	6.2
YORK	[illegible]	3.1	4.1	4.9	5.5	6.2	6.9

DIVERSIONS CONTINUED

STAGE 3 DIVERSIONS

DURING STAGE 3 CONSTRUCTION, LINER WILL BE PLACED IN THREE SEQUENTIAL OPERATIONS - 3A, 3B, AND 3C. THESE AREAS ARE SHOWN ON THE DRAWING PRESENTED ON THE NEXT PAGE.

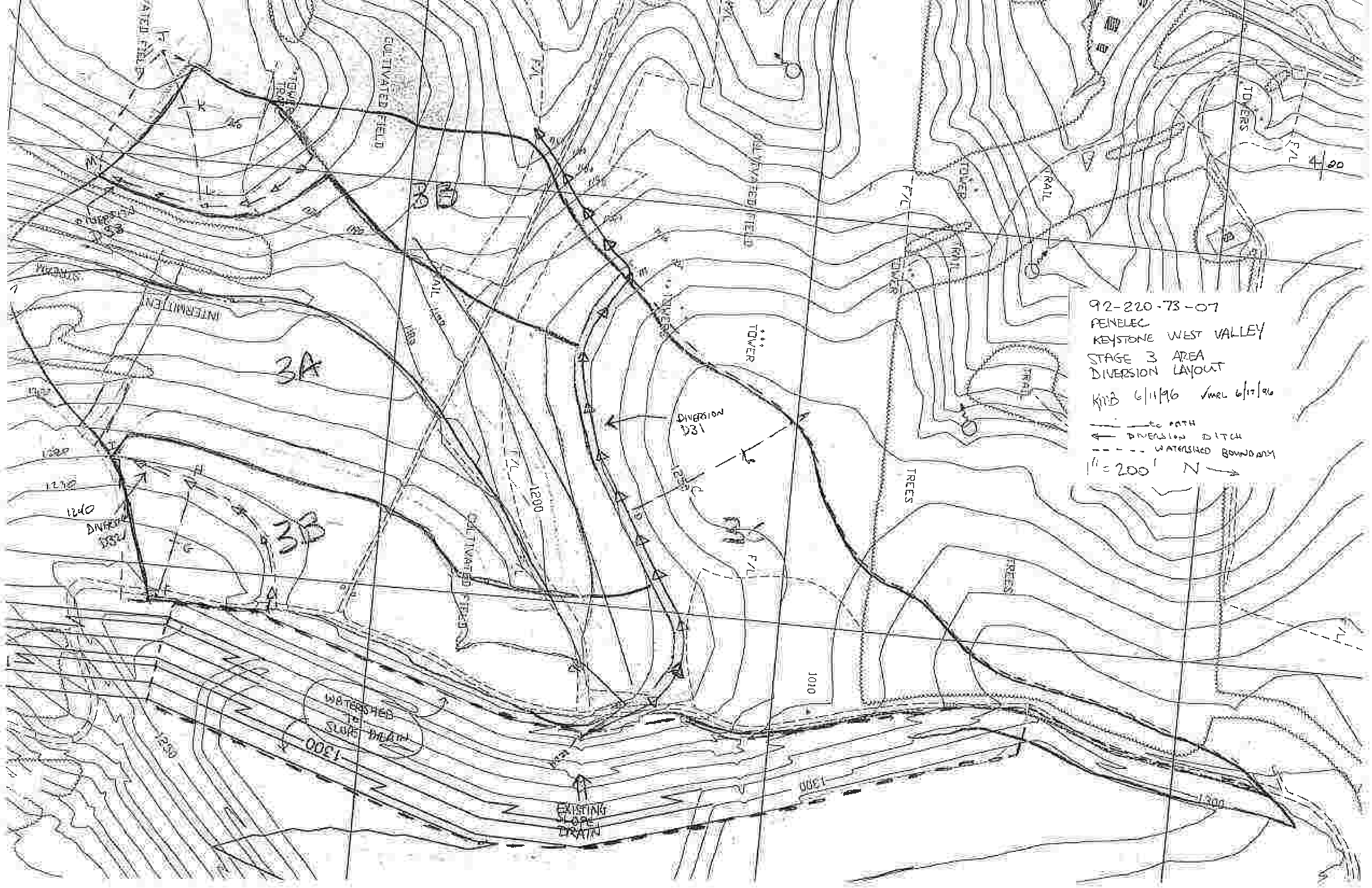
3A WILL BE LINED FIRST. THE THREE DIVERSION CHANNELS D31, D32, AND D33 WILL BE CONSTRUCTED TO DIVERT RUNOFF FROM ADJACENT AREAS AROUND AREA 3A. WHILE 3B IS BEING CONSTRUCTED, DITCHES D32 AND D33 WILL BE ELIMINATED, WHILE D31 WILL NOT BE AFFECTED UNTIL AREA 3C.

DIVERSION DITCH D31 WILL ALSO BE CARRYING RUNOFF FROM THE EAST VALLEY. A SLOPE DRAIN COMES OFF THE EAST VALLEY PILE FACE; WATER FROM THIS SLOPE DRAIN WILL THEN FLOW INTO DITCH D31.

THE WATERSHEDS TO THE DIVERSION DITCHES ARE DELINEATED ON THE NEXT PAGE. THE ACREAGE TO THE SLOPE DRAIN IN THE EAST VALLEY IS 14.4 acres, AS REPORTED IN THE HAUL ROAD DITCH DESIGN CALCULATION SECTION (RH SEL, GAI PROJECT 92-220-73-07, DATED 5/24/96) FOR KEYSTONE WEST VALLEY.

THE OTHER DRAINAGE AREAS ARE:

D31 (NOT INCLUDING AREA FROM SLOPE DRAIN) 14.1 acres
D32 2.4 acres
D33 2.9 acres


92-220-73-07
PENELEC
KEYSTONE WEST VALLEY
STAGE 3 AREA
DIVERSION LAYOUT
KMB 6/11/96 VMRL 6/17/96
DIVERSION DITCH
WATERSHED BOUNDARY
1" = 200' N
3A
3B
DIVERSION D31
WATERSHED TO SLOPE DRAIN
EXISTING SLOPE DRAIN
CULTIVATED FIELD
INTERMITTENT
TOWER
TOWERS
TRAIL
TREES
F/L
1200
1300
1010
1240

SUBJECT: Penelec - Keystone West Valley
Phase II Permitting - Stage 3
BY: KMB DATE: 06/12/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KRL DATE: 6/17/96 SHEET NO. 5 OF 20

Time of Concentration Worksheet - SCS Methods Reference: "Urban Hydrology for Small Watersheds",
Watershed to Diversion D31 TR-55, Soil Conservation Service, June 1986
Postdevelopment Conditions

SHEET FLOW	Flowpath: A-B	units
1. Surface description (table 3-1)	Dense Grass	
2. Manning's roughness coeff., n_{st} (table 3-1)	$n_{st} := 0.24$	
3. Flow length, L_{st} (total $L_{st} \le 150$ feet)	$L_{st} := 150$	feet
4. Two-year, 24-hour rainfall, P_2	$P_2 := 2.6$	inches
5. Land Slope, $S_{st} := \frac{1255 - 1250}{310}$	$S_{st} = 0.016$	
6. Sheet Flow Time, $T_{st} := \frac{0.007 \cdot (n_{st} \cdot L_{st})^{0.8}}{P_2^{0.5} \cdot S_{st}^{0.4}}$	$T_{st} = 0.398$	hours

SHALLOW CONCENTRATED FLOW	Flowpath: B-C		Flowpath: C-D	
7. Surface description (paved or unpaved)	Unpaved			Unpaved
8. Flow length, L_{sc}	$L_{sc} := 160$ feet			$L_{sc1} := 160$ feet
9. Watercourse Slope, $S_{sc} := \frac{1255 - 1250}{310}$	$S_{sc} = 0.016$		$S_{sc1} := \frac{1250 - 1230}{115}$	$S_{sc1} = 0.174$
10. Average Velocity, $V_{sc} := 16.1345 \cdot S_{sc}^{0.5}$	$V_{sc} = 2.049$	fps	$V_{sc1} := 16.1345 \cdot S_{sc1}^{0.5}$	$V_{sc1} = 6.729$ fps
11. Shallow Conc. Flow time, $T_{sc} := \left(\frac{L_{sc}}{3600 \cdot V_{sc}}\right)$	$T_{sc} = 0.022$	hour	$T_{sc1} := \left(\frac{L_{sc1}}{3600 \cdot V_{sc1}}\right)$	$T_{sc1} = 0.007$ hour

CHANNEL FLOW	Flowpath: D-E	
12. Bottom width, b	$b := 2$	feet
13. Side slopes, z $z := 2$	$z = 2$	
14. Flow depth, d	$d := 1$	feet
15. Cross sectional area, $a := (b + z \cdot d) \cdot d$	$a = 4$	ft^2
16. Wetted perimeter, $P_w := \left[b + 2 \cdot d \cdot (1 + z^2)^{0.5}\right]$	$P_w = 6.472$	feet
17. Hydraulic radius, $r := \frac{a}{P_w}$	$r = 0.618$	feet
18. Channel Length, L_{ch}	$L_{ch} := 630$	feet
19. Channel Slope, $S_{ch} := .01$	$S_{ch} = 0.01$	
20. Channel lining	GRASS	
21. Manning's roughness coeff., n	$n := 0.045$	
22. Velocity, $V_{ch} := \left[\left(\frac{1.49}{n}\right) \cdot \left[r^{\left(\frac{2}{3}\right)}\right] \cdot S_{ch}^{\left(\frac{1}{2}\right)}\right]$	$V_{ch} = 2.402$	fps
22. Channel Flow time, $T_{ch} := \left(\frac{L_{ch}}{3600 \cdot V_{ch}}\right)$	$T_{ch} = 0.073$	hour

Total Watershed Time-of-Concentration, $T_c := (T_{st} + T_{sc} + T_{sc1} + T_{ch})$ $T_c = 0.499$ hour

SUBJECT: Penelec - Keystone West Valley
Phase II Permitting - Stage 3
BY: KMB DATE: 06/12/96 PROJ. NO.: 92-220-73-07
CHKD. BY: MRL DATE: 6/17/96 SHEET NO. 6 OF 20

Time of Concentration Worksheet - SCS Methods Reference: "Urban Hydrology for Small Watersheds",
Watershed to Diversion D32 TR-55, Soil Conservation Service, June 1986
Postdevelopment Conditions

SHEET FLOW	Flowpath: F-G		units
1. Surface description (table 3-1)		Dense Grass	
2. Manning's roughness coeff., n_{st} (table 3-1)		$n_{st} := 0.24$	
3. Flow length, L_{st} (total $L_{st} \le 150$ feet)		$L_{st} := 150$	feet
4. Two-year, 24-hour rainfall, P_2		$P_2 := 2.6$	inches
5. Land Slope, $S_{st} := \frac{1250 - 1240}{160}$		$S_{st} = 0.063$	
6. Sheet Flow Time, $T_{st} := \frac{0.007 \cdot (n_{st} \cdot L_{st})^{0.8}}{P_2^{0.5} \cdot S_{st}^{0.4}}$		$T_{st} = 0.231$	hours
SHALLOW CONCENTRATED FLOW	Flowpath: G-H		
7. Surface description (paved or unpaved)		Unpaved	
8. Flow length, L_{sc}		$L_{sc} := 150$	feet
9. Watercourse Slope, $S_{sc} := \frac{(1240 - 1220)}{160}$		$S_{sc} = 0.125$	
10. Average Velocity, $V_{sc} := 16.1345 \cdot S_{sc}^{0.5}$		$V_{sc} = 5.704$	fps
11. Shallow Conc. Flow time, $T_{sc} := \left(\frac{L_{sc}}{3600 \cdot V_{sc}}\right)$		$T_{sc} = 0.007$	hour
CHANNEL FLOW	Flowpath: H-I		
12. Bottom width, b		$b := 2$	feet
13. Side slopes, z $z := 2$		$z = 2$	
14. Flow depth, d		$d := 1.0$	feet
15. Cross sectional area, $a := (b + z \cdot d) \cdot d$		$a = 4$	ft^2
16. Wetted perimeter, $P_w := \left[b + 2 \cdot d \left(1 + z^2\right)^{0.5}\right]$		$P_w = 6.472$	feet
17. Hydraulic radius, $r := \frac{a}{P_w}$		$r = 0.618$	feet
18. Channel Length, L_{ch}		$L_{ch} := 220$	feet
19. Channel Slope, $S_{ch} := 0.01$		$S_{ch} = 0.01$	
20. Channel lining		Grass	
21. Manning's roughness coeff., n		$n := 0.045$	
22. Velocity, $V_{ch} := \left[\left(\frac{1.49}{n}\right) \cdot \left[r^{\left(\frac{2}{3}\right)}\right] \cdot S_{ch}^{\left(\frac{1}{2}\right)}\right]$		$V_{ch} = 2.402$	fps
22. Channel Flow time, $T_{ch} := \left(\frac{L_{ch}}{3600 \cdot V_{ch}}\right)$		$T_{ch} = 0.025$	hour

Total Watershed Time-of-Concentration, $T_c := T_{st} + T_{sc} + T_{ch}$ $T_c = 0.264$ hour

SUBJECT: Penelec - Keystone West Valley
Phase II Permitting - Stage 3
BY: KMB DATE: 06/12/96 PROJ. NO.: 92-220-73-07
CHKD. BY: MRL DATE: 6/17/96 SHEET NO. 7 OF 20

Time of Concentration Worksheet - SCS Methods Reference: "Urban Hydrology for Small Watersheds",
Watershed to Diversion D33 TR-55, Soil Conservation Service, June 1986
Postdevelopment Conditions

SHEET FLOW	Flowpath: J-K	units
1. Surface description (table 3-1)	Dense Grass	
2. Manning's roughness coeff., n_{st} (table 3-1)	$n_{st} := 0.24$	
3. Flow length, L_{st} (total $L_{st} \le 150$ feet)	$L_{st} := 150$	feet
4. Two-year, 24-hour rainfall, P_2	$P_2 := 2.6$	inches
5. Land Slope, $S_{st} := \frac{1260 - 1250}{130}$	$S_{st} = 0.077$	
6. Sheet Flow Time, $T_{st} := \frac{0.007 \cdot (n_{st} \cdot L_{st})^{0.8}}{P_2^{0.5} \cdot S_{st}^{0.4}}$	$T_{st} = 0.213$	hours
SHALLOW CONCENTRATED FLOW	Flowpath: K-L	
7. Surface description (paved or unpaved)	Unpaved	
8. Flow length, L_{sc}	$L_{sc} := 250$	feet
9. Watercourse Slope, $S_{sc} := \frac{(1250 - 1220)}{160}$	$S_{sc} = 0.188$	
10. Average Velocity, $V_{sc} := 16.1345 \cdot S_{sc}^{0.5}$	$V_{sc} = 6.986$	fps
11. Shallow Conc. Flow time, $T_{sc} := \left(\frac{L_{sc}}{3600 \cdot V_{sc}}\right)$	$T_{sc} = 0.01$	hour
CHANNEL FLOW	Flowpath: L-M	
12. Bottom width, b	$b := 2$	feet
13. Side slopes, z $z := 2$	$z = 2$	
14. Flow depth, d	$d := 1.0$	feet
15. Cross sectional area, $a := (b + z \cdot d) \cdot d$	$a = 4$	ft^2
16. Wetted perimeter, $P_w := \left[b + 2 \cdot d \cdot (1 + z^2)^{0.5}\right]$	$P_w = 6.472$	feet
17. Hydraulic radius, $r := \frac{a}{P_w}$	$r = 0.618$	feet
18. Channel Length, L_{ch}	$L_{ch} := 260$	feet
19. Channel Slope, $S_{ch} := 0.01$	$S_{ch} = 0.01$	
20. Channel lining	Grass	
21. Manning's roughness coeff., n	$n := 0.045$	
22. Velocity, $V_{ch} := \left[\left(\frac{1.49}{n}\right) \cdot \left[r^{\left(\frac{2}{3}\right)}\right] \cdot S_{ch}^{\left(\frac{1}{2}\right)}\right]$	$V_{ch} = 2.402$	fps
22. Channel Flow time, $T_{ch} := \left(\frac{L_{ch}}{3600 \cdot V_{ch}}\right)$	$T_{ch} = 0.03$	hour

Total Watershed Time-of-Concentration, $T_c := T_{st} + T_{sc} + T_{ch}$ $T_c = 0.253$ hour

STAGE 3 DIVERSIONS CONTINUED

THE TIME-OF-CONCENTRATION FLOW PATH FOR EACH DIVERSION IS SHOWN ON THE DIVERSION LAYOUT DRAWING. CONSIDER THAT THE t_c FOR THE EXISTING SLOPE DRAIN WILL BE ACCOUNTED FOR BY THE CHOSEN FLOW PATH FOR D31.

FOR ALL 3 DIVERSIONS (EXCEPT THE EXISTING SLOPE DRAIN AREA) RUNOFF WILL BE FROM CULTIVATED FIELDS. FOR THE KEYSTONE STATION, THIS TERRAIN TYPE HAS BEEN ASSIGNED A RUNOFF CURVE NUMBER OF 80. THE EAST VALLEY SLOPE DRAIN AREA WILL USE THE CN FOR REVEGETATED PILE, BENCH FACE (CN = 78) [REF. "PROJECT DESIGN PARAMETERS OUTLINE, KEYSTONE STATION ..., GAI PROJECT 85-376-4, SEPTEMBER 1987]

FOR D31, COMPOSITE $CN = (14.1 \times 80 + 14.4 \times 78) / (14.1 + 14.4)$

$$= 79$$

TR-20 WILL BE RUN TO CALCULATE PEAK FLOW:

DITCH	WATERSHED AREA acres	sq miles	CN	t_c (hr)	RESULTING PEAK FLOW cfs
D31	28.5	0.0445	79	0.50	54.5
D32	2.4	0.0038	80	0.26	6.6
D33	2.9	0.0045	80	0.25	7.9

```
JOB TR-20                    FULLPRINT          SUMMARY    NOPLOTS
TITLE 111 KEYSTONE STAGE 3 DIVERSION                       KMB 6/12/96  /MRL 6/17/96
 6 RUNOFF 1 010      5 0.0445      79.0        0.50        1          D31
 6 RUNOFF 1 010      6 0.0038      80.0        0.26        1          D32
 6 RUNOFF 1 010      1 0.0045      80.0        0.25        1          D33
   ENDATA
 7 LIST
 7 INCREM 6             0.10
 7 COMPUT 7 010    010  0.         4.40        1.          2 2  01  03 25-YR
   ENDCMP 1
   ENDJOB 2
```

9/20

SUMMARY TABLE 1 - SELECTED RESULTS OF STANDARD AND EXECUTIVE CONTROL INSTRUCTIONS IN THE ORDER PERFORMED
(A STAR(*) AFTER THE PEAK DISCHARGE TIME AND RATE (CFS) VALUES INDICATES A FLAT TOP HYDROGRAPH
A QUESTION MARK(?) INDICATES A HYDROGRAPH WITH PEAK AS LAST POINT.)

SECTION/ STRUCTURE ID	STANDARD CONTROL OPERATION	DRAINAGE AREA (SQ MI)	RAIN TABLE #	ANTEC MOIST COND	MAIN TIME INCREM (HR)	PRECIPITATION BEGIN (HR)	PRECIPITATION AMOUNT (IN)	PRECIPITATION DURATION (HR)	RUNOFF AMOUNT (IN)	PEAK DISCHARGE ELEVATION (FT)	PEAK DISCHARGE TIME (HR)	PEAK DISCHARGE RATE (CFS)	PEAK DISCHARGE RATE (CSM)
ALTERNATE 1 STORM 3													
XSECTION 10	RUNOFF	.04	2	2	.10	.0	4.40	24.00	2.29	---	12.20	54.50	1224.8
XSECTION 10	RUNOFF	.00	2	2	.10	.0	4.40	24.00	2.38	---	12.06	6.60	1735.7
XSECTION 10	RUNOFF	.00	2	2	.10	.0	4.40	24.00	2.38	---	12.06	7.93	1761.2

SUBJECT PENELEC - KEYSTONE WEST VALLEY
PHASE II PERMITTING

BY KMB DATE 6/12/96 PROJ. NO. 92-220-73-07
CHKD. BY MRL DATE 6/17/96 SHEET NO. 10 OF 20

GAI CONSULTANTS, INC.
Engineers • Geologists • Planners
Environmental Specialists

STAGE 3 DIVERSIONS CONTINUED

HYDRAULICALLY SIZE THE CHANNELS.
FROM THE LAYOUT DRAWING, THE FOLLOWING RANGE OF SLOPES OCCURS:

D31 — FIXED AT 1% UNTIL POINT E IS REACHED.
THEN, AVERAGE SLOPE IS $\frac{1210-1160}{310} = 0.161$

D32, D33 — FIXED AT 1%.

CHANNEL D31 WAS ANALYZED WITH TWO DIFFERENT LININGS AS A CONSIDERATION FOR THE GREAT INCREASE IN SLOPE AT THE DOWNSTREAM END. BOTH LININGS WERE ANALYZED FOR BOTH SLOPES (THE STEEP SECTION FLOWS TOO FAST FOR GRASS). THE HYDRAULIC CALCULATIONS FOLLOW.
ALL CHANNELS HAVE 2:1 SIDE SLOPES

SUMMARY:

CHANNEL	BOTTOM WIDTH (ft)	TOTAL DEPTH (ft)	LINING
D31	3	2.5	GRASS/NECM/GROUTED ROCK
D32	2	1.5	GRASS
D33	2	1.5	GRASS

SUBJECT: Keystone Station
Phase II Permitting
BY: KMB DATE: 06/13/96 PROJ. NO.: 92-220-73-07
CHKD. BY: MRL DATE: 6/17/96 SHEET NO. 11 OF 20

GAI CONSULTANTS, INC.
Engineers Geologists Planners
Environmental Specialists

Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right) \cdot a \cdot r^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right) \cdot (r)^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$

Diversion Ditch D31

Design Flow, $Q_d = 54.5 \cdot ft^3 \cdot sec^{-1}$ from sheet 8 of 20

Bottom Width, $b = 3 \cdot ft$

Side Slopes, $z = 2$

Channel Lining Is ~~Grass and/or~~ Grass with Nylon Erosion Control Matting. $n = 0.045$

Channel Minimum Slope, $S_{min} := 0.01$ (from Sheet 10) or $S_{min} = 0.01 \cdot \frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 2.053 \cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 14.6 \cdot ft^2$

Minimum Velocity, $V_{min} = 3.7 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 11.2 \cdot ft$

Freeboard, $F_b = 0.4 \cdot ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990, and engineering judgement.

Total depth, $D = 2.5 \cdot ft$

Top Width at Total Depth, $T_D = 13 \cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 83 \cdot ft^3 \cdot sec^{-1}$

Channel Maximum Slope, $S_{max} = \frac{(1210 \cdot ft - 1160 \cdot ft)}{310 \cdot ft}$ (from Sheet 10) or $S_{max} = 0.161 \cdot \frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 1.036 \cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 5.3 \cdot ft^2$

Maximum Velocity, $V_{max} = 10.4 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 7.1 \cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 334 \cdot ft^3 \cdot sec^{-1}$

TYPE B-4 CHANNEL

SUBJECT: Keystone Station
Phase II Permitting
BY: KMB DATE: 06/13/96 PROJ. NO.: 92-220-73-07
CHKD. BY: MRL DATE: 6/17/96 SHEET NO. 12 OF 20

GAI CONSULTANTS, INC.
Engineers Geologists Planners
Environmental Specialists

Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right) \cdot a \cdot r^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right) \cdot (r)^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$

Diversion Ditch D31

Design Flow, $Q_d = 54.5 \cdot ft^3 \cdot sec^{-1}$ from sheet 8 of 20

Bottom Width, $b = 3 \cdot ft$

Side Slopes, $z = 2$

Channel Lining is Grouted Rock with Manning's roughness coefficient, $n = 0.025$

Channel Minimum Slope, $S_{min} := 0.01$ (from Sheet 10) or $S_{min} = 0.01 \cdot \frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 1.55 \cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 9.5 \cdot ft^2$

Minimum Velocity, $V_{min} = 5.8 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 9.2 \cdot ft$

Freeboard, $F_b = 1 \cdot ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990

Total depth, $D = 2.5 \cdot ft$

Top Width at Total Depth, $T_D = 13 \cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 150 \cdot ft^3 \cdot sec^{-1}$

Channel Maximum Slope, $S_{max} := \frac{(1210 \cdot ft - 1160 \cdot ft)}{310 \cdot ft}$ (from Sheet 10) or $S_{max} = 0.161 \cdot \frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 0.761 \cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 3.4 \cdot ft^2$

Maximum Velocity, $V_{max} = 15.8 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 6 \cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 602 \cdot ft^3 \cdot sec^{-1}$

TYPE C-6 CHANNEL

SUBJECT: Keystone Station
Phase II Permitting
BY: KMB DATE: 06/13/96 PROJ. NO.: 92-220-73-07
CHKD. BY: MKL DATE: 6/17/96 SHEET NO. 13 OF 20

GAI CONSULTANTS, INC.
Engineers Geologists Planners
Environmental Specialists

Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right)\cdot a \cdot r^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right)\cdot (r)^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$

Diversion Ditch D32

Design Flow, $Q_d = 6.6\ ft^3\ sec^{-1}$ from sheet 8 of 20

Bottom Width, $b = 2\cdot ft$

Side Slopes, $z = 2$

Channel Lining is Grass with Manning's roughness coefficient, $n = 0.045$

Channel Minimum Slope, $S_{min} := 0.01$ (from Sheet 10) or $S_{min} = 0.01 \cdot \frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 0.83\ ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 3\cdot ft^2$

Minimum Velocity, $V_{min} = 2.2\ ft\ sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 5.3\cdot ft$

Freeboard, $F_b = 0.7\ ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990

Total depth, $D = 1.5\cdot ft$

Top Width at Total Depth, $T_D = 8\cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 22\cdot ft^3 \cdot sec^{-1}$

Channel Maximum Slope, $S_{max} := 0.01$ (from Sheet 10) or $S_{max} = 0.01 \cdot \frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 0.83\ ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 3\cdot ft^2$

Maximum Velocity, $V_{max} = 2.2\ ft\ sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 5.3\cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 22\cdot ft^3 \cdot sec^{-1}$

TYPE A-2 CHANNEL

SUBJECT: Keystone Station
Phase II Permitting
BY: KMB DATE: 06/13/96 PROJ. NO.: 92-220-73-07
CHKD. BY: MRC DATE: 6/17/96 SHEET NO. 14 OF 80

CONSULTANTS, INC.
Engineers Geologists Planners
Environmental Specialists

Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right) \cdot a \cdot r^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right) \cdot (r)^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$

Diversion Ditch D33

Design Flow, $Q_d = 7.9 \ ft^3 \ sec^{-1}$ from sheet 8 of 80

Bottom Width, $b = 2 \cdot ft$

Side Slopes, $z = 2$

Channel Lining is Grass with Manning's roughness coefficient, $n = 0.045$

Channel Minimum Slope, $S_{min} := 0.01$ (from Sheet 10) or $S_{min} = 0.01 \cdot \frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 0.908 \ ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 3.5 \cdot ft^2$

Minimum Velocity, $V_{min} = 2.3 \ ft \ sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 5.6 \cdot ft$

Freeboard, $F_b = 0.6 \ ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990

Total depth, $D = 1.5 \cdot ft$

Top Width at Total Depth, $T_D = 8 \cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 22 \cdot ft^3 \cdot sec^{-1}$

Channel Maximum Slope, $S_{max} = 0.01$ (from Sheet 10) or $S_{max} = 0.01 \cdot \frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 0.908 \ ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 3.5 \cdot ft^2$

Maximum Velocity, $V_{max} = 2.3 \ ft \ sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 5.6 \cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 22 \cdot ft^3 \cdot sec^{-1}$

TYPE A-2 CHANNEL

SUBJECT PENELEC – KEYSTONE WEST VALLEY
PHASE II PERMITTING

BY KMB DATE 6/12/96 PROJ. NO. 92-220-73-07
CHKD. BY MRL DATE 6/18/96 SHEET NO. 15 OF 20

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DIVERSIONS CONTINUED

STAGE 4 DIVERSIONS

DURING STAGE 4 CONSTRUCTION, LINER WILL BE PLACED IN THREE SEQUENTIAL OPERATIONS – 4A, 4B, AND 4C. THESE AREAS ARE SHOWN ON THE DRAWING PRESENTED ON THE NEXT PAGE.

THE ONLY DIVERSION THAT WILL BE CONSTRUCTED WILL BE TO THE WEST OF AREA 4A TO DIVERT RUNOFF FROM AREA 4B. THE AREA OF 4B TO THE EAST OF 4A IS VERY SMALL AND ANY CHANNEL CONSTRUCTED WILL ENTAIL MUCH CULVERTING UNDER THE HAUL ROAD. AREA 4C, TO THE EAST OF 4B, WILL BE DIVERTED BY THE HAUL ROAD CLEAN WATER DITCH.

THE WATERSHED AREA TO THE CHANNEL = 26.5 acres (4.9 acres OF REVEG. BENCH)

THE TIME-OF-CONCENTRATION CALCULATION IS ON PAGE 17

TR-20 WILL BE RUN WITH THE FOLLOWING:

AREA = 26.5 acres = 0.0414 mi^2
t_c = 0.48 hr
CN = 80, STANDARD CN FOR OFFSITE CONDITIONS
78, BENCH FACE, REVEGETATED

$$\overline{CN} = \frac{78 \times 4.9 + 80 \times 21.6}{26.5} = 80$$

THE PEAK FLOW IS 52.8 cfs

92-220-73-07
PENELEC 15/20
KEYSTONE WEST VALLEY
STAGE 4 AREA
DIVERSION LAYOUT
KMB 6/12/96 ✓MRL 6/18/96
tc PATH
DIVERSION DITCH
WATERSHED BOUNDARY
1" = 200' 5' CONTOURS N
4B
4A
4C
1200
1250
1300
1350
1400

SUBJECT: Penelec - Keystone West Valley
Phase II Permitting - Stage 4
BY: KMB DATE: 06/13/96 PROJ. NO.: 92-220-73-07
CHKD. BY: MRL DATE: 6/17/96 SHEET NO. 17 OF 20

Time of Concentration Worksheet - SCS Methods Reference: "Urban Hydrology for Small Watersheds", TR-55, Soil Conservation Service, June 1986
Watershed to Diversion Ditch D41
Postdevelopment Conditions

SHEET FLOW — Flowpath: A-B — units

1. Surface description (table 3-1) — Dense Grass
2. Manning's roughness coeff., n_{st} (table 3-1) — $n_{st} := 0.24$
3. Flow length, L_{st} (total $L_{st} \le 150$ feet) — $L_{st} := 150$ feet
4. Two-year, 24-hour rainfall, P_2 — $P_2 := 2.6$ inches
5. Land Slope, $S_{st} := \frac{1263 - 1260}{140}$ — $S_{st} = 0.021$
6. Sheet Flow Time, $T_{st} := \frac{0.007 \cdot (n_{st} \cdot L_{st})^{0.8}}{P_2^{0.5} \cdot S_{st}^{0.4}}$ — $T_{st} = 0.355$ hours

SHALLOW CONCENTRATED FLOW — Flowpath: B-C

7. Surface description (paved or unpaved) — unpaved
8. Flow length, L_{sc} — $L_{sc} := 350$ feet
9. Watercourse Slope, $S_{sc} := \frac{1260 - 1190}{350}$ — $S_{sc} = 0.2$
10. Average Velocity, $V_{sc} := 16.1345 \cdot S_{sc}^{0.5}$ — $V_{sc} = 7.216$ fps
11. Shallow Conc. Flow time, $T_{sc} := \left(\frac{L_{sc}}{3600 \cdot V_{sc}}\right)$ — $T_{sc} = 0.013$ hour

CHANNEL FLOW — Flowpath: C-D

12. Bottom width, b — $b := 2$ feet
13. Side slopes, z $z := 2$ — $z = 2$
14. Flow depth, d — $d := 2.5$ feet
15. Cross sectional area, $a := (b + z \cdot d) \cdot d$ — $a = 17.5$ ft^2
16. Wetted perimeter, $P_w := \left[b + 2 \cdot d \cdot (1 + z^2)^{0.5}\right]$ — $P_w = 13.18$ feet
17. Hydraulic radius, $r := \frac{a}{P_w}$ — $r = 1.328$ feet
18. Channel Length, L_{ch} — $L_{ch} := 1600$ feet
19. Channel Slope, $S_{ch} := 0.01$ — $S_{ch} = 0.01$
20. Channel lining — GRASS
21. Manning's roughness coeff., n — $n := 0.045$
22. Velocity, $V_{ch} := \left[\left(\frac{1.49}{n}\right) \cdot r^{\left(\frac{2}{3}\right)}\right] \cdot S_{ch}^{\left(\frac{1}{2}\right)}$ — $V_{ch} = 4$ fps
22. Channel Flow time, $T_{ch} := \left(\frac{L_{ch}}{3600 \cdot V_{ch}}\right)$ — $T_{ch} = 0.111$ hour

Total Watershed Time-of-Concentration, $T_c := (T_{st} + T_{sc} + T_{ch})$ — $T_c = 0.48$ hour

DIVTC.MCD, 6/17/96, page 1

```
JOB TR-20                    FULLPRINT          SUMMARY   NOPLOTS
TITLE 111 KEYSTONE WEST VALLEY STAGE 4 DIVERSION DITCHES       KMB 06/18/96  /MRL  6/18/96
 6 RUNOFF 1 010        5 0.0414     80.0        0.48        1
   INDATA
 , LIST
 7 INCREM 6           0.10
 7 COMPUT 7 010   010   0.         4.40        1.          2 2  01   03 25-YR
   ENDCMP 1
   ENDJOB 2
```

SUMMARY TABLE 1 - SELECTED RESULTS OF STANDARD AND EXECUTIVE CONTROL INSTRUCTIONS IN THE ORDER PERFORMED
(A STAR(*) AFTER THE PEAK DISCHARGE TIME AND RATE (CFS) VALUES INDICATES A FLAT TOP HYDROGRAPH
A QUESTION MARK(?) INDICATES A HYDROGRAPH WITH PEAK AS LAST POINT.)

SECTION/ STRUCTURE ID	STANDARD CONTROL OPERATION	DRAINAGE AREA (SQ MI)	RAIN TABLE #	ANTEC MOIST COND	MAIN TIME INCREM (HR)	PRECIPITATION BEGIN (HR)	PRECIPITATION AMOUNT (IN)	PRECIPITATION DURATION (HR)	RUNOFF AMOUNT (IN)	PEAK DISCHARGE ELEVATION (FT)	PEAK DISCHARGE TIME (HR)	PEAK DISCHARGE RATE (CFS)	PEAK DISCHARGE RATE (CSM)
ALTERNATE 1 STORM 3													
+ XSECTION 10	RUNOFF	.04	2	2	.10	.0	4.40	24.00	2.37	---	12.19	52.79	1275.2

1

SUBJECT PENELEC - KEYSTONE WEST VALLEY
PHASE II PERMITTING

BY KMB DATE 6/13/96 PROJ. NO. 92-220-73-07
CHKD. BY MRL DATE 6/17/96 SHEET NO. 19 OF 20

CONSULTANTS, INC.
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STAGE 4 DIVERSIONS CONTINUED

HYDRAULICALLY SIZE CHANNEL DA-1. THE SLOPE OF THE CHANNEL IS FIXED AT 1%. THE NEXT PAGE SHOWS A SUMMARY OF THE CHANNEL HYDRAULIC CALCULATIONS.

SUMMARY

CHANNEL DA-1

BOTTOM WIDTH	3 ft
TOTAL DEPTH	2.5 ft
LINING	GRASS

SUBJECT: Keystone Station
Phase II Permitting
BY: KMB DATE: 06/18/96 PROJ. NO.: 92-220-73-07
CHKD. BY: MRL DATE: 6/18/96 SHEET NO. 20 OF 20

CONSULTANTS, INC.
Engineers Geologists Planners
Environmental Specialists

Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right) \cdot a \cdot r^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right) \cdot (r)^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$

Diversion Ditch D41

Design Flow, $Q_d = 52.8 \cdot ft^3 \cdot sec^{-1}$ from sheet 19 of 20

Bottom Width, $b = 3 \cdot ft$

Side Slopes, $z = 2$

Channel Lining is Grass with Manning's roughness coefficient, $n = 0.045$

Channel Minimum Slope, $S_{min} := 0.01$ (from Sheet 19) or $S_{min} = 0.01 \cdot \frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 2.023 \cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 14.3 \cdot ft^2$

Minimum Velocity, $V_{min} = 3.7 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 11.1 \cdot ft$

Freeboard, $F_b = 0.5 \cdot ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990

Total depth, $D = 2.5 \cdot ft$

Top Width at Total Depth, $T_D = 13 \cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 83 \cdot ft^3 \cdot sec^{-1}$

Channel Maximum Slope, $S_{max} := 0.01$ (from Sheet 19) or $S_{max} = 0.01 \cdot \frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 2.023\ ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 14.3 \cdot ft^2$

Maximum Velocity, $V_{max} = 3.7\ ft\ sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 11.1 \cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 83 \cdot ft^3 \cdot sec^{-1}$

TYPE A-7 CHANNEL

SUBJECT KEYSTONE
PHASE II PERMITTING

BY SER DATE 7/18/96 PROJ. NO. 92-220-73-7
CHKD. BY MRL DATE 7/26/96 SHEET NO. 1 OF 9

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SLOPE PIPE

A SLOPE PIPE WILL BE REQUIRED TO PASS FLOW FROM AN ISOLATED BENCH. THE BENCH WILL BE ISOLATED FROM DRAINING TO THE WEST SLOPE DRAIN AND THE HAUL ROAD DIRTY WATER DITCH BY THE HAUL ROAD. IT IS PROPOSED TO CONSTRUCT A SLOPE PIPE FROM THE ISOLATED BENCH TO THE BENCH BELOW AS SHOWN ON SHEET 2.

THE DRAINAGE AREA TO THE SLOPE PIPE IS SHOWN ON SHEET 2.

$AREA = 1.21\ AC = 0.00189\ MI^2$

THE TIME-OF-CONCENTRATION, t_c PATH IS SHOWN ON SHEET 2 AND THE t_c IS ESTIMATED ON SHEET 3.

$t_c = 0.15\ HR$

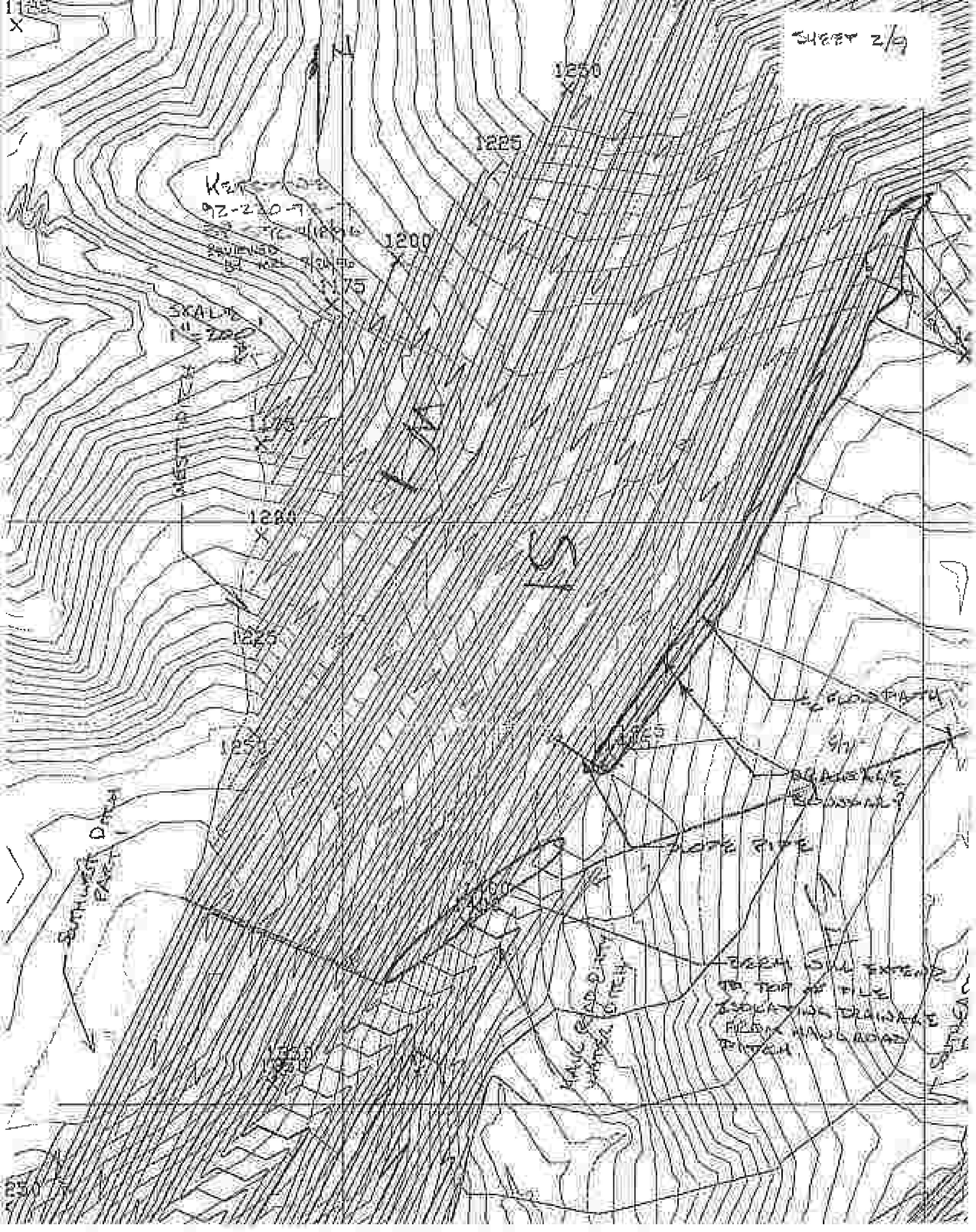
REFERENCE "ULTIMATE CONDITIONS - DRAINAGE FACILITIES" CALC BY SER 3/19/96 FOR BACKGROUND DATA INCLUDING RAINFALL, DESIGN EVENT, CN, ETC.

USE CN = 78
$P_{25,24} = 4.4\ in$

A TR-20 RUN WAS COMPLETED AND THE PEAK FLOW FOR THE DESIGN EVENT (25-YR, 24-HR STORM) IS 3.4 CFS SEE SHEETS 4 & 5

A CULVERT DESIGN HAS BEEN COMPLETED ON SHEETS 6 & 7 WITH A PROFILE SHOWN ON SHEET 8.

USE A 12"ϕ CMP CULVERT AS SHOWN ON SHEET 8.



SHEET 2/9
1250
1225
1200
1175
SCALE 1" = 200'
1225
1250

SUBJECT: Genco - Kestone West Valley
Phase II Permitting
BY: SER DATE: 7/18/96 PROJ. NO.: 92-220-73-07
CHKD. BY: MRL DATE: 7/26/96 SHEET NO. 3 OF 9

Time of Concentration Worksheet - SCS Methods
Watershed - Slope Pipe
Postdevelopment Conditions

Reference: "Urban Hydrology for Small Watersheds", TR-55, Soil Conservation Service, June 1986

SHEET FLOW — Flowpath: a-b — units — Flowpath: a-b

1. Surface description (table 3-1) — Grass — Grass
2. Manning's roughness coeff., n_{st} (table 3-1) — $n_{st} := 0.24$ — $n_{st} := 0.24$
3. Flow length, L_{st} (total $L_{st} \le 150$ feet) — $L_{st} := 25$ feet — $L_{st1} := 40$
4. Two-year, 24-hour rainfall, P_2 — $P_2 := 2.6$ inches — $P_2 := 2.6$
5. Land Slope, $S_{st} := \frac{2}{25}$ — $S_{st} = 0.08$ — $S_{st1} := \frac{17}{40}$ — $S_{st1} = 0.425$
6. Sheet Flow Time, $T_{st} := \frac{0.007 \cdot (n_{st} \cdot L_{st})^{0.8}}{P_2^{0.5} \cdot S_{st}^{0.4}}$ — $T_{st} = 0.05$ hours — $T_{st1} := \frac{0.007 \cdot (n_{st} \cdot L_{st1})^{0.8}}{P_2^{0.5} \cdot S_{st1}^{0.4}}$ — $T_{st1} = 0.037$

SHALLOW CONCENTRATED FLOW — Flowpath: N/A

7. Surface description (paved or unpaved) — ______
8. Flow length, L_{sc} — $L_{sc} := 0$ feet
9. Watercourse Slope, $S_{sc} := 0$ — $S_{sc} = 0$
10. Average Velocity, $V_{sc} := 16.1345 \cdot S_{sc}^{0.5}$ — $V_{sc} = 0$ fps
11. Shallow Conc. Flow time, $T_{sc} := \left(\frac{L_{sc}}{3600 \cdot V_{sc}}\right)$ — $T_{sc} = 0$ hour

CHANNEL FLOW — Flowpath: b-c

12. Bottom width, b — $b := 0$ feet
13. Side slopes, $z := \frac{15 + 2.5}{2}$ — $z = 8.75$
14. Flow depth, d — $d := 1$ feet
15. Cross sectional area, $a := (b + z \cdot d) \cdot d$ — $a = 8.75$ ft^2
16. Wetted perimeter, $P_w := \left[b + 2 \cdot d \cdot (1 + z^2)^{0.5}\right]$ — $P_w = 17.614$ feet
17. Hydraulic radius, $r := \frac{a}{P_w}$ — $r = 0.497$ feet
18. Channel Length, L_{ch} — $L_{ch} := 970$ feet
19. Channel Slope, $S_{ch} := 0.02$ — $S_{ch} = 0.02$
20. Channel lining — Grass
21. Manning's roughness coeff., n — $n := 0.045$
22. Velocity, $V_{ch} := \left[\left(\frac{1.49}{n}\right) \cdot \left[r^{\left(\frac{2}{3}\right)}\right] \cdot S_{ch}^{\left(\frac{1}{2}\right)}\right]$ — $V_{ch} = 2.937$ fps
22. Channel Flow time, $T_{ch} := \left(\frac{L_{ch}}{3600 \cdot V_{ch}}\right)$ — $T_{ch} = 0.092$ hour

Total Watershed Time-of-Concentration, $T_c := T_{st} + T_{st1} + T_{sc} + T_{ch}$ — $T_c = 0.18$ hour

```
JOB TR-20                    FULLPRINT          SUMMARY   NOPLOTS
TITLE 111   KEYSTONE WEST VALLEY - SLOPE PIPE - 92-220-73-7
 6 RUNOFF 1 00101     1 0.00189     78.         0.18        1
   ENDATA
     ST
 . .NCREM 6              0.1
 7 COMPUT 7 001      01 0.          4.4         1.          2 2        25 YR
   ENDCMP 1
   ENDJOB 2
```

SHEET 4/9

✓MRL 7/26/96

SHEET 5A

SUMMARY TABLE 1 - SELECTED RESULTS OF STANDARD AND EXECUTIVE CONTROL INSTRUCTIONS IN THE ORDER PERFORMED
(A STAR(*) AFTER THE PEAK DISCHARGE TIME AND RATE (CFS) VALUES INDICATES A FLAT TOP HYDROGRAPH
A QUESTION MARK(?) INDICATES A HYDROGRAPH WITH PEAK AS LAST POINT.)

[illegible]ION/ [illegible]UCTURE ID	STANDARD CONTROL OPERATION	DRAINAGE AREA (SQ MI)	RAIN TABLE #	ANTEC MOIST COND	MAIN TIME INCREM (HR)	PRECIPITATION BEGIN (HR)	AMOUNT (IN)	DURATION (HR)	RUNOFF AMOUNT (IN)	PEAK DISCHARGE ELEVATION (FT)	TIME (HR)	RATE (CFS)	RATE (CSM)
ALTERNATE 0	STORM 0												
XSECTION 1	RUNOFF	.00	2	2	.10	.0	4.40	24.00	2.22	---	12.01	3.40	1797.8

1

SUBJECT: Keystone Station
Phase II Permitting
BY: SER DATE: 7/18/96 PROJ. NO.: 92-220-73-07
CHKD. BY: MRL DATE: 7/26/96 SHEET NO. 6 OF 9

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Environmental Specialists

CULVERT DESIGN - SLOPE PIPE

Purpose: Design the slope pipe which will carry flow from an isolated bench to a bench below.

Methodology: "Hydraulic Design of Highway Culverts", HDS No. 5, Federal Highway Administration, September 1985

Data Input Section

Design Flow, $Q := 3.4 \cdot \frac{ft^3}{sec}$ see sheet 5.

Inlet invert elevation, $EL_i := 1413.2 \cdot ft$ See sheet 8.

Outlet invert elevation, $EL_o := 1389.8 \cdot ft$ See sheet 8.

Limiting headwater elevation, $EL_1 := 1414.8 \cdot ft$

Pipe Length, $L := 82 \cdot ft$

Pipe diameter, $D := \frac{12 \cdot in}{12 \cdot \frac{in}{ft}}$ $D = 1 \cdot ft$

Pipe material is BCCMP projecting from fill.

Flow Area, $A := \frac{D^2 \cdot \pi}{4}$ $A = 0.785 \cdot ft^2$

Flow Velocity, $V := \frac{Q}{A}$ $V = 4.329 \cdot ft \cdot sec^{-1}$

Hydraulic Radius, $R := \frac{D}{4}$ $R = 0.25 \cdot ft$

Entrance Loss Coefficient, $k_e := 0.9$ from HDS No. 5 for CMP projecting from fill.

Bend Loss Coefficient, $k_b := 2$ Two minor bends, conservative assumption

Manning's loss Coefficient $n := 0.022$

Critical Depth, $d_c := 1.0 \cdot ft$ from chart in HDS No. 5.

Parameters for use in Equation 28 of HDS No. 5, for Submerged Conditions Inlet Control

$c := 0.0553 \cdot \frac{sec^2}{ft}$ from HDS No. 5 for CMP pipe projecting from fill, units by dimensional analysis of Equation (28) below.

$Y := 0.54$ from HDS No. 5 for given pipe material and entrance type

$S := 0.01$ Slope at pipe inlet.

SUBJECT: Keystone Station
Phase II Permitting
BY: SER DATE: 7/18/96 PROJ. NO.: 92-220-73-07
CHKD. BY: MRL DATE: 7/26/96 SHEET NO. 7 OF 9

CONSULTANTS, INC.
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Environmental Specialists

Inlet Control Calculation Section

Submerged Equation (28) from HDS No. 5,

$$HW_i := D \cdot \left[c \cdot \left(\left(\frac{Q}{A \cdot D^{0.5}} \right) \right)^2 + Y - 0.5 \cdot S \right] \qquad HW_i = 1.6 \cdot ft$$

Inlet Control Headwater Elevation,

$$EL_{hi} := EL_i + HW_i \qquad EL_{hi} = 1414.8 \cdot ft$$

Outlet Control Calculation Section

Pipe Head Loss Equation from HDS No. 5,

$$H := \left(1 + k_e + k_b + \frac{29 \cdot n^2 \cdot L}{R^{1.33}} \cdot ft^{0.33} \right) \cdot \frac{V^2}{2 \cdot g} \qquad H = 3.3 \cdot ft$$

$$h_0 := \frac{D + d_c}{2}$$

$$h_0 = 1 \cdot ft$$

Outlet Control Headwater Elevation,

$$EL_{ho} := EL_o + H + h_0 \qquad EL_{ho} = 1394.1 \cdot ft$$

Controlling Headwater Elevation

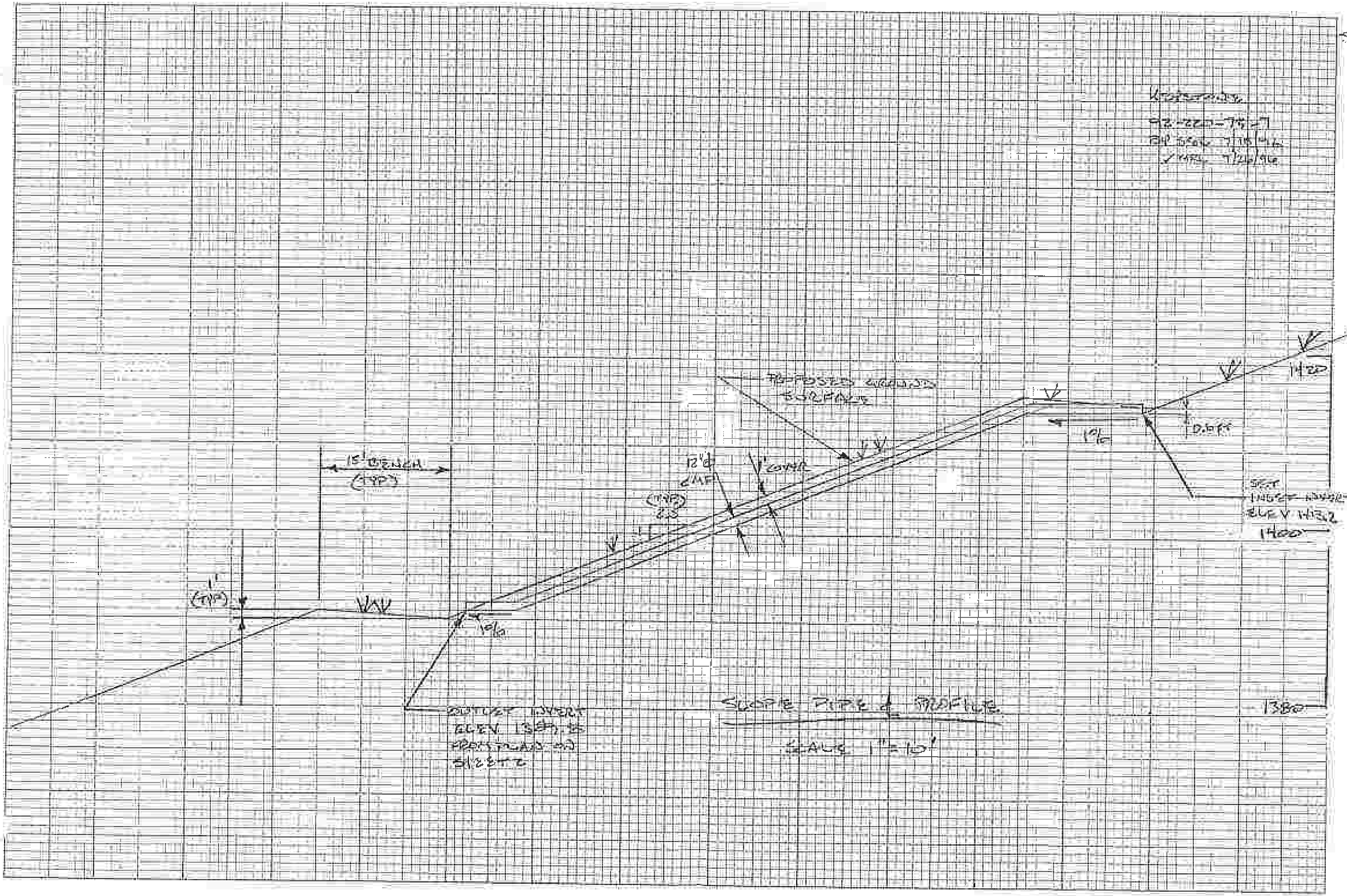
$$EL_{hc} := \max \left(\begin{pmatrix} EL_{hi} \\ EL_{ho} \end{pmatrix} \right) \qquad EL_{hc} = 1414.8 \cdot ft$$

Compare to the limiting headwater elevation,

$$EL_l = 1414.8 \cdot ft$$

$E_{hc} := EL_l$, Therefore Pipe design is OK

SHEET 5/9



K+E 10 X 10 TO THE INCH • 7 X 10 INCHES KEUFFEL & ESSER CO. MADE IN U.S.A.

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SUBJECT: Keystone Station
Phase II Permitting
BY: SER DATE: 7/18/96 PROJ. NO.: 92-220-73-07
CHKD. BY: MRL DATE: 7/26/96 SHEET NO. 9 OF 9

GAI CONSULTANTS, INC.
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Estimate velocity at outlet

$S = 0.01$

$Q = 3.4 \cdot ft^3 \cdot sec^{-1}$

$n = 0.022$

$K := \frac{Q \cdot n}{D^{\frac{8}{3}} \cdot S^{\frac{1}{2}}}$ $K = 0.748 \cdot ft^{0.333333} \cdot sec^{-1}$ Conveyance divided by diameter to the 8/3 power

This value is greater than that for y/D=1 in Appendix A of Chow "open Channel Hydraulics", 1959. Therefore flow is pressure flow and velocity is full flow velocity as follows.

$V = 4.329 \cdot ft \cdot sec^{-1}$

Provide NECM near outlet of pipe extending 10 feet upstream and downstream.

SUBJECT KEYSTONE
PHASE II PERMITTING

BY SER DATE 7/23/96 PROJ. NO. 93-220-73-7

CHKD. BY MRL DATE 7/23/96 SHEET NO. 1 OF 4

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WEIR BOX OUTLET CHANNEL

DESIGN A CHANNEL TO CARRY FLOW FROM THE GROUNDWATER SIDE OF THE WEIR BOX.

DESIGN FLOW = 1 CFS AS PER JMJ

SEE SHEET 2 FOR PLAN VIEW OF CHANNEL

SEE SHEET 3 AND 4 FOR CHANNEL DESIGNS.

CONCLUSION:

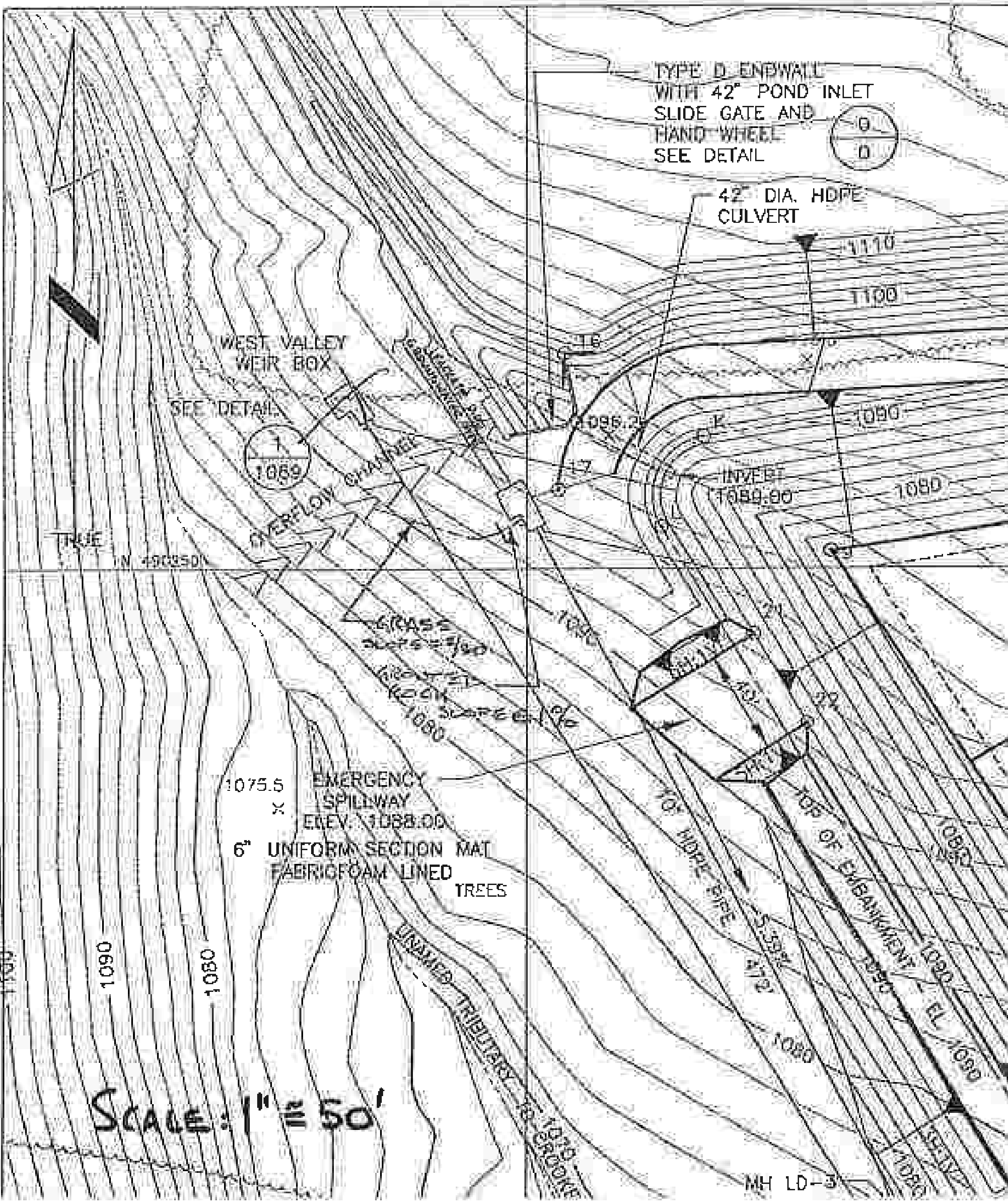
USE A 1' DEEP TRIANGULAR CHANNEL WITH:
GROUTED ROCK FOR FIRST 10' AT 1%
GRASS FOR REMAINDER AT NATURAL SLOPE (3'/30')

USE 2:1 SIDE SLOPES

SHEET 2/4

14 13 12

KEYSTONE
92-220-73-7
BY JGR 7/23/96 Reviewed by MEL 7/23/96



SUBJECT: Keystone Station
Phase II Permitting
BY: SER DATE: 7/23/96 PROJ. NO.: 92-220-73-07
CHKD. BY: MRL DATE: 7/23/96 SHEET NO. 3 OF 4

GAI CONSULTANTS, INC.
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Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right)\cdot a\cdot r^{\left(\frac{2}{3}\right)}\cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right)\cdot (r)^{\left(\frac{2}{3}\right)}\cdot s^{\left(\frac{1}{2}\right)}$

Weir Box Outlet Channel - Grouted Rock Portion

Design Flow, $Q_d = 1\cdot ft^3\cdot sec^{-1}$

Bottom Width, $b = 0\cdot ft$

Side Slopes, $z = 2$

Channel Lining is Grouted Rock with Manning's roughness coefficient, $n = 0.025$

Channel Minimum Slope, $S_{min} := 0.01$

Maximum Flow Depth, $d_{max} = 0.483\cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 0.5\cdot ft^2$

Minimum Velocity, $V_{min} = 2.1\cdot ft\cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 1.9\cdot ft$

Freeboard, $F_b = 0.5\cdot ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990.

Total depth, $D = 1\cdot ft$

Top Width at Total Depth, $T_D = 4\cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 7\cdot ft^3\cdot sec^{-1}$

Channel Maximum Slope, $S_{max} := 0.01$

Minimum Flow Depth, $d_{min} = 0.483\cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 0.5\cdot ft^2$

Maximum Velocity, $V_{max} = 2.1\cdot ft\cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 1.9\cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 7\cdot ft^3\cdot sec^{-1}$

TYPE C-8 CHANNEL

SUBJECT: Keystone Station
Phase II Permitting
BY: SER DATE: 7/23/96 PROJ. NO.: 92-220-73-07
CHKD. BY: MRC DATE: 7/23/96 SHEET NO. 4 OF 4

GAI CONSULTANTS, INC.
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Environmental Specialists

Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right) \cdot a \cdot r^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right) \cdot (r)^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$

Weir Box Outlet Channel - Grass Portion

Design Flow, $Q_d = 1 \cdot ft^3 \cdot sec^{-1}$

Bottom Width, $b = 0 \cdot ft$

Side Slopes, $z = 2$

Channel Lining is Grass with Manning's roughness coefficient, $n = 0.045$

Channel Minimum Slope, $S_{min} := \frac{2 \cdot ft}{30 \cdot ft}$ or $S_{min} = 0.067 \cdot \frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 0.422 \cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 0.4 \cdot ft^2$

Minimum Velocity, $V_{min} = 2.8 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 1.7 \cdot ft$

Freeboard, $F_b = 0.6 \cdot ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990.

Total depth, $D = 1 \cdot ft$

Top Width at Total Depth, $T_D = 4 \cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 10 \cdot ft^3 \cdot sec^{-1}$

Channel Maximum Slope, $S_{max} := \frac{2 \cdot ft}{30 \cdot ft}$ or $S_{max} = 0.067 \cdot \frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 0.422 \cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 0.4 \cdot ft^2$

Maximum Velocity, $V_{max} = 2.8 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 1.7 \cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 10 \cdot ft^3 \cdot sec^{-1}$

TYPE A-B CHANNEL

SUBJECT: Keystone Station
Phase II Permitting
BY: SER DATE: 6/17/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KMB DATE: 7/24/96 SHEET NO. 1 OF 4

West Dirty Water Ditch Bypass

GSI CONSULTANTS, INC.
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Purpose: Design the West Dirty Water Ditch (DWD) Bypass.

Description: The West DWD will discharge into the Equilization Pond through a pipe with a sluice gate at its entrance. When the sluice gate is closed, the flow from the West DWD will flow to the stream in the West DWD Bypass. The inlet to the Bypass will be a spillway with a five foot long crest which in turn will discarge to the bypass channel. Design the spillway and the bypass channel.

Methodology: "Earth Spillways", TR-2, US Soil Conservation Service, October 1, 1956 and Manning's Equation.

Design Flow: Design for the 25-year, 24-hour peak flow of 91 cfs for the West DWD, reference "Dirty Water and Related Facilities" calc by SER 5/24/96.

Bypass Channel

See sheet 4 for design. The flow is supercritical since the velocity of 29.4 fps is greater than the square root of (gd_m)

$$V := 29.4 \cdot \frac{ft}{sec} \qquad d_m := \frac{(2 \cdot ft + 0.841 \cdot ft \cdot 2) \cdot 0.841 \cdot ft}{2 \cdot ft + 2 \cdot 2 \cdot 0.841 \cdot ft} \qquad d_m = 0.577 \cdot ft \qquad \sqrt{g \cdot d_m} = 4.31 \cdot ft \cdot sec^{-1}$$

Control Section and Inlet Channel

The control section and inlet channel will be trapezoidal with a total depth of 2 feet, a flow depth of 1.5 feet, and side slopes of 2:1. The lining will be uniform section mat.

Find the required bottom width.

Assume the critical depth at the control section is $d_c := 1.0 \cdot ft$

$z := 2$

For critical flow to occur, velocity equals square root of (gd_m)

$$d_m := \frac{(2 \cdot ft + 2 \cdot 1 \cdot ft) \cdot 1 \cdot ft}{2 \cdot ft + 2 \cdot 2 \cdot 1 \cdot ft} \qquad d_m = 0.667 \cdot ft \qquad V := \sqrt{g \cdot d_m} \qquad V = 4.631 \cdot ft \cdot sec^{-1}$$

Flow $Q := 100 \cdot \frac{ft^3}{sec}$

Area $a(b) := (b + z \cdot d_c) \cdot d_c$

and Area $A := \frac{Q}{V} \qquad A = 21.592 \cdot ft^2$

Find b, $b := \frac{A}{d_c} - z \cdot d_c \qquad b = 19.592 \cdot ft$

use $b := 18 \cdot ft$

SUBJECT: Keystone Station
Phase II Permitting
BY: SER DATE: 6/10/96 PROJ. NO.: 92-220-73-07
CHKD. BY: [signature] DATE: 7/24/96 SHEET NO. 2 OF 4

CONSULTANTS, INC.
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Find the actual critical depth for a bottom width of 18 feet.

Area $a(d_c) := (b + z \cdot d_c) \cdot d_c$

Velocity $v(d_c) := \dfrac{Q}{(b + z \cdot d_c) \cdot d_c}$

Define a function f(d) and find its root

$$f(d_c) := \frac{Q}{(b + z \cdot d_c) \cdot d_c} - \sqrt{\frac{g \cdot (2 \cdot ft + 2 \cdot d_c) \cdot d_c}{2 \cdot ft + 2 \cdot 2 \cdot d_c}}$$

Trial depth $d_c := 0.5 \cdot ft$

$solution := root(f(d_c), d_c)$

$d_c := solution$

$d_c = 1.052 \cdot ft$

Proof

$v(d_c) = 4.730 \cdot ft \cdot sec^{-1}$ $\qquad d_m := \dfrac{(2 \cdot ft + 2 \cdot d_c) \cdot d_c}{2 \cdot ft + 2 \cdot 2 \cdot d_c}$ $\qquad d_m = 0.695 \cdot ft$

$\sqrt{g \cdot d_m} = 4.730 \cdot ft \cdot sec^{-1}$

$F := \dfrac{v(d_c)}{\sqrt{g \cdot d_m}}$ $\qquad F = 1$

Therefore $d_c = 1.052 \cdot ft$ is the critical depth at the control section.

Backwater Calculation

Critical depth at the control section is $d_c = 1.052 \cdot ft$

The length of the inlet channel is $L := 5 \cdot ft$

The bottom width of the inlet channel is $b = 18 \cdot ft$

The side slopes of the inlet channel are $z = 2$

The inlet channel is level at elevation $EL_{control} := 1098 \cdot ft$

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Find H_{ec} at the control section

$$a(d_c) = 21.141 \cdot ft^2$$

$$v(d_c) = 4.73 \cdot ft \cdot sec^{-1}$$

$$H_{ec} := d_c + \frac{(v(d_c))^2}{2 \cdot g} \qquad H_{ec} = 1.399 \cdot ft$$ formula from TR-2, p.13

Find α

$n := 0.015$ for uniform section mat concrete revetment

$$\alpha := \frac{4.315 \cdot ft^{\frac{1}{3}} \cdot n^2}{H_{ec}^{\frac{4}{3}}} \qquad \alpha = 0.00062 \cdot ft^{-1}$$ formula from TR-2, p.15., Eq. 15

The head on the weir crest is

$$H_p := H_{ec} \cdot (1 + \alpha \cdot L)$$ formula from TR-2, p.15., Eq. 14

$$H_p = 1.4 \cdot ft$$

The elevation of the water in the headwater pool is

$$EL_{pool} := EL_{control} + H_p \qquad EL_{pool} = 1099.4 \cdot ft$$

The embankment crest elevation is 1100, therefore the freeboard is

$$F_b := 1100 \cdot ft - EL_{pool} \qquad F_b = 0.6 \cdot ft$$ which is considered acceptable.

Backcalculate a weir discharge coefficient.

The effective length of the weir is

$$L_{eff} := b + d_c \cdot z \qquad L_{eff} = 20.103 \cdot ft$$

$$C := \frac{Q}{L_{eff} H_p^{\frac{3}{2}}} \qquad C = 2.99 \cdot ft^{0.5} \cdot sec^{-1}$$ This is reasonable.

SUBJECT: Keystone Station
Phase II Permitting - Ultimate Conditions
BY: SER DATE: 4/12/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KRB DATE: 7/?/96 SHEET NO. 4 OF 4

CONSULTANTS, INC.
Engineers Geologists Planners Environmental Specialists

Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right)\cdot a\cdot r^{\left(\frac{2}{3}\right)}\cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right)\cdot(r)^{\left(\frac{2}{3}\right)}\cdot s^{\left(\frac{1}{2}\right)}$

West Dirty Water Ditch - Bypass Channel

Design Flow, $Q_d = 91\cdot ft^3\cdot sec^{-1}$ from "Dirty Water Ditches and Related Facilities" calc by SER 5/24/96

Bottom Width, $b = 2\cdot ft$

Side Slopes, $z = 2$

Channel Lining is Grouted Rock with Manning's roughness coefficient, $n = 0.025$

Channel Minimum Slope, $S_{min} := \frac{2\cdot ft}{10\cdot ft}$ or $S_{min} = 0.2\cdot\frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 1.082\cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 4.5\cdot ft^2$

Minimum Velocity, $V_{min} = 20.2\cdot ft\cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 6.3\cdot ft$

Freeboard, $F_b = 0.9\cdot ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990

Total depth, $D = 2\cdot ft$

Top Width at Total Depth, $T_D = 10\cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 340\cdot ft^3\cdot sec^{-1}$

Channel Maximum Slope, $S_{max} := \frac{2\cdot ft}{10\cdot ft}$ or $S_{max} = 0.2\cdot\frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 1.082\cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 4.5\cdot ft^2$

Maximum Velocity, $V_{max} = 20.2\cdot ft\cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 6.3\cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 340\cdot ft^3\cdot sec^{-1}$

TYPE C-2 CHANNEL

SUBJECT GENCO — Keystone

BY MRL DATE 6/18/96 PROJ. NO. 92-220-73-07

CHKD. BY KMB DATE 7/12/96 SHEET NO. 1 OF 7

GAI CONSULTANTS, INC.
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FABRICFORM CHANNELS

PURPOSE: Determine the thickness of fabricform required for slope drains and off-liner dirty water ditches. The fabricform is to be thick enough so that the channels are stable against sliding when they are flowing at maximum discharge. Use "Uniform Section Mat" for the fabricform.

References: ① Armorform Design Manual, Nicolon Corporation. Prepared by Bowser-Morner Associates, Inc. September 25, 1989.

② Armorform Design Theory Manual, Nicolon Corporation. Prepared by Bowser-Morner Associates, Inc. 9/25/89

③ "Dirty Water Ditches and Related Facilities" calculations by SER, 5/24/96

④ "Ultimate Conditions - Drainage Facilities" calculations by SER, 3/19/96

SUBJECT Genco - Keystone

BY MRL DATE 6/18/96 PROJ. NO. 92-220-73-07

CHKD. BY [illegible] DATE 7/12/96 SHEET NO. 2 OF 7

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Channel Input Parameters

- Slope Drains (Reference 4)

Q_{MAX} = 71 CFS
V_{MAX} = 35.3 FPS
$S_{o\,MAX}$ = 0.40 ft/ft.

b = 2' (bottom width)
z = 2 (sideslopes H:V)
y = 0.62' (flow depth at Q_{MAX} and S_{MAX})
D = 2' (Total channel depth)

- Dirty Water Ditches (Reference 3)

Q_{MAX} = 91 cfs
V_{MAX} = 31.9 FPS
$S_{o\,MAX}$ = 0.25 ft/ft.

b = 2'
z = 2
y = 0.795'
D = 2.5'

- For all Fabriform channels, use an angle of friction between mat and soil (δ) of 30°. This is the minimum angle for Fabriform placed directly on silty sand, sandy silt, clayey sand, low cohesion materials, silty clay, or cohesive materials. See reference 2, page 15.

SUBJECT GENCO — Keystone

BY MRL DATE 6/18/96 PROJ. NO. 92-220-73-07

CHKD. BY [illegible] DATE 7/12/96 SHEET NO. 3 OF 7

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• Dirty Water Ditch

Use chart No. 4 (attached) — Simplified Design Method

$$R_h = \frac{y(b + yz)}{b + 2y\sqrt{1+z^2}} = \frac{0.795(2 + 0.795(2))}{2 + 2(.795)\sqrt{1+2^2}} = 0.514'$$

$F_s \times R_h = 1.5 \times 0.514' = 0.77'$

Use 1.5 (see reference ①, page 2)

At $S_o = 25\%$ and $F_s x R = 0.77$, 6" USM adequate

USE 6" USM — changed MRL 7/15/96

• Slope Drains

Use chart No. 4 (attached) — Simplified Design Method

$$R_h = \frac{0.62(2 + 0.62(2))}{2 + 2(0.62)\sqrt{1+2^2}} = 0.421'$$

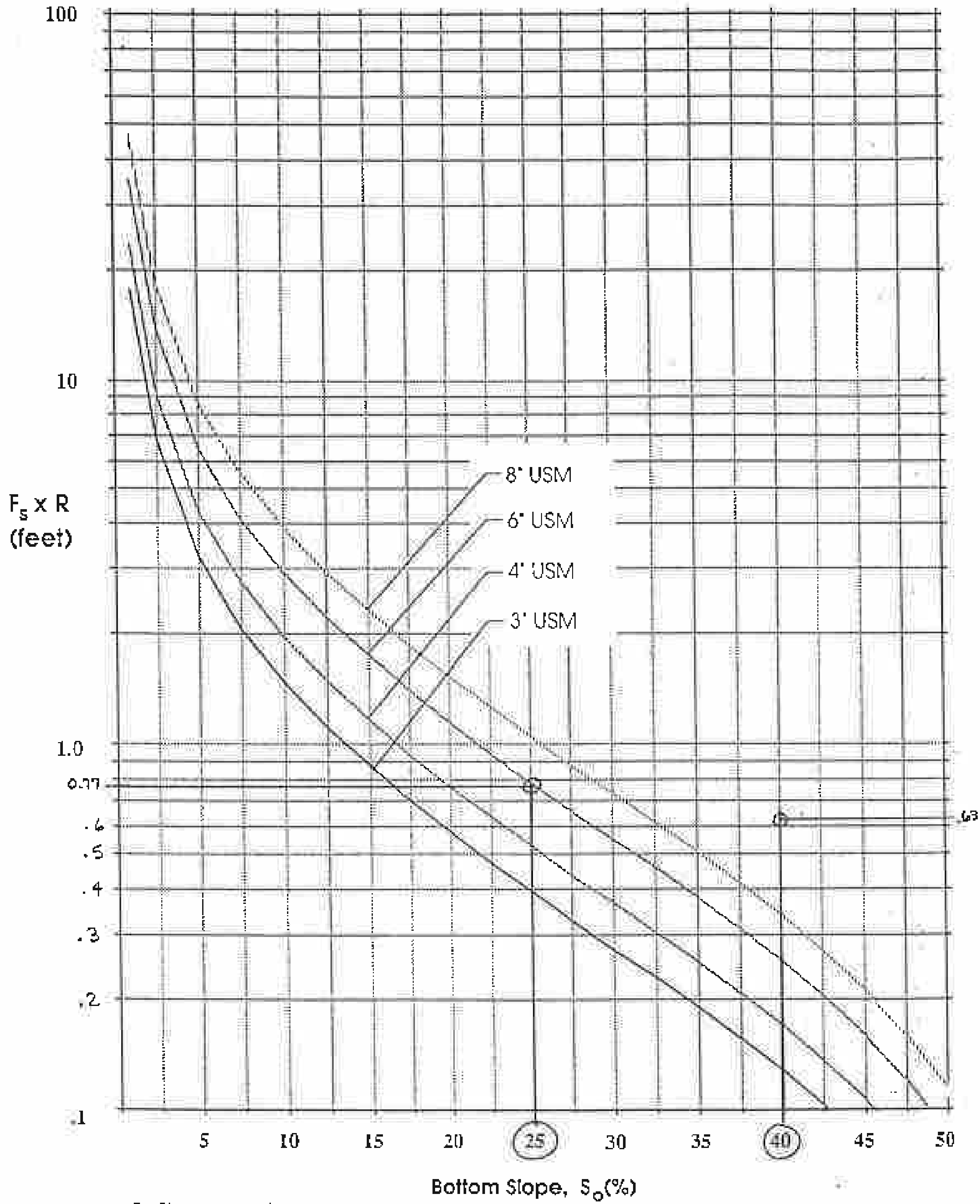
$F_s \times R_h = 1.5 \times 0.421' = 0.63'$

At $S_o = 40\%$ and $F_s x R$, Fabriform thickness > 8"

Therefore, use "general method" to determine thickness.

USM Friction Angle, δ = 30°

Chart No. 4



Reference : Armorform Design Manual

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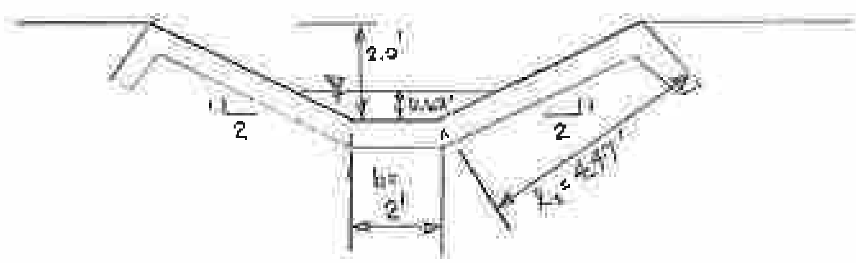
SLOPE DRAINS ... continued

Use the "general method" to determine Fabriform thickness. Assume that the channel bottom and sideslopes function as an integral unit. (i.e. the Fabriform has sufficient shear strength to prevent the channel bottom from sliding while the sideslopes remain in place) To be conservative, ignore the buried ends of the channel Fabriform.

Calculate the resisting shear stress and the tractive shear stress separately for the channel bottom and sideslopes. Determine the Fabriform thickness required from the weighted average of the resisting and tractive shear stresses.

See reference ①, page 4, for the "general method" procedure.



SUBJECT GENCO - Keystone

BY MRL DATE 6/16/96 PROJ. NO. 92-720-73-07

CHKD. BY KMB DATE 7/12/96 SHEET NO. 6 OF 7

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CHANNEL BOTTOM

Resisting shear stress : $M = \dfrac{(\gamma_c - \gamma_w)(\tan\delta - S_o)}{\sqrt{1 + S_o^2}}$

$$M = \frac{(140 - 62.4)(\tan 30^\circ - 0.40)}{\sqrt{1 + .4^2}} = \frac{(77.6)(0.1774)}{1.077} = 12.78 \text{ lbs}/\text{ft}^3$$

Tractive shear stress : $T = F_S R_h \gamma_w S_o = 1.5(0.421)(62.4)(0.40) = 15.76 \text{ lb}/\text{ft}^2$

CHANNEL SIDESLOPES

Resisting shear stress : $M_Z = \dfrac{(\gamma_c - \gamma_w)\left[\dfrac{z \tan\delta}{\sqrt{1+z^2}} - S_o\right]}{C_F \sqrt{1 + S_o^2}}$

$$M_Z = \frac{(140 - 62.4)\left[\dfrac{2 \tan 30^\circ}{\sqrt{1 + 2^2}} - 0.4\right]}{0.79\sqrt{1 + 0.4^2}} = \frac{(77.6)(0.1164)}{0.851} = 10.61 \text{ lbs}/\text{ft}^3$$

($C_F = 0.79$ for $Z = 2$, ref. 1, page 5)

$M^* = M_Z (d/y)$ where d = depth of Fabriform channel
y = flow depth

$$M^* = 10.61\left(\frac{2}{0.62}\right) = 34.23 \text{ lbs}/\text{ft}^3$$

Tractive shear stress : $T = 15.76 \text{ lb}/\text{ft}^2$

SUBJECT GENCO - Keystone

BY MPL DATE 6/18/96 PROJ. NO. 92-220-73-07

CHKD. BY KMB DATE 7/12/96 SHEET NO. 7 OF 7

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Weighted average resisting shear stress

$$M_w = \frac{(M \times b) + (M^* \times 2\ell_s)}{b + 2\ell_s} = \frac{\left(12.78 \frac{lb}{ft^3} \times 2'\right) + \left(34.23 \frac{lb}{ft^3} \times 2(4.47')\right)}{2' + 2(4.47')}$$

$$= \frac{25.56 + 306.02}{10.94} = 30.31 \ lb/ft^3$$

Weighted average tractive shear stress

$$T_w = 15.76 \ lb/ft^2$$

$$\text{Thickness} = t = \frac{T_w}{M_w} = \frac{15.76 \ lb/ft^2}{30.31 \ lb/ft^3} = 0.52 \ ft = \underline{\underline{6.2''}}$$

USE 8" USM

SUBJECT GENCO : Keystone Station

BY MRL DATE 6/7/96 PROJ. NO. 92-220-73-07

CHKD. BY KJB DATE 6/21/96 SHEET NO. 1 OF 8

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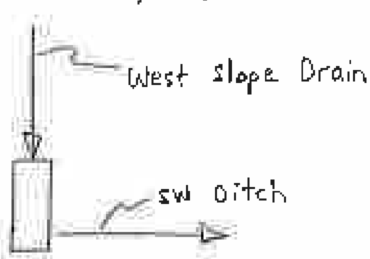
ENERGY DISSIPATOR

PURPOSE : Design an energy dissipator for the base of the West slope drain. Discharge flows from the energy dissipator into the southwest ditch.

INLET AND OUTLET DITCHES DESIGN PARAMETERS

Reference: "Ultimate Conditions Drainage Facilities" calculations by SER, sheets 5, 25, 36, and 42.



West Slope Drain

$Q = 60$ cfs

$S_{max} = 0.4$ ft./ft. (2.5H:1V) $S_{min} = 0.05$ ft./ft.

$n = 0.015$ (uniform section mat)

$b = 2'$

s.s. = 2H:1V

D(total depth) = 2.0'

SW Ditch

$Q = 60$ cfs

$s = 0.01$ ft./ft.

$n = 0.045$ (grassed)

$b = 2'$

s.s. = 2H:1V

D(total depth) = 4.0'

SUBJECT: Keystone Station
Phase II Permitting - Ultimate Conditions
BY: SER DATE: 4/12/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KMB DATE: 6/11/98 SHEET NO. 2 OF 8

GAI CONSULTANTS, INC.
Engineers Geologists Planners
Environmental Specialists

Purpose: Ditch Design (WEST SLOPE DRAIN)

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right) \cdot a \cdot r^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right) \cdot (r)^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$

West Slope Drain with Uniform Section Mat

Design Flow, $Q_d = 60 \cdot ft^3 \cdot sec^{-1}$ from sheet 1 of 8

Bottom Width, $b = 2 \cdot ft$

Side Slopes, $z = 2$

Channel Lining is Concrete Revetment, Uniform Section Mat with Manning's roughness coefficient, $n = 0.015$

Channel Minimum Slope, $S_{min} := \frac{5 \cdot ft}{100 \cdot ft}$ (from Sheet 1) or $S_{min} = 0.05 \cdot \frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 0.965 \cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 3.8 \cdot ft^2$

Minimum Velocity, $V_{min} = 15.8 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 5.9 \cdot ft$

Freeboard, $F_b = \cancel{0.5}\ 1.0 \cdot ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990

Total depth, $D = \cancel{1.5}\ 2.0 \cdot ft$

Top Width at Total Depth, $T_D = \cancel{8}\ 10 \cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 151 \cdot ft^3 \cdot sec^{-1}$

Channel Maximum Slope, $S_{max} := \frac{1 \cdot ft}{2.5 \cdot ft}$ (from Sheet 1) or $S_{max} = 0.4 \cdot \frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 0.568 \cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 1.8 \cdot ft^2$

Maximum Velocity, $V_{max} = 33.7 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 4.3 \cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 427 \cdot ft^3 \cdot sec^{-1}$

vKrG 5/4/05

INPUT INFORMATION:

FLOW RATE (cfs.)	MANNING'S 'N'	CHANNEL GRADE (ft./ft.)	SIDESLOPE (_H:1V)	BOTTOM WIDTH (ft.)
60.00	0.015	0.4000	2.00	2.0

WEST SLOPE DRAIN SOLUTION:

THE NORMAL DEPTH IN THE CHANNEL IS 0.57 ft. OR 6.8 in.

AREA (ft^2)	WETTED PREIMETER (ft)	HYDRAULIC RADIUS (ft)	FROUDE NUMBER	VELOCITY (ft/sec)	VELOCITY HEAD (ft)	TOTAL ENERGY (ft)	RIP-RAP SIZE (D50) (in)
1.79	4.54	0.39	~~83.98~~ 9.2	33.60	17.53	18.10	70.6

SUBJECT: Keystone Station
Phase II Permitting - Ultimate Conditions
BY: SER DATE: 4/12/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KMB DATE: 6/11/96 SHEET NO. 4 OF 8

GAI CONSULTANTS, INC.
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Environmental Specialists

Purpose: Ditch Design (SW DITCH)

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right) \cdot a \cdot r^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right) \cdot (r)^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$

Southwest Ditch - Part 1a (at entrance to Southwest Ditch)

Design Flow, $Q_d = 60 \cdot ft^3 \cdot sec^{-1}$ from sheet 1 of 8

Bottom Width, $b = 2 \cdot ft$

Side Slopes, $z = 2$

Channel Lining is Grass with Manning's roughness coefficient, $n = 0.045$

Channel Minimum Slope, $S_{min} := \frac{1 \cdot ft}{100 \cdot ft}$ (from Sheet 1) or $S_{min} = 0.01 \cdot \frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 2.337 \cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 15.6 \cdot ft^2$

Minimum Velocity, $V_{min} = 3.8 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 11.3 \cdot ft$

Freeboard, $F_b = \cancel{1.2} \cdot ft$ 1.7 by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990

Total depth, $D = \cancel{3.5} \cdot ft$ 4.0

Top Width at Total Depth, $T_D = \cancel{16} \cdot ft$ 18

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 153 \cdot ft^3 \cdot sec^{-1}$

Channel Maximum Slope, $S_{max} := \frac{1 \cdot ft}{100 \cdot ft}$ (from Sheet 1) or $S_{max} = 0.01 \cdot \frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 2.337 \cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 15.6 \cdot ft^2$

Maximum Velocity, $V_{max} = 3.8 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 11.3 \cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 153 \cdot ft^3 \cdot sec^{-1}$

SUBJECT GENCO: Keystone Station

BY MRL DATE 6/7/96 PROJ. NO. 92-220-73-07

CHKD. BY [illegible] DATE 6/12/96 SHEET NO. 5 OF 8

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CALCULATIONS:

As flow enters the energy dissipator from the West Slope Drain, a hydraulic jump will occur.

Determine the length of the hydraulic jump using Figure 15-4 (sheet 6)

Froude number for the West Slope Drain = 9.2 (see sheet 3)

Using Figure 15-4, $L/y_2 = 6.12 \rightarrow L = 6.12\, y_2$

Based on the proposed energy dissipator design (see sheet 8),

y_2 = tailwater = [(depth of flow in SW ditch) + (6" depression of dissipator floor)]

$y_2 = 2.34' + 0.5' = 2.84'$ This tailwater will control the jump.

↳ see sheet 4

Length of jump = $L = 6.12(y_2) = 6.12(2.84') = 17.38'$

$\boxed{\text{Use } L = 18'}$

SUBJECT GENCO: Keystone Station

BY MRC DATE 6/7/96 PROJ. NO. 92-220-73-07

CHKD. BY KMB DATE 6/4/96 SHEET NO. 6 OF 8

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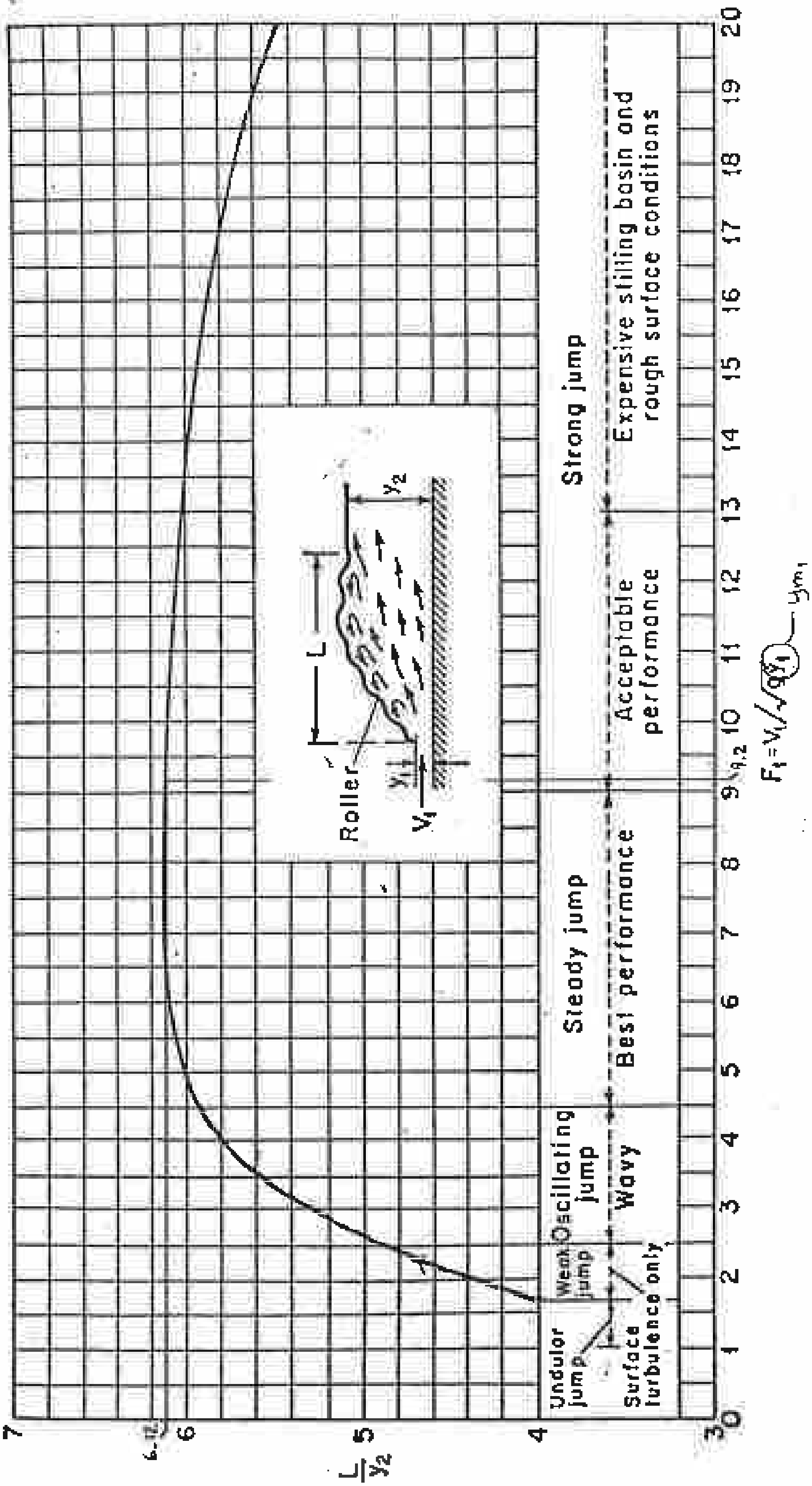

FIG. 15-4. Length in terms of sequent depth y_2 of jumps in horizontal channels. (*Based on data and recommendations of U.S. Bureau of Reclamation* [341].)

Reference: Open-channel Hydraulics
Ven Te Chow, McGraw-Hill Book Company, 1959

SUBJECT GENCO: Keystone Station

BY MRC DATE 6/10/96 PROJ. NO. 92-220-73-07

CHKD. BY [illegible] DATE 6/11/96 SHEET NO. 7 OF 8

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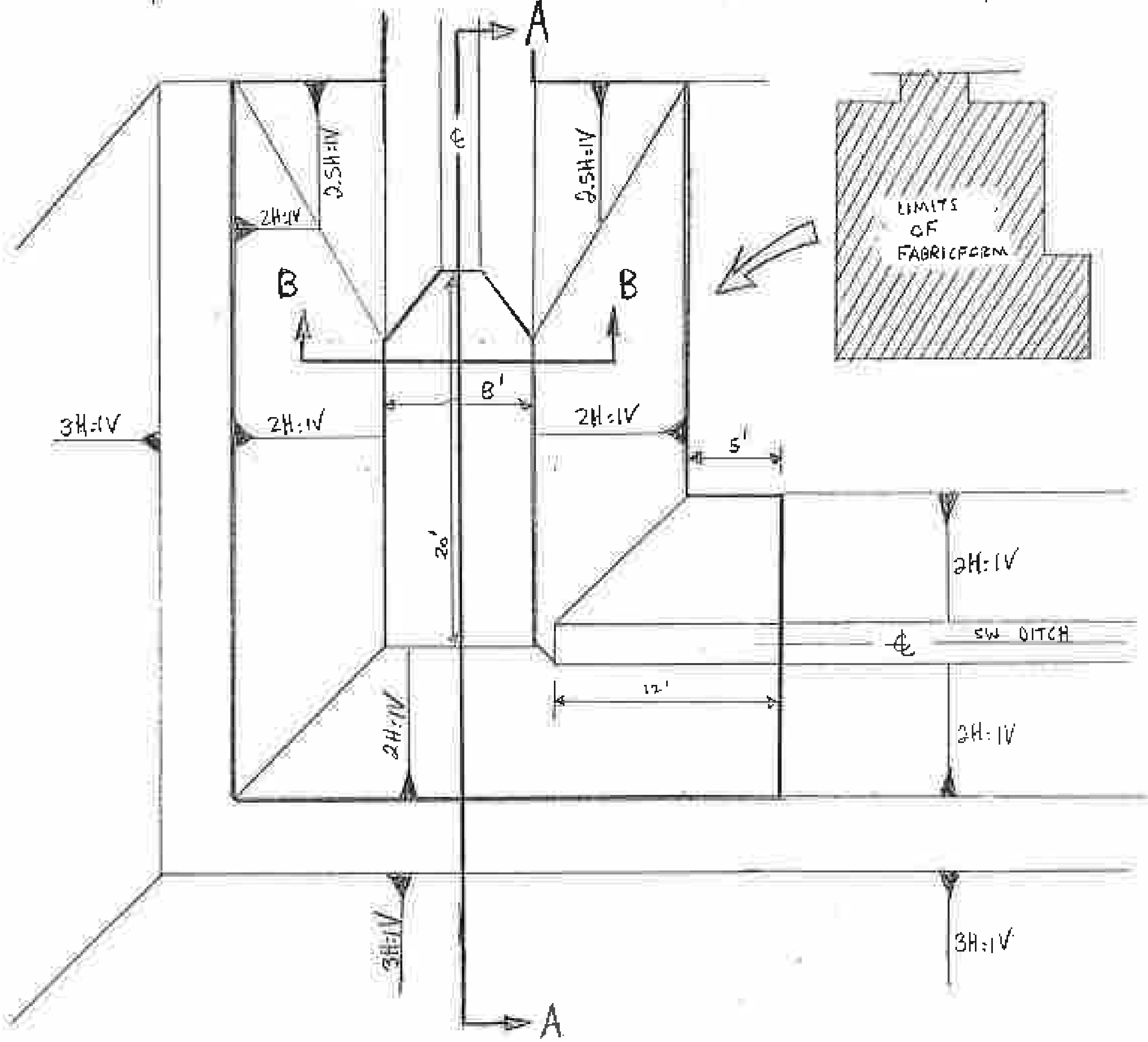

PLAN VIEW - ENERGY DISSIPATOR

N.T.S.

SUBJECT GENCO : Keystone Station

BY MRL DATE 6/10/96 PROJ. NO. 92-226-73-07

CHKD. BY [illegible] DATE 6/21/96 SHEET NO. 8 OF 8

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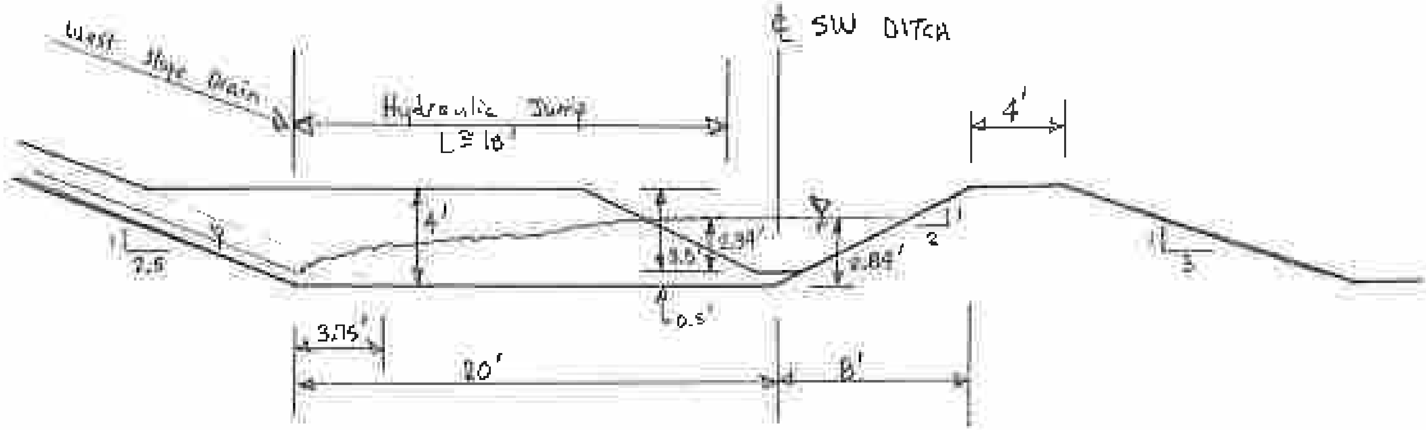
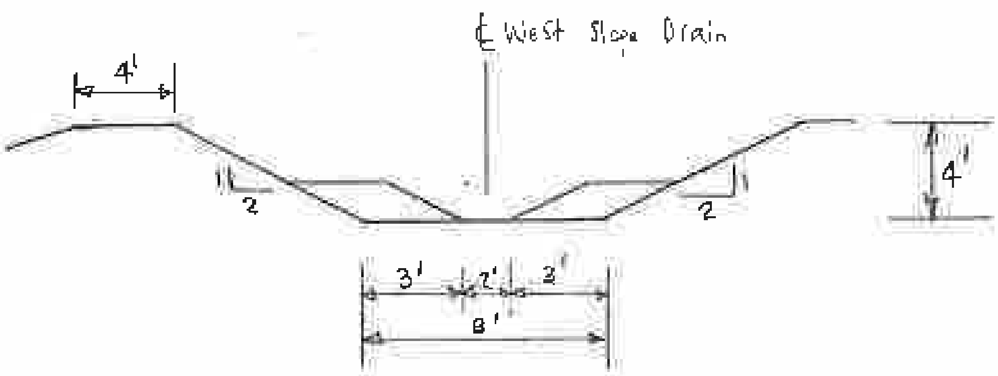


SECTION A-A

N.T.S.



SECTION B-B

N.T.S.

SUBJECT GENCO : Keystone

BY MRL DATE 6/13/96 PROJ. NO. 92-220-73-07

CHKD. BY KMB DATE 7/15/96 SHEET NO. 1 OF 8

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CHANNEL / CULVERT OUTLET PROTECTION

PURPOSE: Design outlet protection for the channels and culverts proposed for the site.

PROCEDURE: Develop two standard outlet protection designs. One design is for discharges less than or equal to 50 cfs and the other design is for discharges greater than 50 cfs but less than or equal to 100 cfs.

REFERENCES:

1. PA DER Erosion and Sediment Pollution Control Program Manual, April 1990.
2. NSA - Quarried Stone for Erosion and Sediment Control, March 1987.
3. Civil Engineering Review Manual, 3rd edition, Michael R. Lindeburg, 1981.
4. "Stage 3 - Drainage Facilities" calculations by SER, 4/25/96.
5. "Ultimate Conditions - Drainage Facilities" calculations by SER, 3/19/96

Channel and culvert outlet discharges vary from 29 CFS to 96 CFS. All channels and culverts ultimately discharge into streams / swales that have minimal tailwater (TW).

Use the attached Figure 1 as a guide to size riprap aprons for outlet protection. To use this figure, calculate the equivalent pipe diameter for the channels.

For trapezoidal channels, equivalent diameter (De)

$$= \frac{2h(a+b)}{(b+2s)}$$

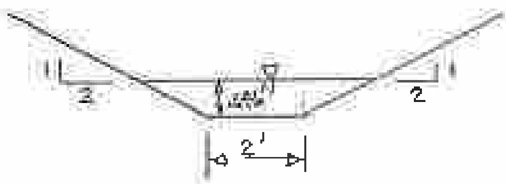
Reference (3)

where h = flow depth
a = flow top width
b = bottom width
s = length of side slope

Two sizes of riprap aprons will be designed. One will be for Q's ≤ 50 CFS and the other will be for Q's > 50 CFS and ≤ 100 CFS. For both cases, the outlet protection will be sized for the maximum slope case occurring in the "template" channel.

CALCULATE De FOR $Q \leq 50$ CFS

Use the stage 3 SW ditch flow characteristics to calculate a typical De. (Reference 3)



$Q_{SW\ DITCH} = 42$ CFS

$V = 19.8$ FPS

$h = 0.646'$

$a = [(.646) \times 2 \times 2] + 2 = 4.58'$

$b = 2'$

$s = \sqrt{.646^2 + 1.292^2} = 1.44'$

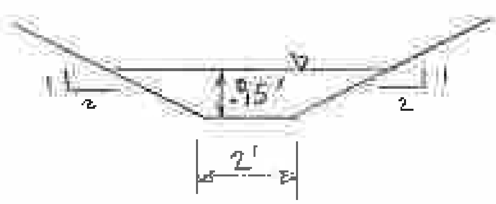
$$De = \frac{2(.646)(4.58 + 2)}{(2 + 2(1.44))} = \frac{8.50}{4.88} = 1.74'$$

$De = 20.9'' \cong 21''$

USE 22"

CALCULATE De FOR $50 < Q \leq 100$ CFS

Use the stage 4/ultimate SW ditch flow characteristics to calculate a typical De. (Reference 4)



$Q_{SW\ DITCH} = 90$ CFS

$V = 24.3$ FPS

$h = 0.95'$

$a = [(.95) \times 2 \times 2] + 2 = 5.8'$

$b = 2'$

$s = \sqrt{.95^2 + 1.9^2} = 2.12'$

$$De = \frac{2(.95)(5.8 + 2)}{(2 + 2(2.12))} = \frac{14.82}{6.24} = 2.38'$$

$De = 28.5'$

USE 29"

SUBJECT GENCO : Keystone

BY MRL DATE 6/13/96 PROJ. NO. 92-220-73-07

CHKD. BY KMB DATE 7/15/96 SHEET NO. 4 OF 8

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REFERENCE (1)

DESIGN OF OUTLET PROTECTION FROM A ROUND PIPE FLOWING FULL
MINIMUM TAILWATER CONDITION (T_w < 0.5 DIAMETER)

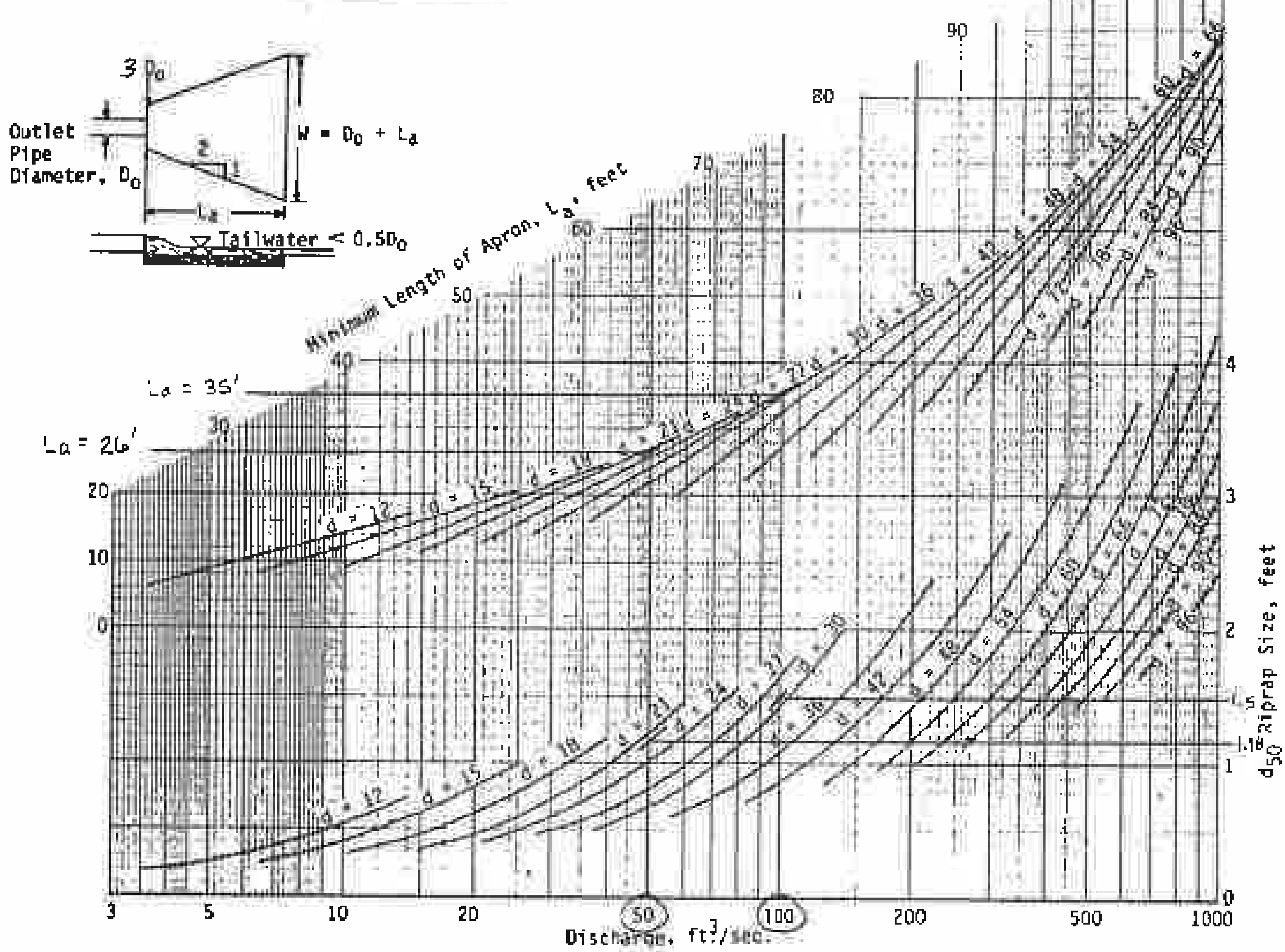

FIGURE 1

At Q = 50 cfs, De = d = 22", d_{50} = 1.18' = 14.2", La = 26'

At Q = 100 cfs, De = d = 29", d_{50} = 1.5' = 18", La = 35'

SUBJECT GENCO: Keystone

BY MRL DATE 6/14/96 PROJ. NO. 92-226-73-07

CHKD. BY KMB DATE 7/1/96 SHEET NO. 5 OF 8

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• ANALYZE FLOW CONDITIONS IN THE APRON ...

INPUT INFORMATION:

FLOW RATE (cfs.)	MANNING'S 'N'	CHANNEL GRADE (ft./ft.)	SIDESLOPE (_H:1V)	BOTTOM WIDTH (ft.)
Q = 50.00	0.040	0.0050	2.00	15.0

SOLUTION:

THE NORMAL DEPTH IN THE CHANNEL IS d = 1.13 ft. OR 13.5 in.

AREA (ft^2)	WETTED PREIMETER (ft)	HYDRAULIC RADIUS (ft)	FROUDE NUMBER	VELOCITY (ft/sec)	VELOCITY HEAD (ft)	TOTAL ENERGY (ft)	RIP-RAP SIZE (D50) (in)
19.43	20.04	0.97	0.21	2.57	0.10	1.23	NA

With d = 1.13', make D = 2.0' (See sheets 7 & 8 for apron design)

INPUT INFORMATION:

FLOW RATE (cfs.)	MANNING'S 'N'	CHANNEL GRADE (ft./ft.)	SIDESLOPE (_H:1V)	BOTTOM WIDTH (ft.)
Q = %100.00	0.040	0.0050	2.00	15.0

SOLUTION:

THE NORMAL DEPTH IN THE CHANNEL IS d = 1.68 ft. OR 20.2 in.

AREA (ft^2)	WETTED PREIMETER (ft)	HYDRAULIC RADIUS (ft)	FROUDE NUMBER	VELOCITY (ft/sec)	VELOCITY HEAD (ft)	TOTAL ENERGY (ft)	RIP-RAP SIZE (D50) (in)
30.86	22.52	1.37	0.23	3.24	0.16	1.84	NA

With d = 1.68', make D = 2.5' (See sheets 7 & 8 for apron design)

SUBJECT GENCO : Keystone

BY MRL DATE 6/14/96 PROJ. NO. 92-220-73-07

CHKD. BY [illegible] DATE 7/5/96 SHEET NO. 6 OF 8

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RIPRAP SIZING :

Table A REFERENCE (2)

Quarried Stone for Erosion & Sediment Control (5)

GRADED RIPRAP STONE

NSA No.	Size Inches (sq. openings) Max.	Avg. (1)	Min. (2)	Wave Height (3) (ft.)	Velocity (4) (ft./sec.)	Filter Stone NSA Size No.
R-1	1½	¾	(No. 8)	—	2.5	FS-1
R-2	3	1½	1	0.3	4.5	FS-1
R-3	6	3	2	0.5	6.5	FS-2
R-4	12	6	3	1.0	9.0	FS-2
R-5	18	9	5	1.5	11.5	FS-2
→ R-6	24	12	7	2.0	13.0	FS-3
R-7	30	15	12	2.5	14.5	FS-3
R-8	48	24	15	4.0	—	FS-3

Channel exit velocities range up to 20 fps @ 42 CFS and up to 24.3 fps @ 90 CFS. With these velocities, grout the riprap for the first 8' of the riprap apron. This will reduce the size of the required d_{50} rock.
For the 15' wide section of the riprap apron, velocities have been reduced substantially. (See sheet 5) However, since a hydraulic jump will occur on the riprap apron, maintain riprap for the entire apron.

USE NSA R-6 riprap

Place riprap 30" thick (2.5')

SUBJECT GENCO: Keystone

BY MRC DATE 6/11/96 PROJ. NO. 92-220-73-07

CHKD. BY KWPS DATE 7/15/96 SHEET NO. 7 OF 8

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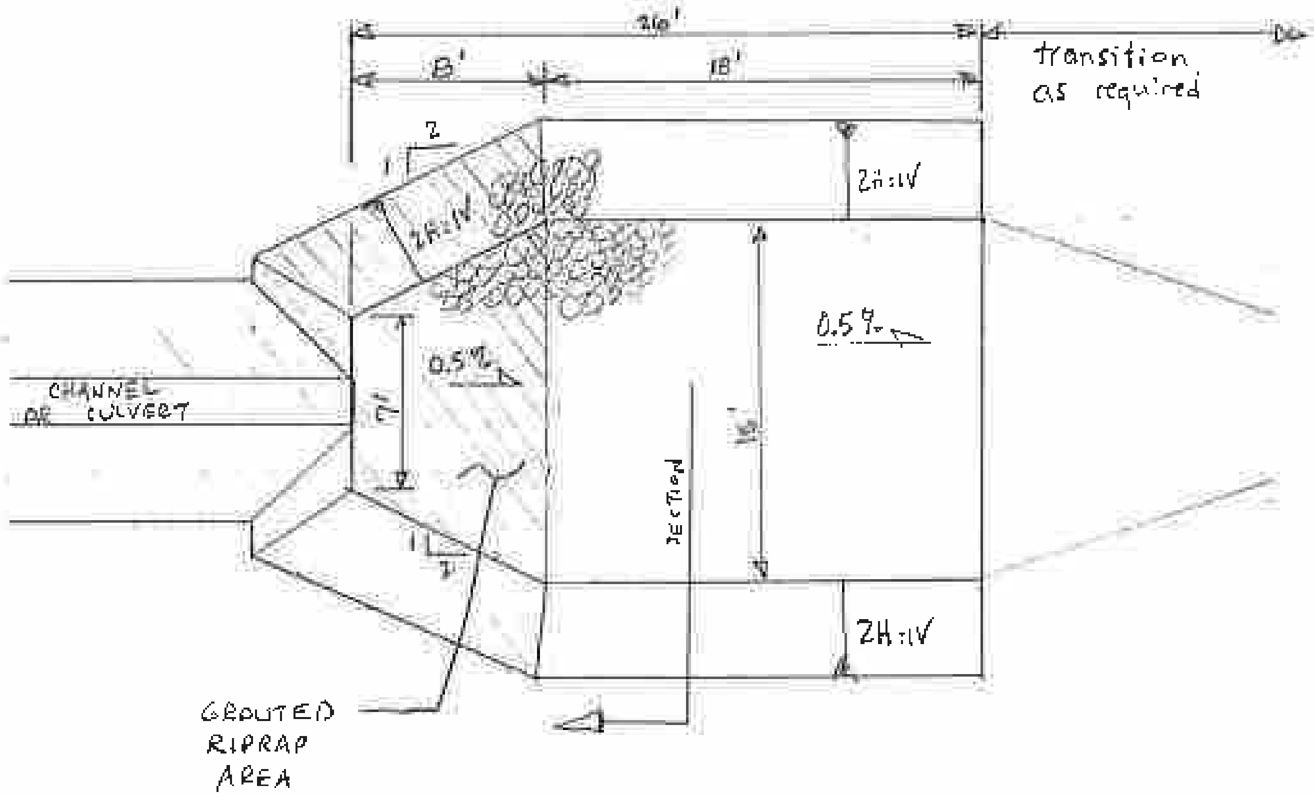
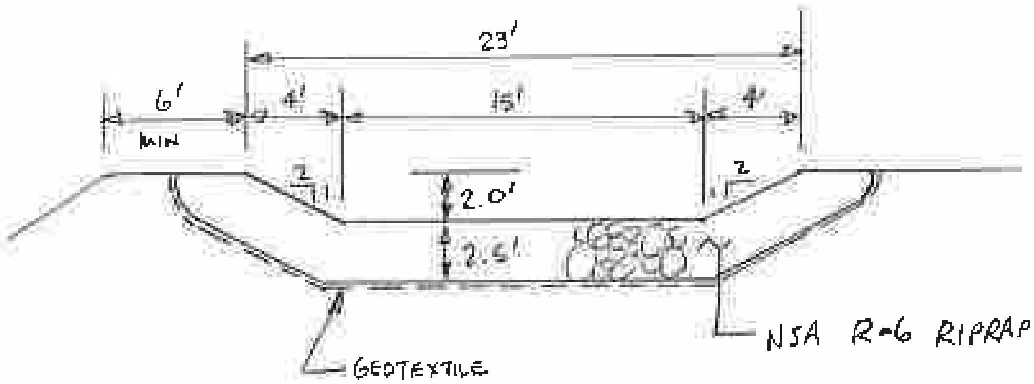
RIPRAP APRON PROTECTION

Q ≤ 50 CFS





SUBJECT GENCO : Keystone

BY MRL DATE 6/14/96 PROJ. NO. 92-220-73-07

CHKD. BY [illegible] DATE 7/15/96 SHEET NO. 8 OF 8

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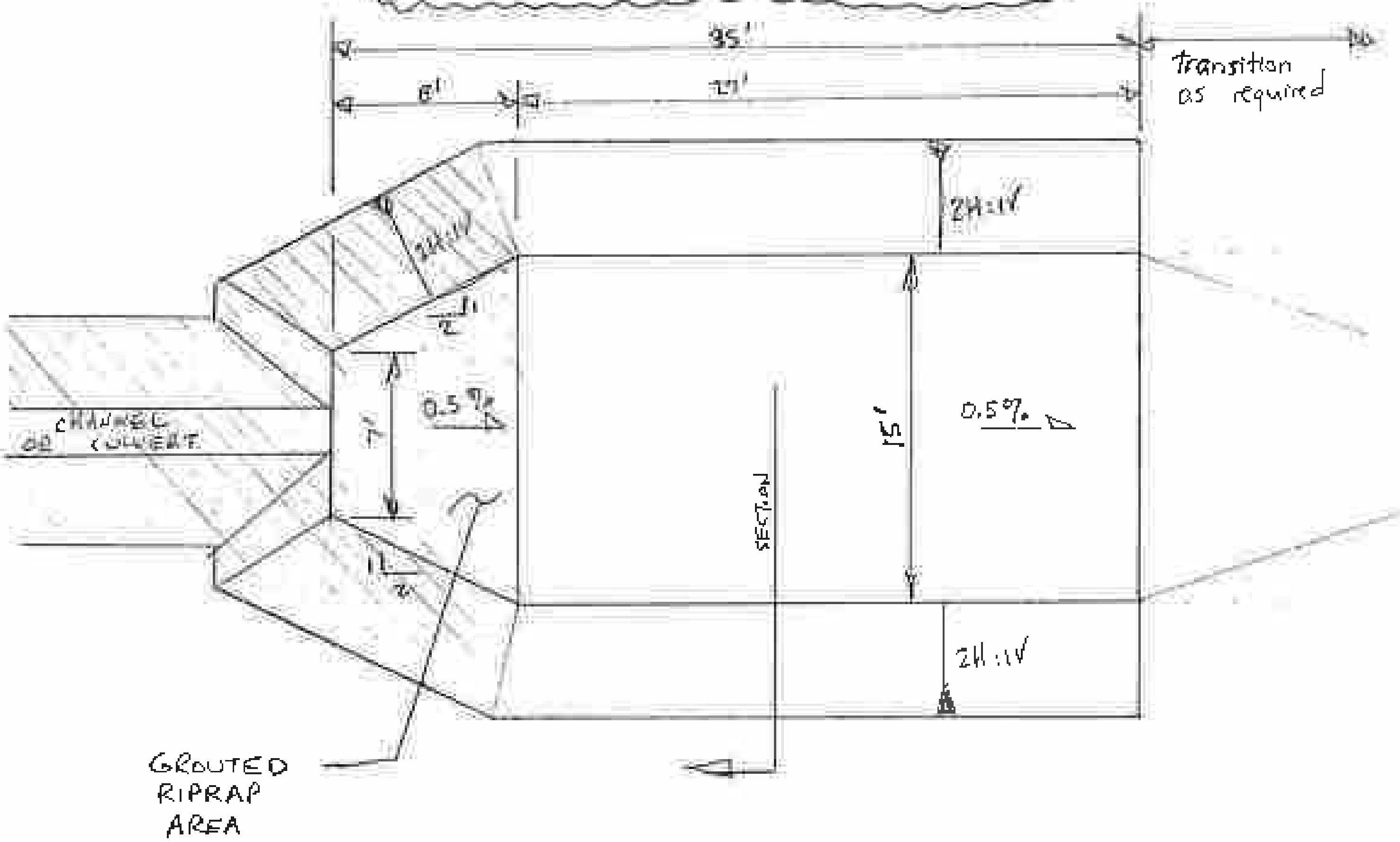
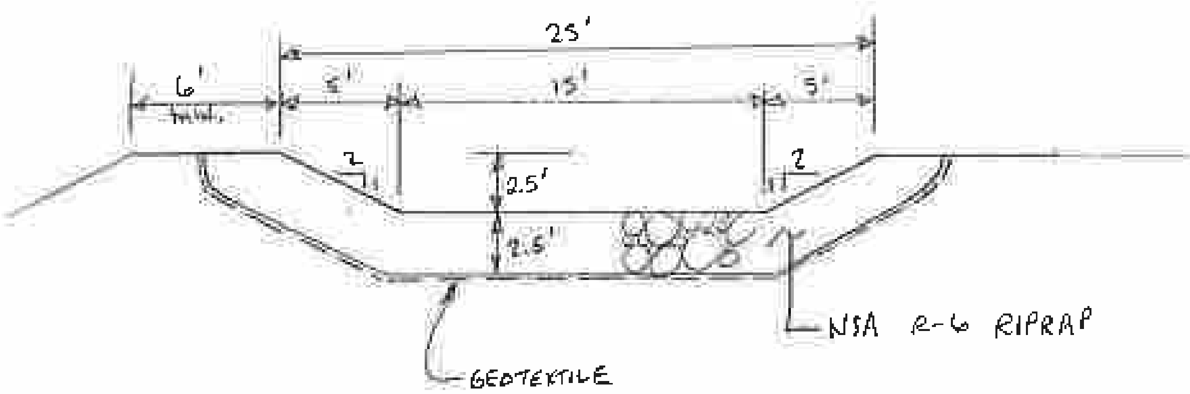
RIPRAP APRON PROTECTION

50 CFS < Q ≤ 100 CFS





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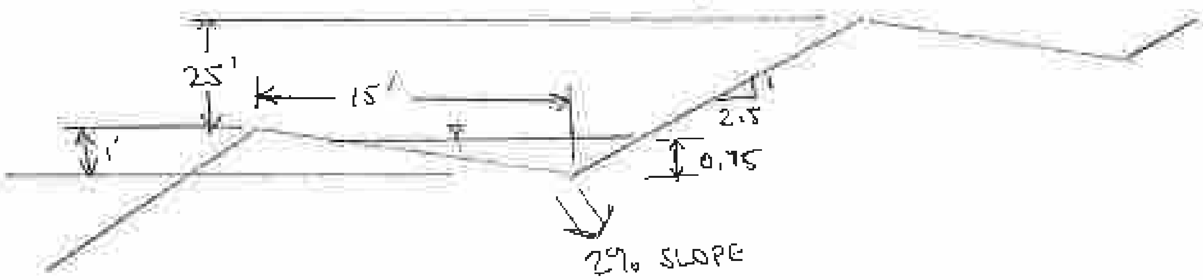
BENCH CAPACITY

DETERMINE THE MAXIMUM LENGTH OF BENCH THAT CAN BE ACHIEVED TO CONVEY RUNOFF FROM THE 25-YR 24-HR STORM. BENCHES WILL NOT RECEIVE RUNOFF FROM AN ACTIVE DISPOSAL AREA. THE PROPOSED BENCH LAYOUT / DIMENSIONS IS :



NTS

THE FLOW DEPTH WILL BE TAKEN AS 0.75' TO ALLOW FOR 0.25' OF FREEBOARD.

CALCULATE BENCH CAPACITY USING MANNING'S EQUATION

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

USE $n = 0.045$ FOR GRASSED PILE (KEYSTONE DESIGN PARAMETERS

CALCULATE A AND R FROM THE FOLLOWING :

$A = \frac{1}{2}(11.25)(0.75) + \frac{1}{2}(1.88 \times 0.75) = 4.92 \text{ ft}^2$

P = WETTED PERIMETER $= \sqrt{0.75^2 + 11.25^2} + \sqrt{0.75^2 \times 1.88^2} = 13.3$ FT

$R = A/P = 4.92 \text{ ft}^2 / 13.3 \text{ ft} = 0.37$ ft

$S = 0.02$

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BENCH CAPACITY CONTINUED

$$Q = \frac{1.49}{0.045}(4.92)(0.37)^{2/3}(0.02)^{1/2} = 11.9 \text{ cfs}$$

USING THIS ALLOWABLE CAPACITY, CALCULATE THE MAXIMUM LENGTH OF BENCH THAT MAY BE DRAINED DURING THE 25-YR 24-HR STORM

USE TR-55 METHODS
USE CN = 78 FOR VEGETATED PILE, BENCH FACE (KEYSTONE DESIGN PARAMETERS)
25-YR 24-HR PRECIPITATION FOR THE SITE = 4.4 inches (ARMSTRONG COUNTY)

CALCULATE THE RUNOFF FROM TR-55:

$$S = \frac{1000}{CN} - 10 = \frac{1000}{78} - 10 = 2.82$$

$$Q = \text{RUNOFF} = \frac{(P - 0.2S)^2}{(P + 0.8S)} = \frac{(4.4 - 0.2 \times 2.82)^2}{(4.4 + 0.8 \times 2.82)} = 2.21 \text{ in}$$

$$I_a = 0.2S = 0.2 \times 2.82 = 0.564$$
$$I_a/P = 0.564/4.4 = 0.13$$

FROM TR-55, q_p = PEAK DISCHARGE = $q_u A_m Q F_p$

$Q = 2.21$ inches FROM ABOVE
USE F_p = PONDING FACTOR = 1.0
A_m AND q_u WILL DEPEND ON BENCH LENGTH.

$$q_p = q_u A_m \times 2.21 \times 1 = 2.21\, q_u A_m$$

A_m = AREA IN SQUARE MILES
q_u WILL BE DETERMINED FROM EXHIBIT 4-II

SUBJECT KEYSTONE WEST VALLEY
PHASE II PERMITTING

BY KMB DATE 6/4/96 PROJ. NO. 92-220-73-07
CHKD. BY MRL DATE 6/19/96 SHEET NO. 3 OF 4

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BENCH CAPACITY CONTINUED

THE UNIT AREA THAT WILL BE DRAINED BY THE BENCH IS :

1' LONG x (15' BENCH SECTION + (26' VERTICAL x 2.5:1))
= 80 ft^2/ft

THE FACTOR q_u WILL DEPEND ON TIME OF CONCENTRATION.
FROM STAGE 3 DRAINAGE HYDROLOGY CALCULATIONS, THE t_c FOR FLOW FOR SHEET FLOW TO THE BENCH = 0.056 hr.

$\therefore t_c = 0.056 +$ TRAVEL TIME ALONG BENCH LENGTH

BENCH VELOCITY = CAPACITY / FLOW AREA = 11.9 cfs / 4.92 ft^2 = 2.4 ft/s

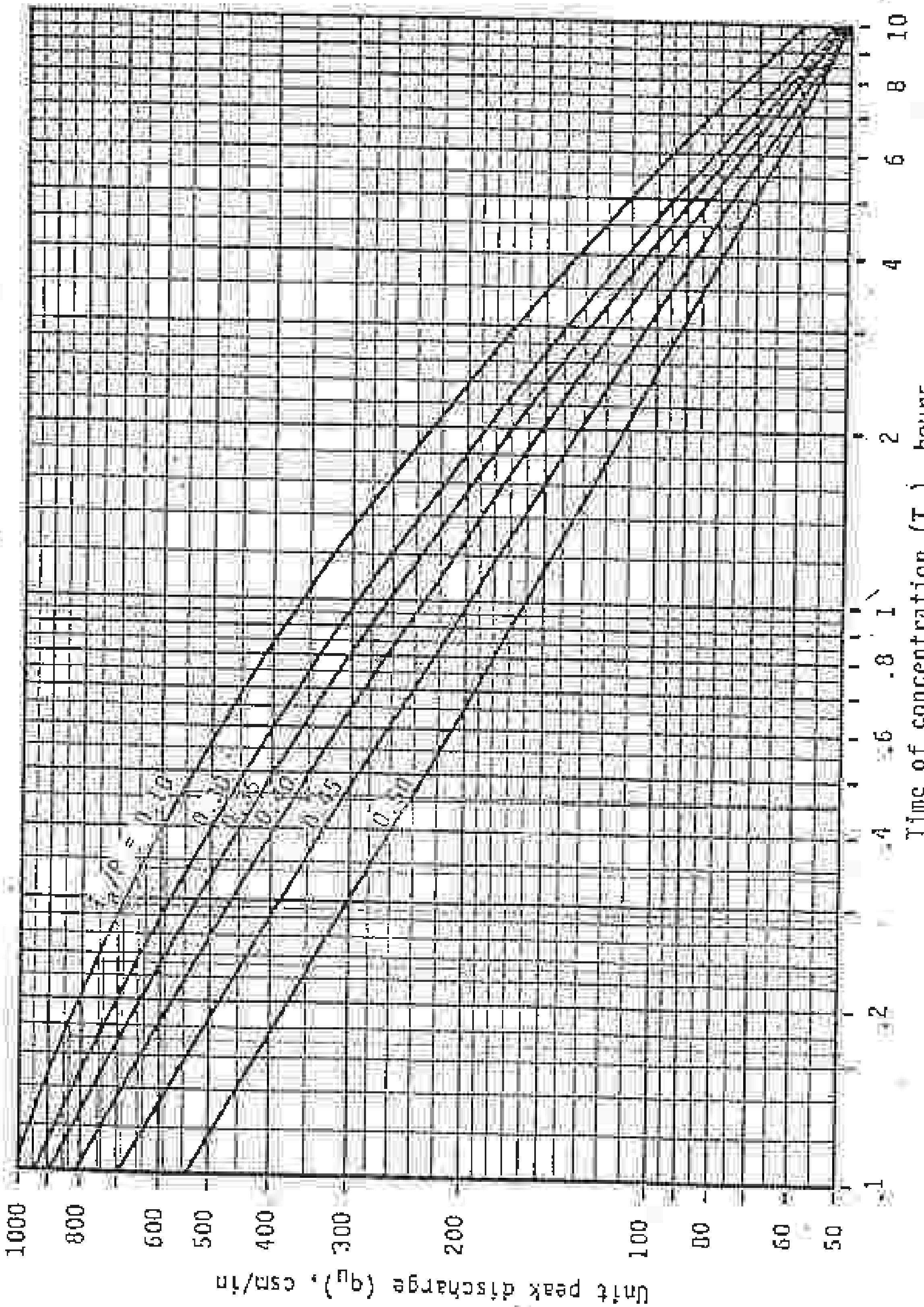
$\therefore t_c = 0.056 + \frac{L}{3600 \times 2.4} = 0.056 + \frac{L}{8640}$

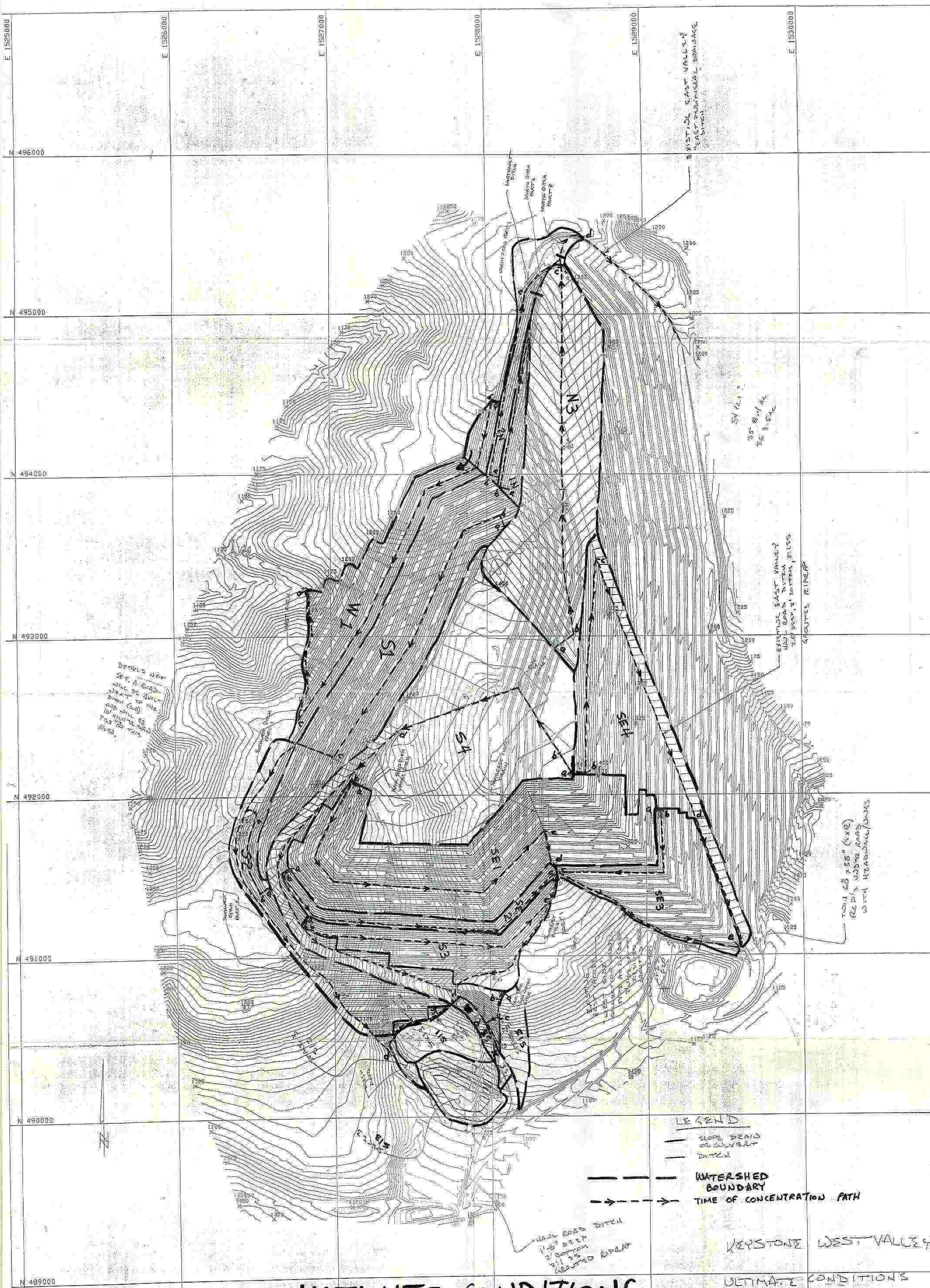
BENCH LENGTH (ft)	BENCH DRAINAGE AREA (ft²)	(Am) BENCH DRAINAGE AREA (mi²)	t_c (hr)	q_u (csm/in)	q_p (cfs)
1000	80,000	0.00287	0.17	840	5.3
1500	120,000	0.00430	0.23	740	7.0
2000	160,000	0.00574	0.29	690	8.8
2500	200,000	0.00717	0.34	640	10.1
3000	240,000	0.00861	0.40	590	11.2
3500	280,000	0.0100	0.46	550	12.2

INTERPOLATING THE CAPACITY OF 11.9 cfs, THE MAXIMUM ALLOWABLE BENCH LENGTH = 3350 ft

KEYSTONE - WEST VALLEY
PHASE II PERMITTING
BENCH CAPACITY
92-220-73-07
KMB 6/4/96
✓MRL 6/19/96 7/4

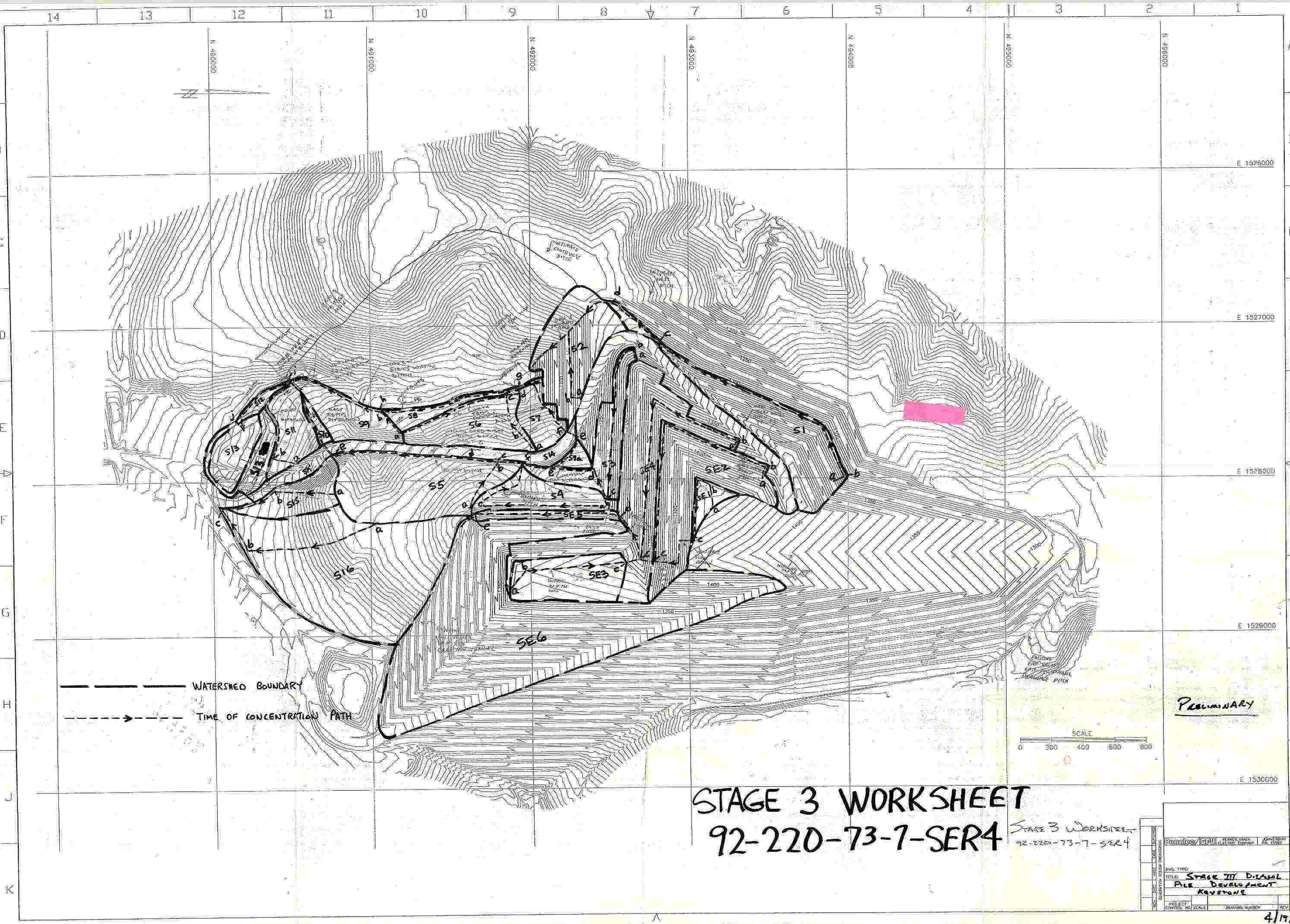
Exhibit 4-II: Unit peak discharge (q_u) for SCS type II rainfall distribution





ULTIMATE CONDITIONS
WORKSHEET 92-220-73-7-SER 1


EV-A
S3-A
a
b
c
WATERSHED BOUNDARY
TIME OF CONCENTRATION PATH
SCALE 1" = 200'
200 0 200 400
STAGE 3 HAUL ROAD
DIRTY WATER DITCH
WORKSHEET 92-220-73-7-5ER2
GAI Dwg. File No.
Penelec/GPU PENNSYLVANIA ELECTRIC COMPANY JOHNSTOWN PA 15907
KEYSTONE GENERATING STATION
DISPOSAL SITE
PLUM CREEK TOWNSHIP
DWG. TYPE: CIVIL
TITLE:
1" = 200'
D-000-0000
PROJECT CONTROL NO
SCALE
DRAWING NUMBER
REV
GENERATION DESIGN ENGINEERING
EAST VALLEY EQUALIZATION PONDS
CONCRETE FLUME
INTERMITTENT STREAM
CULTIVATED FIELD
TRAIL
TOWER
TREES



WATERSHED BOUNDARY
TIME OF CONCENTRATION PATH
PRELIMINARY
SCALE
0 200 400 600 800
N 490000
N 491000
N 492000
N 493000
N 494000
N 495000
N 496000
E 1526000
E 1527000
E 1528000
E 1529000
E 1530000
S1
S2
S3
S4
S5
S6
S7
S8
S9
S10
S11
S12
S13
S14
S15
S16
SE1
SE2
SE3
SE5
SE6
STAGE 3 WORKSHEET
92-220-73-7-SER4
Stage 3 Worksheet
92-220-73-7-SER4
Penelec/GPU
PENNSYLVANIA ELECTRIC COMPANY
JOHNSTOWN PA 15907
TITLE: Stage III Disposal Pile Development Keystone
PROJECT CONTROL NO
SCALE
DRAWING NUMBER
REV
4/19/96

APPENDIX I-1-B

FORM I

WEST CLEAN STORMWATER MANAGEMENT
POND AND DRAINAGE TO PLUM CREEK

SUBJECT ______

BY ______ DATE ______ PROJ. NO. ______

CHKD. BY ______ DATE ______ SHEET NO. ______ OF ______

GAI CONSULTANTS, INC.
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WEST CLEAN STORMWATER MANAGEMENT POND AND DRAINAGE TO PLUM CREEK

TABLE OF CONTENTS

SUBJECT KEYSTONE FORM I, APPENDIX I-1-B
DISPOSAL SITE

BY SER DATE 6/24/96 PROJ. NO. 92-220-73-07

CHKD. BY KMR DATE 7/11/96 SHEET NO. 1 OF 25

RVD SER 1/22/97, Chkd JMW

GAI CONSULTANTS, INC.
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STORMWATER MANAGEMENT - WEST CLEAN STORMWATER MANAGEMENT POND

PURPOSE: DESIGN A POND TO CONTROL STORMWATER THAT FLOWS TO THE WEST TO A CULVERT BENEATH ROUTE 210. PROVIDE CONTROLS WHICH LIMIT POST-DEVELOPMENT PEAK FLOWS TO THE EXISTING PRE-DEVELOPMENT PEAK FLOWS FOR THE 2, 10, 25 AND 100 YEAR 24 HOUR STORM EVENTS. PROVIDE A MINIMUM OF 1 FOOT FREEBOARD.

ADDITIONAL DESIGN CRITERIA: PROVIDE STORMWATER CONTROLS ~~WHERE POSSIBLE~~ FOR INTERMEDIATE CONDITIONS FOR THE 2, 10, AND 25 YEAR STORM EVENTS. THAT IS, PROVIDE CONTROLS FOR PHASES OF CONSTRUCTION WHICH HAVE SIGNIFICANTLY DIFFERENT DRAINAGE PATTERNS THAN THE POST-DEVELOPMENT CONDITION. ~~SINCE THESE INTERMEDIATE CONDITIONS WILL BE SHORT TERM, 2-5 YRS IN DURATION, CONTROLS FOR LOW FREQUENCY EVENTS ARE NOT CONSIDERED NECESSARY~~. ANALYZE THE FOLLOWING INTERMEDIATE CONDITIONS IN ADDITION TO THE POST-DEVELOPMENT CONDITION;

1) STAGE 3 CONDITIONS - THE HAUL ROAD CLEAN WATER DITCH PART 1 WILL TEMPORARILY FLOW TO THE WEST DITCH WHICH DISCHARGES TO THE POND.

2) STAGE 3A CONDITIONS - THE DIVERSION DITCH D31 WILL FLOW TO THE POND UNTIL STAGE 3C IS CONSTRUCTED.

DESIGN THE OUTLET STRUCTURE TO MANAGE FLOW FOR THE 25-YEAR EVENT FOR ALL CONDITIONS. PROVIDE A MINIMUM OF 1 FOOT FREEBOARD FOR ALL CONDITIONS FOR THE 25-YEAR EVENT.

SEE SHEET 22 FOR FLOW AND ELEV SUMMARY AND CONCLUSIONS.

SUBJECT KEYSTONE

BY SER DATE 6/24/96 PROJ NO. 92-220-73-7

CHKD. BY KMR DATE 7/11/96 SHEET NO. 2 OF 25

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DESIGN STORM RAINFALL (FROM PA EROSION AND SEDIMENT POLLUTION CONTROL PROGRAM MANUAL, PENNDER, APRIL 1990)

RETURN PERIOD YEARS	24 HOUR RAINFALL
2	2.6
10	3.9
25	4.4
100	5.2

METHODOLOGY: USE 1) THE PC VERSION OF SCS'S TR-20, 2) TR-55 "URBAN HYDROLOGY FOR SMALL WATERSHEDS", SCS, JUNE, 1986 AND 3) HDS NO 5, "HYDRAULIC DESIGN OF HIGHWAY CULVERTS", FHWA SEPT. 1985

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PRE-DEVELOPMENT CONDITIONS

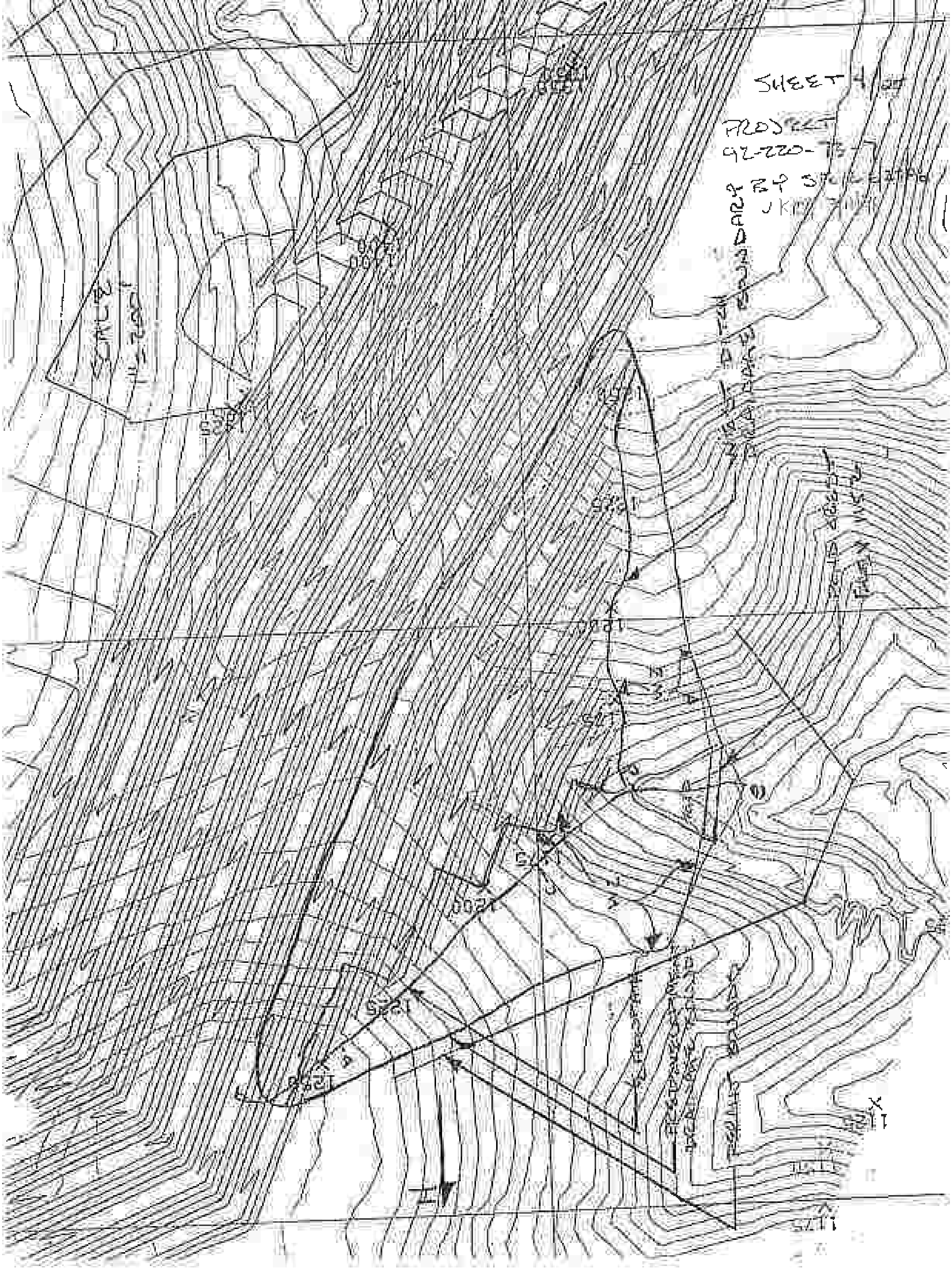
THE PREDEVELOPMENT DRAINAGE AREA AND t_c FLOW PATH IS SHOWN ON SHEET 4.

AREA = 12.6 ACRE = 0.0197 SQ. MI

THE TIME-OF-CONCENTRATION IS ESTIMATED ON SHEET 5

t_c = 0.24 HR

THE LAND USE FOR THIS AREA IS STRAIGHT ROW CROPS OR PASTURE, USE CN=80 AS PER THE KEYSTONE STATION, PROJECT DESIGN PARAMETERS OUTLINE, 85-376-4, SEPTEMBER 1987


SHEET 4
92-220-

SUBJECT: Keystone West Valley
Phase II Permitting
BY: SER DATE:6/24/96 PROJ. NO.: 92-220-73-07
CHKD. BY: [handwritten initials] DATE: 7/1/96 SHEET NO. 5 OF 25

Time of Concentration Worksheet - SCS Methods
Watershed - Predevelopment West Clean SWM Pond
Postdevelopment Conditions

Reference: "Urban Hydrology for Small Watersheds", TR-55, Soil Conservation Service, June 1986

SHEET FLOW — Flowpath: a-b — units

1. Surface description (table 3-1) — Cultivated Field with cover > 20%
2. Manning's roughness coeff., n_{st} (table 3-1) — $n_{st} := 0.17$
3. Flow length, L_{st} (total $L_{st} \le 150$ feet) — $L_{st} := 150$ feet
4. Two-year, 24-hour rainfall, P_2 — $P_2 := 2.6$ inches
5. Land Slope, $S_{st} := \frac{1254 - 1247}{L_{st}}$ — $S_{st} = 0.047$
6. Sheet Flow Time, $T_{st} := \frac{0.007 \cdot (n_{st} \cdot L_{st})^{0.8}}{P_2^{0.5} \cdot S_{st}^{0.4}}$ — $T_{st} = 0.197$ hours

SHALLOW CONCENTRATED FLOW — Flowpath: b-c — Flowpath: c-d

7. Surface description (paved or unpaved) — unpaved — unpaved
8. Flow length, L_{sc} — $L_{sc} := 470$ feet — $L_{sc1} := 220$
9. Watercourse Slope, $S_{sc} := \frac{1247 - 1190}{L_{sc}}$ — $S_{sc} = 0.121$ — $S_{sc1} := \frac{1190 - 1150}{L_{sc1}}$ — $S_{sc1} = 0.182$
10. Average Velocity, $V_{sc} := 16.1345 \cdot S_{sc}^{0.5}$ — $V_{sc} = 5.619$ fps — $V_{sc1} := 16.1345 \cdot S_{sc1}^{0.5}$ — $V_{sc1} = 6.88$
11. Shallow Conc. Flow time, $T_{sc} := \left(\frac{L_{sc}}{3600 \cdot V_{sc}}\right)$ — $T_{sc} = 0.023$ hour — $T_{sc1} := \left(\frac{L_{sc1}}{3600 \cdot V_{sc1}}\right)$ — $T_{sc1} = 0.009$

CHANNEL FLOW — Flowpath: d-e

12. Bottom width, b — $b := 2$ feet
13. Side slopes, z — $z := 2$
14. Flow depth, d — $d := 1.5$ feet
15. Cross sectional area, $a := (b + z \cdot d) \cdot d$ — $a = 7.5$ ft^2
16. Wetted perimeter, $P_w := \left[b + 2 \cdot d \cdot (1 - z^2)^{0.5}\right]$ — $P_w = 8.708$ feet
17. Hydraulic radius, $r := \frac{a}{P_w}$ — $r = 0.861$ feet
18. Channel Length, L_{ch} — $L_{ch} := 200$ feet
19. Channel Slope, $S_{ch} := \frac{1150 - 1138}{L_{ch}}$ — $S_{ch} = 0.06$
20. Channel lining — Grass
21. Manning's roughness coeff., n — $n := 0.045$
22. Velocity, $V_{ch} := \left[\left(\frac{1.49}{n}\right) \cdot \left[r^{\left(\frac{2}{3}\right)}\right] \cdot S_{ch}^{\left(\frac{1}{2}\right)}\right]$ — $V_{ch} = 7.342$ fps
22. Channel Flow time, $T_{ch} := \left(\frac{L_{ch}}{3600 \cdot V_{ch}}\right)$ — $T_{ch} = 0.0076$ hour

Total Watershed Time-of-Concentration, $T_c := T_{st} + T_{sc} + T_{sc1} + T_{ch}$ — $T_c = 0.24$ hour

SUBJECT KEYSTONE

BY SER DATE 6/24/96 PROJ. NO. 92-220-73-7

CHKD. BY KMR DATE 7/11/96 SHEET NO. 6 OF 25

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POST-DEVELOPMENT

THE POST-DEVELOPMENT CONDITION IS THE ULTIMATE CONDITION.

THE DRAINAGE AREA TO THE POND CONSISTS OF THE DRAINAGE AREA TO THE "ULTIMATE CONDITIONS" WEST DITCH (CALLED DRAINAGE AREA W1) AND AN INCREMENTAL DRAINAGE AREA (CALLED W2) (SEE SHEET 4). USE A COMPOSITE AREA FOR THE DRAINAGE INTO THE POND. THE AREA AND CN FOR THIS COMPOSITE AREA IS DOCUMENTED ON SHEET 10. THE TIME-OF-CONCENTRATION, t_c IS ASSUMED TO BE EQUAL TO THE t_c OF THE ULT. COND. WEST DITCH t_c = 0.31 HR

THE AREA DOWNSTREAM OF THE POND CREST TO THE POINT OF INTEREST @, WHICH IS ESSENTIALLY THE DOWNSTREAM SLOPE OF THE EMBANKMENT, WILL BE NEGLECTED.

REFERENCE "ULTIMATE CONDITIONS - DRAINAGE FACILITIES" CALC BY SER 3/19/96 FOR AREA, CN, AND t_c VALUES.

SUBJECT KEYSTONE

BY SER DATE 6/24/96 PROJ. NO. 92-220-73-7

CHKD. BY KMK DATE 7/11/96 SHEET NO. 7 OF 25

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STAGE 3 CONDITIONS

THE DRAINAGE AREA TO THE POND FOR STAGE 3 CONDITIONS CONSISTS OF THE ULTIMATE CONDITIONS DRAINAGE AREAS W1 AND W2 AND THE STAGE 3 CONDITIONS AREA S1. USE A COMPOSITE AREA FOR THE DRAINAGE INTO THE POND. THE AREA AND CN FOR THIS COMPOSITE AREA IS DOCUMENTED ON SHEET 10. THE t_c IS ASSUMED TO BE EQUAL TO THE MAXIMUM OF THE t_c FOR THE ULT. COND. WEST DITCH, t_c = 0.31 HR AND THE t_c FOR THE STAGE 3 HAUL ROAD CLEAN WATER DITCH, PART 1 PLUS PATH C-d OF THE ULT. COND. WEST DITCH, t_c = 0.216 + 0.017 = 0.23 HR

USE t_c = 0.31 HR.

REFERENCE "STAGE 3 - DRAINAGE FACILITIES" CALC BY SER 4/25/96 FOR AREA, CN, AND t_c VALUES.

SUBJECT KEYSTONE

BY SER DATE 6/24/96 PROJ. NO. 92-220-73-7
CHKD. BY KMB DATE 7/11/96 SHEET NO. 8 OF 25

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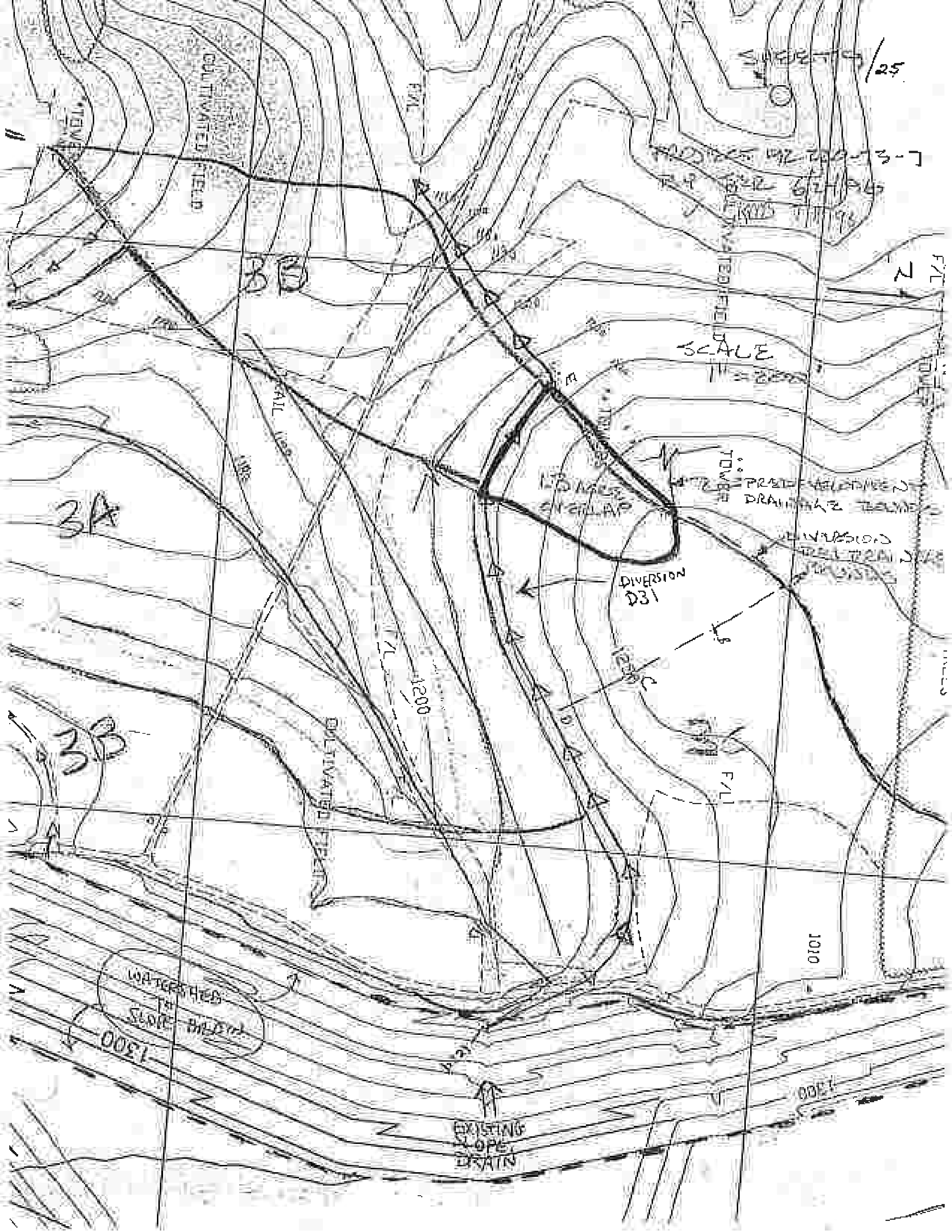
STAGE 3A CONDITIONS

THE DRAINAGE AREA TO THE POND DURING INITIAL CONSTRUCTION OF STAGE 3, BEFORE STAGE 3C IS CONSTRUCTED, CONSISTS OF THE PREDEVELOPMENT DRAINAGE AREA AND AN AREA DIVERTED BY DIVERSION D31. USE A COMPOSITE AREA FOR THE DRAINAGE INTO THE POND. THE AREA AND CN FOR THIS COMPOSITE IS DOCUMENTED ON SHEET 10. THE t_c IS ASSUMED TO BE EQUAL TO THE t_c FOR DIVERSION D31 $t_c = 0.50$ HOUR.

REFERENCE "STAGE 3 AND STAGE 4 TEMPORARY DIVERSIONS" CALC BY KMB 6/11/96.

THE DRAINAGE AREA FROM THE ABOVE REFERENCE OVERLAPS THE PRE DEV. DRAINAGE AREA. THIS IS SHOWN ON SHEET 9 AND ACCOUNTED FOR ON SHEET 10.



CULTIVATED FIELD
3B
3A
3B
SCALE
DIVERSION
D31
TOWER
F/L
1200
CULTIVATED FIELD
1010
1300
1300
EXISTING
SLOPE
DRAIN

Keystone West Valley
Phase II Permitting

By : SER Date: 8/24/98
Chkd By: [illegible] Date: 5/11/96

Project No. 92-220-73-7
Sheet No. 10 of 25

West Clean Stormwater Management Pond

Area and Curve Number Summary

Watershed	Total Area (Acres)	Total Area (SQ. MILES)	Composite CN	Areas of Individual Land Covers (Acres) CN =	Revegetated Pile Top 75	Revegetated Pile Bench Face 78	Active Area or Bottom Ash Haul Road 85	Paved Haul Road 98	Ponds 100	Pasture Offsite 80
Ultimate/ Postdevelopment Conditions										
Ultimate Conditions W1	12.3	0.0192	78		0.0	12.3	0.0	0.0	0.0	0.0
Ultimate Conditions W2	5.4	0.0084	81		0.0	0.0	0.0	0.0	0.3	5.1
Ult. Cond. Composite	17.7	0.0277	79							
Stage 3 Conditions										
Stage 3 Conditions S1	8.6	0.0134	78		0.0	8.6	0.0	0.0	0.0	0.0
Ultimate Conditions W1	12.3	0.0192	78		0.0	12.3	0.0	0.0	0.0	0.0
Ultimate Conditions W2	5.4	0.0084	81		0.0	0.0	0.0	0.0	0.3	5.1
Stage 3 Cond. Composite	26.3	0.0411	79							
Stage 3A Conditions										
Diversion D31 Drainage Area = 1.3 acre	27.2	0.0425	79		0.0	14.4	0.0	0.0	0.0	12.8
Predevelopment with pond	12.6	0.0197	80		0.0	0.0	0.0	0.0	0.3	12.3
Stage 3A Cond. Composite	39.8	0.0622	79							

d:\penciloc\keystone\phase2\ksph2acn.wk3

SUBJECT KEYSTONE

BY SER DATE 6/24/96 PROJ. NO. 92-220-73-7

CHKD BY KMB DATE 7/11/96 SHEET NO. 11 OF 25

RVD BY SER 10/27/97, Chkd JMJ

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POND DESIGN (Enlarged during 10/27/97 Revision)

THE POND IS SHOWN ON SHEET 12 IN PLAN.

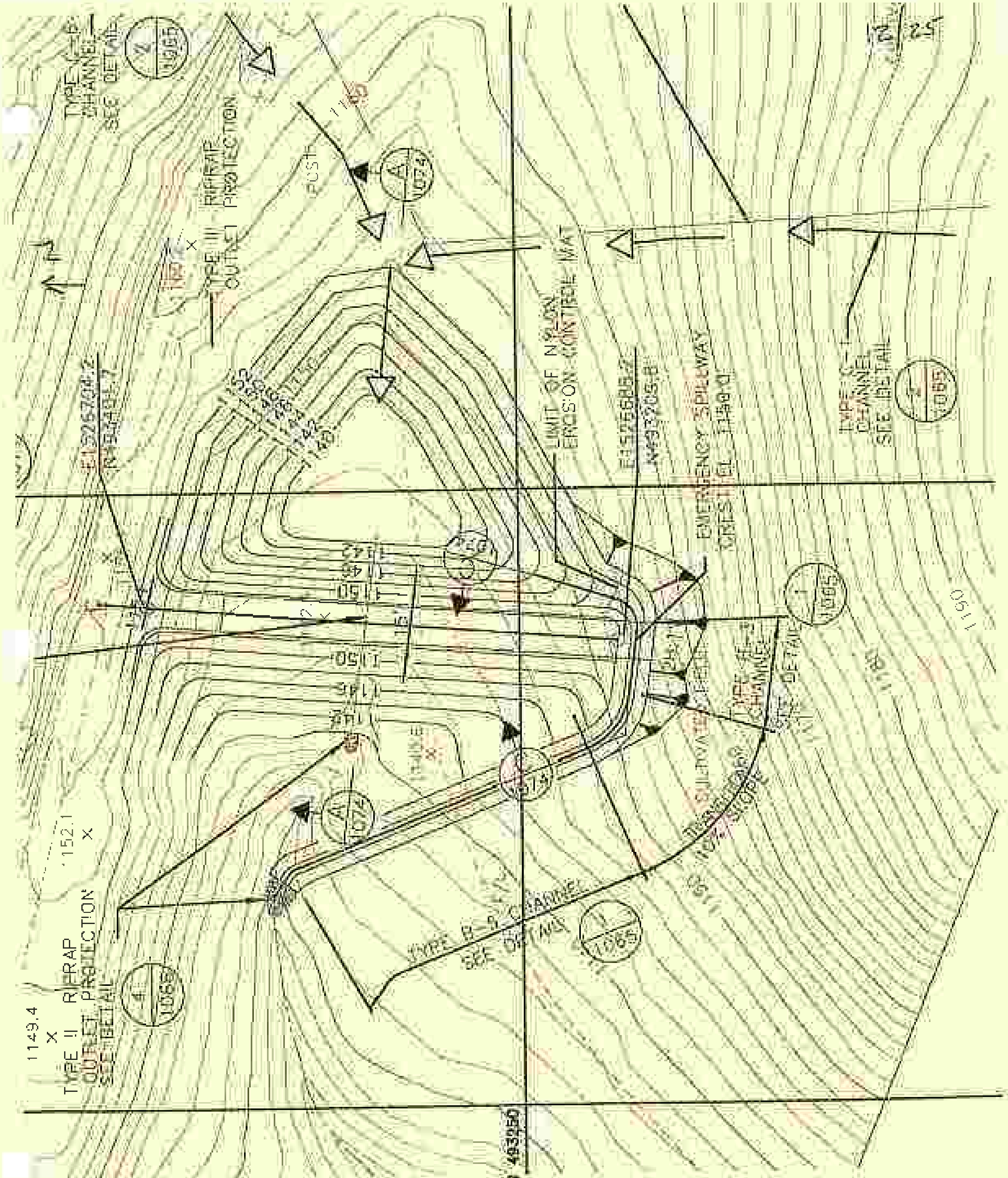
THE FOLLOWING ELEVATION - AREA DATA WILL BE USED TO ESTIMATE STORAGE VOLUMES. (SEE SHEET 12A)

ELEVATION	AREA (ACRE) (Revised - see Sheet 12A)	
1138	0	
1140	0.069	
1142	0.11	PLANIMETERED FROM SHEET 12
1144	0.15	
1146	0.20	INTERPOLATE OR EXTRAPOLATE
1148	0.26	FOR OTHER ELEVATIONS
1150	0.32	
1152	0.39	

VOLUME ESTIMATE SHOWN ON SHEET ~~13~~ 12A, BY AVERAGE END AREA METHOD.

THE POND WILL HAVE A PRINCIPAL SPILLWAY, AN 18" Ø HDPE 85 FT LONG WITH AN ORIFICE PLATE WITH A 14" Ø HOLE. ANALYZE INLET CONDITIONS USING THE ORIFICE EQUATION WITH C=0.6 AND ANALYZE OUTLET CONDITIONS USING THE METHODS IN "HYDRAULIC DESIGN OF HIGHWAY CULVERTS", FHWA, SEPT. 1985.

THE POND WILL ALSO HAVE AN EMERGENCY SPILLWAY WITH A 15' LONG INLET CHANNEL AT ELEVATION 1150.0 WITH A WIDTH OF 4' AND SIDE SLOPES OF 2:1. THE EXIT CHANNEL WILL HAVE A SLOPE OF 10% UNTIL IT MEETS NATURAL GROUND. THE EXIT CHANNEL WILL TRANSITION FROM THE INLET CHANNEL CROSS SECTION TO A TRAP. SECTION WITH A BOTTOM WIDTH OF 2 FT SIDE SLOPES OF 2:1 AND A DEPTH OF 2 FEET.



POND PLAN

1" = 50'

Keystone West Valley Phase II Permitting
Project 92-220-73-7
BY: SER 10/27/96
Ckd By: JMJ 12/2/97

12A/25

West Clean Stormwater Management Pond
Stage - Storage Rating

Elevation	Area acres	Average Area acres	Increm. Volume acft	Cumul. Volume acft
1138.0	0.000			
1139.0	0.035	0.017	0.017	0.0173
1140.0	0.069	0.052	0.052	0.0690
1141.0	0.090	0.079	0.079	0.148
1142.0	0.110	0.100	0.100	0.248
1143.0	0.130	0.120	0.120	0.368
1144.0	0.150	0.140	0.140	0.508
1145.0	0.175	0.163	0.163	0.671
1146.0	0.200	0.188	0.188	0.858
1147.0	0.230	0.215	0.215	1.07
1148.0	0.260	0.245	0.245	1.32
1149.0	0.290	0.275	0.275	1.59
1150.0	0.320	0.305	0.305	1.90
1150.4	0.334	0.327	0.131	2.03
1150.5	0.338	0.336	0.034	2.06
1151.0	0.355	0.346	0.173	2.24
1152.0	0.390	0.373	0.373	2.61
1152.5	0.408	0.399	0.199	2.81

SHEET 13/25

✓ KMS 7/17/96

NOTE: LENGTH OF PRINCIPAL SPILLWAY HAS CHANGED TO 90 FT. THIS WILL NOT AFFECT THE HYDRAULICS. SER 7/24/96

Keystone West Valley Phase II Permitting
Project 92-220-73-7
BY: SER 6/24/96
RVD BY SER 10/27/97

West Clean Stormwater Management Pond
Stage - Discharge Rating

Barrel

16.616	inch inside diameter of SDR 26 18" dia. (OD) **HDPE** pipe
1138	feet inlet invert elevation
0.01	feet/feet slope
85	feet length
1137.15	feet outlet invert elevation
1.5058	Area of Pipe (Full Flow)
0.9	k_e for pipe projecting from fill
0.346167	Hydraulic Radius of Pipe (Full flow)
0.011	Manning's n
3.123	(1+k_e+(29*n^2*L)/R^1.33)

H=(1+ke+(29*n^2*L)/R^1.33)*(Q^2)/(A^2)*(1/(2*g))

Orifice Plate

14	inch diameter hole
0.6	orifice discharge coefficient
1.069014	Area of Orifice (sq ft)
1138.583	elevation of center of orifice
1138	invert of orifice

FOR VOLUMES SEE SHEET 12A

Elevation	Area acres	Average Area acres	Increm. Volume ac-ft	Cumul. Volume ac-ft	Critical Depth for Outlet Control Discharge feet	*h_0= $(D+d_c)/2$ feet	H, Head Loss feet	Outlet Control Discharge cfs	Orifice Plate Discharge cfs	Controlling Discharge cfs	Emergency Spillway Discharge cfs	Total Discharge cfs
1138.0	0.000				0	0.69	0.16	0.0	0.0	0.0	0.0	0.0
1140.0	0.031	0.0155	0.0310	0.0310	1.2	1.29	1.56	8.5	6.1	6.1	0.0	6.1
1141.0	0.047	0.0388	0.0388	0.0698	1.30	1.34	2.51	10.8	8.0	8.0	0.0	8.0
1142.0	0.062	0.0543	0.0543	0.1240	1.38	1.38	3.47	12.7	9.5	9.5	0.0	9.5
1143.0	0.084	0.0730	0.0730	0.1970	1.38	1.38	4.47	14.4	10.8	10.8	0.0	10.8
1144.0	0.106	0.0950	0.0950	0.2920	1.38	1.38	5.47	16.0	12.0	12.0	0.0	12.0
1145.0	0.134	0.1198	0.1198	0.4118	1.38	1.38	6.47	17.4	13.0	13.0	0.0	13.0
1146.0	0.161	0.1473	0.1473	0.5590	1.38	1.38	7.47	18.7	14.0	14.0	0.0	14.0
1147.0	0.195	0.1780	0.1780	0.7370	1.38	1.38	8.47	19.9	14.9	14.9	0.0	14.9
1148.0	0.229	0.2120	0.2120	0.9490	1.38	1.38	9.47	21.0	15.8	15.8	0.0	15.8
1149.0	0.271	0.2498	0.2498	1.1988	1.38	1.38	10.47	22.1	16.6	16.6	0.0	16.6
1150.0	0.312	0.2913	0.2913	1.4900	1.38	1.38	11.47	23.1	17.4	17.4	0.0	17.4
1150.4	0.330	0.3208	0.1283	1.6183	1.38	1.38	11.87	23.5	17.7	17.7	0.7	18.4
1150.5	0.334	0.3318	0.0332	1.6515	1.38	1.38	11.97	23.6	17.8	17.8	2.1	19.9
1151.0	0.366	0.3350	0.1725	1.8240	1.38	1.38	12.47	24.1	18.1	18.1	12.4	30.5
1152.0	0.400	0.3780	0.3780	2.2020	1.38	1.38	13.47	25.1	18.8	18.8	53.8	72.6
1152.5	0.422	0.4110	0.2055	2.4075	1.38	1.38	13.97	25.5	19.2	19.2	85.8	105.0

WSWMPOND.XLS

SHEET 13A/25

DRISCOPIPE DIRECT BURIAL - 07-10-1996

PROJECT: WEST VALLEY WEST CLEAN STORMWATER MANAGEMENT POND

REMARKS:

Calculated By SHR 7/10/96 CHKD BY: KDM 7/11/96

WORST CASE BELOW POND CREST

POND CREST ELEV = 1152.5
PIPE CREST ELEV ≐ 1137.5+1.5
D = 13.5 FT ROUND TO 14 FT

DRISCOPIPE 1000 Product Series

Dimension Ratio (DR)	=	26.00
Burial Depth	=	14 Feet
Soil Density	=	120 Pounds/Cu Ft
Water Table	=	0 Feet Above Pipe
Other Loads	=	144 Pounds/Sq Ft
Soil Modulus	=	2000 psi
Pipe Modulus	=	35000 psi
S(A) (Stress in Pipe Wall)	=	153.1 psi
P(T) (Pressure @ Pipe Crown)	=	12.3 psi
P(CB)(Critical Buckling Pressure)	=	76.9 psi
Maximum Ring Deflection	=	6.50 %
CRUSHING SAFETY FACTOR	=	9.8 to 1
WALL BUCKLING SAFETY FACTOR	=	6.3 to 1
CALCULATED RING DEFLECTION	=	0.61 %

CALCULATED RING DEFLECTION IS ACCEPTABLE.

ASSUMED, ENGINEERING JUDGEMENT

ASSUMED, WORST CASE

CONSERVATIVE ASSUMPTION ACTUAL LOAD IS LESS USE H20 LOAD ≈ 1 psi SEE FIGURE 2, SH. 13B

SEE FIGURE 1, SH 13B SPECS WILL BE MADE TO MATCH CONDITIONS FOR THIS MODULUS.

DEFAULT VALUE OF PROGRAM.

WARNING!

THE USE OF THIS PROGRAM TO DESIGN POLYETHYLENE PIPING SYSTEMS USING PRODUCTS NOT MANUFACTURED BY PHILLIPS DRISCOPIPE MAY RESULT IN SERIOUS DESIGN ERRORS.

These programs provide accurate and reliable information to the best of Phillips Driscopipe's knowledge, but our suggestions and recommendations cannot be guaranteed because the conditions of use are beyond our control. Each project has it's own set of variables and conditions. Interpretation of these variables is important. The user must apply proper engineering judgement when selecting values for input into these programs. Phillips Petroleum Company and Phillips Driscopipe assume no responsibility for the information presented herein and hereby expressly disclaim all liability relating to the use of this information.

For Additional Information on DRISCOPIPE Products Contact:
PHILLIPS DRISCOPIPE Richardson, Tx. - 800/527-0662

ID = 16.616 INCH FOR SDR 26 18"φ(OD) HDPE
SEE SHEET 13C

SHEET 13B / 25

Plexco/Spirolite™

Chevron

COPIED FROM PLEXCO/SPIROLITE "APPLICATION NOTE No 1"

Figure 1
Bureau of Reclamation Values of E′ for Iowa Formula
(For Initial Flexible Pipe Deflection)

Soil type-pipe bedding material (Unified Classification System[2/])	E′ for degree of compaction of bedding (lb/in²)[5/]			
	Dumped	Slight <85% Proctor <40% rel. den.	Moderate 85-95% Proctor 40-70% rel. den.	High >95% Proctor >70% rel. den.
<u>Fine-grained Soils (LL>50)</u>[3/] Soils with medium to high plasticity CH, MH, CH-MH	No data available; consult a competent Soils Engineer; Otherwise use E′ = 0			
<u>Fine-grained Soils (LL<50)</u> Soils with medium to no plasticity CL, ML, ML-CL, with less than 25% coarse-grained particles	50	200	400	1,000
<u>Fine-grained Soils (LL<50)</u> Soils with medium to no plasticity CL, ML, ML-CL, with more than 25% coarse-grained particles <u>Coarse -grained Soils with Fines</u> GM, GC, SM, SC[4/] contains more than 12% fines	100	400	1,000	2,000
<u>Coarse-grained Soils with Little or No Fines</u> GW, GP, SW, SP[4/] contains less than 12% fines	200	1,000	2,000	3,000
<u>Crushed Rock</u>	1,000	3,000		

[2/] ASTM Designation D2487, USBR Designation E-3.

[3/] LL = Liquid limit.

[4/] Or any borderline soil beginning with one of these symbols (i.e., GM-GC, GC-SC.).

[5/] 1 lb/in² = 0.07 kg/cm².

FIGURE 2
Variation In Soil Stress With Cover Depth Below H-20 Wheel Load

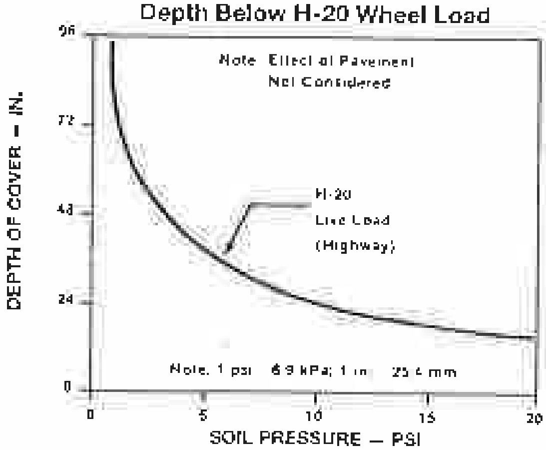
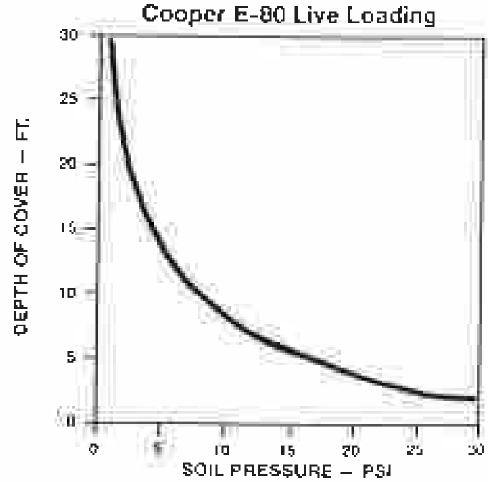


FIGURE 3
Cooper E-80 Live Loading



SHEET 13 C/25

10" (10.750 OD)

SDR 7	267 psi	19.32 lbs./ft.	7.678 ID	1.536 wall
SDR 9	200 psi	15.61	8.362	1.194
SDR 11 ●	**160 psi**	**13.09**	**8.796**	**.977**
SDR 13.5	128 psi	10.87	9.158	.796
SDR 15.5	110 psi	9.58	9.362	.694
SDR 17 ●	**100 psi**	**8.78**	**9.486**	**.632**
SDR 19	89 psi	7.92	9.618	.566
SDR 21 ●	**80 psi**	**7.21**	**9.726**	**.512**
SDR 26 ●	**64 psi**	**5.87**	**9.924**	**.413**
SDR 32.5 ●	**51 psi**	**4.75**	**10.088**	**.331**

12" (12.750 OD)

SDR 7	267 psi	27.16 lbs./ft.	9.108 ID	1.821 wall
SDR 9	200 psi	21.97	9.916	1.417
SDR 11 ●	**160 psi**	**18.41**	**10.432**	**1.159**
SDR 13.5	128 psi	15.29	10.862	.944
SDR 15.5 ●	**110 psi**	**13.48**	**11.104**	**.823**
SDR 17 ●	**100 psi**	**12.36**	**11.250**	**.750**
SDR 19	89 psi	11.14	11.408	.671
SDR 21 ●	**80 psi**	**10.13**	**11.536**	**.607**
SDR 26 ●	**64 psi**	**8.26**	**11.770**	**.490**
SDR 32.5 ●	**51 psi**	**6.67**	**11.966**	**.392**

13" (13.386 OD)

SDR 7	267 psi	29.24 lbs./ft.	9.562 ID	1.912 wall
SDR 9	200 psi	23.62	10.412	1.487
SDR 11	160 psi	20.30	10.952	1.217
SDR 13.5	128 psi	16.87	11.402	.992
SDR 15.5	110 psi	14.85	11.658	.864
SDR 17	100 psi	13.62	11.812	.787
SDR 19	89 psi	12.28	11.976	.705
SDR 21	80 psi	11.16	12.112	.637
SDR 26	64 psi	9.12	12.356	.515
SDR 32.5	51 psi	7.36	12.562	.412

14.000 OD

SDR 7	267 psi	32.76 lbs./ft.	10.00 ID	2.000 wall
SDR 9	200 psi	26.50	10.888	1.556
SDR 11 ●	**160 psi**	**22.20**	**11.454**	**1.273**
SDR 13.5	128 psi	18.44	11.926	1.037
SDR 15.5	110 psi	16.24	12.194	.903
SDR 17 ●	**100 psi**	**14.91**	**12.352**	**.824**
SDR 19	89 psi	13.43	12.526	.737
SDR 21	80 psi	12.22	12.666	.667
SDR 26 ●	**64 psi**	**9.96**	**12.924**	**.538**
SDR 32.5	51 psi	8.05	13.138	.431

16.000 OD

SDR 9	200 psi	34.60 lbs./ft.	12.444 ID	1.778 wall
SDR 11 ●	**160 psi**	**29.00**	**13.090**	**1.455**
SDR 13.5	129 psi	24.09	13.630	1.185
SDR 15.5	110 psi	21.21	13.936	1.032
SDR 17 ●	**100 psi**	**19.46**	**14.118**	**.941**
SDR 19	89 psi	17.54	14.316	.842
SDR 21 ●	**80 psi**	**15.96**	**14.476**	**.762**
SDR 26 ●	**64 psi**	**13.01**	**14.770**	**.615**
SDR 32.5	51 psi	10.50	15.016	.492

18.000 OD

SDR 9	200 psi	43.79 lbs./ft.	14.000 ID	2.000 wall
SDR 11 ●	**160 psi**	**36.69**	**14.728**	**1.636**
SDR 13.5	128 psi	30.48	15.334	1.333
SDR 15.5 ●	**110 psi**	**26.84**	**15.678**	**1.161**
SDR 17 ●	**100 psi**	**24.64**	**15.882**	**1.059**
SDR 19	89 psi	22.19	16.106	.947
SDR 21	80 psi	20.19	16.286	.857
SDR 26 ●	**64 psi**	**16.47**	**16.616**	**.692**
SDR 32.5	51 psi	13.30	16.892	.554

20.000 OD

SDR 9	200 psi	54.05 lbs./ft.	15.556 ID	2.222 wall
SDR 11 ●	**160 psi**	**45.30**	**16.364**	**1.818**
SDR 13.5	128 psi	37.63	17.038	1.481
SDR 15.5	110 psi	33.14	17.420	1.290
SDR 17 ●	**100 psi**	**30.41**	**17.648**	**1.176**
SDR 19	89 psi	27.42	17.894	1.053
SDR 21	80 psi	24.93	18.096	.952
SDR 26 ●	**64 psi**	**20.34**	**18.462**	**.769**
SDR 32.5 ●	**51 psi**	**16.41**	**18.770**	**.615**

21.500 OD

SDR 9	200 psi	62.47 lbs./ft.	16.722 ID	2.389 wall
SDR 11	160 psi	52.37	17.590	1.955
SDR 13.5	128 psi	43.51	18.314	1.593
SDR 15.5	110 psi	38.30	18.726	1.387
SDR 17	100 psi	35.16	18.970	1.265
SDR 19	89 psi	31.68	19.236	1.132
SDR 21	80 psi	28.82	19.452	1.024
SDR 26	64 psi	23.51	19.846	.827
SDR 32.5	51 psi	18.98	20.176	.662

● denotes standard sizes

SUBJECT KEYSTONE

BY KMB DATE 7/11/96 PROJ. NO. 97-220-73-09

CHKD. BY SER DATE 7/16/96 SHEET NO. 13 D OF 25

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VERIFY THE RESULTS OF THE DRISCOPIPE BURIED PIPE ANALYSIS COMPUTER PROGRAM. THE PROCEDURE FROM THE DRISCOPIPE BINDER WILL BE USED.

- DESIGN BY WALL CRUSHING

$$S_A = \frac{(SDR - 1)}{2} P_T$$

SDR = 26

P_T = EXTERNAL PRESSURE, psi

P_T = SOIL DENSITY x DEPTH = 120 lb/ft³ x 13.5 ft = 1620 psf = 11.25 psi
+ LIVE LOAD = 144 lb/ft² = 1 psi

12.25 psi

$$S_A = \frac{(26-1)}{2}(12.25) = 153.1 \text{ psi}$$

SAFETY FACTOR = 1500/153.1 = 9.8 (OK)

- DESIGN BY WALL BUCKLING

$$P_{CB} = 0.8\sqrt{E' \times P_c}$$

$$P_c = \frac{2.32\ E}{(SDR)^3} = \frac{2.32 \times 35000}{26^3} = 4.62 \text{ psi}$$

$$P_{CB} = 0.8\sqrt{2000 \times 4.62} = 76.9 \text{ psi}$$

$$FS = \frac{P_{cb}}{P_t} = \frac{76.9}{12.25} = 6.3 \quad \text{(OK)}$$

% SOIL STRAIN = RING DEFLECTION = $\frac{P_t}{E'_{min}} \times 100 = \frac{12.25}{2000} \times 100 = 0.61\%$ (OK)

SUBJECT KEYSTONE

BY SER DATE 6/24/96 PROJ. NO. 92-230-73-7

CHKD. BY JWB DATE 7/11/96 SHEET NO. 14 OF 25

GAI CONSULTANTS, INC.

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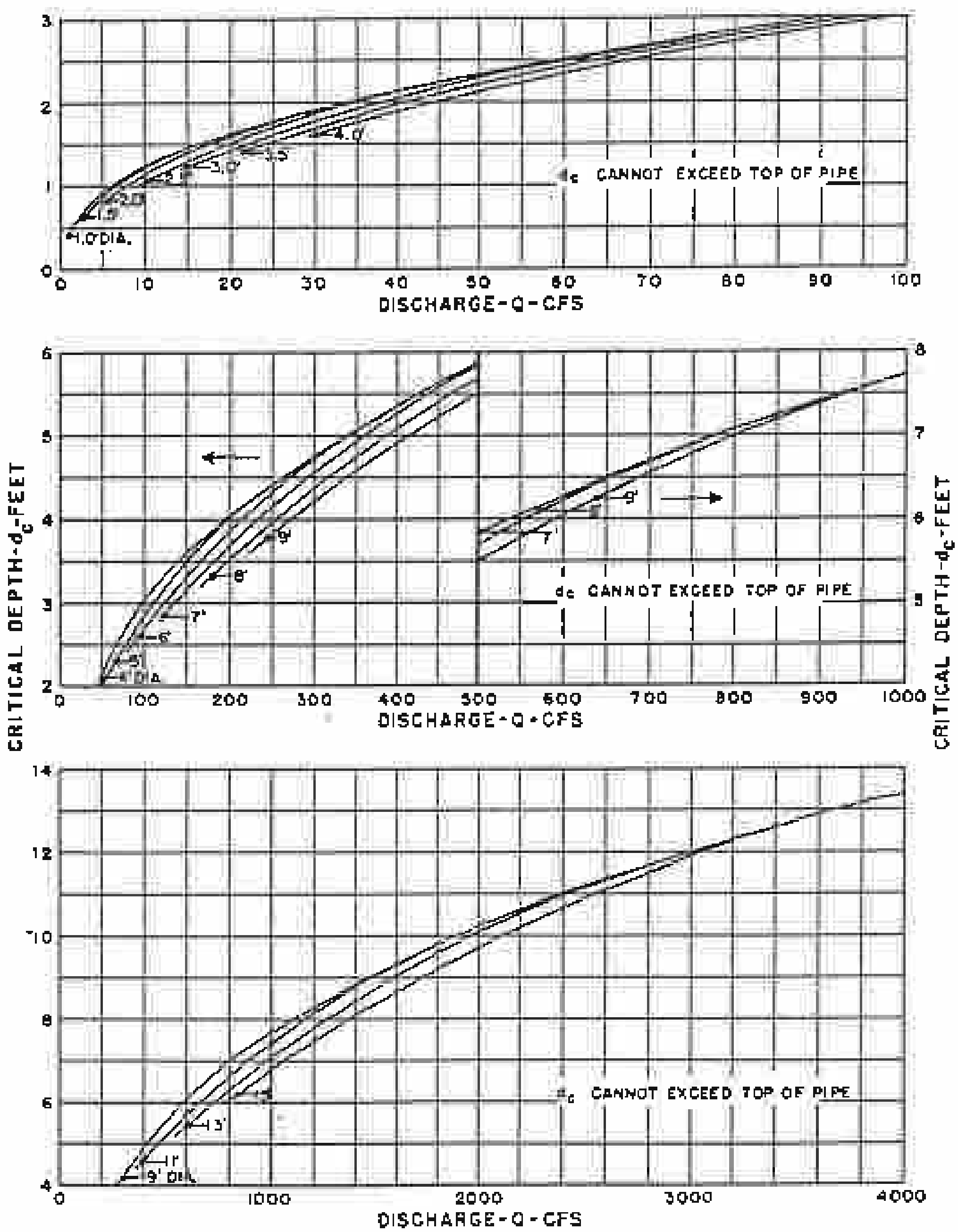
CHART 4



BUREAU OF PUBLIC ROADS
JAN. 1964

CRITICAL DEPTH
CIRCULAR PIPE

SUBJECT: Keystone Station
Phase II Permitting
BY: SER DATE: 6/26/96 PROJ. NO.: 92-220-73-07
CHKD. BY: [illegible] DATE: 7/11/96 SHEET NO. 5 OF 25

West Clean Stormwater Management Pond Emergency Spillway

GAI CONSULTANTS, INC.
Engineers Geologists Planners
Environmental Specialists

Purpose: Estimate the Stage - Discharge Rating of the West Clean Stormwater Management Pond Emergency Spillway.

Methodology: "Earth Spillways", TR-2, US Soil Conservation Service, October 1, 1956 and Manning's Equation.

Control Section and Inlet Channel

The control section and inlet channel will be trapezoidal with a total depth of 2.5 feet, a bottom width of 4 feet and side slopes of 2:1. The lining will grass with nylon erosion control matting. The length of the inlet channel will be 15 feet.

$b := 4 \qquad z := 2 \qquad n := 0.045 \qquad L := 15$

Create an index variable i and vary it between 0 and 19.

$i := 0, 1 .. 19$

Put the critical depth in the "zeroeth" column of an array A. Set equal to 0.1 feet times (i+1) therefore critical depth is varied from 0.1 to 2.0 feet by increments of 0.1 feet.

$A_{i,0} := (i+1)\cdot 0.1 \qquad\qquad d_c(i) := (i+1)\cdot 0.1$

Put the area at critical depth in the first column of the array A.

$A_{i,1} := \left(b + z\cdot A_{i,0}\right)\cdot A_{i,0} \qquad\qquad Area(i) := \left(b + z\cdot d_c(i)\right)\cdot d_c(i)$

Put the top width at critical depth in the second column of the array A.

$A_{i,2} := b + 2\cdot z\cdot A_{i,0} \qquad\qquad T(i) := b + 2\cdot z\cdot d_c(i)$

Put the mean hydraulic depth at critical depth in the third column of the array A.

$A_{i,3} := \dfrac{A_{i,1}}{A_{i,2}} \qquad\qquad d_m(i) := \dfrac{Area(i)}{T(i)}$

Put the velocity at critical depth in the fourth column of the array A.

$A_{i,4} := \sqrt{\dfrac{g}{\left(\dfrac{ft}{sec^2}\right)}\cdot A_{i,3}} \qquad\qquad Velocity(i) := \sqrt{g\cdot d_m(i)}$

Feet per second squared units are used to make array values unitless.

Put the specific head at critical depth in the fifth column of the array A.

$A_{i,5} := A_{i,0} + \dfrac{\left(A_{i,4}\right)^2}{2\cdot\dfrac{g}{\left(\dfrac{ft}{sec^2}\right)}} \qquad\qquad H_{ec}(i) := d_c(i) + \dfrac{Velocity(i)^2}{2\cdot g}$

Put the discharge at critical depth in the sixth column of the array A.

$A_{i,6} := A_{i,1}\cdot A_{i,4} \qquad\qquad Q_c(i) := Velocity(i)\cdot Area(i)$

SUBJECT: Keystone Station
Phase II Permitting
BY: SER DATE: 6/26/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KMB DATE: 7/11/96 SHEET NO. 16 OF 25

GAI CONSULTANTS, INC.
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Put the discharge coefficient α at critical depth in the seventh column of the array A.

$$A_{i,7} := \frac{4.315 \cdot n^2}{\left(A_{i,5}\right)^{\frac{4}{3}}} \qquad \alpha(i) := \frac{4.315 \cdot n^2}{H_{ec}(i)^{\frac{4}{3}}}$$

Put the total head in the eighth column of the array A.

$$A_{i,8} := \left(1 + A_{i,7} \cdot L\right) \cdot A_{i,5} \qquad H_p(i) := (1 + \alpha(i) \cdot L) \cdot H_{ec}(i)$$

Put the wetted perimeter at the critical depth in the ninth column of the array A.

$$A_{i,9} := b + 2 \cdot A_{i,0} \cdot \sqrt{1 - z^2} \qquad P(i) := b - 2 \cdot d_c(i) \cdot \sqrt{1 - z^2}$$

Put the hydraulic radius at the critical depth in the tenth column of the array A.

$$A_{i,10} := \frac{A_{i,1}}{A_{i,9}} \qquad R(i) := \frac{\text{Area}(i)}{P(i)}$$

Put the critical slope in the eleventh column of the array A.

$$A_{i,11} := \left[\frac{A_{i,6} \cdot n}{1.49 \cdot A_{i,1} \cdot \left(A_{i,10}\right)^{\frac{2}{3}}}\right]^2 \qquad S_c(i) := \left[\frac{Q_c(i) \cdot n}{1.49 \cdot \text{Area}(i) \cdot R(i)^{\frac{2}{3}}}\right]^2 \text{manning's equation}$$

Display the array A with headings for the columns.

A =

d_c	a	t	d_m	v	H_{ec}	q	α	H_p	p	r	S_c
ft	ft^2	ft	ft	$\frac{ft}{sec}$	ft	$\frac{ft^3}{sec}$		ft	ft	ft	$\frac{ft}{ft}$
0.1	0.42	4.4	0.095	1.752	0.148	0.736	0.112	0.396	4.447	0.094	0.065
0.2	0.88	4.8	0.183	2.429	0.292	2.137	0.045	0.489	4.894	0.18	0.053
0.3	1.38	5.2	0.265	2.922	0.433	4.032	0.027	0.606	5.342	0.258	0.047
0.4	1.92	5.6	0.343	3.321	0.571	6.377	0.018	0.729	5.789	0.332	0.044
0.5	2.5	6	0.417	3.661	0.708	9.153	0.014	0.855	6.236	0.401	0.041
0.6	3.12	6.4	0.488	3.96	0.844	12.356	0.011	0.982	6.683	0.467	0.04
0.7	3.78	6.8	0.556	4.229	0.978	15.986	$9.002\cdot10^{-3}$	1.11	7.13	0.53	0.038
0.8	4.48	7.2	0.622	4.474	1.111	20.045	$7.593\cdot10^{-3}$	1.238	7.578	0.591	0.037
0.9	5.22	7.6	0.687	4.701	1.243	24.539	$6.535\cdot10^{-3}$	1.365	8.025	0.65	0.036
1	6	8	0.75	4.912	1.375	29.474	$5.715\cdot10^{-3}$	1.493	8.472	0.708	0.035
1.1	6.82	8.4	0.812	5.111	1.506	34.857	$5.062\cdot10^{-3}$	1.62	8.919	0.765	0.034
1.2	7.68	8.8	0.873	5.299	1.636	40.696	$4.531\cdot10^{-3}$	1.748	9.367	0.82	0.033
1.3	8.58	9.2	0.933	5.478	1.766	46.999	$4.092\cdot10^{-3}$	1.875	9.814	0.874	0.033
1.4	9.52	9.6	0.992	5.649	1.896	53.774	$3.724\cdot10^{-3}$	2.002	10.261	0.928	0.032
1.5	10.5	10	1.05	5.812	2.025	61.029	$3.411\cdot10^{-3}$	2.129	10.708	0.981	0.032
1.6	11.52	10.4	1.108	5.97	2.154	68.772	$3.141\cdot10^{-3}$	2.255	11.155	1.033	0.031
1.7	12.58	10.8	1.165	6.122	2.282	77.013	$2.908\cdot10^{-3}$	2.382	11.603	1.084	0.031
1.8	13.68	11.2	1.221	6.269	2.411	85.758	$2.703\cdot10^{-3}$	2.508	12.05	1.135	0.03
1.9	14.82	11.6	1.278	6.411	2.539	95.016	$2.523\cdot10^{-3}$	2.635	12.497	1.186	0.03
2	16	12	1.333	6.55	2.667	104.795	$2.363\cdot10^{-3}$	2.761	12.944	1.236	0.029

Legend
critical depth = d_c
area = a
top width = t
mean depth = d_m
velocity = v
specific head at critical depth = H_{ec}
discharge = q
loss coefficient = α
total head = H_p
wetted perimeter = p
hydraulic radius = r
critical slope = S_c

Exit Channel

The exit channel should be designed to cause supercritical flow such that critical flow occurs at the control section. Assume that this condition is not required for flows below the minimum total depth calculated.

Set critical slope as

$S_c = A_{0,11}$ $S_c = 0.065\cdot\frac{ft}{ft}$ or greater. Use S_c = 0.10 ft/ft.

SUBJECT: Keystone Station
Phase II Permitting
BY: SER DATE: 6/27/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KMB DATE: 7/11/96 SHEET NO. 18 OF 25

GAI CONSULTANTS, INC.
Engineers Geologists Planners
Environmental Specialists

Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right) \cdot a \cdot r^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right) \cdot (r)^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$

West Stormwater Management Pond Emergency Spillway Exit Channel near control section

Design Flow, $Q_d = 40 \cdot ft^3 \cdot sec^{-1}$ 100- year peak flow (max. of Stage 3 and Post-development Conditions) see sheet 22

Bottom Width, $b = 4 \cdot ft$

Side Slopes, $z = 2$

Channel Lining is Grass with nylon erosion control mat Manning's roughness coefficient, $n = 0.045$

Channel Minimum Slope, $S_{min} := \frac{10 \cdot ft}{100 \cdot ft}$ (from Sheet 17) or $S_{min} = 0.1 \cdot \frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 0.888 \cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 5.1 \cdot ft^2$

Minimum Velocity, $V_{min} = 7.8 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 7.6 \cdot ft$

Freeboard, $F_b = 1.6$ ft by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990.

Total depth, $D = 2.5 \cdot ft$

Top Width at Total Depth, $T_D = 14 \cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 306 \cdot ft^3 \cdot sec^{-1}$

Channel Maximum Slope, $S_{max} := \frac{10 \cdot ft}{100 \cdot ft}$ (from Sheet 17) or $S_{max} = 0.1 \cdot \frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 0.888 \cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 5.1 \cdot ft^2$

Maximum Velocity, $V_{max} = 7.8 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 7.6 \cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 306 \cdot ft^3 \cdot sec^{-1}$

SUBJECT: Keystone Station
Phase II Permitting
BY: SER DATE: 6/27/96 PROJ. NO.: 92-220-73-07
CHKD. BY: [illegible] DATE: 7/11/96 SHEET NO. 19 OF 25

GAI CONSULTANTS, INC.
Engineers Geologists Planners
Environmental Specialists

Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right)\cdot a\cdot r^{\left(\frac{2}{3}\right)}\cdot s^{\left(\frac{1}{2}\right)}$ or $V = \left(\frac{1.49}{n}\right)\cdot(r)^{\left(\frac{2}{3}\right)}\cdot s^{\left(\frac{1}{2}\right)}$

West Stormwater Management Pond Emergency Spillway Exit Channel downstream of transition

Design Flow, $Q_d = 40\cdot ft^3\cdot sec^{-1}$ 100- year peak flow (max. of Stage 3 and Post-development Conditions) see sheet 22

Bottom Width, $b = 2\cdot ft$

Side Slopes, $z = 2$

Channel Lining is Grass with nylon erosion control mat Manning's roughness coefficient, $n = 0.045$

Channel Minimum Slope, $S_{min} := \frac{10\cdot ft}{100\cdot ft}$ (from Sheet 7) or $S_{min} = 0.1\cdot\frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 1.143\cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 4.9\cdot ft^2$

Minimum Velocity, $V_{min} = 8.2\cdot ft\cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 6.6\cdot ft$

Freeboard, $F_b = 0.9$ ft by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990.

Total depth, $D = 2\cdot ft$

Top Width at Total Depth, $T_D = 10\cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 134\cdot ft^3\cdot sec^{-1}$

Channel Maximum Slope, $S_{max} = \frac{2\cdot ft}{12.5\cdot ft}$ (from Sheet 12) or $S_{max} = 0.16\cdot\frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 1.02\cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 4.1\cdot ft^2$

Maximum Velocity, $V_{max} = 9.7\cdot ft\cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 6.1\cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 169\cdot ft^3\cdot sec^{-1}$

20/25

✓KMB 3/11/96

Rev. by SER 10/27/97
✓ by JMJ 12/2/97

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JOB TR-20                       FULLPRINT          SUMMARY    NOPLOTS
TITLE 111 KEYSTONE WEST SWM POND-92-220-73-7
 3 STRUCT       01                                                       WSWMPOND
                             1138.0          0.0       0.0
                             1140.0          6.1       0.069
 8                           1141.0          8.0       0.148
 8                           1142.0          9.5       0.248
 8                           1143.0         10.8       0.368
 8                           1144.0         12.0       0.508
 8                           1145.0         13.0       0.671
 8                           1146.0         14.0       0.858
 8                           1147.0         14.9       1.07
 8                           1148.0         15.8       1.32
 8                           1149.0         16.6       1.59
 8                           1150.0         17.4       1.90
 8                           1150.4         18.4       2.03
 8                           1150.5         19.9       2.06
 8                           1151.0         30.5       2.24
 8                           1152.0         72.6       2.61
 8                           1152.5        105.0       2.81
 9 ENDTBL
 3 STRUCT       02                                                       WSWMPOND
 8                           1138.0          0.0       0.0
 8                           1140.0          6.1       0.069
 8                           1141.0          8.0       0.148
 8                           1142.0          9.5       0.248
 8                           1143.0         10.8       0.368
 8                           1144.0         12.0       0.508
 8                           1145.0         13.0       0.671
                             1146.0         14.0       0.858
                             1147.0         14.9       1.07
 8                           1148.0         15.8       1.32
 8                           1149.0         16.6       1.59
 8                           1150.0         17.4       1.90
 8                           1150.4         18.4       2.03
 8                           1150.5         19.9       2.06
 8                           1151.0         30.5       2.24
 8                           1152.0         72.6       2.61
 8                           1152.5        105.0       2.81
 9 ENDTBL
 6 RUNOFF 1 001        1 0.0197      80.         0.24         1          PREDEV
 6 RUNOFF 1 001        2 0.0411      79.         0.31         1          STAGE3
 6 RESVOR 2    01 2    3                                      1 1 1      S3ROUTED
 6 RUNOFF 1 001        4 0.0277      79.         0.31         1          POST/ULT
 6 RESVOR 2    01 4    5                                      1 1 1      POSTROUTED
 6 RUNOFF 1 001        6 0.0622      79.         0.50         1          STAGE3A
 6 RESVOR 2    02 6    7                                      1 1 1      S3AROUTED
   ENDATA
 7 LIST
 7 INCREM 6              0.05
 7 COMPUT 7 001       02 0.            2.6        1.          2 2        2 YR
   ENDCMP 1
 7 COMPUT 7 001       02 0.            3.9        1.          2 2        10 YR
   ENDCMP 1
 7 COMPUT 7 001       02 0.            4.4        1.          2 2        25 YR
   ENDCMP 1
 7 COMPUT 7 001       02 0.            5.2        1.          2 2        100 YR
   ENDCMP 1
   ENDJOB 2
```

21/25

Rev. by SER 10/27/97
✓ by JMJ 12/2/97

SUMMARY TABLE 1 - SELECTED RESULTS OF STANDARD AND EXECUTIVE CONTROL INSTRUCTIONS IN THE ORDER PERFORMED
(A STAR(*) AFTER THE PEAK DISCHARGE TIME AND RATE (CFS) VALUES INDICATES A FLAT TOP HYDROGRAPH
A QUESTION MARK(?) INDICATES A HYDROGRAPH WITH PEAK AS LAST POINT.)

SECTION/ STRUCTURE ID	STANDARD CONTROL OPERATION	DRAINAGE AREA (SQ MI)	RAIN TABLE #	ANTEC MOIST COND	MAIN TIME INCREM (HR)	PRECIPITATION BEGIN (HR)	PRECIPITATION AMOUNT (IN)	PRECIPITATION DURATION (HR)	RUNOFF AMOUNT (IN)	PEAK DISCHARGE ELEVATION (FT)	PEAK DISCHARGE TIME (HR)	PEAK DISCHARGE RATE (CFS)	PEAK DISCHARGE RATE (CSM)
ALTERNATE 0 STORM 0													
XSECTION 1	RUNOFF	.02	2	2	.05	.0	2.60	24.00	.69	---	12.06	14.22	722.0
XSECTION 1	RUNOFF	.04	2	2	.05	.0	2.60	24.00	.65	---	12.10	24.69	600.8
STRUCTURE 1	RESVOR	.04	2	2	.05	.0	2.60	24.00	.64	1143.24	12.34	11.09	269.8
XSECTION 1	RUNOFF	.03	2	2	.05	.0	2.60	24.00	.65	---	12.10	16.64	600.8
STRUCTURE 1	RESVOR	.03	2	2	.05	.0	2.60	24.00	.64	1141.75	12.30	9.12	329.4
XSECTION 1	RUNOFF	.06	2	2	.05	.0	2.60	24.00	.64	---	12.22	28.64	460.5
STRUCTURE 2	RESVOR	.06	2	2	.05	.0	2.60	24.00	.63	1144.81	12.60	12.81	206.0
XSECTION 1	RUNOFF	.02	2	2	.05	.0	3.90	24.00	1.49	---	12.05	29.41	1492.7
XSECTION 1	RUNOFF	.04	2	2	.05	.0	3.90	24.00	1.42	---	12.09	52.69	1281.9
STRUCTURE 1	RESVOR	.04	2	2	.05	.0	3.90	24.00	1.40	1147.55	12.45	15.39	374.6
XSECTION 1	RUNOFF	.03	2	2	.05	.0	3.90	24.00	1.42	---	12.09	35.51	1281.9
STRUCTURE 1	RESVOR	.03	2	2	.05	.0	3.90	24.00	1.40	1145.15	12.38	13.15	474.6
XSECTION 1	RUNOFF	.06	2	2	.05	.0	3.90	24.00	1.40	---	12.21	62.02	997.1
STRUCTURE 2	RESVOR	.06	2	2	.05	.0	3.90	24.00	1.26	1150.21	12.77	17.94	288.4
XSECTION 1	RUNOFF	.02	2	2	.05	.0	4.40	24.00	1.82	---	12.05	35.60	1807.3
XSECTION 1	RUNOFF	.04	2	2	.05	.0	4.40	24.00	1.75	---	12.09	64.23	1562.7
STRUCTURE 1	RESVOR	.04	2	2	.05	.0	4.40	24.00	1.71	1148.99	12.48	16.59	403.7
XSECTION 1	RUNOFF	.03	2	2	.05	.0	4.40	24.00	1.75	---	12.09	43.29	1562.7
STRUCTURE 1	RESVOR	.03	2	2	.05	.0	4.40	24.00	1.73	1146.33	12.41	14.30	516.3
XSECTION 1	RUNOFF	.06	2	2	.05	.0	4.40	24.00	1.73	---	12.20	75.84	1219.3
STRUCTURE 2	RESVOR	.06	2	2	.05	.0	4.40	24.00	1.53	1151.11	12.56	35.07	563.9
XSECTION 1	RUNOFF	.02	2	2	.05	.0	5.20	24.00	2.39	---	12.05	45.74	2321.6
XSECTION 1	RUNOFF	.04	2	2	.05	.0	5.20	24.00	2.30	---	12.09	83.18	2023.8
STRUCTURE 1	RESVOR	.04	2	2	.05	.0	5.20	24.00	2.14	1150.77	12.42	25.60	622.9
XSECTION 1	RUNOFF	.03	2	2	.05	.0	5.20	24.00	2.30	---	12.09	56.06	2023.8
STRUCTURE 1	RESVOR	.03	2	2	.05	.0	5.20	24.00	2.28	1148.05	12.44	15.84	572.0
XSECTION 1	RUNOFF	.06	2	2	.05	.0	5.20	24.00	2.28	---	12.20	98.58	1585.0
STRUCTURE 2	RESVOR	.06	2	2	.05	.0	5.20	24.00	2.01	1151.85	12.42	66.35	1066.7

1

Keystone Station
West Valley Phase II Permitting
Project 92-220-73-7

BY: SER 10/27/96
Chkd By: JHJ 10/2/97

West Clean Stormwater Management Pond
Summary of Peak Flows and Water Surface Elevations

Condition	Storm Event 2-year Peak Flow (cfs)	2-year Peak Elevation (ft. NGVD)	10-year Peak Flow (cfs)	10-year Peak Elevation (ft. NGVD)	25-year Peak Flow (cfs)	25-year Peak Elevation (ft. NGVD)	100-year Peak Flow (cfs)	100-year Peak Elevation (ft. NGVD)
Pre- Development	14.2	N/A	29.4	N/A	35.6	N/A	45.7	N/A
Stage 3	11.1	1143.2	15.4	1147.6	16.6	1149.0	25.6	1150.8
Post- Development	9.1	1141.8	13.2	1145.2	14.3	1146.3	15.8	1148.1
Stage 3A	12.8	1144.8	17.9	1150.2	35.1	1151.1	66.4	1151.9

Pond Crest Elevation = 1152.5 feet NGVD

Conclusion: The Post- development flows and the Stage 3 flows will be controlled to be less than the pre- development flows for the 2, 10, 25, and 100 year events with a minimum freeboard of 1.7 feet.

The Stage 3A flows will be controlled to be less than the pre- development flows for the 2, 10, and 25 year events with a minimum freeboard of 1.4 feet. The freeboard for the 100 year event will be 0.6 feet.

Note that freeboard is 1.0 feet minimum for all cases and all events, except for the 100 year storm event during Stage 3A conditions, which will be a short term condition.

22/25

SHEET 23 TO 25 COPIED FROM 1994 MARYLAND STANDARDS AND SPECS FOR SOIL EROSION AND SEDIMENT CONTROL MARYLAND DOE

SHEET 23/25

Designing Anti-Seep Collars (Refer to Detail 13)

1. Determine the length of pipe within the saturation zone of the embankment (L_s) either graphically or by using the following equation, assuming that the upstream slope of the embankment intersects the invert of the pipe at its upstream end and that the slope of the pipe (S_o) is constant.

$$L_s = \frac{Y(Z+4)}{(1-4 S_o)}$$

Y = 13.5 FT
Z = 3
S_o = 0.01 FT/FT

L_s = 98.4 FT
USE L = 85 FT

2. Determine the vertical projection (P_t) required to increase L_s by 15% either graphically as shown on C-10-22 or by using the equation:

$$P_t = 0.075\ L_s$$

P_t = 6.4 FT.

3. Choose the actual vertical projection (2' minimum) of each anti-seep collar (P) by rounding up P_t or rounding down P_t and using multiple collars.

USE P = 3.5 FT N = 2

4. Determine the number of anti-seep collars (N) required of the chosen vertical projection (P) using equation:

$$\frac{P_t}{P} = N$$

5. Either round up N or repeat steps 3 and 4 to determine optimum P/N relationship.

6. Provide construction specifications relative to the materials to be used and method for anchoring the anti-seep collar(s) to the pipe in a water tight manner.

7. Anti-seep collar spacing shall be between 5 and 14 times the vertical projection of each collar.

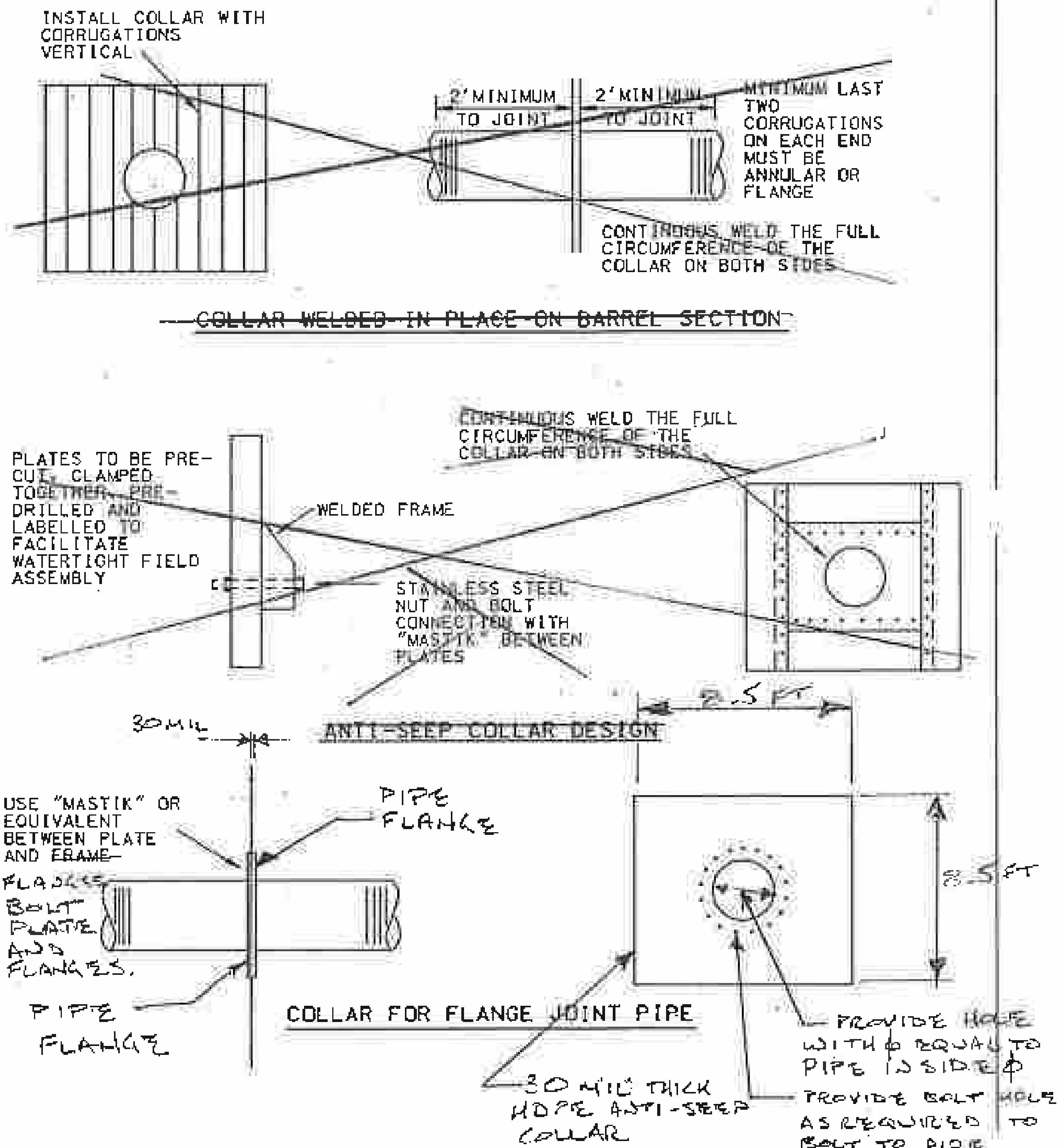
SPACE AT 20' AND 40' FROM INLET

8. Anti-seep collar dimensions shall extend a minimum of 2 feet in all directions around the pipe.

9. Anti-seep collars shall be placed a minimum of two feet from pipe joints.

10. Anti-seep collars should be placed within the saturation zone. In cases where the spacing limit will not allow this, at least one collar shall be placed in the saturation zone.

DETAIL 14 – TYPICAL ANTI-SEEP COLLARS

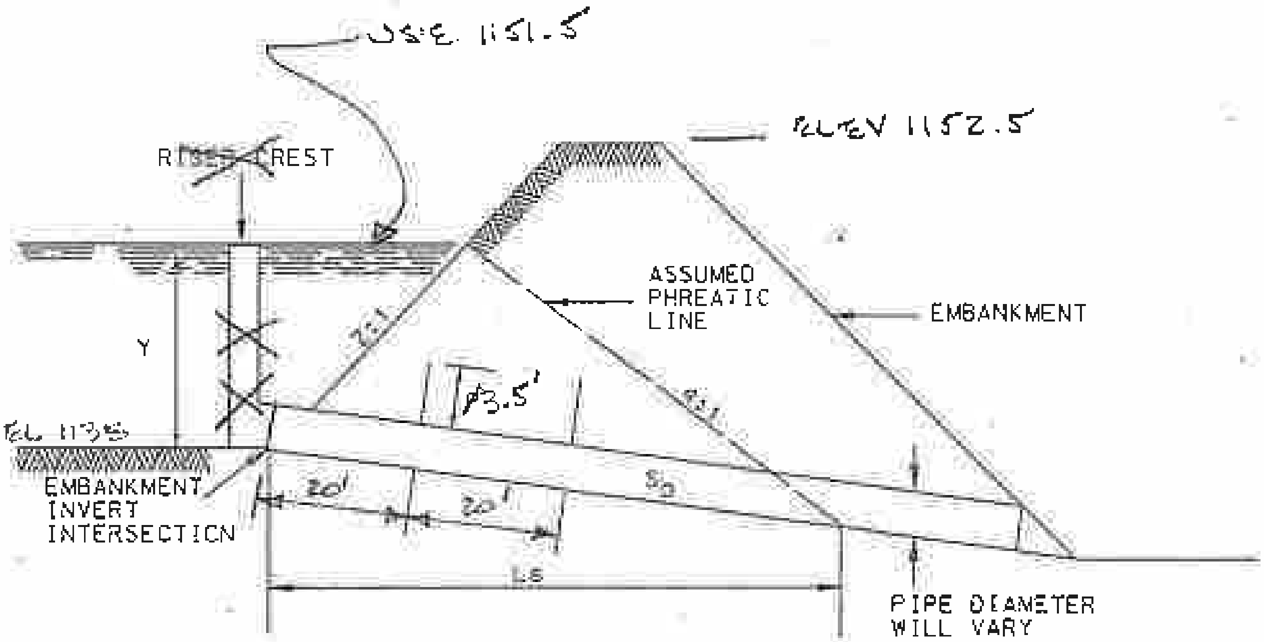


~~COLLAR WELDED IN PLACE ON BARREL SECTION~~

~~ANTI-SEEP COLLAR DESIGN~~

COLLAR FOR FLANGE JOINT PIPE

DETAIL 13 - ANTI-SEEP COLLAR DESIGN



ANTI-SEEP COLLAR DESIGN

where: p= vertical projection of anti-seep collar(ft).

Ls = length of pipe in the saturated zone (ft.)

y= distance in feet from upstream invert of pipe to highest normal water level expected to occur during the life of the structure. usually the top of the riser.

z= slope of upstream embankment as a ratio of z ft. horizontal to one ft. vertical.

s_o= slope of pipe in feet per foot.

This procedure is based on the phreatic line as shown in the drawing above.

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.AST STORMWATER MANAGEMENT

East Valley Outlets - Analyze proposed drainage to east and compare to previous permit conditions.

References:

1) "Ultimate Conditions - Drainage Facilities" calc. by SER 3/19/96, GAI Project 92-220-73-7
2) "Drainage Design Computations for Keystone Station, East Valley Ash Disposal Site, East Peripheral Drainage Ditch", H.F. Lenz Co., June 1985.
3) "Closure Plan Hydraulics", calc. by EHK 3/20/85, GAI Project 85-205-7
4) "Hydrologic Parameters for Channel Design" by EHK 2/7/85, GAI Project 85-205-7

The proposed drainage pattern has been altered for ultimate conditions with respect to the previous permit.The peak flows from the calcs. for the previous permit are as follows,

Drainage Structure	100 - Year, 24 - Hour Peak Flow (cfs)
East Valley West Side Collection Channel (EVWSCC) (see reference 3)	108
East Valley East Peripheral Drainage Ditch (EVEPDD) (see reference 2)	190
Total (not considering timing effects)	298

The flows reported above were estimated using various methods. Model the previous permit drainage and the currently proposed drainage with the SCS computer program TR-20 considering the drainage breakdown shown on the drainage schematics shown on sheets 2 and 3.

Time-of-concentration discussion

The previous permit's design calculations were completed in 1985. The times-of-concentration, t_c's, used for the permit design were estimated using the current, that is current in 1985, US Soil Conservation Service's (SCS) method for non-channelized portions of the t_c flowpath. The SCS now recommends another method for non-channelized portions of t_c flowpaths as documented in the SCS's TR-55, " Urban Hydrology for Small Watersheds", June 1986, which is now the accepted method. Estimates of t_c's obtained from the previous method and the current method for any particular drainage area could be significantly different, as will be demonstrated below. The maximum time-of-concentration used for the 1985 design is 1.1 hour for EVWSCC and 1.5 hour for EVEPDD.

Since the purpose of these calcs. is to compare the proposed drainage to the previously permitted drainage, t_c's for the previously permitted drainage pattern will be estimated using the current method. The estimates for this are shown on sheets 4 to 10. The maximum time-of-concentration using the current method for the previous permit's drainage pattern is 1.1 hour for EVEPDD which is significantly different from the previous permit value listed above. Note that all drainage structures under consideration for this permit will be designed/ or analyzed using current methods.

SUBJECT KEYSTONE STATION
PHASE II PERMITTING

BY SER DATE 4/22/96 PROJ. NO. 92-220-73-07

CHKD. BY PWC DATE 7/23/96 SHEET NO. 2 OF 23

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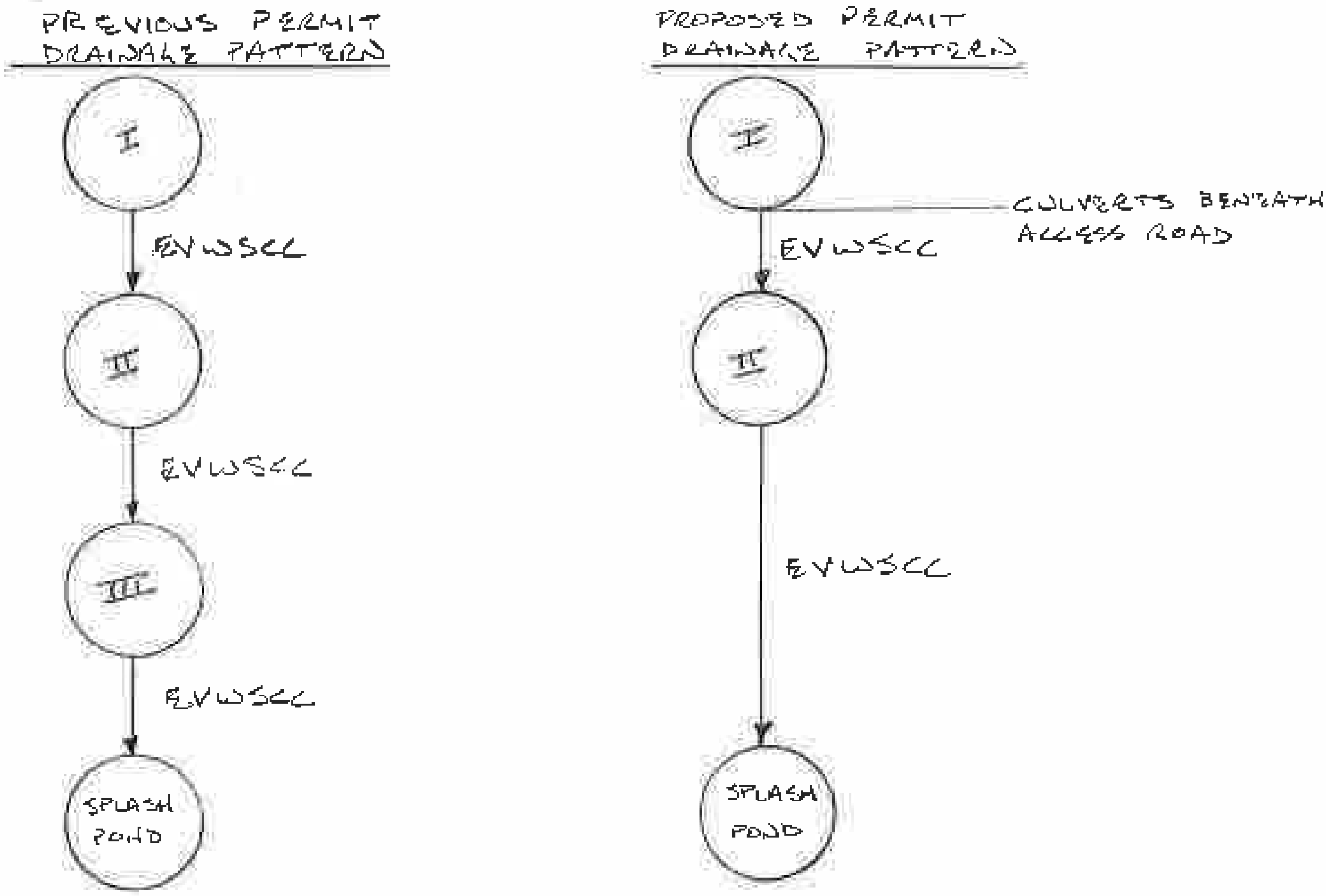
EAST VALLEY
WEST SIDE COLLECTION CHANNEL
DRAINAGE SCHEMATICS



AREAS I, II AND III ARE SHOWN ON WORKSHEET 92-220-73-7-SER 6.
AREA I INCLUDES AREAS 15, 16, 17, 18 AND 19 FROM REFERENCE 4.
AREA II INCLUDES AREAS 20, 21, 22, 23 AND 24 FROM REFERENCE 4.
AREA III INCLUDES AREAS 25, 26 AND 27 FROM REFERENCE 4.

AREA I INCLUDES ALL AREAS DRAINING TO ACCESS ROAD CULVERTS THAT IS SE1, SE2, SE3, AND SE4 SEE SHEET 4 AND 5 OF REFERENCE 1.

AREA II IS THE AREA REFERRED TO AS LOCAL SEE SHEET 45 OF REFERENCE 1.

SUBJECT Keystone Station

Phase II Permitting

BY SER DATE 4/22/96 PROJ. NO. 92-220-73-07

CHKD. BY PWC DATE 7/23/96 SHEET NO. 3 OF 23

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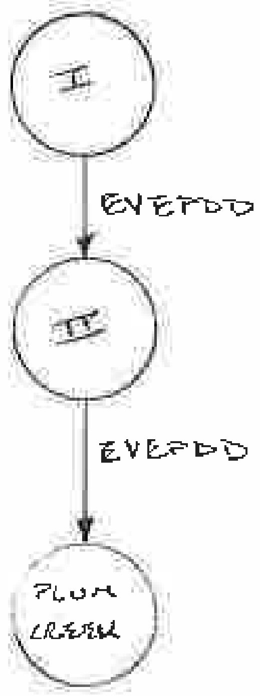
East Valley
East Peripheral Drainage Ditch
Drainage Schematic

EVEPDD
Drainage Pattern



FOR THE PREVIOUS PERMIT
AREA I AND AREA II ARE SHOWN ON WORKSHEET 92-220-73-7-SRB6
AREA I INCLUDES (LENZ) AREAS 1 AND 2 FROM REFERENCE 2.
AREA II INCLUDES (LENZ) AREAS 3, 4, 5, 6, 7, 8, 9, 10 AND 11 FROM REFERENCE 2.

FOR THE PROPOSED PERMIT
AREA I INCLUDES AREAS N1, N2 AND N3 FROM REFERENCE 1.
AREA II IS EQUIVALENT TO LENZ AREAS 3, 4, 5, 6, 7, 8, 9, 10 AND 11 FROM REFERENCE 2.

SUBJECT: Penelec - Keystone West Valley
Phase II Permitting
BY: SER DATE: 4/18/96 PROJ. NO.: 92-220-73-07
CHKD. BY: PWC DATE: 7/23/96 SHEET NO. 4 OF 23

GAI CONSULTANTS, INC.
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...e of Concentration Worksheet - SCS Methods
East Valley West Side Collection Channel - AREA I
Postdevelopment - Previous Permit Conditions

Reference: "Urban Hydrology for Small Watersheds", TR-55, Soil Conservation Service, June 1986

SHEET FLOW — Flowpath: a-b — units

1. Surface description (table 3-1) — Dense Grass
2. Manning's roughness coeff., n_{st} (table 3-1) — $n_{st} := 0.24$
3. Flow length, L_{st} (total $L_{st} \le 150$ feet) — $L_{st} := 30$ feet
4. Two-year, 24-hour rainfall, P_2 — $P_2 := 2.6$ inches — see reference 1, sheet 7.
5. Land Slope, $S_{st} := 0.50$ — $S_{st} = 0.5$
6. Sheet Flow Time, $T_{st} := \dfrac{0.007\cdot(n_{st}\cdot L_{st})^{0.8}}{P_2^{0.5}\cdot S_{st}^{0.4}}$ — $T_{st} = 0.028$ hours

SHALLOW CONCENTRATED FLOW — Flowpath: NA

7. Surface description (paved or unpaved)
8. Flow length, L_{sc} — $L_{sc} := 0$ feet
9. Watercourse Slope, $S_{sc} := 0$ — $S_{sc} = 0$
10. Average Velocity, $V_{sc} := 16.1345\cdot S_{sc}^{0.5}$ — $V_{sc} = 0$ fps
11. Shallow Conc. Flow time, $T_{sc} := \left(\dfrac{L_{sc}}{3600\cdot V_{sc}}\right)$ — $T_{sc} = 0$ hour

Note: see reference 1, sheet 7 for manning's n values for channels (typical).

Note: assume flow depths for channel flow (typical).

CHANNEL FLOW

	Flowpath: b-c	units	Flowpath: c-d
12. Bottom width, b	$b := 0$	feet	$b_1 := 4$
13. Side slopes, z $\quad z := \dfrac{2+100}{2}$	$z = 51$		$z_1 := 2$
14. Flow depth, d	$d := .15$	feet	$d_1 := 1$
15. Cross sectional area, $a := (b + z\cdot d)\cdot d$	$a = 1.148$	ft^2	$a_1 := (b_1 + z_1\cdot d_1)\cdot d_1$ $\quad a_1 = 6$
16. Wetted perimeter, $P_w := \left[b + 2\cdot d\cdot(1+z^2)^{0.5}\right]$	$P_w = 15.303$	feet	$P_{w1} := \left[b_1 + 2\cdot d_1\cdot(1+z_1^2)^{0.5}\right]$ $\quad P_{w1} = 8.472$
17. Hydraulic radius, $r := \dfrac{a}{P_w}$	$r = 0.075$	feet	$r_1 := \dfrac{a_1}{P_{w1}}$ $\quad r_1 = 0.708$
18. Channel Length, L_{ch}	$L_{ch} := 1040$	feet	$L_{ch1} := 330$
19. Channel Slope, $S_{ch} := 0.01$	$S_{ch} = 0.01$		$S_{ch1} := \dfrac{15 - .01\cdot 15}{45}$ $\quad S_{ch1} = 0.33$
20. Channel lining	GRASS		Fabric Formed Grout
21. Manning's roughness coeff., n	$n := 0.045$		$n_1 := 0.030$
22. Velocity, $V_{ch} := \left[\left(\dfrac{1.49}{n}\right)\cdot\left[r^{\left(\frac{2}{3}\right)}\right]\cdot S_{ch}^{\left(\frac{1}{2}\right)}\right]$	$V_{ch} = 0.589$	fps	$V_{ch1} := \left[\left(\dfrac{1.49}{n_1}\right)\cdot\left[r_1^{\left(\frac{2}{3}\right)}\right]\cdot S_{ch1}^{\left(\frac{1}{2}\right)}\right]$ $\quad V_{ch1} = 22.669$
22. Channel Flow time, $T_{ch} := \left(\dfrac{L_{ch}}{3600\cdot V_{ch}}\right)$	$T_{ch} = 0.491$	hour	$T_{ch1} := \left(\dfrac{L_{ch1}}{3600\cdot V_{ch1}}\right)$ $\quad T_{ch1} = 4.044\cdot 10^{-3}$

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...t Valley West Side Collection Channel - AREA I (Continued)
...tdevelopment - Previous Permit Conditions

CHANNEL FLOW — Flowpath: d-e

12. Bottom width, b — $b_2 := 2$ feet
13. Side slopes, z — $z_2 := 2$ — $z_2 = 2$
14. Flow depth, d — $d_2 := 2$ feet
15. Cross sectional area, $a_2 := (b_2 + z_2 \cdot d_2) \cdot d_2$ — $a_2 = 12$ ft^2
16. Wetted perimeter, $P_{w2} := \left[b_2 + 2 \cdot d_2 \cdot \left(1 + z_2^{\,2}\right)^{0.5}\right]$ — $P_{w2} = 10.944$ feet
17. Hydraulic radius, $r_2 := \frac{a_2}{P_{w2}}$ — $r_2 = 1.096$ feet
18. Channel Length, L_{ch} — $L_{ch2} := 1900$ feet
19. Channel Slope, $S_{ch2} := \frac{1215 - 1207}{L_{ch2}}$ — $S_{ch2} = 4.211 \cdot 10^{-3}$
20. Channel lining — GRASS
21. Manning's roughness coeff., n — $n_2 := 0.045$
22. Velocity, $V_{ch2} := \left[\left(\frac{1.49}{n_2}\right) \cdot \left[r_2^{\left(\frac{2}{3}\right)}\right] \cdot S_{ch2}^{\left(\frac{1}{2}\right)}\right]$ — $V_{ch2} = 2.285$ fps

Channel Flow time, $T_{ch2} := \left(\frac{L_{ch2}}{3600 \cdot V_{ch2}}\right)$ — $T_{ch2} = 0.231$ hour

Total Watershed Time-of-Concentration, $T_c := T_{st} + T_{sc} + T_{ch} + T_{ch1} + T_{ch2}$

$T_c = 0.75$ hour

SUBJECT: Penelec - Keystone West Valley
Phase II Permitting
BY: SER DATE: 4/18/96 PROJ. NO.: 92-220-73-07
CHKD. BY: PMK DATE: 7/23/96 SHEET NO. 6 OF 23

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...e of Concentration Worksheet - SCS Methods
...t Valley West Side Collection Channel - AREA II
Postdevelopment - Previous Permit Conditions

Reference: "Urban Hydrology for Small Watersheds", TR-55, Soil Conservation Service, June 1986

SHEET FLOW	Flowpath: a-b	units
1. Surface description (table 3-1)	Dense Grass	
2. Manning's roughness coeff., n_{st} (table 3-1)	$n_{st} := 0.24$	
3. Flow length, L_{st} (total $L_{st} \le 150$ feet)	$L_{st} := 30$	feet
4. Two-year, 24-hour rainfall, P_2	$P_2 := 2.6$	inches
5. Land Slope, $S_{st} := 0.50$	$S_{st} = 0.5$	
6. Sheet Flow Time, $T_{st} := \dfrac{0.007\cdot(n_{st}\cdot L_{st})^{0.8}}{P_2^{0.5}\cdot S_{st}^{0.4}}$	$T_{st} = 0.028$	hours

SHALLOW CONCENTRATED FLOW	Flowpath: NA	
7. Surface description (paved or unpaved)		
8. Flow length, L_{sc}	$L_{sc} := 0$	feet
9. Watercourse Slope, $S_{sc} := 0$	$S_{sc} = 0$	
10. Average Velocity, $V_{sc} = 16.1345\cdot S_{sc}^{0.5}$	$V_{sc} = 0$	fps
Shallow Conc. Flow time, $T_{sc} := \left(\dfrac{L_{sc}}{3600\cdot V_{sc}}\right)$	$T_{sc} = 0$	hour

CHANNEL FLOW	Flowpath: b-c		Flowpath: c-e
12. Bottom width, b	$b := 0$	feet	$b_1 := 2$
13. Side slopes, z $\quad z := \dfrac{2 - 100}{2}$	$z = 51$		$z_1 := 2$
14. Flow depth, d	$d := .15$	feet	$d_1 := 0.75$
15. Cross sectional area, $a := (b + z\cdot d)\cdot d$	$a = 1.148$	ft^2	$a_1 := (b_1 + z_1\cdot d_1)\cdot d_1 \quad a_1 = 2.625$
16. Wetted perimeter, $P_w := \left[b + 2\cdot d\cdot(1+z^2)^{0.5}\right]$	$P_w = 15.303$	feet	$P_{w1} := \left[b_1 + 2\cdot d_1\cdot(1+z_1^2)^{0.5}\right] \quad P_{w1} = 5.354$
17. Hydraulic radius, $r := \dfrac{a}{P_w}$	$r = 0.075$	feet	$r_1 := \dfrac{a_1}{P_{w1}} \quad r_1 = 0.49$
18. Channel Length, L_{ch}	$L_{ch} := 1880$	feet	$L_{ch1} := 650$
19. Channel Slope, $S_{ch} := 0.01$	$S_{ch} = 0.01$		$S_{ch1} := \dfrac{1260 - 1207}{650} \quad S_{ch1} = 0.082$
20. Channel lining	GRASS		Grouted Rock
21. Manning's roughness coeff., n	$n := 0.045$		$n_1 := 0.025$
22. Velocity, $V_{ch} := \left[\left(\dfrac{1.49}{n}\right)\cdot\left[r^{\left(\frac{2}{3}\right)}\right]\cdot S_{ch}^{\left(\frac{1}{2}\right)}\right]$	$V_{ch} = 0.589$	fps	$V_{ch1} := \left[\left(\dfrac{1.49}{n_1}\right)\cdot\left[r_1^{\left(\frac{2}{3}\right)}\right]\cdot S_{ch1}^{\left(\frac{1}{2}\right)}\right] \quad V_{ch1} = 10.582$
Channel Flow time, $T_{ch} := \left(\dfrac{L_{ch}}{3600\cdot V_{ch}}\right)$	$T_{ch} = 0.887$	hour	$T_{ch1} := \left(\dfrac{L_{ch1}}{3600\cdot V_{ch1}}\right) \quad T_{ch1} = 0.017$

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CHANNEL FLOW Flowpath: e-f

12. Bottom width, b $b_2 := 3$ feet

13. Side slopes, z $z_2 := 2$ $z_2 = 2$

14. Flow depth, d $d_2 := 2.5$ feet

15. Cross sectional area, $a_2 := (b_2 + z_2 \cdot d_2) \cdot d_2$ $a_2 = 20$ ft^2

16. Wetted perimeter, $P_{w2} := \left[b_2 + 2 \cdot d_2 \cdot \left(1 + z_2^{2}\right)^{0.5}\right]$ $P_{w2} = 14.18$ feet

17. Hydraulic radius, $r_2 := \frac{a_2}{P_{w2}}$ $r_2 = 1.41$ feet

18. Channel Length, L_{ch} $L_{ch2} := 2600$ feet

19. Channel Slope, $S_{ch2} := \frac{1207 - 1188}{2600}$ $S_{ch2} = 0.0073$

20. Channel lining Grouted Rock

21. Manning's roughness coeff., n $n_2 := 0.025$

22. Velocity, $V_{ch2} := \left[\left(\frac{1.49}{n_2}\right) \cdot \left[r_2^{\left(\frac{2}{3}\right)}\right] \cdot S_{ch2}^{\left(\frac{1}{2}\right)}\right]$ $V_{ch2} = 6.408$ fps

Channel Flow time, $T_{ch2} := \left(\frac{L_{ch2}}{3600 \cdot V_{ch2}}\right)$ $T_{ch2} = 0.113$ hour

Total Watershed Time-of-Concentration, $T_c := T_{st} + T_{sc} + T_{ch} + T_{ch1} + T_{ch2}$ $T_c = 1.04$ hour

SUBJECT: Penelec - Keystone West Valley
Phase II Permitting
BY: SER DATE: 4/18/96 PROJ. NO.: 92-220-73-07
CHKD. BY: PWC DATE: 7/23/96 SHEET NO. 1 OF 23

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...ge of Concentration Worksheet - SCS Methods
...t Valley West Side Collection Channel - AREA III
Postdevelopment - Previous Permit Conditions

Reference: "Urban Hydrology for Small Watersheds", TR-55, Soil Conservation Service, June 1986

SHEET FLOW	Flowpath: a-b	units
1. Surface description (table 3-1)	Dense Grass	
2. Manning's roughness coeff., n_{st} (table 3-1)	$n_{st} := 0.24$	
3. Flow length, L_{st} (total $L_{st} \le 150$ feet)	$L_{st} := 30$	feet
4. Two-year, 24-hour rainfall, P_2	$P_2 := 2.6$	inches
5. Land Slope, $S_{st} := 0.50$	$S_{st} = 0.5$	
6. Sheet Flow Time, $T_{st} := \dfrac{0.007\cdot(n_{st}\cdot L_{st})^{0.8}}{P_2^{\,0.5}\cdot S_{st}^{\,0.4}}$	$T_{st} = 0.028$	hours

SHALLOW CONCENTRATED FLOW	Flowpath: NA	
7. Surface description (paved or unpaved)		
8. Flow length, L_{sc}	$L_{sc} := 0$	feet
9. Watercourse Slope, $S_{sc} := 0$	$S_{sc} = 0$	
10. Average Velocity, $V_{sc} := 16.1345\cdot S_{sc}^{\,0.5}$	$V_{sc} = 0$	fps
Shallow Conc. Flow time, $T_{sc} := \left(\dfrac{L_{sc}}{3600\cdot V_{sc}}\right)$	$T_{sc} = 0$	hour

CHANNEL FLOW	Flowpath: b-c		Flowpath: c-f	
12. Bottom width, b	$b := 0$	feet	$b_1 := 2$	
13. Side slopes, z $\quad z := \dfrac{2+100}{2}$	$z = 51$		$z_1 := 2$	
14. Flow depth, d	$d := .15$	feet	$d_1 := 0.75$	
15. Cross sectional area, $a := (b + z\cdot d)\cdot d$	$a = 1.148$	ft^2	$a_1 := (b_1 + z_1\cdot d_1)\cdot d_1$	$a_1 = 2.625$
16. Wetted perimeter, $P_w := \left[b + 2\cdot d\cdot(1+z^2)^{0.5}\right]$	$P_w = 15.303$	feet	$P_{w1} := \left[b_1 + 2\cdot d_1\cdot(1+z_1^{\,2})^{0.5}\right]$	$P_{w1} = 5.354$
17. Hydraulic radius, $r := \dfrac{a}{P_w}$	$r = 0.075$	feet	$r_1 := \dfrac{a_1}{P_{w1}}$	$r_1 = 0.49$
18. Channel Length, L_{ch}	$L_{ch} = 1750$	feet	$L_{ch1} := 700$	
19. Channel Slope, $S_{ch} := 0.01$	$S_{ch} = 0.01$		$S_{ch1} := \dfrac{1269 - 1188}{L_{ch1}}$	$S_{ch1} = 0.116$
20. Channel lining	GRASS		Grouted Rock	
21. Manning's roughness coeff., n	$n := 0.045$		$n_1 := 0.025$	
22. Velocity, $V_{ch} := \left[\left(\dfrac{1.49}{n}\right)\cdot\left[r^{\left(\frac{2}{3}\right)}\right]\cdot S_{ch}^{\left(\frac{1}{2}\right)}\right]$	$V_{ch} = 0.589$	fps	$V_{ch1} := \left[\left(\dfrac{1.49}{n_1}\right)\cdot\left[r_1^{\left(\frac{2}{3}\right)}\right]\cdot S_{ch1}^{\left(\frac{1}{2}\right)}\right]$	$V_{ch1} = 12.606$
Channel Flow time, $T_{ch} := \left(\dfrac{L_{ch}}{3600\cdot V_{ch}}\right)$	$T_{ch} = 0.826$	hours	$T_{ch1} := \left(\dfrac{L_{ch1}}{3600\cdot V_{ch1}}\right)$	$T_{ch1} = 0.015$

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st Valley West Side Collection Channel - AREA III (Continued)
Postdevelopment - Previous Permit Conditions

CHANNEL FLOW — Flowpath: f-g

12. Bottom width, b — $b_2 := 3$ feet

13. Side slopes, z $z_2 := 2$ — $z_2 = 2$

14. Flow depth, d — $d_2 := 2$ feet

15. Cross sectional area, $a_2 := (b_2 + z_2 \cdot d_2) \cdot d_2$ — $a_2 = 14$ ft^2

16. Wetted perimeter, $P_{w2} := \left[b_2 + 2 \cdot d_2 \cdot \left(1 + z_2^{\,2}\right)^{0.5}\right]$ — $P_{w2} = 11.944$ feet

17. Hydraulic radius, $r_2 := \dfrac{a_2}{P_{w2}}$ — $r_2 = 1.172$ feet

18. Channel Length, L_{ch} — $L_{ch2} := 1930$ feet

19. Channel Slope, $S_{ch2} := \dfrac{1188 - 1005}{L_{ch2}}$ — $S_{ch2} = 0.095$

20. Channel lining — Grouted Rock

21. Manning's roughness coeff., n — $n_2 := 0.025$

22. Velocity, $V_{ch2} := \left[\left(\dfrac{1.49}{n_2}\right) \cdot \left[r_2^{\left(\frac{2}{3}\right)}\right] \cdot S_{ch2}^{\left(\frac{1}{2}\right)}\right]$ — $V_{ch2} = 20.402$ fps

22. Channel Flow time, $T_{ch2} := \left(\dfrac{L_{ch2}}{3600 \cdot V_{ch2}}\right)$ — $T_{ch2} = 0.026$ hours

Total Watershed Time-of-Concentration, $T_c := T_{st} - T_{sc} + T_{ch} + T_{ch1} + T_{ch2}$ — $T_c = 0.90$ hour

SUBJECT: Penelec - Keystone West Valley
Phase II Permitting
BY: SER DATE: 4/18/96 PROJ. NO.: 92-220-73-07
CHKD. BY: PWC DATE: 7/23/96 SHEET NO. 9 OF 23

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...e of Concentration Worksheet - SCS Methods

East Valley East Peripheral
Drainage Ditch - AREA I
Postdevelopment - Previous Permit Conditions
SHEET FLOW

Reference: "Urban Hydrology for Small Watersheds", TR-55, Soil Conservation Service, June 1986

Flowpath: a-b units

1. Surface description (table 3-1) — Dense Grass
2. Manning's roughness coeff., n_{st} (table 3-1) — $n_{st} := 0.24$
3. Flow length, L_{st} (total $L_{st} \le 150$ feet) — $L_{st} := 150$ feet
4. Two-year, 24-hour rainfall, P_2 — $P_2 := 2.6$ inches
5. Land Slope, $S_{st} := \dfrac{1325 - 1320}{500}$ — $S_{st} = 0.01$
6. Sheet Flow Time, $T_{st} := \dfrac{0.007 \cdot (n_{st} \cdot L_{st})^{0.8}}{P_2^{0.5} \cdot S_{st}^{0.4}}$ — $T_{st} = 0.482$

SHALLOW CONCENTRATED FLOW

	Flowpath: a-b		Flowpath: b-c	
7. Surface description (paved or unpaved)	unpaved		unpaved	
8. Flow length, L_{sc}	$L_{sc} := 2250$	feet	$L_{sc1} := 1400$	
9. Watercourse Slope, $S_{sc} := \dfrac{1325 - 1305}{1930}$	$S_{sc} = 0.01$		$S_{sc1} := \dfrac{1305 - 1293}{1400}$	$S_{sc1} = 8.571 \cdot 10^{-3}$
10. Average Velocity, $V_{sc} := 16.1345 \cdot S_{sc}^{0.5}$	$V_{sc} = 1.642$	fps	$V_{sc1} := 16.1345 \cdot S_{sc1}^{0.5}$	$V_{sc1} = 1.494$
11. Shallow Conc. Flow time, $T_{sc} := \left(\dfrac{L_{sc}}{3600 \cdot V_{sc}}\right)$	$T_{sc} = 0.381$	hour	$T_{sc1} := \left(\dfrac{L_{sc1}}{3600 \cdot V_{sc1}}\right)$	$T_{sc1} = 0.26$

Total Watershed Time-of-Concentration, $T_c := T_{st} + T_{sc} + T_{sc1}$ — $T_c = 1.12$ hour

SUBJECT: Penelec - Keystone West Valley
Phase II Permitting
BY: SER DATE: 4/18/96 PROJ. NO.: 92-220-73-07
CHKD. BY: PWC DATE: 7/23/96 SHEET NO. 10 OF 23

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Time of Concentration Worksheet - SCS Methods
West Valley East Peripheral
Drainage Ditch - AREA II
Postdevelopment - Previous Permit Conditions

Reference: "Urban Hydrology for Small Watersheds", TR-55, Soil Conservation Service, June 1986

SHEET FLOW — Flowpath: a-b — units

1. Surface description (table 3-1) — Dense Grass
2. Manning's roughness coeff., n_{st} (table 3-1) — $n_{st} := 0.24$
3. Flow length, L_{st} (total $L_{st} \le 150$ feet) — $L_{st} := 30$ feet
4. Two-year, 24-hour rainfall, P_2 — $P_2 := 2.6$ inches
5. Land Slope, $S_{st} := 0.50$ — $S_{st} = 0.5$
6. Sheet Flow Time, $T_{st} := \dfrac{0.007 \cdot (n_{st} \cdot L_{st})^{0.8}}{P_2^{0.5} \cdot S_{st}^{0.4}}$ — $T_{st} = 0.028$ hours

SHALLOW CONCENTRATED FLOW — Flowpath: NA

7. Surface description (paved or unpaved)
8. Flow length, L_{sc} — $L_{sc} := 0$ feet
9. Watercourse Slope, $S_{sc} := 0$ — $S_{sc} = 0$
10. Average Velocity, $V_{sc} := 16.1345 \cdot S_{sc}^{0.5}$ — $V_{sc} = 0$ fps
11. Shallow Conc. Flow time, $T_{sc} := \left(\dfrac{L_{sc}}{3600 \cdot V_{sc}}\right)$ — $T_{sc} = 0$ hour

CHANNEL FLOW — Flowpath: b-c

12. Bottom width, b — $b := 0$ feet
13. Side slopes, z — $z := \dfrac{2 + 100}{2}$ — $z = 51$
14. Flow depth, d — $d := .15$ feet
15. Cross sectional area, $a := (b + z \cdot d) \cdot d$ — $a = 1.148$ ft^2
16. Wetted perimeter, $P_w := \left[b + 2 \cdot d \cdot (1 + z^2)^{0.5}\right]$ — $P_w = 15.303$ feet
17. Hydraulic radius, $r := \dfrac{a}{P_w}$ — $r = 0.075$ feet
18. Channel Length, L_{ch} — $L_{ch} := 1020$ feet
19. Channel Slope, $S_{ch} := 0.01$ — $S_{ch} = 0.01$
20. Channel lining — GRASS
21. Manning's roughness coeff., n — $n := 0.045$
22. Velocity, $V_{ch} := \left[\left(\dfrac{1.49}{n}\right) \cdot \left[r^{\left(\frac{2}{3}\right)}\right] \cdot S_{ch}^{\left(\frac{1}{2}\right)}\right]$ — $V_{ch} = 0.589$ fps
23. Channel Flow time, $T_{ch} := \left(\dfrac{L_{ch}}{3600 \cdot V_{ch}}\right)$ — $T_{ch} = 0.481$ hours

Flowpath c-h is equivalent to the flowpath downstream of (Lenz) Area 2, see next sheet

t_t for flow path c-h is

$T_{ch1} := 0.133$ hour

Total Watershed Time-of-Concentration, $T_c := T_{st} + T_{sc} + T_{ch} + T_{ch1}$ — $T_c = 0.64$ hour

EASTSWM6.MCD, 7/18/96, 9

SUBJECT: Keystone Station
Phase II Permitting
BY: SER DATE: 4/18/96 PROJ. NO.: 92-220-73-07
CHKD. BY: Pw;c DATE: 7/23/96 SHEET NO. 11 OF 23

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East Valley East Peripheral Drainage Ditch (EVEPDD)

Previous permit design calc. parameters (see reference 2).

at downstream outlet at Plum Creek (Lenz areas 1 thru 11)

$t_c := 1.803 \cdot hr$ which was rounded to 1.5 hour for design

$Area := 0.25$ square miles

$CN := 78$

at outlet of (Lenz) area 2, which is equivalent to the outlet of area N3 (see sheet ___)

$t_c := 1.67 \cdot hr$

$Area := 0.169$ square miles

$CN := 78$

Note that the time difference between these two points was estimated using a channel flow method which (was in March 1995 and) is an accepted method. Therefore the time difference of 1.803-1.67= 0.133 hours can be added to the time of concentration calculated for the proposed drainage pattern (t_c = 0.32 hour = maximum of areas N1, N2, and N3, see sheet 24 of reference 1). The time difference of 0.133 hours can also be added to the t_c calculated on sheet 10 for the previous permit drainage pattern, the time of 0.133 hours was used as the tc between points c and h on sheet 10.

EVEPDD Data for this analysis

Previous Permit Drainage Pattern
AREA I
Drainage area = 0.169 square miles
CN= 78
$t_c := 1.12 \cdot hr$ from sheet 9

AREA II
Drainage area = $0.25 - 0.169$ square miles
CN= 78
$t_c := 0.64 \cdot hr$ from sheet 10

Currently Proposed Drainage Pattern (see reference 1)
AREA I
Drainage area = $0.0036 + 0.0072 + 0.04 = 0.051$ square miles, sum of areas N1, N2, and N3

$$CN = \frac{0.0036 \cdot 78 + 0.0072 \cdot 79 + 0.04 \cdot 75}{0.0036 + 0.0072 + 0.04} = 76$$ composite of areas N1, N2, and N3

$t_c := 0.32 \cdot hr$ maximum from areas N1, N2, and N3

AREA II
All data same as Previous Permit Drainage Pattern

SUBJECT: Keystone Station
Phase II Permitting
BY: SER DATE: 4/18/96 PROJ. NO.: 92-220-73-07
CHKD. BY: PWC DATE: 7/23/96 SHEET NO. 12 OF 23

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: Valley West Side Collection Channel (EVWSCC)

EVWSCC Data for this analysis

Previous Permit Drainage Pattern (see reference 4)

AREA I

Drainage area = $0.0019 + 0.025 + 0.014 = 0.041$ square miles, sum of areas 15,16,17,18, &19

$$CN = \frac{0.0019 \cdot 70 + 0.025 \cdot 78 + 0.014 \cdot 70}{0.0019 + 0.025 + 0.014} = 75$$ composite of areas 15,16,17,18, &19

$t_c := 0.75 \cdot hr$ from sheet 5

AREA II

Drainage area = $0.0165 + 0.0082 + 0.023 = 0.048$ square miles, sum of areas 20,21,22,23 &24

$$CN = \frac{0.0165 \cdot 78 + 0.0082 \cdot 75.1 + 0.023 \cdot 70}{0.0165 + 0.0082 + 0.023} = 74$$ composite of areas 20,21,22,23 &24

$t_c := 1.04 \cdot hr$ from sheet 6A

AREA III

Drainage area = $0.023 + 0.0031 + 0.00024 = 0.026$ square miles, sum of areas 25, 26 &27

$$CN = \frac{0.023 \cdot 78 + 0.0031 \cdot 70 + 0.00024 \cdot 70}{0.023 + 0.0031 + 0.00024} = 77$$ composite of areas 25, 26 &27

$t_c := 0.90 \cdot hr$ from sheet 8

Currently Proposed Drainage Pattern (see reference 1)

AREA I

Drainage area = $0.0448 + 0.0061 + 0.0166 + 0.0275 = 0.095$ square miles, sum of areas SE1, SE2, SE3 and SE4

$$CN = \frac{0.0448 \cdot 78 + 0.0061 \cdot 78 + 0.0166 \cdot 78 + 0.0275 \cdot 80}{0.0448 + 0.0061 + 0.0166 + 0.0275} = 79$$ composite of areas SE1, SE2, SE3 and SE4

$t_c := 0.28 \cdot hr$ maximum from areas SE1, SE2, SE3 and SE4

AREA II (see sheet 24 of reference 1)

Drainage area = 0.0044 square miles

CN = 80

$t_c := 0.10 \cdot hr$

SUBJECT: Keystone Station
Phase II Permitting
BY: SER DATE: 4/18/96 PROJ. NO.: 92-220-73-07
CHKD. BY: PWC DATE: 7/23/96 SHEET NO. 13 OF 23

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Hydrology Summary

A TR-20 run has been completed using the data above. The input file is shown on sheet 14 and an output summary is shown on sheet 15. The total flows for the 100-year, 24- hour storm event are

337 cfs for the previous permit drainage pattern and
377 cfs for the currently proposed drainage pattern.

The increase in flow is $\frac{377-337}{337}\cdot 100 = 12$ percent or $377 - 337 = 40$ cfs. This increase is negligible when compared to the entire Plum Creek watershed.

The 100- year flood flow in Plum Creek has been estimated to be between 9800 cfs and 12,600 cfs near the outlet of the two channels see sheet 18.

Therefore, the increase in flow in Plum Creek is between $\left(\frac{9800+40}{9800} - 1\right)\cdot 100 = 0.4$ percent

+ $\left(\frac{12600+40}{12600} - 1\right)\cdot 100 = 0.3$ percent. Therefore the increase is insignificant.

Note that this estimate does not account for the effects of timing which would further reduce the effects of the increase in flow from the site.

Conclusion: The channels and their outlets to Plum Creek are located within the permit boundary, therefore the increase in flow in the channels will not affect other property owners. The increase in flow in Plum Creek is negligible, therefore the project will not affect downstream property owners along Plum Creek.

SHEET 14/23
Ckd : PWC 7/23/90

```
JOB TR-20                    FULLPRINT          SUMMARY   NOPLOTS
TITLE 111   KEYSTONE WEST VALLEY - EAST SWM - 92-220-73-7
 6 RUNOFF 1 001        1 0.169       78.         1.12         1           AIPPEP
 6 RUNOFF 1 001        2 0.081       78.         0.64         1           AIIPPEP
 6 ADDHYD 4 001    1 2 3                                      1           TPPEP
 6 RUNOFF 1 001        4 0.051       76.         0.32         1           AICPEP
 6 RUNOFF 1 001        2 0.081       78.         0.64         1           AIICPEP
 6 ADDHYD 4 001    4 2 5                                      1           TCPEP
 6 RUNOFF 1 001        6 0.041       75.         0.75         1           AIPPWS
 6 RUNOFF 1 001        7 0.048       74.         1.04         1           AIIPPWS
 6 RUNOFF 1 001        1 0.026       77.         0.90         1           AIIIPPWS
 6 ADDHYD 4 001    6 7 2                                      1
 6 ADDHYD 4 001    2 1 4                                      1           TPPWS
 6 RUNOFF 1 001        6 0.095       79.         0.28         1           AICPWS
 6 RUNOFF 1 001        7 0.0044      80.         0.10         1           AIICPWS
 6 ADDHYD 4 001    6 7 1                                      1           TCPWS
 6 ADDHYD 4 001    3 4 2                                      1           TPP
 6 ADDHYD 4     01 5 1 3                                      1           TCP
   ENDATA
 7 LIST
 7 INCREM 6            0.1
 7 COMPUT 7 001     01 0.             5.2         1.          2 2        100 YR
   ENDCMP 1
   ENDJOB 2
```

SHEET 15/23

SUMMARY TABLE 1 - SELECTED RESULTS OF STANDARD AND EXECUTIVE CONTROL INSTRUCTIONS IN THE ORDER PERFORMED CKD: PWC 7/23/94
(A STAR(*) AFTER THE PEAK DISCHARGE TIME AND RATE (CFS) VALUES INDICATES A FLAT TOP HYDROGRAPH
A QUESTION MARK(?) INDICATES A HYDROGRAPH WITH PEAK AS LAST POINT.)

...ION/ STRUCTURE ID		STANDARD CONTROL OPERATION	DRAINAGE AREA (SQ MI)	RAIN TABLE #	ANTEC MOIST COND	MAIN TIME INCREM (HR)	PRECIPITATION BEGIN (HR)	PRECIPITATION AMOUNT (IN)	PRECIPITATION DURATION (HR)	RUNOFF AMOUNT (IN)	PEAK DISCHARGE ELEVATION (FT)	PEAK DISCHARGE TIME (HR)	PEAK DISCHARGE RATE (CFS)	PEAK DISCHARGE RATE (CSM)
ALTERNATE	0	STORM	0											
XSECTION	1	RUNOFF	.17	2	2	.10	.0	5.20	24.00	2.88	---	12.60	150.76	892.1
XSECTION	1	RUNOFF	.08	2	2	.10	.0	5.20	24.00	2.88	---	12.28	105.53	1302.8
XSECTION	1	ADDHYD	.25	2	2	.10	.0	5.20	24.00	2.88	---	12.43	231.15	924.6
XSECTION	1	RUNOFF	.05	2	2	.10	.0	5.20	24.00	2.70	---	12.10	92.40	1811.7
XSECTION	1	RUNOFF	.08	2	2	.10	.0	5.20	24.00	2.88	---	12.28	105.53	1302.8
XSECTION	1	ADDHYD	.13	2	2	.10	.0	5.20	24.00	2.81	---	12.15	178.61	1353.1
XSECTION	1	RUNOFF	.04	2	2	.10	.0	5.20	24.00	2.61	---	12.34	43.95	1072.0
XSECTION	1	RUNOFF	.05	2	2	.10	.0	5.20	24.00	2.52	---	12.56	39.04	813.3
XSECTION	1	RUNOFF	.03	2	2	.10	.0	5.20	24.00	2.79	---	12.44	26.34	1012.9
XSECTION	1	ADDHYD	.09	2	2	.10	.0	5.20	24.00	2.56	---	12.43	79.59	894.3
XSECTION	1	ADDHYD	.12	2	2	.10	.0	5.20	24.00	2.61	---	12.43	105.91	921.0
XSECTION	1	RUNOFF	.09	2	2	.10	.0	5.20	24.00	2.98	---	12.07	200.03	2105.6
XSECTION	1	RUNOFF	.00	2	2	.10	.0	5.20	24.00	3.03	---	11.97	12.62	2868.3
XSECTION	1	ADDHYD	.10	2	2	.10	.0	5.20	24.00	2.98	---	12.06	206.50	2077.4
XSECTION	1	ADDHYD	.37	2	2	.10	.0	5.20	24.00	2.80	---	12.43	337.06	923.5
...UCTURE	1	ADDHYD	.23	2	2	.10	.0	5.20	24.00	2.88	---	12.10	377.28	1630.4

SUBJECT KEYSTONE
PHASE II PERMITTING

BY SER DATE 7/17/96 PROJ. NO 92-220-73-7

CHKD. BY MRL DATE 7/18/96 SHEET NO. 16 OF 23

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100-YEAR FLOOD FLOW ON PLUM CREEK

ESTIMATE THE 100-YEAR FLOOD FLOW ON PLUM CREEK USING THE PSU-IV METHOD.

THE WATERSHED IS SHOWN ON THE NEXT SHEET

$$AREA = 32.5\ IN^2 \cdot \left(\frac{100{,}000\ IN}{IN}\right)^2 \cdot \left(\frac{FT}{12\ IN}\right)^2 \cdot \left(\frac{MI}{5280\ FT}\right)^2 = 81.0\ MI^2$$

THE WATERSHED IS IN REGION 4 SEE SHEET 19

$$\therefore \hat{Y} = 2.60 + 0.85 \cdot (\log A) - 0.44\ (\log FOR)$$

SEE SHEET 20

$A = 81\ MI^2$

USE FOR = 40, WATERSHED IS APPROX 40% FORESTED

$$\therefore \hat{Y} = 2.60 + 0.85\ (\log 81) - 0.44\ (\log 40)$$

$$\hat{Y} = 3.52$$

$\bar{S}_Y = 0.232$ SEE SHEET 22

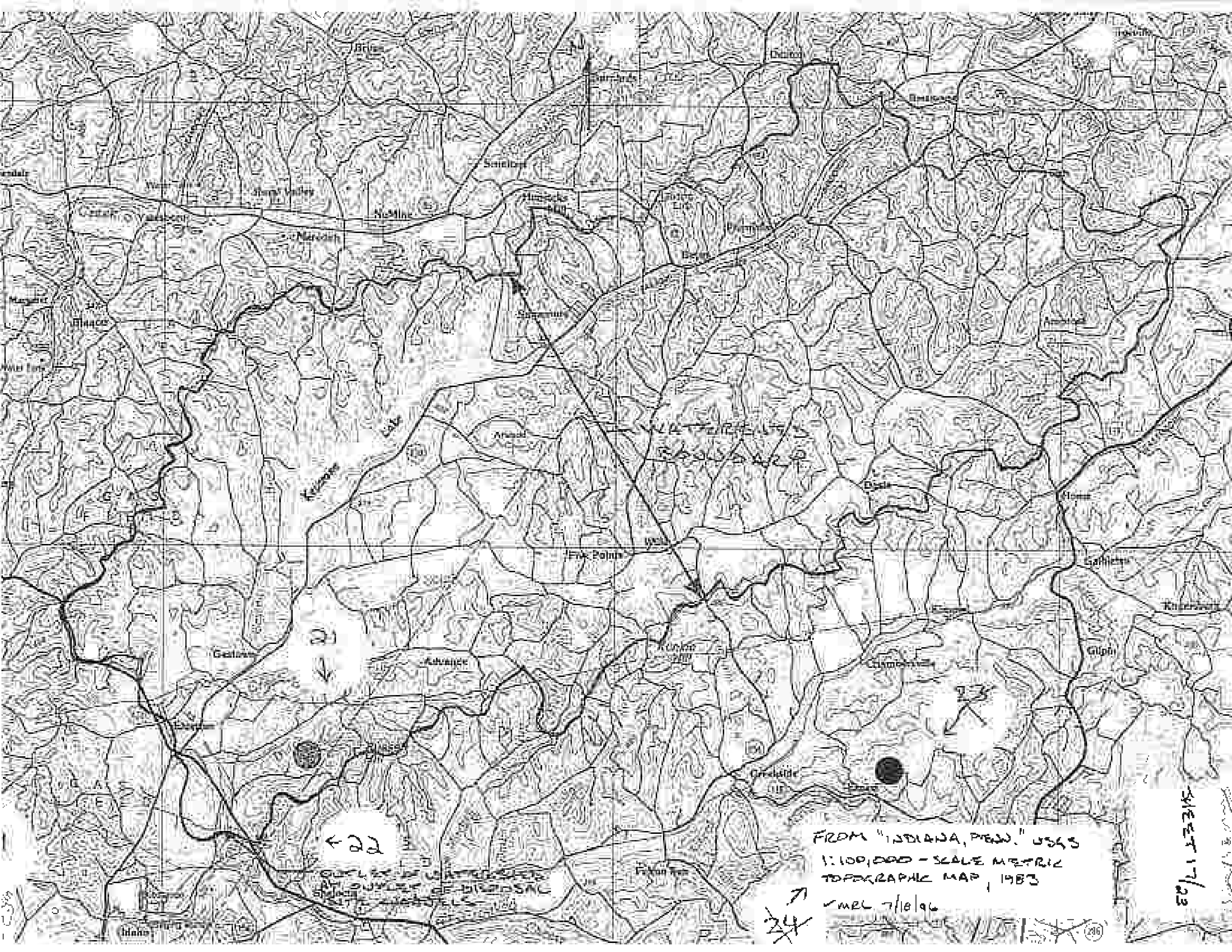
$\bar{G} = 0.22$ SEE SHEET 23

$K_Y = 2.486$ FOR 100-YEAR SEE SHEET 21

$$Y_{100} = \hat{Y} + K_Y \cdot \bar{S}_Y = 3.52 + 2.486 \cdot 0.232 = 4.10$$

$$Q_{100} = 10^{Y_{100}} = 10^{4.10} = 12{,}600\ CFS$$

THIS DOES NOT ACCOUNT FOR THE DAMPING EFFECTS OF THE KEYSTONE LAKE.


FROM "INDIANA, PENN." USGS
1:100,000 - SCALE METRIC
TOPOGRAPHIC MAP, 1983
✓MRL 7/10/96
24
←22
21
23
WATERSHED BOUNDARY
Five Points
Advance
Keystone Lake
Sheet 17/23

SUBJECT Keystone
Phase II Permitting
BY SER DATE 7/17/96 PROJ. NO. 92-220-73-7
CHKD. BY MRL DATE 7/18/96 SHEET NO. 18 OF 23

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KEYSTONE LAKE

KEYSTONE LAKE IS A WATER SUPPLY RESERVOIR FOR KEYSTONE STATION. IT IS LOCATED ON THE NORTH BRANCH OF PLUM CREEK 0.5± MILE UPSTREAM OF THE CONFLUENCE OF THE NORTH AND SOUTH BRANCHES OF PLUM CREEK.

THE EFFECT OF THE RESERVOIR ON THE 100-YEAR FLOOD WILL NOT BE ESTIMATED HEREIN, EXCEPT AS FOLLOWS:

ASSUME THAT THE RESERVOIR TOTALLY CONTROLS THE 100-YEAR EVENT AND ESTIMATE THE FLOW FROM THE REMAINING WATERSHED TO THE DISPOSAL SITE LOCATION. THE ACTUAL 100-YEAR FLOOD WILL BE MUCH GREATER SINCE THE RESERVOIR IS FOR WATER SUPPLY NOT FLOOD CONTROL.

KEYSTONE LAKE DRAINAGE AREA = 20.6 MI^2 FROM "KEYSTONE STATION DAM, NDI No. PA. -275, PHASE I INSPECTION REPORT, NATIONAL DAM INSPECTION PROGRAM", BY GAI, JUNE 1978

$\therefore$ REMAINING DRAINAGE AREA = 81.0 − 20.6 = 60.4 MI^2

$\therefore$ REMAINING DA'S $\hat{y} = 2.60 + 0.85\,(\log A) - 0.44\,(\log FOR)$
USE FOR = 40 AS BEFORE

$\hat{y} = 2.60 + 0.85\,(\log(60.4)) - 0.44\,(\log 40)$

$\hat{y} = 3.41$

$K_y = 2.486$ AND $S_y = 0.232$ AS BEFORE

$\therefore Y_{100} = \hat{y} + K_y S_y = 3.41 + 2.486 \cdot 0.232$

$Y_{100} = 3.99$

$Q_{100} = 10^{Y_{100}} = 10^{3.99} = 9800$ CFS

PLUM CREEK
$\therefore$ 100-YEAR FLOOD FLOW AT PROJECT SITE IS BETWEEN 9800 CFS AND 12,600 CFS

SHEETS 19 TO 23 FROM "FIELD MANUAL OF PROCEDURE PSU-IV, ESTIMATING DESIGN FLOODS PEAKS ON UNGAGED PENN WATERSHEDS" PENN STATE, APRIL 1981

SHEET 19/23

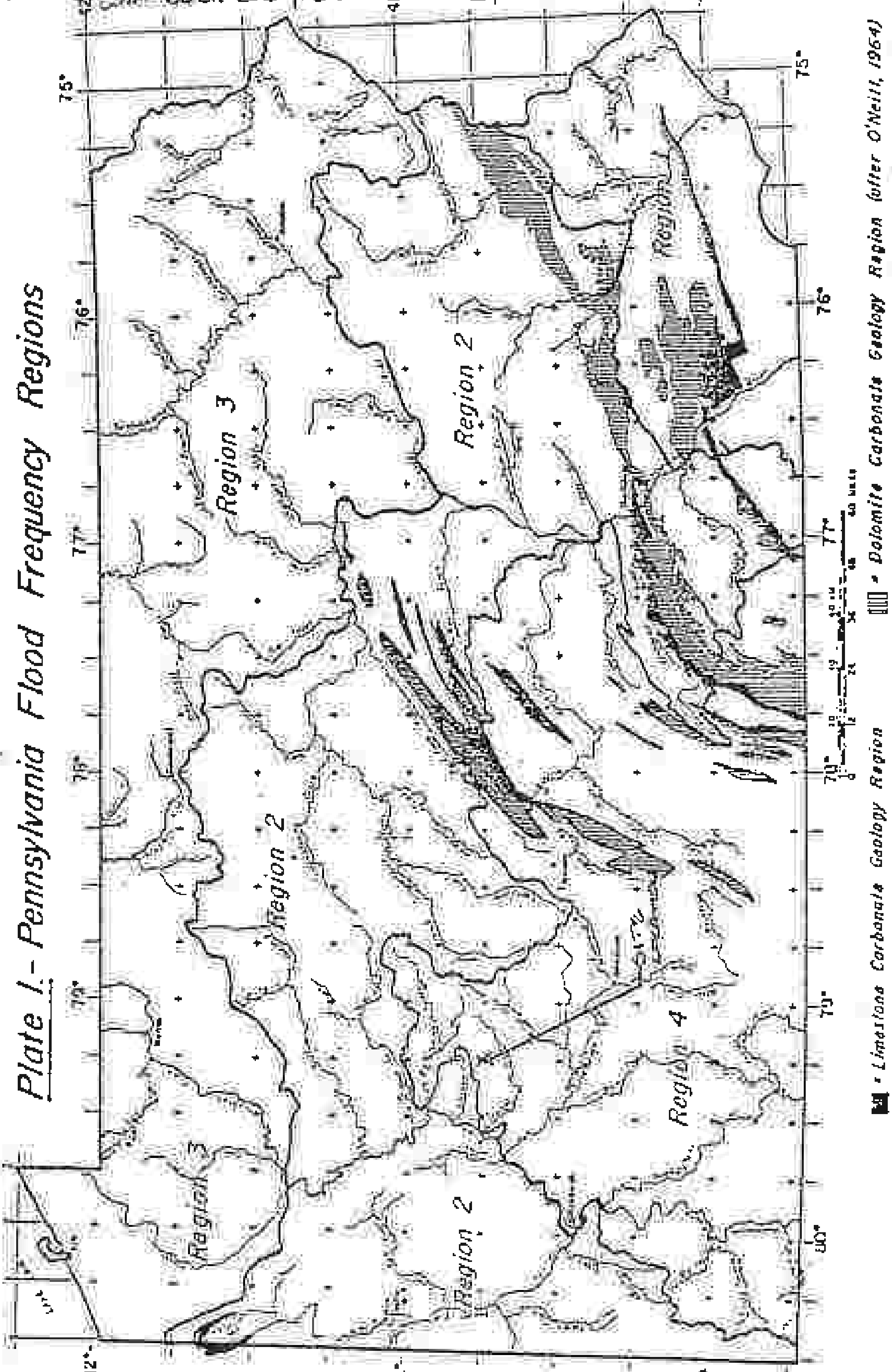


Table 1.1 Prediction Equations for $\hat{y}$ = Mean log Q

Region	Equation	Ranges of Applicability
1	$\hat{y} = 2.55 + 0.71 \log A - 0.00039\ DEL$	$1.5\ mi^2 \leq A \leq 250\ mi^2$ $0\ ft. \leq DEL \leq 1000\ ft$
2	$\hat{y} = 1.90 + 0.81 \log A - 0.0021\ FOR$	$1.5\ mi^2 \leq A \leq 250\ mi^2$ $0\% \leq FOR \leq 100\%$
3	$\hat{y} = 2.04 + 0.83 \log A - 0.0025\ FOR$	$1.5\ mi^2 \leq A \leq 250\ mi^2$ $0\% \leq FOR \leq 100\%$
4	$\hat{y} = 2.60 + 0.85 \log A - 0.44 \log FOR$	$1.5\ mi^2 \leq A \leq 250\ mi$ $10\% \leq FOR \leq 100\%$

Definitions:

A = Drainage area, in mi^2, measured from any convenient map. For applications to areas less than 1.5 mi^2 see Section 5.

DEL = Divide elevation, in feet, determined from a topographic map. If DEL $\geq$ 1000 feet, use DEL = 1000 feet.

FOR = Percentage of drainage area covered by forests, measured as green area on a 7½ minute USGS topographic map. If FOR $\leq$ 10% in Region 4, use FOR = 10.

Example 1:

The 25-year flood peak is to be found for a 20 square mile drainage area located at coordinates 40° 31' and 76° 00'. The percent forest cover has been determined from USGS topographic maps or aerial photos as 60 percent.

The drainage area is found on Plate 1 to be located in Region 2. Following the arrows in Figure 1.1, a value of $\hat{y}$ = 2.83 is found, which corresponds to a $Q_{2.33}$ of 676 cfs. From Plates 2 and 3, the standard deviation S_y = 0.28 and the skew coefficient G = 0.39 are obtained. With these values, Figure 1.2 is entered at T_r = 25 years. The value $\hat{y}$ is expressed as 2 + 0.83, and a flood peak, $Q_{25} = 22.5 \times 10^2 = 2250$ is obtained.

Using equation 1.1 in conjunction with Tables 1.1 and 1.2 instead of the graphical solutions the following results would be obtained.

$$\hat{y} = 1.90 + 0.81 \log 20 - 0.0021 \times 60 = 2.83$$

Entering Table 1.2 with T_r = 25 years and G = 0.39, the coefficient, K_y = 1.377 is obtained, and

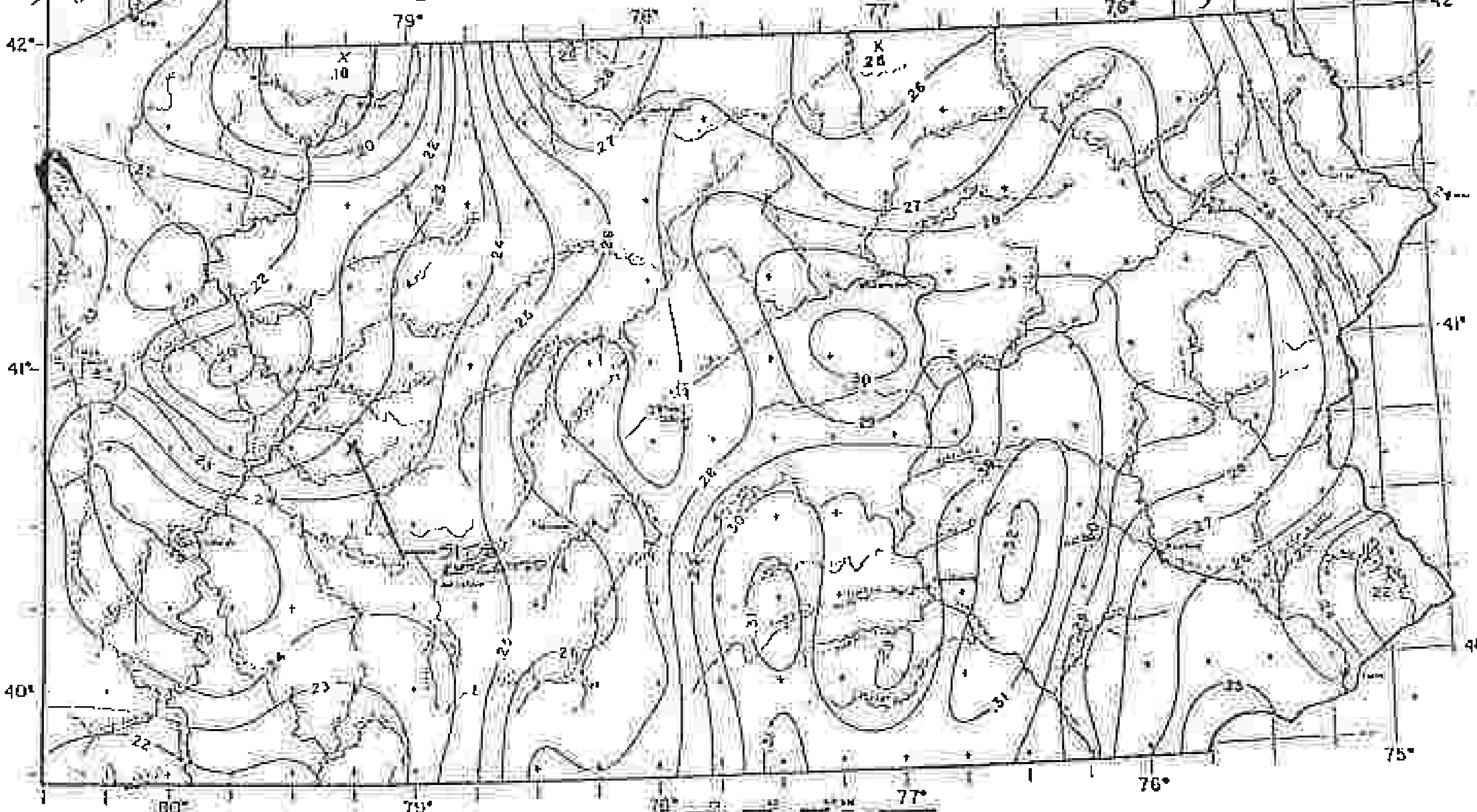
$$Y_{25} = 2.83 + 1.377 \times 0.28 = 3.35$$

which is the logarithm of Q = 2256 cfs

Table 1.2. K_y Values for Log Pearson Type III Distribution in Eq. 1.1.

Skew Coeff., G	1.1111	1.2500	2	5	10	25	50	100	200
	Recurrence Interval in Years								
	Percent Probability of Exceedence								
	90	80	50	20	10	4	2	1	0.5
	Positive Skew								
1.0	-1.128	-0.852	-0.164	0.758	1.340	2.043	2.542	3.022	3.489
.9	-1.147	-0.854	-0.148	0.769	1.339	2.018	2.498	2.957	3.401
.8	-1.166	-0.856	-0.132	0.780	1.336	1.993	2.453	2.891	3.312
.7	-1.183	-0.857	-0.116	0.790	1.333	1.967	2.407	2.824	3.223
.6	-1.200	-0.857	-0.099	0.800	1.328	1.939	2.359	2.755	3.132
.5	-1.216	-0.856	-0.083	0.808	1.323	1.910	2.311	2.686	3.041
.4	-1.231	-0.855	-0.066	0.816	1.317	1.880	2.261	2.615	2.949
.3	-1.245	-0.853	-0.050	0.824	1.309	1.849	2.211	2.544	2.856
.2	-1.258	-0.850	-0.033	0.830	1.301	1.818	2.159	2.472	2.763
.1	-1.270	-0.846	-0.017	0.836	1.202	1.785	2.107	2.400	2.670
.0	-1.282	-0.842	0	0.842	1.282	1.751	2.054	2.325	2.576
	Negative Skew								
-.1	-1.292	-0.836	0.017	0.846	1.270	1.716	2.000	2.252	2.482
-.2	-1.301	-0.830	0.033	0.850	1.258	1.680	1.945	2.178	2.388
-.3	-1.309	-0.824	0.050	0.853	1.245	1.643	1.890	2.104	2.294
-.4	-1.317	-0.816	0.066	0.855	1.231	1.606	1.834	2.029	2.201
-.5	-1.323	-0.808	0.083	1.216	1.216	1.567	1.777	1.955	2.108
-.6	-1.328	-0.800	0.099	0.857	1.200	1.528	1.720	1.880	2.016
-.7	-1.333	-0.790	0.116	0.857	1.183	1.488	1.663	1.806	1.926
-.8	-1.336	-0.780	0.132	0.856	1.166	1.488	1.606	1.733	1.837
-.9	-1.339	-0.769	0.148	0.854	1.147	1.407	1.549	1.660	1.749
-.10	-1.340	-0.758	0.164	0.852	1.128	1.366	1.492	1.588	1.664

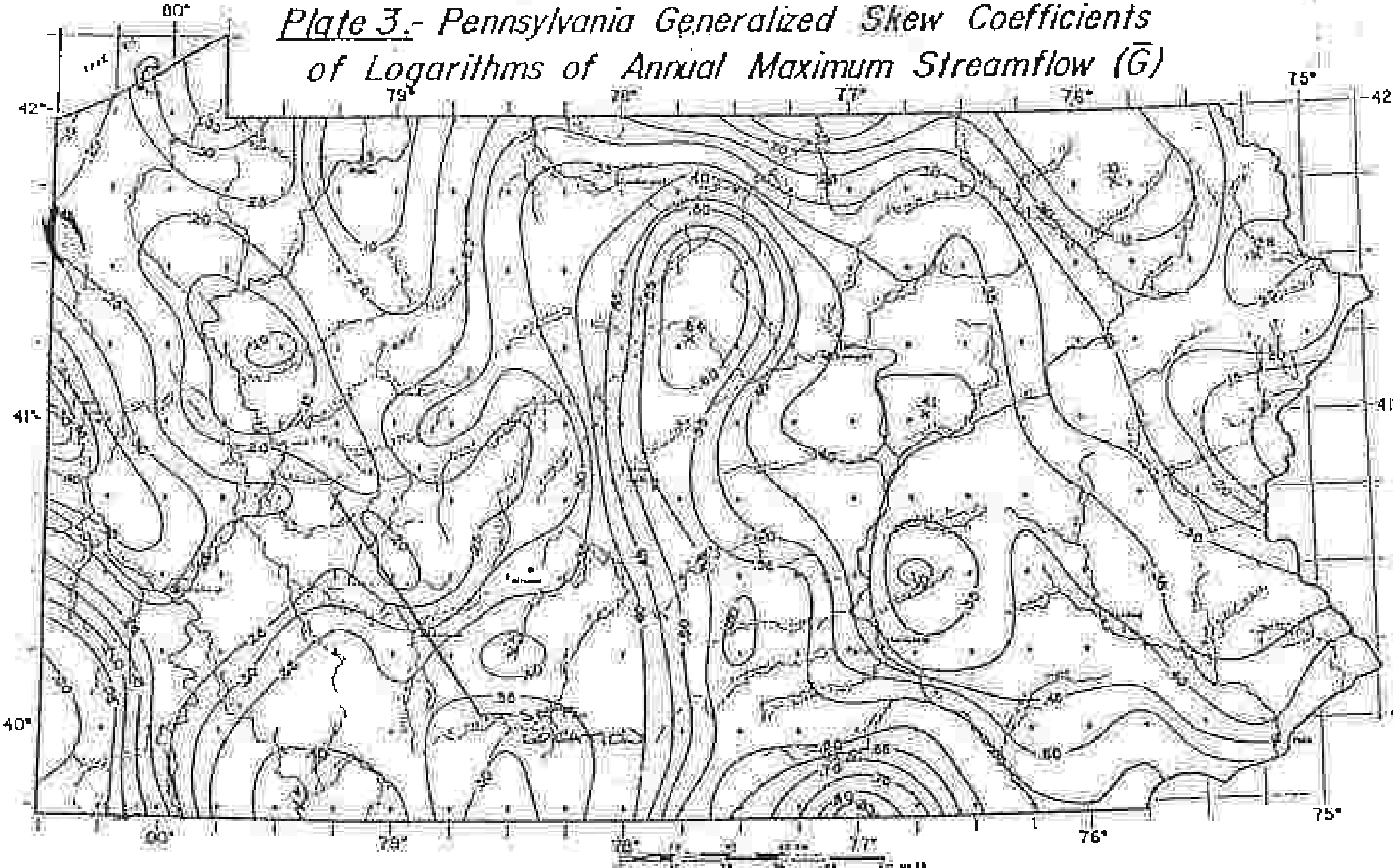
Plate 2.- Pennsylvania Generalized Standard Deviations of Logarithms of Annual Maximum Streamflow ($\bar{S}_y$)



Based on annual series records through water year 1977 from 139 Pennsylvania stream gages and 148 gages from adjoining states

SHEET 23/23

Plate 3.- Pennsylvania Generalized Skew Coefficients of Logarithms of Annual Maximum Streamflow ($\bar{G}$)



Based on annual series records through water year 1977 from 139 Pennsylvania stream gages and 148 gages from adjoining states.

APPENDIX I-1-C

FORM I

EXISTING DRAINAGE FACILITIES -
CALCULATIONS FROM PREVIOUS SOLID WASTE PERMIT APPLICATION

SUBJECT

BY DATE PROJ. NO.

CHKD. BY DATE SHEET NO. OF

GAI CONSULTANTS, INC.
Engineers • Geologists • Planners
Environmental Specialists

EXISTING DRAINAGE FACILITIES

CALCULATIONS FROM PREVIOUS SOLID WASTE PERMIT APPLICATION

DESCRIPTION	NO. OF SHEETS
DRAINAGE DESIGN COMPUTATIONS FOR KEYSTONE STATION EAST VALLEY ASH DISPOSAL SITE EAST PERIPHERAL DRAINAGE DITCH, REPORT BY H.F. LENZ CO JUNE 1985	68
STAGE I HYDROLOGY	10
HYDROLOGIC PARAMETERS FOR CHANNEL DESIGN	32
TR-20 INPUT FORMS	9
STAGE I HYDRAULICS SHEET FLOW OFF OF ACTIVE SURFACE	4
STAGE I HYDRAULICS	22
CLOSURE HYDRAULICS	27
EMERGENCY SPILLWAY FOR EQUALIZATION POND AND TYPE Q CHANNEL	4
CLOSURE PLAN HYDRAULICS	14
SLOPE DRAIN ON EAST SIDE - STAGE I	5
REVISED CLOSURE HYDRAULIS CHANNEL S	3

WORKSHEETS

STAGE I HYDROLOGY WORKSHEET
CLOSURE HYDROLOGY WORKSHEET
CLOSURE HYDRAULICS WORKSHEET

PENNSYLVANIA ELECTRIC COMPANY

JOHNSTOWN, PENNSYLVANIA

DRAINAGE DESIGN COMPUTATIONS

FOR

KEYSTONE STATION

EAST VALLEY ASH DISPOSAL SITE

EAST PERIPHERAL DRAINAGE DITCH

JUNE 1985

H.F. Lenz Company

CONSULTING ENGINEERS • JOHNSTOWN, PA.

DRAINAGE DESIGN COMPUTATIONS
FOR
KEYSTONE STATION
EAST VALLEY ASH DISPOSAL SITE
EAST PERIPHERAL DRAINAGE DITCH

JUNE 1985

Penelec Work Order No. K465

COMMONWEALTH OF PENNSYLVANIA
REGISTERED PROFESSIONAL
JAMES C. KOHLER
ENGINEER
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Prepared By

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H.F. LENZ CO. FILE No. 84-610

TABLE OF CONTENTS

DRAINAGE DESIGN COMPUTATIONS

KEYSTONE STATION

EAST VALLEY ASH DISPOSAL SITE

EAST PERIPHERAL DRAINAGE DITCH

I. Intent

The purpose of these design computations are to formulate the necessary physical and hydraulic parameters of a storm drainage system which will handle runoff from a new Ash Disposal Site at the Keystone Station in Armstrong County, Pennsylvania. The drainage facility will ultimately service the East/West Valley Ash Sites as shown on Figure 1, "Location Map."

II. Methodology

The following sections of computations which establish the drainage system were generated by utilizing two design methods.

The hydrologic analysis was performed using the methodology contained in Technical Release No. 55 "Urban Hydrology for Small Watersheds" USDA-SCS (October 1981).

The overland runoff quantities were computed for a design storm of 24 hour duration with a 100 year recurrence interval. The volumes and rates of runoff were functions of the watershed characteristics which included hydrologic soil-cover complexes (SCS runoff curve number), time of concentration, travel time and drainage area.

The hydraulic design of capacity for the different drainage ditch configurations and the corresponding culvert was completed by use of the Equation of Continuity (Q = AV). The equation is defined as follows:

Q = Discharge of water in cubic feet per second.

A = Net effective area in square feet provided by the drainage facility.

V = Velocity in feet per second. Velocities were calculated using Manning's Equation:

$$V = \frac{1.486}{n} R^{2/3} S^{1/2}$$

Where:

V = Velocity in feet per second

R = Hydraulic radius which is equal to the net effective area (A) divided by the wetted perimeter (W.P.) : R = A/WP. The wetted perimeter is the lineal feet of the drainage facility cross section which is wetted by the water.

S = Slope of drainage facility

n = Roughness coefficient.

In the case of the culvert, hydraulic charts were used to determine the headwater depths for both inlet control and outlet control using the higher value to indicate the type of control and necessary headwater depth. Outlet channel protection was also determined based on the outlet velocity and the use of hydraulic charts.

The appendix of these computations contains tables and figures of which specific values for different steps in this design were obtained. Other references utilized for design are listed in the Reference Section.

East Peripheral Drainage Ditch Design:

The East Peripheral Drainage Ditch will carry runoff from the top of the East/West Valley Ash Sites, portions of the ash pile benches and areas within the immediate vicinity of the ditch, ultimately discharging into Plum Creek.

These runoff areas as shown on the "Drainage Plan", Drawing No. 41-F-0272 were broken down into smaller Drainage Areas (Nos. 1 thru 11) for the purpose of the system design.

Drainage Areas No. 2 and 6 consist partially of runoff from the top of the East/West Valley Ash Pile and the slope drain handling the east face bench flows, respectively. The hydrology for these sections was completed by GAI Consultants as part of their Pile Development package. The information dealing with certain runoff parameters of these areas was supplied to H. F. Lenz Co. for use in this design - refer GAI letter in Appendix.

The calculations for the remaining portions of the drainage facility were generated based on the following design criteria:

1. Use USDA-SCS method as outlined in Technical Release No. 55, "Urban Hydrology for Small Watersheds".

2. Design for 100 year, 24 hour Rainfall Event.

3. Use Type II Rainfall Distribution.

4. Antecedent Soil Moisture Condition II.

5. Rainfall Depth:

Taken from "Rainfall Duration Frequency Tables for Pennsylvania": Mean Annual Rainfall (taken from Mean Annual Rainfall Map of Pennsylvania) = 2.70 in. for 24 hour Duration and 2.33 year Period. Therefore, from Table 6 for 24 hour Duration, Region II, 100 year Period, Rainfall Depth = 5.51 in.

6. Curve Number - CN

Soils: From the SCS "Soil Survey for Armstrong County" the soils within the watershed areas are a combination of Rayne, Cavode, Weikert, Gilpin, Wharton and Ernest soils. From TR-55, a majority of these soils belong to Hydrologic Soils Group "C", Refer "Soils Map" - Figure 2, Soil Descriptions and TR-55 - Table B.I.

The watershed land uses consist of a pasture-woods combination varying from a good to poor condition. Therefore, taken from Table 2-2, for Hydrologic Soils Group "C", a weighted CN of 78 will be used in design.

7. Runoff Depth:

Interpolated from Table 2-1 : TR-55
Runoff Depth @ CN = 78 for 5.51 in.
Rainfall = 3.20 in.

8. Maximum Expected Overland Discharge $= q = q_p$ (DA)(Q) - Taken from TR-55, where:

q = Discharge in cubic feet per second
q_p = Tabular discharge for Type II Storm Distribution (csm/in) taken from TR-55 - Table 5-3
DA = Drainage Area in square miles
Q = Runoff in inches.

<u>DRAINAGE AREA NO. 1</u>

<u>Hydrologic Analysis:</u>

Maximum Expected Overland Discharge $= q = q_p$ (DA)(Q)

q_p: Time of Concentration $= T_c$

Overland Slope $= \dfrac{1299 - 1297.50}{60 \text{ ft.}} (100\%) = 2.50\%$

Velocity - Fig. 3-1 - Forest = 0.40 fps

T_c = 60 ft./0.40 fps/3600 sec/hr = 0.40 hrs.

Say T_c = 0.10 hrs.

q_p for T_c = 0.10 hrs. and T_t = 0 = 991 csm/in. @ 11.8 hrs.

DA: Drainage Area = Area No. 1 - Refer Drainage Plan
DA = 0.003 mi^2

Q = Runoff interpolated from Table 2-1 = 3.20 in.

Maximum Expected Overland Discharge = q = q_p(DA)(Q)
q = (991 csm/in.)(0.003 mi^2)(3.20 in.) = 9.51 cfs
Use q = 10.00 cfs

Hydraulic Analysis of Proposed Ditch:

Ditch Parameters:

Configuration = V - Ditch with side slopes of 2H : 1V
Depth = 3.0 ft. minimum; Design with 0.5 ft. Freeboard
Lining = Grass
Slope = 1.50% ±

Capacity of Proposed Ditch = Q_p = AV

$$A = 2(2.5 \text{ ft.})^2 = 12.50 \text{ sf}$$
$$V = \frac{1.486}{n} R^{2/3} S^{1/2}$$

n = 0.05 (Refer Table 2.10.13.1)
R = A/WP
A = 12.50 sf
WP = 11.18 ft.
R = 12.50 sf/11.18 ft. = 1.12 ft.
S = 1.50% = 0.015 ft./ft.

$$V = \frac{1.486}{0.05} (1.12)^{2/3}(0.015)^{1/2} = 3.85 \text{ fps}$$

Capacity of Proposed Ditch = Q_p = AV
Q_p = (12.50 sf)(3.85 fps) = 48.12 cfs
Q_p = 48.12 cfs > q = 10.00 cfs
Proposed Ditch is adequate
Actual Velocity = V_a = 2.60 fps @ d = 1.40 ft.

DRAINAGE AREA NO. 2

Hydrologic Analysis:

Maximum Expected Overland Discharge = q = q_p(DA)(Q)

q_p: Time of Concentration = T_c

$$\text{Overland Slope} = \frac{1294-1290}{130 \text{ ft.}} (100\%) = 3.00\%$$

Velocity - Fig. 3-1 - Forest = 0.44 fps
T_c = 130 ft./0.44 fps/3600 sec/hr = 0.082 hrs
Within the limits of this drainage area, the flow from the top of the ash pile will enter the drainage ditch. In accordance with the information supplied by GAI Consultants, the T_c from the top of the ash pile to the point of entry into the ditch is 1.67 hrs. Therefore, use T_c = 1.67 hrs. for establishing the Maximum Expected Overland Discharge to the ditch.

Use q_p for T_c = 1.5 hrs. and T_t = 0 = 236 csm/in. @ 12.8 hrs.

DA: Drainage Area = Areas No. 1, 2 and Top of Pile for Ultimate East/West Development as supplied by GAI Consultants - Refer Drainage Plan and GAI letter in Appendix.

DA = 0.169 mi^2

Q = Runoff interpretated from Table 2-1 = 3.20 in.

Maximum Expected Overland Discharge = q = q_p(DA)(Q)
q = (236 csm/in.)(0.169 mi^2)(3.20 in.) = 127.62 cfs
Use q = 130.00 cfs

<u>Hydraulic Analysis of Proposed Ditch:</u>

Ditch Parameters:

Configuration = V-Ditch with side slopes of 2H : 1V
Depth = 4.0 ft. minimum; Design with 0.5 ft. Freeboard
Lining = Grouted Rock
Slope = 1.50%±

Capacity of Proposed Ditch = Q_p = AV

$$A = 2(3.5\text{ ft.})^2 = 24.50\text{ sf}$$
$$V = \frac{1.486}{n} R^{2/3} S^{1/2}$$

n = 0.035 (Refer Table 2.10.13.1)
R = A/WP
A = 24.50 sf
WP = 15.65 ft.

R = 24.50 sf/15.65 ft. = 1.57 ft.
S = 1.50% = 0.015 ft./ft.
$$V = \frac{1.486}{0.035}(1.57)^{2/3}(0.015)^{1/2} = 8.00\text{ fps}$$

Capacity of Proposed Ditch = Q_p = AV
Q_p = (24.50 sf)(8.00 fps) = 196.00 cfs
Q_p = 196.00 cfs > q = 130.00 cfs
Proposed Ditch is adequate
Actual Velocity = V_a = 6.48 fps @ d = 3.20 ft.

DRAINAGE AREA NO. 3

Hydrologic Analysis:

Maximum Expected Overland Discharge = $q = q_p(DA)(Q)$

q_p: Time of Concentration = T_c
Overland Slope = $\frac{1291-1282}{200 \text{ ft.}}$ (100%) = 4.50%
Velocity - Fig. 3-1 - Forest = 0.54 fps
T_c = 200 ft./0.54 fps/3600 sec/hr = 0.103 hrs.
Check Time of Travel from upstream ditch: T_t
$T_t = T_c + D/V$ = 1.67 hrs. + (500 ft./6.48 fps/3600 sec/hr.) = 1.69 hrs.
T_c = 0.103 hrs. $<$ T_t = 1.69 hrs. Use T_c = 1.69 hrs., say T_c = 1.5 hrs.
q_p for T_c = 1.5 hrs. and T_t = 0 = 236 csm/in. @ 12.8 hrs.

DA: Drainage Area = Areas No. 1 thru 3 plus Top of East/West Pile - Refer Drainage Plan
DA = 0.18 mi^2

Q = Runoff interpolated from Table 2-1 = 3.20 in.

Maximum Expected Overland Discharge = $q = q_p(DA)(Q)$
q = (236 csm/in.)(0.18 mi^2)(3.20 in.) = 135.93 cfs
Use q = 136.00 cfs

Hydraulic Analysis of Proposed Ditch:

Ditch Parameters:

Configuration = V-Ditch with side slopes of 2H : 1V
Depth = 4.0 ft. minimum; Design with 0.5 ft. Freeboard
Lining = Grouted Rock
Slope = 6.00%±

Capacity of Proposed Ditch = $Q_p = AV$

$A = 2(3.5 \text{ ft.})^2 = 24.50$ ft.
$V = \frac{1.486}{n} R^{2/3} S^{1/2}$
n = 0.035 (Refer Table 2.10.13.1)
R = A/WP
A = 24.50 sf
WP = 15.65 ft.
R = 24.50 sf/15.65 ft. = 1.57 ft.
S = 6.00% = 0.060 ft./ft.

$$V = \frac{1.486}{0.035}(1.57)^{2/3}(0.060)^{1/2} = 13.79 \text{ fps}$$

Capacity of Proposed Ditch = Q_p = AV
Q_p = (24.50 sf)(13.79 fps) = 337.86 cfs
Q_p = 337.86 cfs $>$ q = 136.00 cfs
Proposed Ditch is adequate
Actual Velocity = V_a = 10.98 fps @ d = 2.50 ft.

DRAINAGE AREA NO. 4

Hydrologic Analysis:

Maximum Expected Overland Discharge = q = q_p(DA)(Q)

q_p: Time of Concentration = T_c
Overland Slope = $\frac{1270 - 1260}{50 \text{ ft.}}$ (100%) = 20.0%
Velocity - Fig. 3-1 - Nearly Bare Ground = 3.20 fps
T_c = 50 ft./3.20 fps/3600 sec/hr. = 0.0043 hrs.
Check Time of Travel from upstream ditch: T_t
T_t = T_c + D/V = 1.69 hrs. + (400 ft./10.98 fps/3600 sec/hr.) = 1.70 hrs.
T_c = 0.0043 hrs. $<$ T_t = 1.70 hrs. Use T_c = 1.70 hrs., say T_c = 1.50 hrs.
q_p for T_c = 1.5 hrs and T_t = 0 = 236 csm/in. @ 12.8 hrs.

DA: Drainage Area = Areas No. 1 thru 4 plus top of East/West Pile - Refer Drainage Plan
DA = 0.183 mi^2

Q = Runoff interpolated from Table 2-1 = 3.20 in.

Maximum Expected Overland Discharge = q = q_p(DA)(Q)
q = (236 csm/in.)(0.183 mi^2)(3.20 in.) = 138.20 cfs
Use q = 140.00 cfs

Hydraulic Analysis of Proposed Ditch:

Ditch Parameters:

Configuration = V-Ditch with side slopes of 2H : 1V
Depth = 4.0 ft. minimum; Design with 0.5 ft. Freeboard
Lining = Grouted Rock
Slope = 7.00% ±

Capacity of Proposed Ditch = Q_p = AV

$A = 2(3.5\ ft.)^2 = 24.50\ sf$

$V = \frac{1.486}{n} R^{2/3} S^{1/2}$

n = 0.035 (Refer Table 2.10.13.1)

R = A/WP

A = 24.50 sf

WP = 15.65 ft.

R = 24.50 sf/15.65 ft. = 1.57 ft.

S = 7.00% = 0.070 ft./ft.

$V = \frac{1.486}{0.035} (1.57)^{2/3}(0.070)^{1/2} = 14.94\ fps$

Capacity of Proposed Ditch = Q_p = AV

Q_p = (24.50 sf)(14.94 fps) = 366.03 cfs

Q_p = 336.03 cfs $>$ q = 140.00 cfs

Proposed Ditch is adequate

Actual Velocity = V_a = 11.58 fps @ d = 2.40 ft.

DRAINAGE AREA NO. 5

Hydrologic Analysis:

Maximum Expected Overland Discharge = q = q_p(DA)(Q)

q_p: Time of Concentration = T_c

Overland Slope = 0, No direct flows from adjacent watersheds according to overland contours. All flows will be from upstream watersheds discharging into upstream ditching.

Time of Travel from upstream ditch: T_t

T_t = T_c + D/V = 1.70 hrs. + (400 ft./11.58 fps/3600 sec/hr) = 1.71 hrs., say T_c = 1.50 hrs.

q_p for T_c = 1.5 hrs. and T_t = 0 = 236 csm/in. @ 12.8 hrs.

DA: Drainage Area = Areas No. 1 thru 5 plus Top of East/West Pile - Refer Drainage Plan

DA = 0.188 mi^2

Q = Runoff interpolated from Table 2-1 = 3.20 in.

Maximum Expected Overland Discharge = q = q_p(DA)(Q)

q = (236 csm/in.)(0.188 mi^2)(3.20 in.) = 141.97 cfs

Use q = 142.00 cfs

Hydraulic Analysis of Proposed Ditch:

Ditch Parameters:

Configuration = Rectangular Channel
Depth = 4.0 ft. minimum; Design with 0.5 ft. Freeboard
Width = 5.0 ft. minimum
Lining = Reinforced Concrete
Slope = 0.5% ±

Capacity of Proposed Ditch = Q_p = AV

$A = 5.0 \text{ ft.}(3.5 \text{ ft.}) = 17.50 \text{ sf}$

$V = \frac{1.486}{n} R^{2/3} S^{1/2}$

n = 0.014 (Refer Table 2.10.13.1)

R = A/WP

A = 17.50 sf

WP = 12.00 ft.

R = 17.50 sf/12.00 ft. = 1.46 ft.

S = 0.50% = 0.005 ft./ft.

$V = \frac{1.486}{0.014} (1.46)^{2/3}(0.005)^{1/2} = 9.57 \text{ fps}$

Capacity of Proposed Ditch = Q_p = AV
Q_p = (17.50 sf)(9.57 fps) = 167.48 cfs
Q_p = 167.48 cfs $\gg$ q = 142.00 cfs
Proposed Ditch is adequate
Actual Velocity = V_a = 9.21 fps @ d = 3.10 ft.

DRAINAGE AREA NO. 6

Hydrologic Analysis:

Maximum Expected Overland Discharge = q = q_p(DA)(Q)

q_p: Time of Concentration = T_c

Overload Slope = $\frac{1260 - 1212}{330 \text{ ft.}}$ (100%) = 14.50%

Velocity - Fig. 3-1 - Forest = 0.97 fps

T_c = 330 ft./0.97 fps/3600 sec/hr = 0.09 hrs.

Check Time of Travel from upstream ditch: T_t

T_t = T_c + D/V = 1.71 hrs. + (1250 ft./9.21 fps/3600 sec/hr) = 1.75 hrs.

Check Time of Travel from Slope Drain: T_s

T_s = 0.89 hrs. as supplied by GAI Consultants based on their design of the Drain

T_c = 0.09 hrs. $\ll$ T_t = 1.75 hrs. $\gg$ T_s = 0.89 hrs.

Use T_c = 1.75 hrs., say T_c = 1.50 hrs.

q_p for T_c = 1.5 hrs. and T_t = 0 = 236 csm/in. @ 12.8 hrs.

DA: Drainage Area = Areas No. 1 thru 6 plus Top of East/West Pile and Slope Drain Area - Refer Drainage Plan and GAI letter in Appendix.

DA = 0.225 mi^2

Q = Runoff interpolated from Table 2-1 = 3.20 in.

Maximum Expected Overload Discharge = q = q_p(DA)(Q)

q = (236 csm/in.)(0.225 mi^2)(3.20 in.) = 169.92 cfs

Use q = 170.00 cfs

Hydraulic Analysis of Proposed Ditch:

Ditch Parameters:

Configuration = Rectangular Channel
Depth = 4.0 ft. minimum; Design with 0.5 ft. Freeboard (approx.)
Width = 5.0 ft. minimum
Lining = Reinforced Concrete
Slope = 0.5% $\pm$

Capacity of Proposed Ditch = Q_p = AV

A = 5.0 ft. (3.55 ft.) = 17.75 sf

$$V = \frac{1.486}{n} R^{2/3} S^{1/2}$$

n = 0.014 (Refer TAble 2.10.13.1)

R = A/WP

A = 17.75 sf

WP = 12.10 ft.

R = 17.75 sf/12.10 ft. = 1.47 ft.

S = 0.50% = 0.005 ft./ft.

$$V = \frac{1.486}{0.014} (1.95)^{2/3}(0.005)^{1/2} = 9.62 \text{ fps}$$

Capacity of Proposed Ditch = Q_p = AV

Q_p = (17.75 sf)(9.62 fps) = 170.76 cfs

Q_p = 170.76 cfs $>$ q = 170.00 cfs

Proposed Ditch is adequate

Actual Velocity = V_a = 9.62 fps (approx.) @ d = 3.55 ft. (approx.)

DRAINAGE AREA NO. 7

Hydrologic Analysis:

Maximum Expected Overland Discharge = $q = q_p(DA)(Q)$

q_p: Time of Concentration = T_c

Overland Slope = $\frac{1260 - 1200}{570 \text{ ft.}}$ (100%) = 10.53%

Velocity - Fig. 3-1 - Forest = 0.80 fps

T_c = 570 ft./0.80 fps/3600 sec/hr. = 0.20 hrs.

Check Time of Travel from upstream ditch: T_t

$T_t = T_c + D/V$ = 1.75 hrs. + (650/9.62 fps/3600 sec/hr.) = 1.77 hrs.

T_c = 0.20 hrs. < T_t = 1.77 hrs. Use T_c = 1.77 hrs, say T_c = 1.50 hrs.

q_p for T_c = 1.5 hrs. and T_t = 0 = 236 csm/in. @ 12.8 hrs.

DA: Drainage Area = Areas No. 1 thru 7 plus Top of East/West Pile and Slope Drain Area - Refer Drainage Plan

DA = 0.226 mi^2

Q = Runoff interpolated from Table 2-1 = 3.20 in.

Maximum Expected Overland Discharge = $q = q_p(DA)(Q)$

q = (236 csm/in.)(0.226 mi^2)(3.20 in.) = 170.67 cfs

Use q = 171.00 cfs

Hydraulic Analysis of Proposed Ditch:

Ditch Parameters:

Configuration = Trapezoidal with side slopes of 2H : 1V

Depth = 4.0 ft. minimum; Design with 0.5 ft. Freeboard

Bottom Width = 2.0 ft. minimum

Lining = Grouted Rock

Slope = 9.00% ±

Capacity of Proposed Ditch = Q_p = AV

A = [2.0 ft. + 2(3.5 ft)](3.5 ft.) = 31.50 sf

$V = \frac{1.486}{n} R^{2/3} S^{1/2}$

n = 0.035 (Refer Table 2.10.13.1)

R = A/WP

A = 31.50 sf

WP = 17.65 ft.

R = 31.50 sf/17.65 ft. = 1.78 ft.

S = 9.00% = 0.090 ft./ft.

$$V = \frac{1.486}{0.035}(1.78)^{2/3}(0.090)^{1/2} = 18.72 \text{ fps}$$

Capacity of Proposed Ditch = Q_p = AV
Q_p = (31.50 sf)(18.72 fps) = 589.68 cfs
Q_p = 589.68 cfs > q = 171.00 cfs
Proposed Ditch is adequate
Actual Velocity = V_a = 13.74 fps @ d = 2.05 ft.

DRAINAGE AREA NO. 8

Hydrologic Analysis:

Maximum Expected Overland Discharge = q = q_p(DA)(Q)

q_p: Time of Concentration = T_c
Overland Slope = $\frac{1226 - 1190}{320 \text{ ft.}}$ (100%) = 11.25%
Velocity - Fig. 3-1 - Forest = 0.83 fps
T_c = 320 ft./0.83 fps/3600 sec/hr. = 0.11 hrs.
Check Time of Travel from upstream ditch: T_t
T_t = T_c + D/V = 1.77 hrs. + (100 ft./13.74 fps/3600 sec/hr.) = 1.78 hrs.
T_c = 0.11 hrs. < T_t = 1.78 hrs. Use T_c = 1.78 hrs., say T_c = 1.50 hrs.

q_p for T_c = 1.5 hrs. and T_t = 0 = 236 csm/in. @ 12.8 hrs.

DA: Drainage Area = Areas No. 1 thru 8 plus Top of East/West Pile and Slope Drain Area - Refer Drainage Plan
DA = 0.228 mi^2

Q = Runoff interpolated from Table 2-1 = 3.20 in.

Maximum Expected Overland Discharge = q = q_p(DA)(Q)
q = (236 csm/in.)(0.228 mi^2)(3.20 in.) = 172.18 cfs
Use q = 173.00 cfs

Hydraulic Analysis of Proposed Ditch:

Ditch Parameters:

Configuration = Trapezoidal with side slopes of 2H : 1V
Depth = 4.0 ft. minimum; Design with 0.5 ft. Freeboard
Bottom Width = 2.0 ft. minimum
Lining = Grouted Rock
Slope = 15.00% ±

Capacity of Proposed Ditch = $Q_p = AV$

$A = [2.0 \text{ ft.} + 2(3.5 \text{ ft.})](3.5 \text{ ft.}) = 31.50 \text{ sf}$

$V = \frac{1.486}{n} R^{2/3} S^{1/2}$

$n = 0.035$ (Refer Table 2.10.13.1)

$R = A/WP$

$A = 31.50$ sf

$WP = 17.65$ ft.

$R = 31.50 \text{ sf}/17.65 \text{ ft.} = 1.78 \text{ ft.}$

$S = 15.00\% = 0.150 \text{ ft./ft.}$

$V = \frac{1.486}{0.035} (1.78)^{2/3}(0.150)^{1/2} = 24.37 \text{ fps}$

Capacity of Proposed Ditch = $Q_p = AV$

$Q_p = (31.50 \text{ sf})(24.37 \text{ fps}) = 767.66 \text{ cfs}$

$Q_p = 767.66 \text{ cfs} \succ q = 173.00 \text{ cfs}$

Proposed Ditch is adequate

Actual Velocity = $V_a = 16.75$ fps @ d = 1.83 ft.

DRAINAGE AREA NO. 9

Hydrologic Analysis:

Maximum Expected Overland Discharge = $q = q_p(DA)(Q)$

q_p = Time of Concentration = T_c

Overland Slope = 0, No direct flows from adjacent watersheds according to overland contours. All flows will be from upstream watersheds discharging into upstream ditching.

Time of Travel from upstream ditch: T_t

$T_t = T_c + D/V = 1.78 \text{ hrs.} + (700 \text{ ft.}/16.75 \text{ fps}/3600 \text{ sec/hr.}) = 1.79$ hrs., say $T_c = 1.50$ hrs.

q_p for $T_c = 1.50$ hrs, and T_t = 0 236 csm/in. @ 12.8 hrs.

DA: Drainage Area = Areas No. 1 thru 9 plus Top of East/West Pile and Slope Drain Area - Refer Drainage Plan

$DA = 0.228 \text{ mi}^2$

Q = Runoff interpolated from Table 2-1 = 3.20 in.

Maximum Expected Overland Discharge = $q = q_p(DA)(Q)$

$q = (236 \text{ csm/in.})(0.228 \text{ mi}^2)(3.20 \text{ in.}) = 172.18 \text{ cfs}$

Use q = 173.00 cfs

Hydraulic Analysis of Proposed Ditch:

Ditch Parameters:

Configuration = Trapezoidal with side slopes of 2H : 1V
Depth = 4.0 ft. minimum; Design with 0.5 ft. Freeboard
Bottom Width = 2.0 ft. minimum
Lining = Grouted Rock
Slope = 7.50% $\pm$

Capacity of Proposed Ditch = Q_p =AV

$A = [2.0 \text{ ft.} + 2(3.5 \text{ ft.})](3.5 \text{ ft.}) = 31.50 \text{ sf}$

$V = \frac{1.486}{n} R^{2/3} S^{1/2}$

$n = 0.035$ (Refer Table 2.10.13.1)

$R = A/WP$

$A = 31.50$ sf

$WP = 17.65$ ft.

$R = 31.50 \text{ sf}/17.65 \text{ ft.} = 1.78$ ft.

$S = 7.50\% = 0.075$ ft./ft.

$V = \frac{1.486}{0.035} (1.78)^{2/3}(0.075)^{1/2} = 16.90$ fps

Capacity of Proposed Ditch = Q_p = AV

Q_p = (31.50sf)(16.90 fps) = 532.35 cfs

Q_p = 532.35 cfs $>$ q = 173.00 cfs

Proposed Ditch is adequate

Actual Velocity = V_a = 12.71 fps @ d = 2.15 ft.

DRAINAGE AREA NO. 10

Hydrologic Analysis:

Maximum Expected Overland Discharge = q = q_p (DA)(Q)

q_p: Time of Concentration = T_c

Overland Slope = $\frac{1210 - 1047}{1020 \text{ ft.}}$ (100%) = 16.00%

Velocity - Fig. 3-1 - Forest = 1.00 fps

T_c = 1020 ft./1.0 fps/3600 sec/hr. = 0.28 hrs.

Check Time of Travel from upstream ditch: T_t

$T_t = T_c + D/V$ = 1.79 hrs. + (300 ft./12.71 fps/3600 sec/hr.) = 1.80 hrs.

T_c = 0.28 hrs. $<$ T_t = 1.80 hrs. Use T_c = 1.80 hrs., say T_c = 1.50 hrs.

q_p for T_c = 1.5 hrs. and T_t = 0 = 236 csm/in. @ 12.8 hrs.

DA: Drainage Area = Areas No. 1 thru 10 plus Top of East/West Pile and Slope Drain Area - Refer Drainage Plan

$DA = 0.233\ mi^2$

Q = Runoff interpolated from Table 2-1 = 3.20 in.

Maximum Expected Overland Discharge = $q = q_p(DA)(Q)$

$q = (236\ csm/in.)(0.233\ mi^2)(3.20\ in.) = 176.19\ cfs$

Use q = 180.00 cfs

Hydraulic Analysis of Proposed Ditch:

Ditch Parameters:

Configuration = Trapezoidal with side slopes of 2H : 1V
Depth = 4.0 ft. minimum; Design with 0.5 ft. Freeboard
Bottom Width = 2.0 ft. minimum
Lining = Grouted Rock
Slope = 26.0% ±

Capacity of Proposed Ditch = $Q_p = AV$

$A = [2.0\ ft. + 2(3.5\ ft.)](3.5\ ft.) = 31.50\ sf$

$$V = \frac{1.486}{n} R^{2/3} S^{1/2}$$

n = 0.035 (Refer Table 2.10.13.1)

R = A/WP

A = 31.50 sf

WP = 17.65 ft.

R = 31.50 sf/17.65 ft. = 1.78 ft.

S = 26.0% = 0.26 ft./ft.

$$V = \frac{1.486}{0.035}(1.78)^{2/3}(0.26)^{1/2} = 31.87\ fps$$

Capacity of Proposed Ditch = $Q_p = AV$

$Q_p = (31.50sf)(31.87\ fps) = 1003.91\ cfs$

$Q_p = 1003.91\ cfs > q = 180.00\ cfs$

Proposed Ditch is adequate

Actual Velocity = $V_a = 20.65\ fps\ @\ d = 1.65\ ft.$

ROCK ENERGY DISSIPATOR:

Due to the outlet velocity of 20.65 fps, it is necessary to construct a form of scour protection within the channel at the base of the hillside. A basin lined with riprap will be used for scour prevention at this location. The basin will also act as an energy dissipator to decrease the flow velocity creating a laminar flow between the location of the basin and ultimate discharge point at the inlet of the culvert. The following two (2) sets of calculations will be used to determine the scour hole and riprap basin geometry based on the following conditions:

1. Inlet Ditch:

   Configuration = Trapezoidal with side slopes of 2H : 1V
   Depth = 4.0 ft. minimum; Design with 0.5 ft. Freeboard
   Bottom Width = 2.0 ft. minimum
   Lining = Grouted Rock
   Slope = 26.0% ±

2. Maximum Expected Discharge = 180.00 cfs
   Based on a 100 year, 24 hour Rainfall Event
   (Used 200.00 cfs in calculations)

3. Normal Depth in Ditch for Q = 200.00 cfs = dn
   dn = 1.75 ft ±

4. Normal Velocity in ditch for Q = 200.00 cfs = Vn
   Vn = 22.00 fps

5. Tailwater = TW = 1.75

The two (2) sets of calculations used were generated from the following sources:

I. Scour Hole Geometry -- Reference to Chapter V of "Hydraulic Design of Energy Dissipators for Culverts and Channels", U.S. Department of Transportation. Federal Highway Administration, 12/75, reprint 12/78.

II. Riprap Basin Geometry -- Reference to Chapter XI of "Hydraulic Design of Energy Dissipators for Culverts and Channels", U.S. Department of Transportation, Federal Highway Administration, 12/75, reprint 12/78.

Calculation Set I:

Scour Hole Geometry:

Scour Geometry $= \alpha (Ye)^{\gamma} (Q/Ye^{5/2})^{\beta} t^{\theta}$
α, γ, β and θ were obtained from Table V-1 and t was estaimated to be 30 minutes.

Tailwater = 1.75 ft. which is greater than 0.5D = 0.5(2 ft.) = 1.00 ft.
TW/D = 1.75 ft./4.00 ft. = 0.44
$Q/BD^{3/2}$ = 200 cfs/(2 ft.)(4 ft.)$^{3/2}$ = 12.50
Figure III - 9 : Yo/D = 1.00
Table III - 2 : $\frac{d}{D} = \frac{1.75 \text{ ft.}}{2.0 \text{ ft.}}$ = 0.88, therefore, A/D^2 = 0.7320

$A = (2 \text{ ft.})^2(0.7320) = 2.93 \text{ sf}$
$Ye = (A/2)^{1/2} = (2.93 \text{ sf}/2)^{1/2} = 1.21 \text{ ft.}$

Depth of Scour:

$$h_s = \alpha(Ye)^{\gamma} \; (Q/Ye^{5/2})^{\beta} \; t^{\theta}$$
$$h_s = 0.76 \; (1.21 \text{ ft.})^{1.0} \; (200 \text{ cfs}/(1.21 \text{ ft.})^{5/2})^{0.375} \; (30 \text{ min.})^{0.10}$$
$$h_s = 7.91 \text{ ft.}$$

Width of Scour:

$$w_s = \alpha(Ye)^{\gamma} \; (Q/Ye^{5/2})^{\beta} \; t^{\theta}$$
$$w_s = 0.39(1.21 \text{ ft.})^{1.0}(200 \text{ cfs}/(1.21 \text{ ft.})^{5/2})^{0.915}(30 \text{ min.})^{0.15}$$
$$w_s = 64.96 \text{ ft.}$$

Length of Scour:

$$L_s = \alpha(Ye)^{\gamma} \; (Q/Ye^{5/2})^{\beta} \; t^{\theta}$$
$$L_s = 2.85(1.21 \text{ ft.})^{1.0}(200 \text{ cfs}/(1.21 \text{ ft.})^{5/2})^{0.71}(30 \text{ min.})^{0.125}$$
$$L_s = 161.87 \text{ ft.}$$

Scour Hole Geometry Adjustments:

Following test data taken from "Hydraulic Design of energy Dissipators for Culverts anbd Channels" by U.S. Department of Transportation, Federal Highway Administration.

Field Test Scour Hole - vs - Calculated Scour Hole Geometry

Geometry	Field	Calculated	%Field/Calc.
Depth (h_s)	3.0 ft.	6.5 ft.	46.15%
Width (w_s)	9.5 ft.	24.3 ft.	39.09%
Length (L_s)	12.0 ft.	37.6 ft.	31.91%

Based on the above testing results, the calculated values for the Scour Hole Geometry will be adjusted as follows:

Geometry	Calculated	%Field/Calc.	Field Design
Depth (h_s)	7.91 ft.	46.15%	3.65 ft.
Width (w_s)	64.96 ft.	39.09%	25.39 ft.
Length (L_s)	161.87 ft.	31.91%	51.65 ft.

Calculation Set II:

Riprap Basin Geometry:

Determine Brink Depth = Yo and Outlet Velocity = Vo

$Q/BD^{3/2}$ = 200 cfs/(2 ft.)(4 ft.)$^{3/2}$ = 12.50

TW/D = 1.75 ft./4.0 ft. = 0.44

Figure III - 9 : Yo/D = 1.00

Yo = 1.00 (4.0 ft.) = 4.00 ft.

TW/Yo = 1.75 ft./4.00 ft. = 0.44

TW/Yo = 0.44 $<$ 0.75 Therefore, Riprap Basin will act as energy dissipator.

Brink Area (A) for Yo/D = 1.00

A = (0.7320)(2.0 ft.)^2 = 2.93 sf

Due to steep slope of inlet ditch, use $V_o = V_n$

$V_o = V_n$ = 22.00 fps

Equivalent Flow Depth at Brink = Ye

Ye = $(A/2)^{1/2}$ = (2.93 sf/2)$^{1/2}$ = 1.21 ft.

Froude Number = Fr

Fr = $V_o/[(32.2 \text{ fps})(Ye)]^{1/2}$

Fr = 22.0 fps/[(32.2 fps)(1.21 ft.)]$^{1/2}$ = 3.52

Try d_{50}/Ye = 0.70

d_{50} = 0.70(1.21 ft.) = 0.85 ft.

From Fig. XI-2; h_s/Ye = 2.60

h_s = 2.60(1.21 ft.) = 3.15 ft.

Check: h_s/d_{50} = 3.15/0.85 = 3.71

Therefore, $2 < h_s/d_{50} < 4$: $2 < 3.71 < 4$

Riprap Basin will act as an energy dissipator

Energy Dissipator Pool Length = $h_s(10)$ or $3(W_o)$: Use Greater

$L_s = h_s(10)$ = (3.15 ft)(10) = 31.50 ft.

or $L_s = 3(W_o)$ = 3(9.0 ft.) = 27.00 ft.

Therefore, use L_s = 31.50 ft.

Width of Basin = 3:1 flare off outlet of ditch

Riprap Basin Geometry - Summary:

Depth of Basin = 3.15 ft.

Length of Basin = 31.50 ft.

Width of Basin = 3:1 flare off outlet of ditch

Rock Energy Dissipator Final Summary:

Based on the pervious two (2) sets of calculations for scour hole and riprap basin geometry, the following will be the design dimensions for the Rock Energy Dissipator at the base on the hillside:

Depth of Basin = 4.00 ft. minimum
Length of Basin = 50.00 ft. minimum (includes dissipator pool and apron)
Width of Basin = 10.00 ft. minimum off outlet of ditch flaring 3:1 over the 30.00 ft. minimum Dissipator Pool.

DRAINAGE AREA NO. 11

Hydrologic Analysis:

Maximum Expected Overland Discharge = $q = q_p(DA)(Q)$

q_p: Time of Concentration = T_c
Overland Slope = $\frac{1224 - 1000}{1080 \text{ ft.}}$ (100%) = 20.74%
Velocity - Fig. 3-1 - Forest = 1.17 fps
T_c = 1080 ft./1.17 fps/3600 sec/hr. = 0.26 hrs.
Check Time of Travel from upstream ditch: T_t
$T_t = T_c + D/V$ = 1.80 hrs. + (200 ft./22.0 fps/3600 sec/hr.) = 1.803 hrs.
T_c = 0.26 hrs. $\angle$ T_t = 1.803 hrs. Use T_c = 1.803 hrs., say T_c = 1.50 hrs.

q_p for T_c = 1.5 hrs and T_t = 0 = 236 csm/in. @ 12.8 hrs.

DA: Drainage Area = Areas No. 1 thru 11 plus Top of East/West Pile and Slope Drain Area - Refer Drainage Plan
DA = 0.25 mi^2.

Q = Runoff interpolated from Table 2-1 = 3.20 in.

Maximum Expected Overland Discharge = $q = q_p(DA)(Q)$
q = (236 csm/in.)(.025 mi^2)(3.20 in.) = 188.80 cfs
Use q = 190.00 cfs

Hydraulic Analysis of Proposed Ditch:

Ditch Parameters:

Configuration = Trapezoidal with side slopes of 2H : 1V
Depth = 4.0 ft. minimum; Design with 0.5 ft. Freeboard.
Bottom Width = 2.0 ft. minimum
Lining = Rock
Slope = 2.50% $\pm$

Capacity of Proposed Ditch = Q_p = AV

$A = [2.0 \text{ ft.} + 2(3.5 \text{ ft.})](3.5 \text{ ft.}) = 31.50 \text{ sf}$

$V = \frac{1.486}{n} R^{2/3} S^{1/2}$

n = 0.035 (Refer Table 2.10.13.1)

R = A/WP

A = 31.50 sf

WP = 17.65 ft.

R = 31.50 sf/17.65 ft. = 1.78 ft.

S = 2.50% = 0.025 ft./ft.

$V = \frac{1.486}{0.035} (1.78)^{2/3}(0.025)^{1/2} = 9.99 \text{ fps}$

Capacity of Proposed Ditch = Q_p = AV

Q_p = (31.50 sf)(9.99 fps) = 314.68 cfs

Q_p = 314.68 cfs $\gg$ q = 190.00 cfs

Proposed ditch is adequate

Actual Velocity = V_a = 8.81 fps (approx.) @ d = 2.82 ft. (approx.)

HYDRAULIC DESIGN OF CULVERT AT DOWNSTREAM END OF DRAINAGE AREA NO. 11

Maximum Expected Overland Discharge to Culvert based on the previous calculations equals 190.00 cfs. Design for 200.00 cfs culvert capacity.

Culvert Parameters:

Diameter = 66 inch

Material = Reinfoced Cement Concrete Pipe (R.C.C.P)

Slope = 1.00% ±

Refer to following Headwater Computation Sheet for the hydraulic analysis of the proposed culvert.

PROJECT: Keystone

DESIGNER: ADL

DATE: 6/4/85

HYDROLOGIC AND CHANNEL INFORMATION

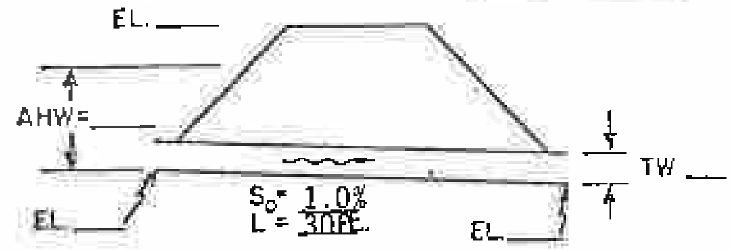
Q_1 = 200 cfs TW_1 = ______

Q_2 = ______ TW_2 = ______

(Q_1 = DESIGN DISCHARGE, SAY Q_{25}
Q_2 = CHECK DISCHARGE, SAY Q_{50} OR Q_{100})

SKETCH

STATION: ______



MEAN STREAM VELOCITY = ______
MAX. STREAM VELOCITY = ______

CULVERT DESCRIPTION (ENTRANCE TYPE)	Q	SIZE	HEADWATER COMPUTATION										CONTROLLING HW	OUTLET VELOCITY	COST	COMMENTS
			INLET CONT.		OUTLET CONTROL $HW = H + h_o - LS_o$											
			$\frac{HW}{D}$	HW	K_e	H	d_c	$\frac{d_c+D}{2}$	TW	h_o	LS_o	HW				
Type D - W Endwall	200	66	1.23	6.76	0.5	0.85	4.1	4.8	2.92	4.8	0.3	5.35	6.76	18		Outlet Protectio Required

SUMMARY & RECOMMENDATIONS:

66 inch diameter R.C.C.P. culvert is adequate
Based on the outlet velocity place approximately 20 ft $\pm$ of grouted rock lining at the outlet of the culvert.

Tailwater and Outlet Velocity Determination:

$$Qact = AV$$

$$A = \pi r^2 = \pi (2.75 \text{ ft})^2 = 23.76 \text{ sf}$$

$$V = \frac{1.486}{n} R^{2/3} S^{1/2} = \frac{1.486}{0.012} \left(\frac{5.5 \text{ ft}}{4} \right)^{2/3} (0.01)^{1/2} = (123.83)(1.24)(0.10) = 15. \text{ fps}$$

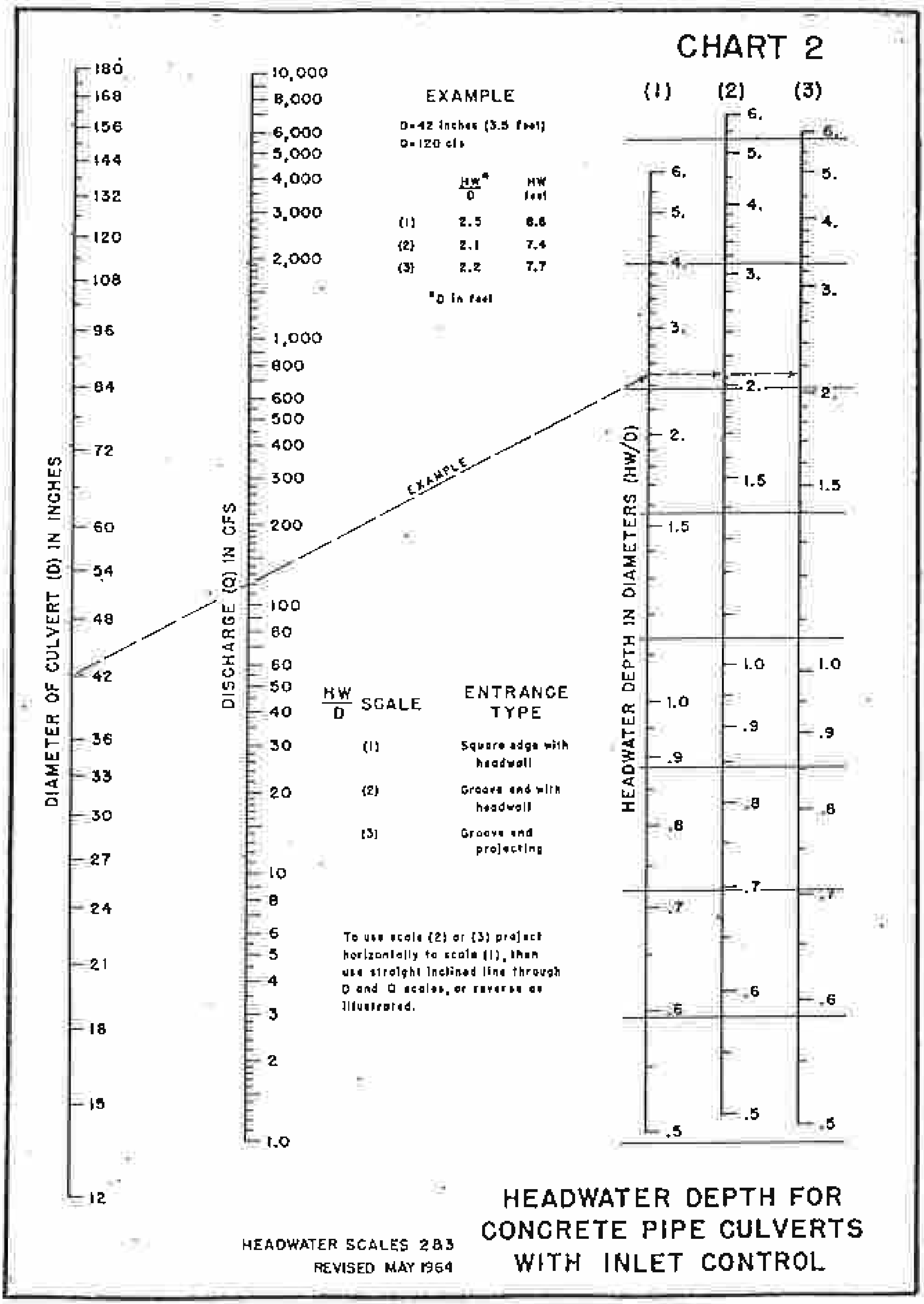
$$Qact = AV = (23.76\text{sf})(15.35\text{fps}) = 364.72 \text{ cfs}$$

$$Qexp/Qact = 200\text{cfs}/364.72\text{cfs} = 0.55$$

Refer Fig. 20: Depth of Flow = 0.53 Therefore TW = 0.53D
TW = 0.53 (5.5 ft) = 2.92 ft

Velocity Proportion = 1.2 therefore, $V_{out} = 1.2V_{act}$

$V_{out} = 1.2(15.35\text{fps}) = 18.42$ fps



Taken from "Hydraulic Charts for the Selection of Highway Culverts," U.S. Department of Transportation, Federal Highway Administration, December 1965.

TABLE 1 - ENTRANCE LOSS COEFFICIENTS

Outlet Control, Full or Partly Full

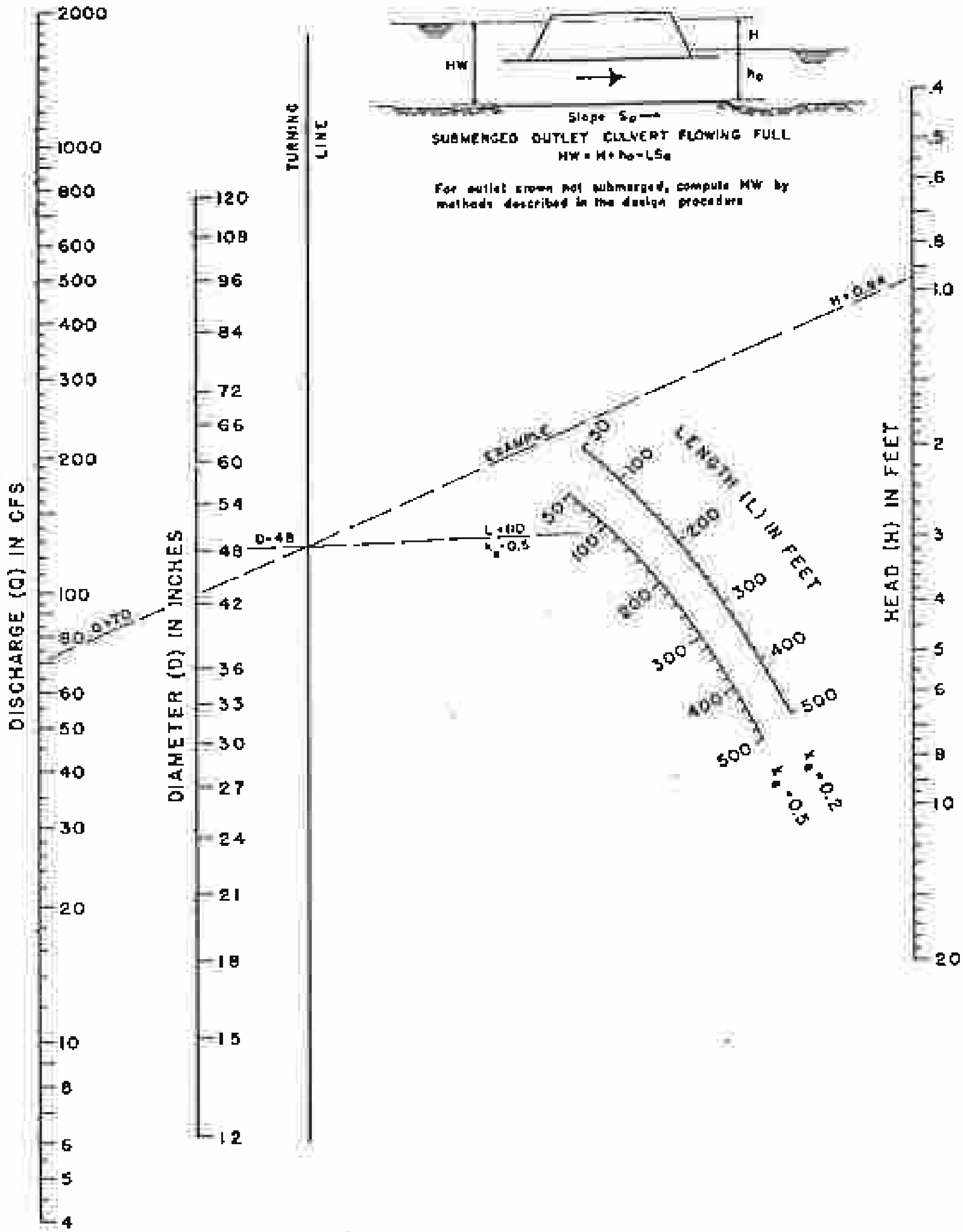
Entrance head loss $H_e = k_e \frac{V^2}{2g}$

Type of Structure and Design of Entrance	Coefficient k_e
Pipe, Concrete	
Projecting from fill, socket end (groove-end) . . .	0.2
Projecting from fill, sq. cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square-edge	0.5
Rounded (radius = 1/12D)	0.2
Mitered to conform to fill slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side-or slope-tapered inlet	0.2
Pipe, or Pipe-Arch, Corrugated Metal	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge . .	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side-or slope-tapered inlet	0.2
Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of 1/12 barrel dimension, or beveled edges on 3 sides . . .	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of 1/12 barrel dimension, or beveled top edge	0.2
Wingwall at 10° to 25° to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side-or slope-tapered inlet	0.2

*Note: "End Section conforming to fill slope," made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both <u>inlet</u> and <u>outlet</u> control. Some end sections, incorporating a <u>closed</u> taper in their design have a superior hydraulic performance. These latter sections can be designed using the information given for the beveled inlet, p. 5-13.

Taken from "Hydraulic Charts for the Selection of Highway Culverts," U.S. Department of Transporation, Federal Highway Administration, December 1965.

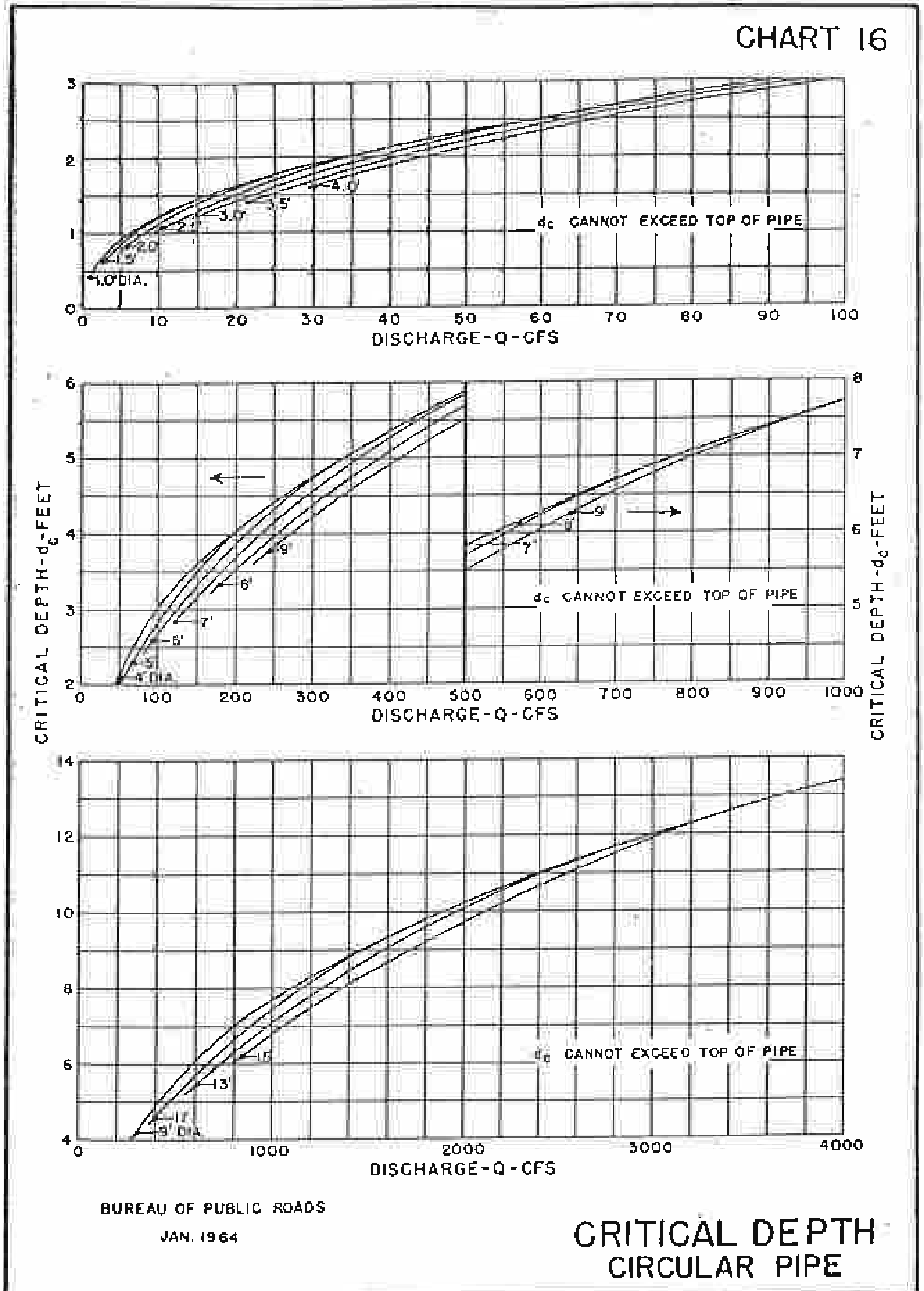
CHART 9



HEAD FOR
CONCRETE PIPE CULVERTS
FLOWING FULL
n = 0.012

BUREAU OF PUBLIC ROADS JAN. 1963

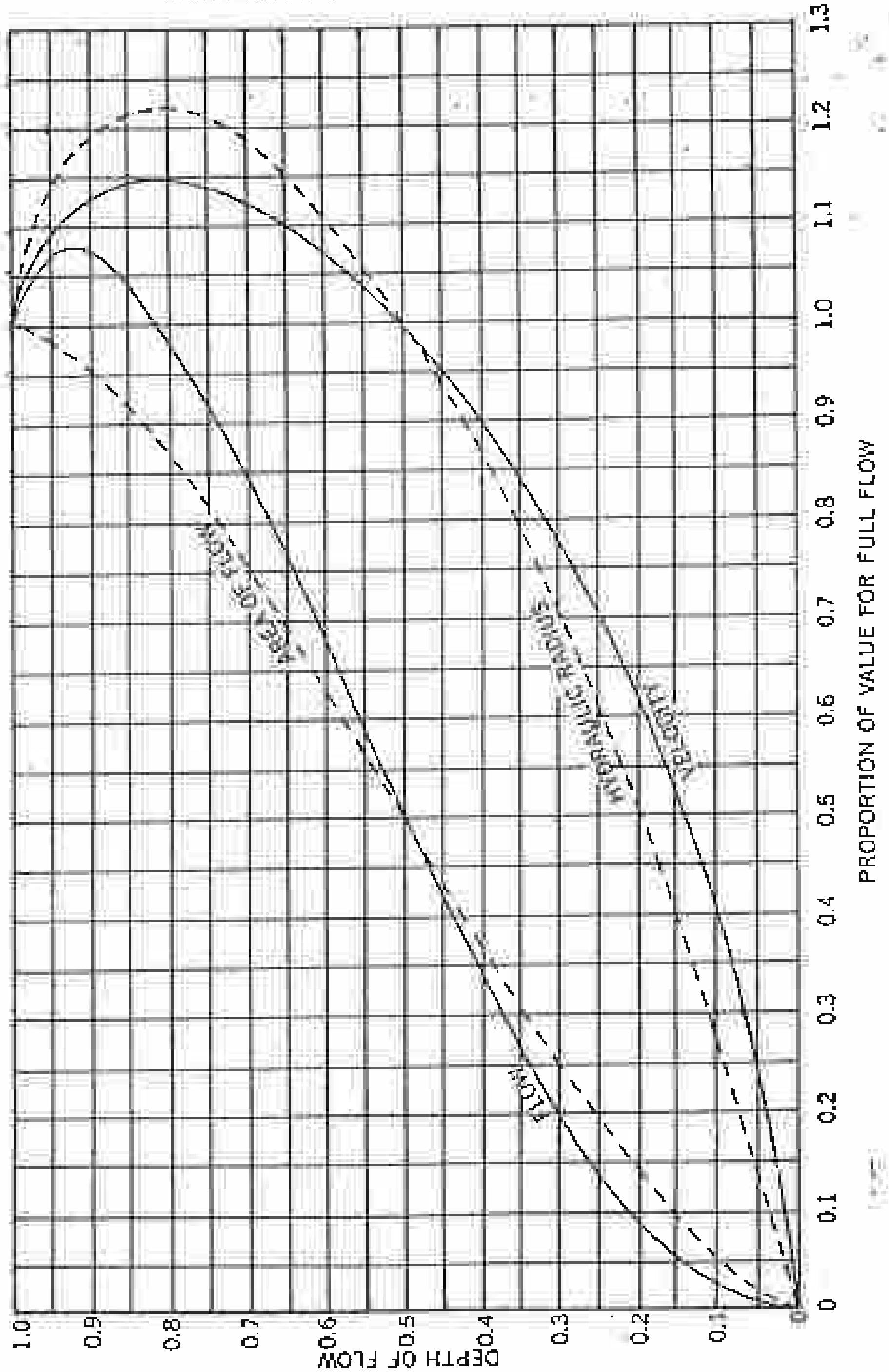
Taken from "Hydraulic Charts for the Selection of Highway Culverts," U.S. Department of Transportation, Federal Highway Administration, Decebmer 1965.



Taken from "Hydraulic Charts for the Selection of Highway Culverts," U.S. Department of Transportation, Federal Highway Administration, December 1965.

FIGURE 20

RELATIVE VELOCITY AND FLOW IN
CIRCULAR PIPE FOR ANY DEPTH OF FLOW



Taken from "Concrete Pipe Design Manual," American Concrete Pipe Association, June 1980

DRAINAGE DITCH DESIGN FROM CULVERT OUTLET TO OUTFALL INTO PLUM CREEK

From the culvert outlet to the point of ultimate discharge into Plum Creek, the alignment of the drainage ditch will follow a nearly level terrain. Due to this fact, additional overland runoff for this section of ditch will be considered negligible. The ditch will be sized for a Maximum Expected Overland Discharge (q) of 190.00 cfs --- refer previous calculations.

Hydraulic Analysis of Proposed Ditch:

Ditch Parameters:

Configuration = Trapezoidal with side slopes of 2H : 1V
Depth = 4.0 ft. minimum; Design with 0.5 ft. Freeboard.
Bottom Width = 5.0 ft. minimum
Lining = Rock
Slope 0.80% ±

Capacity of Proposed Ditch = Q_p = AV

$A = [5.0 \text{ ft.} + 2(3.5 \text{ ft.})](3.5 \text{ ft.}) = 42.00$ sf

$V = \frac{1.486}{n} R^{2/3} S^{1/2}$

n = 0.035 (Refer Table 2.10.13.1)

R = A/WP

A = 42.00 sf

WP = 20.65 ft.

R = 42.00 sf/20.65 ft. = 2.03 ft.

S = 0.80% = 0.008 ft./ft.

$V = \frac{1.486}{0.035} (2.03)^{2/3}(0.008)^{1/2} = 6.08$ fps

Capacity of Proposed Ditch = Q_p = AV

Q_p = (42.00 sf)(6.08 fps) = 255.36 cfs

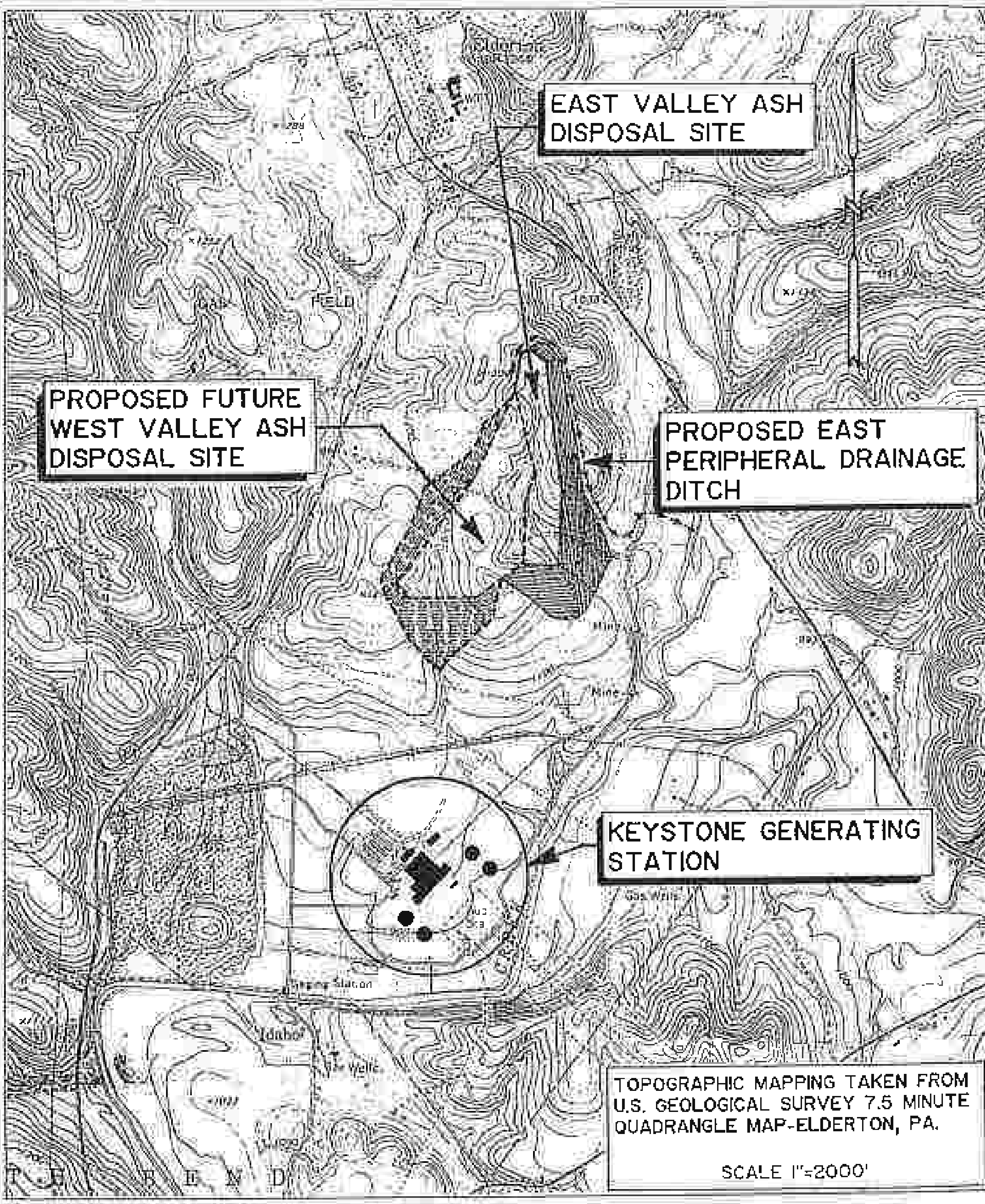
Q_p = 255.36 cfs $>$ q = 190.00 cfs

Proposed ditch is adequate

Actual Velocity = V_a = 5.69 fps (approx.) @ d = 3.10 ft. (approx.)

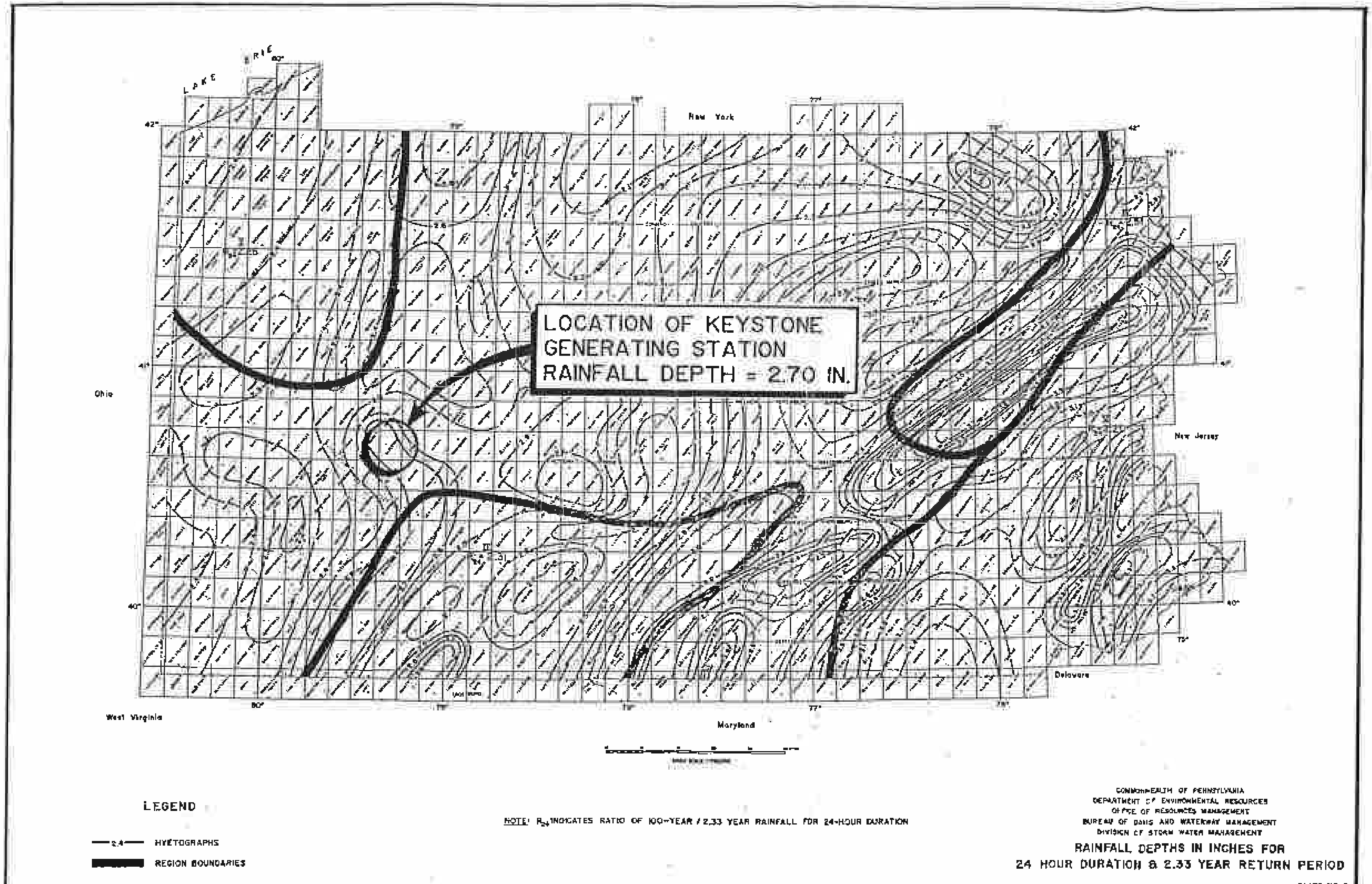
At the time of actual design and construction of the complete drainage facility, a General Outfall Structure Permit for the discharge point into Plum Creek will be secured by the Pennsylvania Electric Company from the Department of Environmental Resources.

APPENDIX



LOCATION MAP

FIGURE 1



MEAN ANNUAL RAINFALL MAP OF PENNSYLVANIA

TABLE 6 (CONTINUED)

24 HOUR RAINFALL FOR REGION II					
MEAN ANNUAL RAINFALL (INCHES)	********RETURN PERIOD********				
	5 YR	10 YR	25 YR	50 YR	100 YR
2.62	3.25	3.76	4.39	4.88	5.34
2.64	3.27	3.79	4.42	4.92	5.39
2.66	3.30	3.82	4.46	4.96	5.43
2.68	3.32	3.85	4.49	4.99	5.47
2.70	3.35	3.88	4.53	5.03	5.51
2.72	3.37	3.91	4.56	5.07	5.55
2.74	3.40	3.94	4.59	5.11	5.59
2.76	3.42	3.97	4.63	5.14	5.63
2.78	3.44	3.99	4.66	5.18	5.67
2.80	3.47	4.02	4.69	5.22	5.71
2.82	3.49	4.05	4.73	5.25	5.75
2.84	3.52	4.08	4.76	5.29	5.79
2.86	3.54	4.11	4.79	5.33	5.83
2.88	3.57	4.14	4.83	5.37	5.88
2.90	3.59	4.17	4.86	5.40	5.92
2.92	3.62	4.20	4.89	5.44	5.96
2.94	3.64	4.22	4.93	5.48	6.00
2.96	3.67	4.25	4.96	5.52	6.04
2.98	3.69	4.28	4.99	5.55	6.08
3.00	3.72	4.31	5.03	5.59	6.12
3.02	3.74	4.34	5.06	5.63	6.16
3.04	3.77	4.37	5.10	5.66	6.20
3.06	3.79	4.40	5.13	5.70	6.24
3.08	3.82	4.43	5.16	5.74	6.28
3.10	3.84	4.45	5.20	5.78	6.32
3.12	3.87	4.48	5.23	5.81	6.36
3.14	3.89	4.51	5.26	5.85	6.41
3.16	3.92	4.54	5.30	5.89	6.45
3.18	3.94	4.57	5.33	5.92	6.49
3.20	3.97	4.60	5.36	5.96	6.53
3.22	3.99	4.63	5.40	6.00	6.57

Taken from "Rainfall Duration Frequency Tables for Pennsylvania," Commonwealth of Pennsylvania Department of Environmental Resources, February 1983

SOILS MAP

FIGURE 2

for community development—Continued

Camp areas		Buildings without basements	Paths and trails	Picnic areas	Playgrounds	Golf fairways
Tents and camp trailers	Travel trailers					
Moderate: slow permeability; slope.	Severe: slope	Moderate: slope.	Slight	Moderate: slope.	Severe: slope	Moderate: slope; bedrock at a depth of 1½ to 3½ feet.
Severe: slope	Severe: slope	Severe: slope	Moderate: slope.	Severe: slope	Severe: slope	Severe: slope.
Moderate: slow permeability; seasonal high water table; moderately fine textured surface layer.	Moderate: slow permeability; slope; seasonal high water table; moderately fine textured surface layer.	Moderate: seasonal high water table.	Moderate: moderately fine textured surface layer; seasonal high water table.	Moderate: moderately fine textured surface layer; seasonal high water table.	Severe: seasonal high water table.	Moderate: seasonal high water table.
Moderate: slow permeability; slope; seasonal high water table; moderately fine textured surface layer.	Severe: slope	Moderate: slope; seasonal high water table.	Moderate: moderately fine textured surface layer; seasonal high water table.	Moderate: moderately fine textured surface layer; slope; seasonal high water table.	Severe: slope; seasonal high water table.	Moderate: slope; seasonal high water table.
Severe: slope	Severe: slope	Severe: slope	Moderate: slope; seasonal high water table; moderately fine textured surface layer.	Severe: slope	Severe: slope; seasonal high water table.	Severe: slope.

Areas of this soil range from 6 to 20 acres and are irregular in shape. Surface runoff is medium, and the hazard of erosion is moderate to high if the soil is cultivated.

Included with this soil in mapping were a few areas of Rainsboro soils and a few areas of a soil that has a sandy loam profile.

This soil is suited to the crops grown in the county and to hay, pasture, trees, and wildlife. Slope is a limitation for many uses. Capability unit IIIe–1.

Cavode Series

The Cavode series consists of deep, somewhat poorly drained, gently sloping to moderately steep soils. These soils are on uplands on ridgetops, benches, and some foot slopes. They formed in material that weathered from acid clay shale interbedded with some thin siltstone. The native vegetation is mixed hardwoods that include red oak, black oak, white oak, and red maple.

In a representative profile, in a wooded area, the surface layer is very dark brown silt loam about 5 inches thick. It is covered with a 2-inch mat of decaying leaves and twigs. The subsoil extends to a depth of 30 inches. The upper 7 inches of the subsoil is mottled, yellowish-brown, friable silty clay loam. The middle part is mottled, light brownish-gray, firm silty clay loam about 14 inches thick. The lower 4 inches is mottled, light brownish-gray, friable shaly silty clay loam. The substratum, at a depth of 30 to 48 inches, is gray, firm shaly silty clay loam. Acid, gray shale is at a depth of about 48 inches.

The available moisture capacity is moderate, and permeability is slow. A seasonal water table rises to within 6 to 18 inches of the surface in wet periods. Seep areas and wet-weather springs are common. If they are adequately drained, these soils are suited to most of the crops grown in the county. Many areas of these soils have been cleared and are used for crops. Other areas are wooded or are idle and reverting to

Taken from "Soil Survey of Armstrong County, Pennsylvania," USDA-SCS, February 1977

TABLE 8.—*Approximate acreage and proportionate extent of the soils*

Soil	Area	Extent
	Acres	*Percent*
Allegheny silt loam, 3 to 8 percent slopes	1,180	0.3
Allegheny silt loam, 8 to 15 percent slopes	2,920	.7
Cavode silt loam, 3 to 8 percent slopes	9,860	2.4
Cavode silt loam, 8 to 15 percent slopes	7,660	1.8
Cavode silt loam, 15 to 25 percent slopes	1,060	.3
Ernest silt loam, 0 to 3 percent slopes	770	.2
Ernest silt loam, 3 to 8 percent slopes	18,960	4.5
Ernest silt loam, 8 to 15 percent slopes	26,030	6.2
Ernest silt loam, 15 to 25 percent slopes	6,380	1.5
Ernest very stony silt loam, 0 to 8 percent slopes	380	.1
Ernest very stony silt loam, 8 to 25 percent slopes	1,110	.3
Gilpin-Weikert complex, 3 to 8 percent slopes	7,020	1.7
Gilpin-Weikert complex, 8 to 15 percent slopes	7,730	1.8
Gilpin-Weikert complex, 15 to 25 percent slopes	42,440	10.1
Hazleton channery loam, 3 to 8 percent slopes	5,380	1.3
Hazleton channery loam, 8 to 15 percent slopes	3,460	.8
Hazleton channery loam, 15 to 25 percent slopes	4,810	1.1
Hazleton very stony loam, 8 to 25 percent slopes	600	.1
Melvin silty clay loam	9,520	2.3
Mine dumps	910	.2
Pope fine sandy loam	1,040	.2
Pope loam	2,100	.5
Rainsboro silt loam, 0 to 3 percent slopes	1,620	.4
Rainsboro silt loam, 3 to 8 percent slopes	6,480	1.5
Rainsboro silt loam, 8 to 15 percent slopes	3,270	.8
Rayne silt loam, 3 to 8 percent slopes	23,640	5.6
Rayne silt loam, 8 to 15 percent slopes	19,520	4.7
Rayne-Gilpin very stony silt loams, 8 to 25 percent slopes	1,200	.3
Steff loam	5,520	1.3
Steff loam, high bottom	850	.2
Strip mines	23,380	5.6
Upshur-Gilpin silt loams, 3 to 8 percent slopes	370	.1
Upshur-Gilpin silt loams, 8 to 15 percent slopes	520	.1
Upshur-Gilpin silt loams, 15 to 25 percent slopes	730	.2
Upshur-Gilpin silt loams, 25 to 35 percent slopes	490	.1
Urban land	1,740	.4
Weikert shaly silt loam, 3 to 8 percent slopes	3,140	.7
Weikert shaly silt loam, 8 to 15 percent slopes	1,600	.4
Weikert and Gilpin soils, 25 to 70 percent slopes	114,010	27.2
Wharton silt loam, 3 to 8 percent slopes	13,460	3.2
Wharton silt loam, 8 to 15 percent slopes	7,310	1.7
Wharton-Gilpin silt loams, 3 to 8 percent slopes	3,060	.7
Wharton-Gilpin silt loams, 8 to 15 percent slopes	7,820	1.9
Wharton-Gilpin silt loams, 15 to 25 percent slopes	10,290	2.5
Wharton-Vandergrift complex, 3 to 8 percent slopes	1,190	.3
Wharton-Vandergrift complex, 8 to 15 percent slopes	1,780	.4
Wharton-Vandergrift complex, 15 to 25 percent slopes	540	.1
Water	4,990	1.2
Total	419,840	100.0

woodland. The seasonal water table, slow permeability, and slope are limitations for some uses.

Representative profile of Cavode silt loam, 3 to 8 percent slopes, in a wooded area 1½ miles north of Tidal along Route T490:

O2—2 to 1 1/2 inches, recent leaf litter.

O1—1 1/2 inches to 0, black (N 2/0) partly decayed leaf litter.

A1—0 to 5 inches, very dark brown (10YR 2/2) silt loam; weak, fine, granular structure; very friable, slightly sticky and nonplastic; very strongly acid; gradual, smooth boundary.

B21t—5 to 12 inches, yellowish-brown (10YR 5/4) silty clay loam; few, fine, faint, brown (10YR 5/3) mottles; moderate, medium, subangular blocky structure; friable, slightly sticky and plastic; thin, discontinuous clay films on peds; 5 percent shale fragments; very strongly acid; gradual, wavy boundary.

B22tg—12 to 26 inches, light brownish-gray (10YR 6/2) silty clay loam; many, medium, distinct, strong-brown (7.5YR 5/6) mottles; moderate, medium, subangular blocky structure; firm, sticky and plastic; thick, continuous clay films on peds; 10 percent shale fragments; very strongly acid; gradual, wavy boundary.

B3g—26 to 30 inches, light brownish-gray (2.5Y 6/2) shaly silty clay loam; many, medium, distinct, gray (10YR 5/1) and strong-brown (7.5YR 5/6) mottles; moderate, medium, subangular blocky structure; friable, sticky and plastic; 20 percent shale fragments; very strongly acid; gradual, wavy boundary.

Cg—30 to 48 inches, gray (5Y 6/1) shaly silty clay loam; many, medium, distinct, light brownish-gray (10YR 6/2) and yellowish-brown (10YR 5/8) mottles; massive; firm, slightly sticky and plastic; 40 percent shale fragments; very strongly acid; clear, wavy boundary.

R—48 inches +, acid, gray shale bedrock.

The solum is 30 to 52 inches thick. The depth to bedrock ranges from 40 to 72 inches. Coarse fragments make up as much as 35 percent of the A1 and B2 horizons and 10 to 80 percent of the B3g and Cg horizons. In some places, there is an Ap horizon that is dark grayish brown to brown. The B horizon ranges from silty clay loam to clay. Mottles in the B horizon range from gray to yellowish brown and yellowish red. The B21t horizon is brown to yellowish brown. The B22tg and B3g horizons are gray to light brownish gray.

Cavode soils occur near the deep, well drained Rayne soils; the moderately deep, well drained Gilpin soils; the shallow, well drained Weikert soils; and the deep, moderately well drained Wharton soils. Drainage of the Cavode soils is similar to that of the Vandergrift and Ernest soils, but the Vandergrift soils have a reddish B horizon and the Ernest soils have a Bx horizon.

CaB—Cavode silt loam, 3 to 8 percent slopes. This soil has the profile described as representative of the series. It is on ridgetops and benches in areas 8 to 30 acres in size. Surface runoff is medium, and the erosion hazard is moderate if the soil is cultivated.

Included with this soil in mapping were a few areas of soils that are medium acid to neutral in the substratum. Also included were some areas of a Cavode

Taken from "Soil Survey of Armstrong County,Pennsylvania", USDA-SCS, February 1977 soil that has many stones scattered on the surface. These areas are indicated on the detailed soil map by the symbol for very stony areas. Also included were small areas of Wharton soils.

This soil is suited to crops that tolerate wetness and to trees and wildlife. Artificial drainage can make it suitable for a wider range of crops. A seasonal water table and slow permeability are limitations for many uses. Capability unit IIIw-2.

CaC—Cavode silt loam, 8 to 15 percent slopes. This soil has a profile similar to the one described as representative of the series, but it is not so deep to bedrock. Areas of this soil range from 6 to 20 acres, are irregular in shape, and are on benches and side slopes. Surface runoff is medium, and the erosion hazard is moderate if the soil is cultivated.

Included with this soil in mapping were a few areas of soils that are medium acid to neutral in the substratum. Also included were some areas of a Cavode soil that has many stones scattered on the surface. These areas are shown on the soil map by the symbol for very stony areas. Also included were small areas of Wharton soils.

This soil is suited to crops that tolerate wetness and to trees and wildlife. Artificial drainage increases the range of suitable crops. A seasonal water table and slow permeability are limitations for many uses. Capability unit IIIe-4.

CaD—Cavode silt loam, 15 to 25 percent slopes. This soil is similar to the one described as representative of the series, but its surface layer has about 10 percent shale fragments. Areas of this soil range from 4 to 15 acres, are irregular in shape, and are on hillsides and some toe slopes. Surface runoff is rapid, and the erosion hazard is high if the soil is cultivated.

Included with this soil in mapping were some small areas of Wharton soils and a few areas of very stony Cavode soils.

This soil is suited to crops that require limited cultivation and that tolerate some wetness. It is also suited to trees and wildlife. Artificial drainage increases its suitability for a wider range of crops. A seasonal water table, slow permeability, and slope are limitations for most uses. Capability unit IVe-3.

Ernest Series

The Ernest series consists of deep, moderately well-drained, nearly level to moderately steep soils. These soils formed in colluvial material that weathered from acid gray shale, siltstone, and some fine sandstone. This material moved downslope from nearby uplands mainly to foot slopes and benches. The native vegetation consists of mixed hardwoods, including oaks, red maple, some sugar maple, black cherry, and hemlock.

In a representative profile the surface layer is brown silt loam about 8 inches thick. The subsoil extends to a depth of 50 inches. In the upper 7 inches, the subsoil is yellowish-brown, friable heavy silt loam. The part below that is mottled, strong-brown, friable light silty clay loam about 9 inches thick. The next part is mottled, brown, firm and brittle shaly heavy silt loam about 12 inches thick. In the lowermost part, the subsoil is mottled, brown, very firm and brittle, shaly heavy silt loam about 14 inches thick. The substratum, between depths of 50 and 74 inches or more, is yellowish-brown, firm shaly silt loam.

The available moisture capacity is moderate, and permeability is moderately slow. A seasonal water table rises to within 18 to 36 inches of the surface in wet periods. If these soils are adequately drained, they are suited to most of the crops grown in the county. Most areas of these soils have been cleared and are used for crops. A few areas are wooded or are idle and reverting to woodland. The seasonal water table, moderately slow permeability, and slope are limitations for many uses.

Representative profile of Ernest silt loam, 8 to 15 percent slopes, in a pasture 2½ miles north of Spring Church at the intersection of Routes T460 and T349:

Ap—0 to 8 inches, brown (10YR 4/3) silt loam; weak, fine, granular structure; friable, slightly sticky and slightly plastic; 5 percent coarse fragments; very strongly acid; clear, smooth boundary.

B1t—8 to 15 inches, yellowish-brown (10YR 5/4) heavy silt loam; moderate, medium and fine, subangular blocky structure; friable, sticky and plastic; thin, discontinuous clay films on peds; 5 percent shale fragments; very strongly acid; clear, wavy boundary.

B2t—15 to 24 inches, strong-brown (7.5YR 5/6) light silty clay loam; few, medium, faint, yellowish-brown (10YR 5/6) and pinkish-gray (7.5YR 6/2) mottles; moderate, medium and coarse, subangular blocky structure; friable, sticky and plastic; thick, continuous clay films on peds; 10 percent shale fragments; very strongly acid; clear, smooth boundary.

Bx1—24 to 36 inches, brown (7.5YR 5/4) shaly heavy silt loam; many, prominent, medium and coarse, pinkish-gray (7.5YR 6/2) and yellowish-brown (10YR 5/6) mottles; strong, very coarse, prismatic structure parting to subangular blocky; firm, brittle, sticky and plastic; thick, discontinuous clay films on peds; 20 percent shale fragments; very strongly acid; clear, smooth boundary.

Bx2—36 to 50 inches, brown (10YR 5/3) shaly heavy silt loam; many, medium to coarse, prominent, light brownish-gray (10YR 6/2) and yellowish-brown (10YR 5/4 and 10YR 5/6) mottles; strong, very coarse, prismatic structure; very firm, brittle, sticky and plastic; thick, continuous clay films on peds; 20 percent shale fragments; very strongly acid; clear, wavy boundary.

C—50 to 74 inches, yellowish-brown (10YR 5/6) shaly silt loam; massive; firm, slightly sticky and nonplastic; 30 percent shale fragments; very strongly acid; clear, wavy boundary.

The solum is 36 to 60 inches thick. The depth to the Bx horizon ranges from 20 to 28 inches. The depth to bedrock is more than 6 feet. Coarse fragments make up 5 to 20 percent of the B1t and B2t horizons and as much as 30 percent of the Bx and C horizons. The Ap horizon is brown, dark brown, dark grayish brown, or dark yellowish brown. The soil material above the Bx1 horizon is silt loam or silty clay loam. The Bx horizon is silt loam, silty clay loam, or clay loam. The C horizon ranges from silt loam to silty clay.

Ernest soils are near the deep, well drained Rayne soils; the moderately deep, well drained Gilpin soils; the shallow, well drained Weikert soils; the deep, moderately well drained Wharton soils; and the deep, somewhat poorly drained Cavode soils. Drainage of the Ernest soils is similar to that of the deep, moderately well drained to somewhat poorly drained Vandergrift soils and the deep, moderately well drained Rainsboro soils. In contrast to Ernest soils, the Wharton and Vandergrift soils lack a Bx horizon, and the Rainsboro soils occupy terraces and have fewer coarse fragments in the upper B horizon.

EnA—Ernest silt loam, 0 to 3 percent slopes. This soil has a profile similar to the one described as repre-

Taken from "Soil Survey of Armstrong County, Pennsylvania," USDA-SCS, February 1977

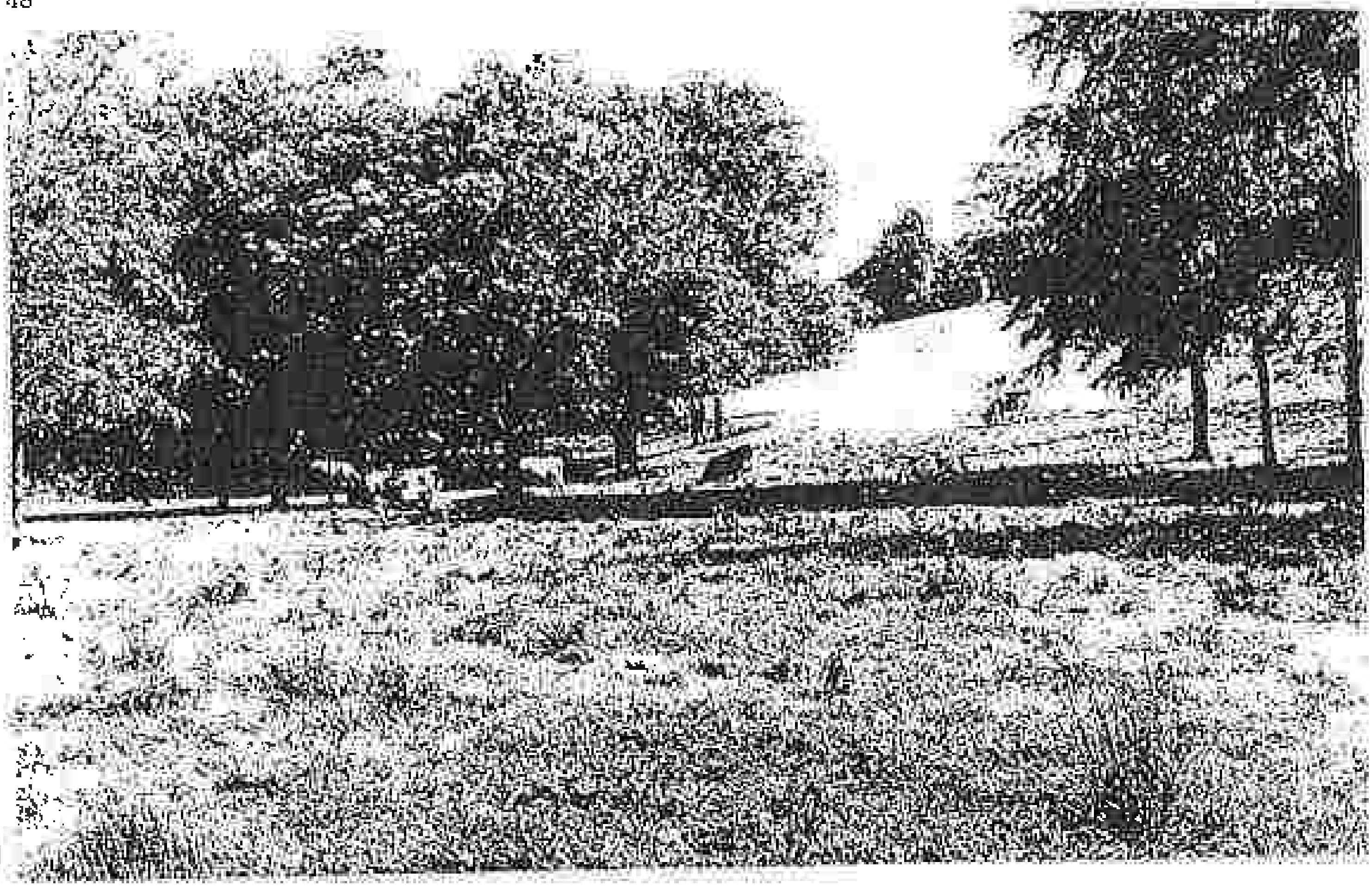


Figure 10.—Ernest silt loam, 3 to 8 percent slopes, is in the foreground, and beyond the trees is Gilpin channery silt loam, 15 to 25 percent slopes.

sentative of the series, but its surface layer is generally thicker. Areas of this soil range from 10 to 35 acres and occupy toe slopes and some benches. Runoff is slow, and the hazard of erosion is slight.

Included in mapping were some wet areas where the water table is closer to the surface and remains there for a longer period. Soils in these areas are mottled with strong brown, brown, or reddish yellow in the upper part of the subsoil. They are shown on the soil map by the symbol for wet spots.

This soil is suited to crops that tolerate some wetness and to trees and wildlife. Artificial drainage can make the soil suitable for a wider range of crops. A seasonal water table and moderately slow permeability are limitations for many uses. Capability unit IIw-2.

EnB—Ernest silt loam, 3 to 8 percent slopes. This soil has a profile similar to the one described as representative of the series, but the upper part of its subsoil is generally slightly thinner. Areas of this soil range from 6 to 20 acres and occupy lower slopes and benches (fig. 10). Runoff is medium, and the hazard of erosion is moderate if the soil is cultivated.

Included with this soil in mapping were some wet areas of soils in which the water table is closer to the surface and remains there for a longer period. In these wet areas the upper part of the subsoil is mottled with strong brown, brown, or reddish yellow. The areas are shown on the soil map by the symbol for wet spots. Also included were some small areas of soils that are coarser textured throughout their profile.

This soil is suited to crops that tolerate some wetness and to trees and wildlife. Artificial drainage can make the soil suitable to a wider range of crops. A seasonal water table and moderately slow permeability are limitations for many uses. Capability unit IIe-3.

EnC—Ernest silt loam, 8 to 15 percent slopes. This soil has the profile described as representative of the series. Areas of this soil range from 6 to 25 acres and occupy benches and lower slopes. Runoff is medium, and the erosion hazard is moderate if the soil is cultivated.

Included in mapping were a few seep areas that are shown on the soil map by the symbol for a wet spot. Also included were some small areas of soil that is coarser textured throughout the profile.

This soil is suited to crops that tolerate some seasonal wetness and to trees and wildlife. Artificial drainage can make it suitable for a wider range of crops. A seasonal water table and moderately slow permeability are limitations for many uses. Capability unit IIIe-4.

EnD—Ernest silt loam, 15 to 25 percent slopes. This soil is similar to the one described as representative of

Taken from "Soil Survey of Armstrong County, Pennsylvania," USDA-SCS, February 1977

the series, but its depth to bedrock is generally 4 to 8 inches less and it has more coarse fragments in the subsoil. Areas of this soil range from 10 to 35 acres and occupy lower foot slopes. Runoff is medium to rapid, and the erosion hazard is high if the soil is cultivated.

Included with this soil in mapping were a few areas of a soil that is not so deep to bedrock and some small areas of a soil that has a higher content of sand.

This soil is suited to crops that require limited cultivation and that tolerate some wetness. It is also suited to hay, pasture, trees, and wildlife. Slope, a seasonal water table, and moderately slow permeability are limitations for most uses. Capability unit IVe–3.

ErB—Ernest very stony silt loam, 0 to 8 percent slopes. This soil is similar to the one described as representative of the series, but its surface layer is covered by a 1- to 2-inch layer of decaying leaves and twigs. Stones cover 2 to 10 percent of the surface. Areas of this soil range from 10 to 20 acres and occupy benches and depressions in woodlands. Runoff is slow to medium and the erosion hazard is moderate if the soil is cleared.

Included in mapping were a few areas of a soil in which the water table is nearer the surface in wet seasons. These areas are shown on the soil map by the symbol for a wet spot.

This soil is well suited to most hardwoods and conifers that tolerate some wetness. It also is suited to recreation uses and wildlife. Stones, a seasonal water table, and moderately slow permeability limit these soils for most uses. Capability unit VIs–1.

ErD—Ernest very stony silt loam, 8 to 25 percent slopes. This soil is similar to the one described as representative of the series, but its surface layer is covered by a 1- to 2-inch layer of decaying leaves and twigs. Stones cover 2 to 10 percent of the surface. Areas of this soil range from 12 to 30 acres, are irregular in shape, and occupy benches and foot slopes. Runoff is slow to rapid, and the erosion hazard is moderate to high if the soil is cleared.

Included with this soil in mapping were a few small areas of a soil that has more sand throughout the profile than typical Ernest soils.

This soil is suited to most hardwoods and conifers that tolerate some wetness. It also is suited to recreation uses and wildlife. Slope, stones, a seasonal water table, and moderately slow permeability are limitations for most uses. Capability VIs–1.

Gilpin Series

The Gilpin series consists of moderately deep, well-drained, gently sloping to very steep soils on uplands. These soils formed in material that weathered from acid shale, siltstone, and fine-grained sandstone. They occur primarily on ridgetops and side slopes in dissected uplands. The native vegetation consists of hardwoods, mainly mixed oaks and red maples. Some black cherry and tulip-poplar are also present.

In a representative profile, in a pasture, the surface layer is dark-brown silt loam about 4 inches thick. The subsoil is yellowish brown and 22 inches thick. The upper part of the subsoil is silt loam, and the lower part is heavy silt loam. The substratum is yellowish-brown, friable shaly silt loam 8 inches thick. Grayish-brown, rippable shale bedrock is at a depth of about 34 inches.

The available moisture capacity and permeability are moderate. Some areas of these soils have been cleared and are used for hay, pasture, and crops. Other areas are idle or reverting to woodland. The steep and very steep soils are wooded. Moderate depth to bedrock and slope are limitations for most uses.

Representative profile of Gilpin silt loam, in an area of Gilpin-Weikert complex, 8 to 15 percent slopes, in a pasture on the west side of Route 03021, 3 miles southeast of Cowansville and 2 miles northwest of the Allegheny River:

A1—0 to 4 inches, dark-brown (10YR 3/3) silt loam; moderate, fine, granular structure; very friable, slightly sticky and nonplastic; 5 percent coarse fragments; strongly acid; clear, smooth boundary.

B21—4 to 9 inches, yellowish-brown (10YR 5/4) silt loam; weak, medium, subangular blocky structure; friable, slightly sticky and slightly plastic; 5 percent coarse fragments; strongly acid; gradual, wavy boundary.

B22t—9 to 26 inches, yellowish-brown (10YR 5/6) heavy silt loam; moderate, medium, subangular blocky structure; friable, slightly sticky and slightly plastic; thin, discontinuous clay films on ped faces; 10 percent coarse fragments; strongly acid; gradual, wavy boundary.

C—26 to 34 inches, yellowish-brown (10YR 5/6) shaly silt loam; weak, medium, subangular blocky structure; friable, slightly sticky and slightly plastic; 30 percent coarse fragments; very strongly acid; gradual, wavy boundary.

R—34 inches +, grayish-brown, rippable shale bedrock.

The solum is 20 to 30 inches thick. The depth to bedrock ranges from 20 to 40 inches. Coarse fragments make up 5 to 40 percent of the solum and 30 to 90 percent of the C horizon. The A1 horizon ranges from black to dark brown and from silt loam to channery loam or shaly silt loam. In some places, there is an Ap horizon that is brown to dark brown. The B21t horizon ranges from yellowish brown to strong brown and is heavy silt loam, heavy loam, or light silty clay loam. The C horizon ranges from shaly silt loam to channery or very channery loam.

Gilpin soils occur near the deep, well drained Rayne soils; the deep, moderately well drained Wharton soils; the shallow, well drained Weikert soils; and the deep, somewhat poorly drained Cavode soils. The Gilpin soils have drainage similar to that of the Hazleton soils but have less sand in the B horizon.

GwB—Gilpin-Weikert complex, 3 to 8 percent slopes. The soils of this mapping unit are so intermingled that it was neither practical nor feasible to map them separately. The Gilpin soil makes up about 50 to 55 percent of the complex. It has a profile similar to the one described as representative of the Gilpin series, but its surface layer is silt loam about 8 inches thick. The Weikert soil makes up about 35 to 40 percent of the complex. It has a profile similar to the one described as representative of the Weikert series, but it has a thicker subsoil. These soils occur on ridges and on knobs in areas 4 to 10 acres in size. Surface runoff is medium, and the hazard of erosion is moderate if the soils are cultivated.

Included with these soils in mapping were a few areas of Rayne soils.

Soils of this complex are suited to crops that tolerate some droughtiness and to pasture (fig. 11), hay, trees, and wildlife. Moderate depth to bedrock is the major limitation for use of the Gilpin soil. Coarse fragments

Taken from "Soil Survey of Armstrong County, Pennsylvania," USDA-SCS, February 1977

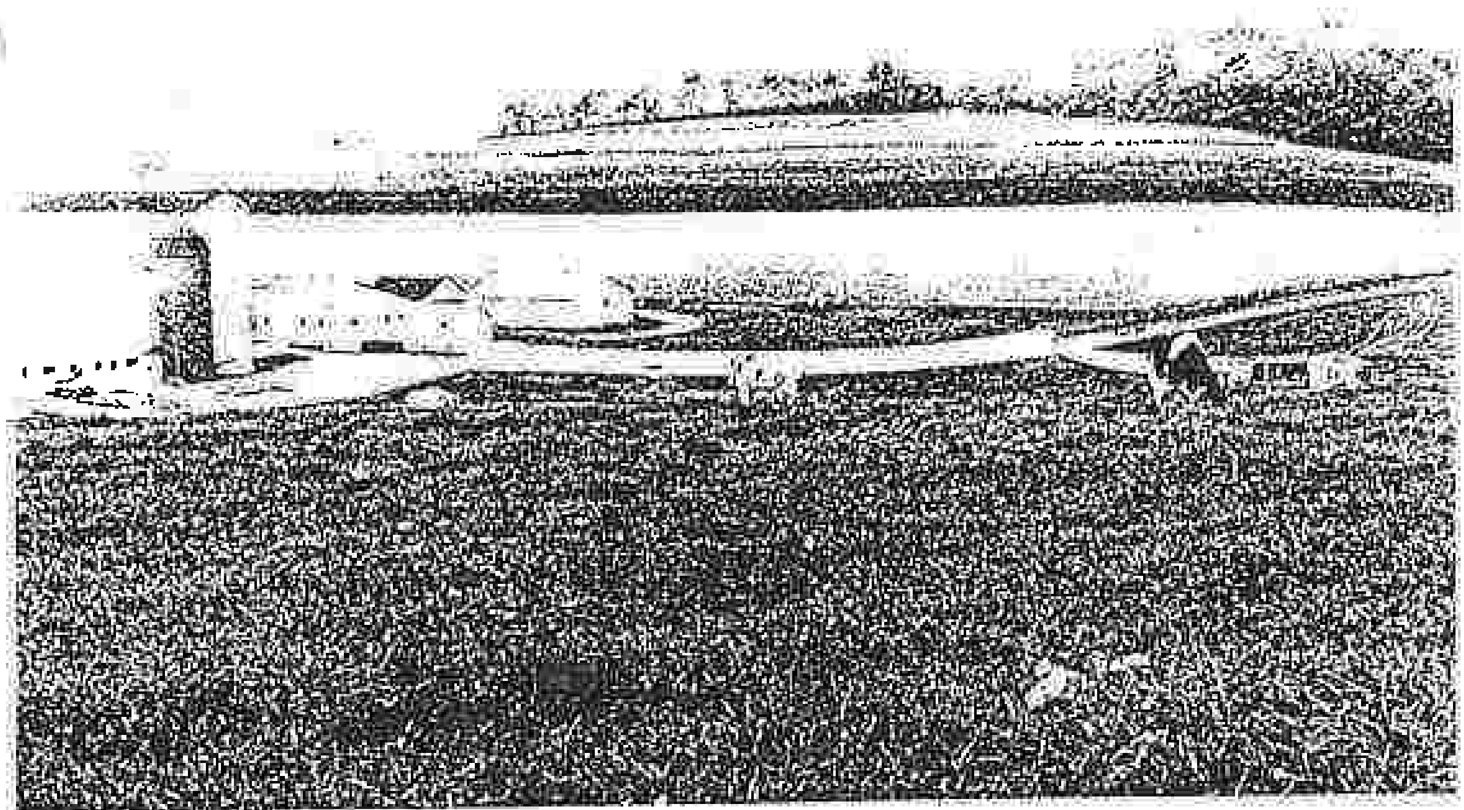


Figure 11.—Pasture in the foreground is on the Gilpin-Weikert complex, 3 to 8 percent slopes. The buildings are on Ernest silt loam, 8 to 15 percent slopes, and the stripcropping in the background is on the Gilpin-Weikert complex, 15 to 25 percent slopes.

and depth to bedrock are limitations of the Weikert soil for many uses. Capability unit IIe-2.

GwC—Gilpin-Weikert complex, 8 to 15 percent slopes. These soils are so intermingled that it was neither practical nor feasible to map them separately. The Gilpin soil makes up about 50 to 55 percent of the complex. It has the profile described as representative of the Gilpin series. The Weikert soil makes up about 35 to 40 percent of the complex. These soils occur on ridges and hillsides in irregularly shaped areas, 6 to 20 acres in size. Surface runoff is medium, and hazard of erosion is moderate if the soils are cultivated.

Included with these soils in mapping were a few small areas of Rayne soils.

Soils of this complex are suited to crops that tolerate some droughtiness and to pasture, hay, trees, and wildlife. Moderate depth to bedrock and slope are the major limitations of the Gilpin soil. Coarse fragments, slope, and depth to bedrock are limitations of the Weikert soil for most uses. Capability unit IIIe-2.

GwD—Gilpin-Weikert complex, 15 to 25 percent slopes. These soils are so intermingled that it was neither practical nor feasible to map them separately. The Gilpin soil makes up about 40 to 55 percent of the complex. It has a profile similar to the one described as representative of the Gilpin series, but it is not so deep to bedrock. The Weikert soil makes up about 35 to 50 percent of the complex. It has a profile similar to the one described as representative of the Weikert series, but its surface layer is thinner and has more thin, flat fragments of sandstone. Areas of these soils are long and narrow and range from 12 to 40 acres. Surface runoff is rapid, and the hazard of erosion is high if these soils are cultivated.

Included with these soils in mapping were a few areas of steep Gilpin and Weikert soils and a few small areas of Rayne soils.

Soils of this complex are suited to crops that tolerate some droughtiness and to hay, pasture, trees, and wildlife. Slope and moderate depth to bedrock are the major limitations of the Gilpin soil. Coarse fragments, slope, and shallow depth to bedrock are limitations of the Weikert soil for most uses. Capability unit IVe-2.

Hazleton Series

The Hazleton series consists of deep, well-drained, gently sloping to moderately steep soils on uplands. These soils formed in material that weathered from acid, gray and brown sandstone. They are mainly on ridgetops and hillsides in dissected areas. The native vegetation is red oak, black oak, white oak, scarlet oak, red maple, black cherry, and sassafras.

In a representative profile, in a cultivated area, the surface layer is dark-brown channery loam about 9 inches thick. The subsoil extends to a depth of 38 inches. In the upper 10 inches it is dark yellowish-brown, friable channery loam. In the lower 19 inches

Taken from "Soil Survey of Armstrong County, Pennsylvania," USDA-SCS, February 1977

RaB—Rainsboro silt loam, 3 to 8 percent slopes. This soil has a profile similar to the one described as representative of the series, but its surface layer is about 2 inches thinner. It is on undulating terraces in areas 8 to 35 acres in size. Runoff is medium, and the erosion hazard is moderate if the soil is cultivated.

Included in mapping were some areas of Allegheny soils. Also included were some areas of a soil that is in depressions and has a seasonal water table that rises to within 10 to 18 inches of the surface.

This soil is suited to crops that tolerate some wetness and to trees and wildlife. Artificial drainage can make it suitable for a wider range of crops. The seasonal water table and moderately slow permeability are limitations for many uses. Capability unit IIe-3.

RaC—Rainsboro silt loam, 8 to 15 percent slopes. This soil has a profile similar to the one described as representative of the series, but its surface layer is about 2 inches thinner. Areas of this soil are 6 to 20 acres in size and are narrow and irregular in shape. Runoff is medium, and the erosion hazard is moderate if the soil is cultivated.

Included with this soil in mapping were some areas of gently sloping Rainsboro soils.

This soil is suited to crops that tolerate seasonal wetness and to trees and wildlife. Artificial drainage can make it suitable for a wider range of crops. A seasonal water table, slope, and moderately slow permeability are limitations for most uses. Capability unit IIIe-4.

Rayne Series

The Rayne series consists of deep, well drained, gently sloping to moderately steep soils on uplands. These soils formed in material that weathered from interbedded shale, siltstone, and some fine-grained sandstone. They are mainly on ridgetops, but also occur on hillsides. The native vegetation is mixed hardwoods, mainly oaks and red maple and some black cherry, ash, and tulip-poplar.

In a representative profile, in a cultivated area, the surface layer is dark-brown silt loam about 6 inches thick. The subsoil is dark yellowish-brown, friable and firm shaly silt loam about 32 inches thick. The substratum, from a depth of 38 to about 60 inches, is dark yellowish-brown, firm very shaly silt loam. Rippable shale bedrock is at a depth of about 60 inches.

The available moisture capacity and permeability are moderate. Most areas of these soils have been cleared and are used for crops, hay, and pasture. A few areas are wooded or are idle. Slope is a limitation for some uses.

Representative profile of Rayne silt loam, 8 to 15 percent slopes, in a cultivated field, ½ mile south of Bryan, Cowanshannock Township:

Ap—0 to 6 inches, dark-brown (10YR 3/3) silt loam, pale brown (10YR 6/3) when dry; moderate, medium and fine, granular structure; very friable, nonsticky and nonplastic; many fine roots; 10 percent shale fragments; strongly acid; gradual, wavy boundary.

B1—6 to 13 inches, dark yellowish-brown (10YR 4/4) shaly silt loam; weak, fine, subangular blocky structure; friable, slightly sticky and slightly plastic; common fine roots; 15 percent shale fragments; strongly acid; gradual, wavy boundary.

B2t—13 to 25 inches, dark yellowish-brown (10YR 4/4) shaly silt loam; moderate, medium and coarse, subangular blocky structure; friable, sticky and plastic; few fine roots; thin, discontinuous clay films on ped faces; 20 percent very fine shale fragments; strongly acid; gradual, wavy boundary.

B3t—25 to 38 inches, dark yellowish-brown (10YR 4/4) shaly silt loam; moderate, medium and coarse, subangular blocky structure; firm, slightly sticky and plastic; few fine roots; thin, discontinuous clay films in old pores; 40 percent very fine shale fragments; very strongly acid; diffuse, wavy boundary.

C—38 to 60 inches, dark yellowish-brown (10YR 4/4) very shaly silt loam; massive; firm, slightly sticky and plastic; 80 percent very fine shale fragments; very strongly acid; gradual, wavy boundary.

R—60 inches +, grayish-brown, rippable shale bedrock.

The solum is 36 to 50 inches thick. The depth to rippable bedrock is 40 to 60 inches. Coarse fragments increase with depth. They make up 5 to 15 percent of the Ap and B1 horizons, 10 to 40 percent of the B2t and B3 horizons, and 20 to 80 percent of the C horizon. The Ap horizon is dark grayish brown to brown. The B horizon ranges from yellowish brown to dark yellowish brown and from channery or shaly loam to silty clay loam. The fine earth in the C horizon is silt loam or loam.

Rayne soils are near the moderately deep, well drained Gilpin soils; the shallow, well drained Weikert soils; the deep, moderately well drained Wharton soils; and the deep, somewhat poorly drained Cavode soils. Drainage of the Rayne soils is similar to that of the Hazleton soils. Hazleton soils have a sandy loam B horizon, which the Rayne soils lack.

RnB—Rayne silt loam, 3 to 8 percent slopes. This soil has a profile similar to the one described as representative of the series, but its surface layer is about 2 inches thicker. It is on ridgetops in areas 4 to 35 acres in size. Surface runoff is medium, and the erosion hazard is moderate if the soil is cultivated.

Included with this soil in mapping were a few acres of nearly level Rayne soils and a few areas of Gilpin soils. Also included were some areas of a soil that is medium acid.

This soil is suited to the crops commonly grown in the county and to trees and wildlife. It has few limitations for most uses. Capability unit IIe-1.

RnC—Rayne silt loam, 8 to 15 percent slopes. This soil has the profile described as representative of the series. It is on ridges and benches in irregularly shaped areas that are 4 to 12 acres in size. Surface runoff is medium, and the erosion hazard is moderate if the soil is cultivated.

Included with this soil in mapping were a few areas of Gilpin soils and a few areas of a soil that has a medium acid substratum.

This soil is suited to most of the crops grown in the county and to trees and wildlife. Slope is a limitation for some uses. Capability unit IIIe-1.

RpD—Rayne-Gilpin very stony silt loams, 8 to 25 percent slopes. The soils of this mapping unit are so intermingled that it was neither practical nor feasible to map them separately. The Rayne soil makes up about 50 to 60 percent of the complex. It has a profile similar to the one described as representative of the Rayne series, but its surface layer is overlain by 1 or 2 inches of partially decayed leaf litter. The Gilpin soil makes up about 30 to 40 percent of the complex. It has a profile similar to the one described as representative of the Gilpin series, but its surface layer also is overlain by 1 or 2 inches of partially decomposed leaf litter. Stones cover 2 to 5 percent of the surface of these soils. Sur-

Taken from "Soil Survey of Armstrong County, Pennsylvania," USDA-SCS, February 1977

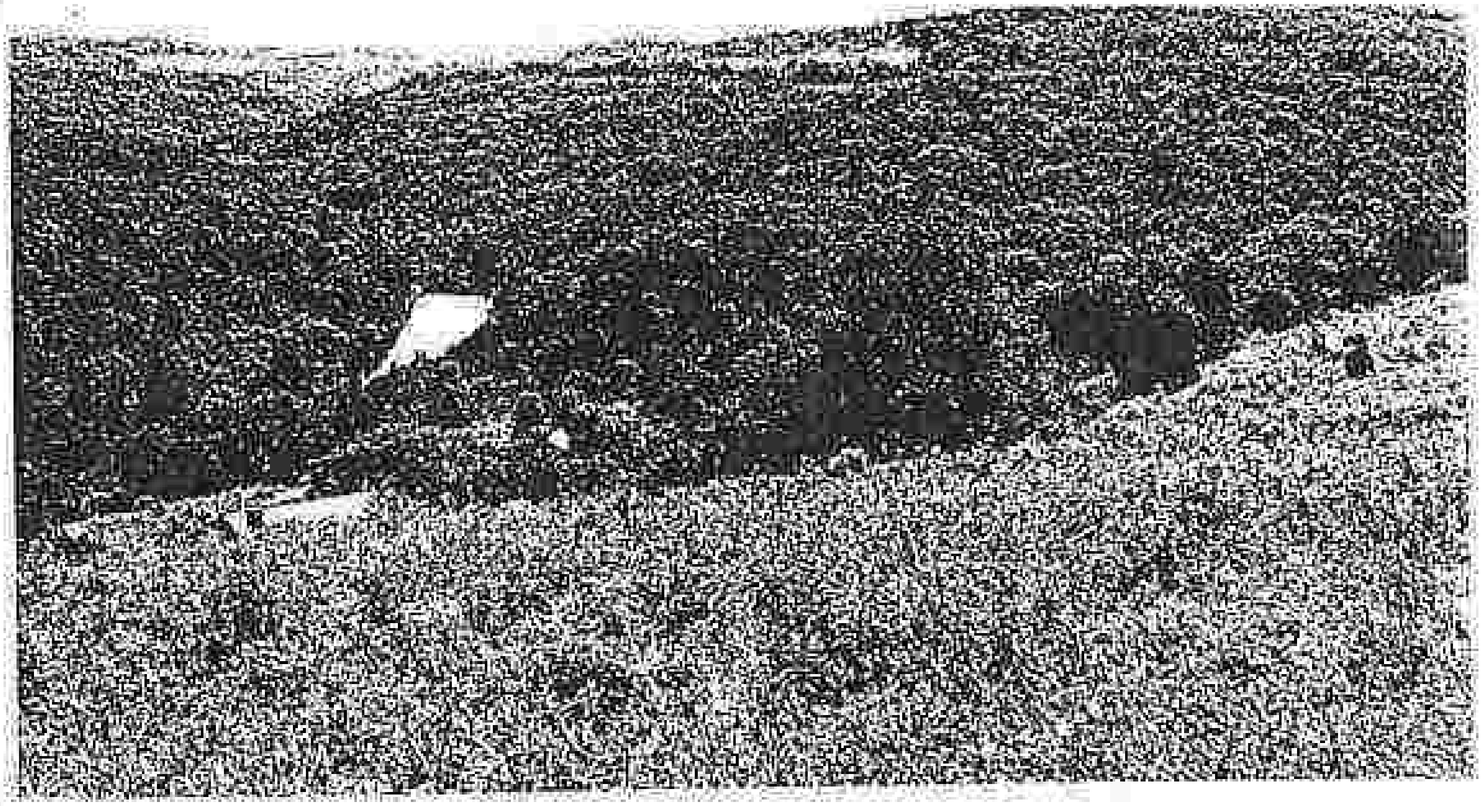


Figure 12.—Typical landscape of Weikert soils. Mahoning Reservoir is on the left.

dark yellowish brown to dark reddish brown. The B horizon ranges from dark reddish brown to weak red or dusky red and is silty clay loam, silty clay, or clay. Mottles with chroma of 2 or less are in the upper 10 inches of the Bt horizon. The C horizon ranges from reddish brown to weak red or yellowish brown.

Vandergrift soils are near the deep, well drained Upshur soils and the deep, moderately well drained Wharton soils. They are similar in drainage to the Cavode and Ernest soils. Unlike Vandergrift soils, Cavode soils have a light brownish-gray and yellowish-brown Bt horizon and Ernest soils have a Bx horizon.

In Armstrong County, Vandergrift soils are mapped only in complexes with Wharton soils.

Weikert Series

The Weikert series consists of shallow, well drained, gently sloping to very steep soils on uplands. These soils formed in material that weathered from interbedded shale, siltstone, and fine-grained sandstone. They occur in areas of complex topography on dissected hillsides and ridges (fig. 12). The native vegetation consists of mixed hardwoods, mainly red oak, scarlet oak, chestnut oak, white oak, red maple, dogwood, and sassafras.

In a representative profile the surface layer is very dark grayish-brown to brown shaly silt loam about 8 inches thick. It is covered by a ¾-inch layer of decaying leaves and twigs. The subsoil is yellowish-brown, friable shaly silt loam about 7 inches thick. The substratum, between depths of 15 and 18 inches, is yellowish-brown very friable, very shaly silt loam. Rippable shale bedrock is at a depth of about 18 inches.

The available moisture capacity is very low, and permeability is moderately rapid. Most areas of these soils are wooded, but some have been cleared and are used for pasture or are idle and reverting to woodland. Shallow depth to bedrock and slope are limitations for most uses.

Representative profile of Weikert shaly silt loam, 8 to 15 percent slopes, in a wooded area 1.1 miles south of Mateer on the west side of State highway 359:

O1—3/4 to 1/4 inch, loose leaf and twig litter.
O2—1/4 inch to 0, decomposed leaf and twig matter.
A1—0 to 4 inches, very dark grayish-brown (10YR 3/2) shaly silt loam; moderate, fine, granular structure; very friable, slightly sticky and slightly plastic; very fine roots; 20 percent shale fragments; very strongly acid; clear, wavy boundary.
A2—4 to 8 inches, brown (10YR 5/3) shaly silt loam; weak, fine, granular structure; friable, slightly sticky and slightly plastic; few fine roots; 40 percent shale fragments; very strongly acid; clear, wavy boundary.
B2—8 to 15 inches, yellowish-brown (10YR 5/4) shaly silt loam; weak, medium, subangular blocky structure; friable, slightly sticky and slightly plastic; few fine and medium roots; 40 percent shale fragments; very strongly acid; clear, wavy boundary.
C—15 to 18 inches, yellowish-brown (10YR 5/4) very shaly silt loam; massive; very friable, slightly sticky and nonplastic; 80 percent shale fragments; very strongly acid; abrupt, wavy boundary.
R—18 inches, grayish-brown, rippable shale bedrock.

Taken from "Soil Survey of Armstrong County, Pennsylvania," USDA-SCS, February 1977

The solum is 8 to 18 inches thick. The depth to bedrock is 12 to 20 inches. Coarse fragments make up 20 to 50 percent of the A and B horizons and as much as 80 percent of the C horizon. In some places there is an Ap horizon that is dark grayish brown to brown. The B horizon ranges from dark yellowish brown to strong brown and from shaly silt loam to channery loam. The C horizon ranges from shaly silt loam to very channery loam.

Weikert soils are near the deep, well drained Rayne and Hazleton soils; the moderately deep, well drained Gilpin soils; the deep, moderately well drained Wharton soils; and the deep, somewhat poorly drained Cavode soils.

WeB—Weikert shaly silt loam, 3 to 8 percent slopes. This soil is similar to the one described as representative of the series, but it has a plow layer about 8 inches thick. It is on ridges and knolls in areas that range from 4 to 12 acres in size. Surface runoff is medium, and the erosion hazard is moderate if the soil is cultivated.

Included with this soil in mapping were a few areas of Gilpin soils and a few small areas of Weikert soils that have coarse fragments of thin, flat sandstone or siltstone in the surface layer.

This soil is suited to crops that tolerate droughtiness and to hay, pasture, trees, and wildlife. Shallowness to bedrock limits this soil for most uses. Capability unit IIIe-2.

WeC—Weikert shaly silt loam, 8 to 15 percent slopes. This soil has the profile described as representative of the series. It is on ridgetops and hillsides in irregularly shaped areas that range from 8 to 25 acres in size. Surface runoff is rapid and the erosion hazard is moderate to high if the soil is cultivated.

Included with this soil in mapping were a few areas of Gilpin soils and a few areas of Weikert soils that have coarse fragments of thin, flat sandstone or siltstone in the surface layer.

This soil is suited to limited cultivation of crops that tolerate some droughtiness and to hay, pasture, trees, and wildlife. Shallow depth to bedrock and slope are limitations for many uses. Capability unit IVe-2.

WkF—Weikert and Gilpin soils, 25 to 70 percent slopes. The soils of this mapping unit occur together and are commonly intermingled, so it was not practical to map them separately. Some mapped areas are entirely Weikert soil or entirely Gilpin soil; others are a mixture of the two. Slope is the dominant characteristic.

Weikert shaly silt loam makes up 50 to 60 percent of the mapping unit. It has a profile similar to the one described as representative of the Weikert series, but generally it is 2 to 5 inches less deep to bedrock. Gilpin channery silt loam makes up 40 to 50 percent of the unit. It has a profile similar to the one described as representative of the Gilpin series, but it has more coarse fragments. Its surface layer is shaly or channery silt loam. Surface runoff is rapid, and the erosion hazard is high if the soils are cultivated.

Included with these soils in mapping were a few areas each of very stony soils and soils that have a gravelly sandy loam profile. Also included were some areas of sandstone, shale, or limestone outcrops and of Hazleton soils.

The soils of this mapping unit are suited to trees and wildlife habitat. Slope and the hazard of erosion are limitations for most uses. Capability unit VIIe-1.

Wharton Series

The Wharton series consists of deep, moderately well drained, gently sloping to moderately steep soils on uplands. These soils formed in material that weathered from acid clay shale interbedded with siltstone. They are mainly on ridgetops, benches, and concave hillsides. The native vegetation consists of mixed hardwoods, mainly red oak, black oak, scarlet oak, and white oak and some black cherry, tulip-poplar, and ash.

In a representative profile, in a cultivated area, the plow layer is dark-brown, friable silt loam about 8 inches thick. The subsoil extends to a depth of about 52 inches. In the upper 3 inches the subsoil is yellowish-brown, friable heavy silt loam; below that, in the next 12 inches, it is yellowish-brown, friable silty clay loam. Next, it is mottled, yellowish-brown, firm silty clay loam for about 8 inches, and in the lowermost part, which is about 21 inches thick, it is mottled, brown, firm silty clay. The substratum, between depths of 52 and 58 inches, is dark grayish-brown, firm very shaly silty clay. Shale bedrock is at a depth of about 58 inches.

The available moisture capacity is high, and permeability is slow. A seasonal water table rises to within 18 to 36 inches of the surface in wet periods. If the soils are adequately drained, they are suited to most of the crops grown in the county. Most areas of these soils have been cleared and are used for crops. A few areas are wooded or are idle and reverting to woodland. The seasonal water table, slow permeability, and slope are limitations for many uses.

Representative profile of Wharton silt loam, 8 to 15 percent slopes, in a cultivated field, 2½ miles northeast of Washington, ½ mile north of intersection of Routes T437 and T416:

Ap—0 to 8 inches, dark-brown (10YR 4/3) silt loam; weak, fine, granular structure; friable, slightly sticky and slightly plastic; strongly acid; abrupt, smooth boundary.

B1—8 to 11 inches, yellowish-brown (10YR 5/4) heavy silt loam; moderate, medium and fine, subangular blocky structure; friable, slightly sticky and slightly plastic; strongly acid; clear, smooth boundary.

B21t—11 to 23 inches, yellowish-brown (10YR 5/6) silty clay loam; moderate, medium and coarse, subangular blocky structure; friable, slightly sticky and slightly plastic; thin discontinuous clay films on ped faces; very strongly acid; gradual, wavy boundary.

B22t—23 to 31 inches, yellowish-brown (10YR 5/4) silty clay loam; many, medium, distinct, pinkish-gray (7.5YR 6/2) mottles; strong, medium and coarse, subangular blocky structure; firm, sticky and plastic; thick continuous clay films on ped faces; very strongly acid; gradual, wavy boundary.

B23t—31 to 52 inches, brown (10YR 5/3) silty clay; many, coarse, prominent, gray (10YR 6/1) mottles; strong, coarse, subangular blocky structure; firm, very sticky and plastic; thick continuous clay films on ped faces; 10 percent coarse fragments; very strongly acid; gradual, wavy boundary.

C—52 to 58 inches, dark grayish-brown (10YR 4/2) very shaly silty clay; massive; firm, slightly sticky and slightly plastic; 55 percent coarse fragments; extremely acid; gradual, wavy boundary.

R—58 inches +, rippable, gray shale bedrock.

The solum is 40 to 60 inches thick. The depth to bedrock ranges from 48 to 72 inches. Coarse fragments make up as much as 15 percent of the Ap, B1, B21t, B22t, and B23t horizons and as much as 90 percent of the C horizon. The

Taken from "Soil Survey of Armstrong County, Pennsylvania," USDA-SCS, February 1977

Ap horizon is dark grayish brown to brown. The matrix of the B horizon ranges from strong brown or brown to yellowish brown and the soil material ranges from clay loam to silty clay.

Wharton soils are on the same landscape as the deep, well-drained Rayne soils; the moderately deep, well-drained Gilpin soils; the shallow, well-drained Weikert soils; the deep, somewhat poorly drained Cavode soils; and the deep, moderately well drained to somewhat poorly drained Vandergrift soils. Wharton soils are similar in drainage to the Ernest and Rainsboro soils, but those soils have a Bx horizon.

WrB—Wharton silt loam, 3 to 8 percent slopes. This soil is similar to the one described as representative of the Wharton series, but its surface layer is about 2 inches thicker. It is on ridgetops and benches in areas that range from 8 to 30 acres. Surface runoff is medium, and the erosion hazard is moderate if the soil is cultivated.

Included with this soil in mapping were a few areas of a soil that has slopes of less than 3 percent and a few areas of a soil that is medium acid or neutral. Some areas of Cavode soils were also included.

This soil is suited to crops that tolerate some wetness and to trees and wildlife habitat. Artificial drainage can make it suitable for a wider range of crops. A seasonal water table and slow permeability are limitations for many uses. Capability unit IIe-3.

WrC—Wharton silt loam, 8 to 15 percent slopes. This soil has the profile described as representative of the series. It is on ridgetops and benches in areas, ranging from 6 to 20 acres, that are irregular in shape. Surface runoff is medium, and the erosion hazard is high if the soil is cultivated.

Included with this soil in mapping were a few areas of a soil that is medium acid or neutral in the substratum.

This soil is suited to crops that tolerate some wetness and to trees and wildlife habitat. Artificial drainage can make it suitable for a wider range of crops. A seasonal water table, slope, and slow permeability are limitations for most uses. Capability unit IIIe-4.

WtB—Wharton-Gilpin silt loams, 3 to 8 percent slopes. The soils of this mapping unit are so intermingled that it was neither practical nor feasible to map them separately. The Wharton soil makes up 50 to 60 percent of the complex. It has a profile similar to the one described as representative of the Wharton series, but it is not so deep to bedrock. The Gilpin soil makes up about 30 to 40 percent of the complex. It has a profile similar to the one described as representative of the Gilpin series, but its surface layer is 6 to 9 inches thick. These soils are on ridgetops and benches. The areas range from 10 to 20 acres and are irregular in shape. Runoff is medium, and the hazard of erosion is moderate if the soils are cultivated.

Included with these soils in mapping were some areas of Cavode soils.

Soils of this complex are suited to most of the crops grown in the county and to trees and wildlife habitat. Artificial drainage can make the Wharton soil suitable for a wider range of crops. A seasonal water table and slow permeability are limitations of the Wharton soil for most uses. Moderate depth to bedrock is the major limitation of the Gilpin soil. Capability unit IIe-3.

WtC—Wharton-Gilpin silt loams, 8 to 15 percent slopes. The soils of this mapping unit are so intermingled that it was neither practical nor feasible to map them separately. The Wharton soil makes up 45 to 55 percent of the complex. It has a profile similar to the one described as representative of the Wharton series, but its surface layer is about 2 inches thinner. The Gilpin soil makes up 35 to 50 percent of the complex. It has a profile similar to the one described as representative of the Gilpin series, but it has a few more coarse fragments. These soils are on ridges and benches in irregularly shaped areas that cover 8 to 35 acres. Runoff is medium, and the erosion hazard is moderate if the soils are cultivated.

Included with these soils in mapping were some small areas of Cavode soils and some very small areas of poorly drained soils around seeps and wet-weather springs.

Soils of this complex are suited to most of the crops grown in the county and to trees and wildlife habitat. Artificial drainage can make the Wharton soil suitable for a wider range of crops. A seasonal water table, slow permeability, and slope are limitations of the Wharton soil for most uses, and moderate depth to bedrock and slope are the major limitations of the Gilpin soil. Capability unit IIIe-4.

WtD—Wharton-Gilpin silt loams, 15 to 25 percent slopes. The soils of this mapping unit are so intermingled that it was neither practical nor feasible to map them separately. The Wharton soils make up 50 to 60 percent of the complex. It has a profile similar to the one described as representative of the Wharton series, but it is not so deep to bedrock. The Gilpin soil makes up about 30 to 40 percent of the complex. It has a profile similar to the one described as representative of the Gilpin series, but it has a few more coarse fragments and is not so deep to bedrock. These soils are on hillsides and foot slopes in areas that are narrow or irregular in shape, covering 12 to 60 acres. Runoff is rapid, and the erosion hazard is high if the soils are cultivated.

Included with these soils in mapping were a few small areas of Weikert soils.

The soils of this complex are suited to limited cultivation of crops and to pasture, hay, trees, and wildlife habitat. Artificial drainage can make the Wharton soil suitable for a wider range of crops. Slope, a seasonal water table, and slow permeability are limitations of the Wharton soil for most uses. Slope and moderate depth to bedrock are the major limitations of the Gilpin soil. Capability unit IVe-3.

WvB—Wharton-Vandergrift complex, 3 to 8 percent slopes. The soils of this mapping unit are so intermingled that it was neither practical nor feasible to map them separately. The Wharton soil makes up 50 to 55 percent of the complex. It has a profile similar to the one described as representative of the Wharton series, but its surface layer is slightly thicker. The Vandergrift soil makes up 35 to 40 percent of the complex and has the profile described as representative of the Vandergrift series. The soils are on ridges and benches, and the areas range from 8 to 35 acres. The surface layer is silt loam or silty clay loam. Runoff is medium, and the hazard of erosion is moderate if the soils are cultivated.

Taken from "Soil Survey of Armstrong County Pennsylvania," USDA-SCS, February 1977

Table B.1--Continued

BRENNER	C/D	BUCKLEY	B/C	CAID	B	CAPUTA	C	CATLIN	B
BRENT	C	BUCKLON	D	CAIRO	D	CARACO	C	CATNIP	D
BRENTON	B	BUCKNER	A	CAJALCO	C	CARALAMPI	B	CATOCTIN	C
BRENTWOOD	B	BUCKNEY	A	CAJON	A	CARBO	C	CATOOSA	B
BRESSER	B	BUCKS	B	CALABAR	D	CARBOL	D	CATSKILL	A
BREVARD	B	BUCKSKIN	C	CALABASAS	C	CARBONDALE	D	CATTARAUGUS	C
BREVORT	B	BUCODA	C	CALAIS	C	CARBURY	B	CAUDLE	B
BREWER	C	BUDD	B	CALAMINE	D	CARDIFF	B	CAVE	D
BREWSTER	D	BUDE	C	CALAPOOYA	C	CARDINGTON	C	CAVE ROCK	A
BREWTON	C	BUDE	C	CALAWAH	B	CARDON	D	CAVO	D
BRICKEL	C	BUELL	A	CALCO	C	CAREY	B	CAVODE	C
BRICKTON	C	BUENA VISTA	B	CALDER	D	CAREY LAKE	B	CAVOUR	D
BRIDGE	C	BUFFINGTON	B	CALDWELL	B	CAREYTOWN	D	CAWKER	B
BRIDGEHAMPTON	B	BUFF PEAK	C	CALEAST	C	CARGILL	C	CAYAGUA	C
BRIDGEPORT	B	BUICK	C	CALEB	B	CARIBE	B	CAYLOR	B
BRIDGER	B	BUKREEK	B	CALERA	C	CARIBEL	B	CAYUGA	C
BRIDGESON	B/C	BULLION	D	CALHI	A	CARIBOU	B	CAZADERO	C
BRIDGEVILLE	B	BULLREY	B	CALHOUN	D	CARLIN	D	CAZADOR	B
BRIDGPORT	B	BULL RUN	B	CALICO	D	CARLINTON	B	CAZENOVIA	B
BRIEDWELL	B	BULL TRAIL	B	CALIFON	C	CARLISLE	A/D	CEBOLIA	C
BRIEF	B	BULLY	B	CALIMUS	B	CARLOTTA	B	CECIL	B
BRIENSBURG		BUMGARD	B	CALITA	B	CARLOW	D	CEDARAN	D
BRIGGS	A	BUNCOMBE	A	CALIZA	A	CARLSBAD	C	CEDAR BUTTE	C
BRIGGSDALE	C	BUNDO	B	CALKINS	C	CARLSBORG	A	CEDAREDGE	B
BRIGGSVILLE	C	BUNEJUG	C	CALLAHAN	C	CARLSON	C	CEDAR MT.	D
BRIGHTON	A/D	BUNKER	D	CALLEGUAS	D	CARLTON	B	CEDARVILLE	B
BRIGHTWOOD	C	BUNSELMEIER	C	CALLINGS	C	CARMI	B	CEDONIA	B
BRILL	B	BUNTINGVILLE	B/C	CALLOWAY	C	CARNEGIE	C	CEDRON	C/D
BRIM	C	BUNYAN	B	CALMAR	B	CARNERO	C	CELAYA	B
BRIMFIELD	C/D	BURBANK	A	CALNEVA	C	CARNEY	D	CELETON	D
BRINKLEY	B	BURCH	B	CALOUSE	B	CAROLINE	C	CELINA	C
BRINEGAR	B	BURCHARD	B	CALPINE	B	CARR	B	CELIO	A
BRINKERTON	D	BURCHELL	B/C	CALVERT	D	CARRISALITOS	D	CELLAR	D
BRISCOT	B	BURDETT	C	CALVERTON	C	CARRIZO	A	CENCOVE	B
BRITE	C	BUREN	C	CALVIN	C	CARSO	D	CENTER	C
BRITTON	C	BURGESS	B	CALVISTA	D	CARSON	D	CENTER CREEK	B
BRIZAM	A	BURGI	B	CAM	B	CARSTAIRS	B	CENTERFIELD	B
BROAD	C	BURGIN	D	CAMAGUEY	D	CARSTUMP	C	CENTERVILLE	D
BROADALBIN	C	BURKE	C	CAMARGO	B	CARTAGENA	D	CENTRALIA	B
BROADAX	B	BURKHARDT	B	CAMARILLO	B/C	CARTECAY	C	CENTRAL POINT	B
BROADBROOK	C	BURLEIGH	D	CAMAS	A	CARUSO	C	CERESCO	B
BROAD CANYON	B	BURLESON	D	CAMASCREEK	B/D	CARUTHERSVILLE	B	CERRILLOS	C
BROADHEAD	C	BURLINGTON	A	CAMBERN	C	CARVER	A	CERRO	C
BROADHURST	D	BURMA		CAMBRIDGE	C	CARVILE	C	CHACRA	C
BROCA	D	BURMESTER	C	CAMDEN	B	CARYVILLE	B	CHAFFEE	C
BROCKLISS	C	BURNAC	C	CAMERON	D	CASA GRANDE	C	CHAGRIN	B
BROCKMAN	C	BURNETTE	D	CAMILLUS	B	CASCADE	C	CHAIX	B
BROCKPORT	D	BURNHAM	D	CAMP	B	CASCAJO	B	CHALFONT	C
BROCKTON	D	BURNSIDE	B	CAMPBELL	B/C	CASCILLA	B	CHALMERS	C
BROCKWAY	B	BURNSVILLE	B	CAMPHORA	B	CASCO	B	CHAMA	B
BRODY	C	BURNT LAKE	B	CAMPIA	B	CASE	B	CHAMBER	C
BROGAN	B	BURRIS	D	CAMPO	C	CASEBIER	D	CHAMBERINO	C
BROGDON	B	BURT	D	CAMPECHE	B/C	CASEY	C	CHAMISE	B
BROLLIAR	D	BURTON	B	CAMPSPASS	C	CASHEL	C	CHAMOKANE	B
BROMO	B	BUSE	B	CAMPUS	B	CASHION	D	CHAMPION	B
BRONAUGH	B	BUSHNELL	C	CAMRODEN	C	CASHMERE	B	CHANCE	B/D
BRONCHO	B	BUSHVALLEY	D	CANA	C	CASHMONT	B	CHANDLER	B
BRONSON	B	BUSTER	C	CANAAN	C/D	CASIAC	A	CHANEY	C
BRONTE	C	BUTANO	C	CANADIAN	B	CASITO	C	CHANNAHON	C
BROOKE	C	BUTLER	D	CANADICE	D	CASPAR	B	CHANNING	B
BROOKFIELD	B	BUTLERTOWN	C	CANANDAIGUA	D	CASPIANA	B	CHANTA	B
BROOKINGS	B	BUTTE	C	CANASERAGA	C	CASS	A	CHANTIER	D
BROOKLYN	D	BUTTERFIELD	C	CANAVERAL	C	CASSADAGA		CHAPIN	C
BROOKSIDE	C	BUXIN	D	CANDELERO	C	CASSIA	C	CHAPMAN	
BROOKSTON	B/D	BUXTON	C	CANE	C	CASSOLARY	B	CHAPPELL	B
BROOKSVILLE	D	BYARS	D	CANEADEA	D	CASSVILLE		CHARD	B
BROSELEY	B	BYRON	B	CANEEK	B	CASTAIC	C	CHARITON	D
BROSS	B			CANEL	B	CASTALIA	C	CHARITY	D
BROUGHTON	D	CABALLO	C	CANELO	C	CASTANA	B	CHARLESTON	C
BROWARD	C	CABARTON	C	CANEY	C	CASTELL	C	CHARLEVOIX	B
BROWNELL	B	CABBA	C	CANEYVILLE	C	CASTILE	B	CHARLOS	A
BROWNFIELD	A	CABBART	C	CANFIELD	C	CASTINE	C	CHARLOTTE	A/D
BROWNLEE	B	CABEZON	D	CANISTEO	C	CASTLE	D	CHARLTON	B
BROYLES	C	CABIN	C	CANNINGER	D	CASTLE VALLEY	D	CHASE	C
BRUCE	D	CABINET	C	CANOE	B	CASTNER	C	CHASEBURG	B
BRUIN	C	CABLE	D	CANONCITO	C	CASTO	C	CHASEVILLE	A
BRUNEEL	B	CABO ROJO	C	CANOVA	B/D	CASTRO	C	CHASKA	C
BRUNO	A	CABOT	C	CANTON	B	CASTROVILLE	B	CHASTAIN	D
BRUNT	C	CACAPON	B	CANTIL	B	CASUSE	C	CHATQUAN	B
BRUSETT	B	CACHE	D	CANTUA	B	CASWELL	D	CHATFIELD	
BRUSH		CACIQUE	D	CANUTIO	B	CATALINA	B	CHATHAM	B
BRUSSETT	B	CADDO	D	CANYON	D	CATALPA	C	CHATSWORTH	D
BRYAN	A	CADEVILLE	D	CAPAC	B	CATANO	A	CHAUNCEY	C
BRYCAN	B	CADMUS	B	CAPAY	D	CATARINA	D	CHAVIES	B
BRYCE	D	CADORA	C	CAPE	D	CATAULA	C	CHAWANAKEE	C
BUCAN	D	CADWELL	C	CAPE FEAR	D	CATAWBA	B	CHEADLE	C
BUCHANAN	C	CAGEY	C	CAPERS	D	CATH	D	CHECKETT	D
BUCHENAU	C	CAGUABO	D	CAPILLO	D	CATHCART	B	CHEDEHAP	B
BUCHER	C	CAHABA	B	CAPLES	C	CATHEDRAL	C	CHEEKTOWAGA	D
BUCKINGHAM		CAHILL	B	CAPPS	C	CATHERINE	B/D	CHEESMAN	B
BUCKLAND	C	CAHONE	C	CAPSHAW	C	CATHRO	D	CHEHALEM	C
BUCKLEBAR	B	CAHTO	C	CAPULIN	C	CATLETT	C/D	CHEHALIS	B

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January 1971

Taken from Technical Release No. 55, "Urban Hydrology for Small Watersheds," USDA-SCS, October 1981

EASTPORT	A
EATONTOWN	
EAUGALLIE	B/D
EBA	C
EBBERT	D
EBON	B
EBENEZER	C
ECCLES	B
ECHARD	C
ECHLIN	B
ECKLEY	B
ECKMAN	B
ECKRANT	D
ECTOR	C
EDALGO	C
EDDS	B
EDDY	C
EDEN	C
EDENTON	C
EDENVALE	D
EDGAR	B
EDGECUMBE	B
EDGELEY	C
EDGEMONT	B
EDGEWATER	C
EDGEWICK	B
EDGEWOOD	B
EDGINGTON	C
EDINA	D
EDINBURG	C
EDISON	B
EDISTO	C
EDITH	B
EDLOE	B
EDMONDS	C
EDMORE	D
EDMUND	C
EDNA	D
EDNEYVILLE	B
EDOM	C
EDSON	C
EDWARDS	B/D
EEL	C
EFFINGTON	C
EFMON	A
EGAM	C
EGAN	D
EGBERT	B/D
EGELAND	B
EGGLESTON	B
EGNAR	C
EICKS	C
EIFORT	
EKAH	C
EKALAKA	B
ELAM	A
ELBERT	D
ELBURN	B
ELCO	B
ELD	B
ELDER	B
ELEVEN POINTS	D
ELFKIN	B
ELDORADO	B
ELDRIDGE	C
ELEPHANT	C
ELFKIN	D
ELFRIDA	B
ELIJAH	C
ELIOAK	C
ELK	B
ELKADER	B
ELKCREEK	C
ELK HOLLOW	B
ELKHORN	A
ELKINS	D
ELKINSVILLE	B
ELKMOUND	C
ELK MOUNTAIN	B
ELKTON	D
ELLABELLE	B/D
ELLEDGE	D
ELLERY	D
ELLETT	D
ELLIBER	A
ELLICOTT	A
ELLINGTON	B
ELLINOR	B
ELLIOTT	C
ELLIS	D
ELLISFORDE	C

ELLISON	A
ELLIAM	C
ELLSBERRY	
ELLSWORTH	C
ELMA	B
ELMDALE	B
ELMIRA	A
ELMO	C
ELMONT	B
ELMORE	B
ELMWOOD	C
ELNORA	B
ELOIKA	B
ELPAN	D
EL PECO	C
EL RANCHO	B
ELRED	B/D
ELRED	B/D
ELS	A
ELSAH	B
ELSINBORO	B
ELSMERE	A
ELSO	D
EL SOLYO	C
ELSTON	B
ELTOPIA	B
ELTREE	B
ELTSAC	D
ELWHA	B
ELWOOD	C
ELY	B
ELYSIAN	B
ELZINGA	B
EMBDEN	B
EMDENT	C
EMER	C
EMERALD	B
EMERSON	B
EMIDA	D
EMIGRANT	B
EMIGRATION	D
EMILY	B
EMLIN	B
EMMA	C
EMMERT	A
EMPEE	D
EMRICKS	C
EMPORT	B
EMPEY	B
EMPEYVILLE	C
EMPIRE	C
EMRICK	B
ENCE	B
ENCIERRO	D
ENCINA	B
ENDERS	C
ENDICOTT	D
ENDIT	B
ENFIELD	B
ENGLE	B
ENGLESTOF	B
ENGLEWOOD	C
ENGLUND	D
ENNIS	B
ENOCHVILLE	B/D
ENOLA	B
ENON	C
ENOS	B
ENSENBURG	D
ENSIGN	D
ENSLEY	D
ENSTROM	B
ENTERPRISE	B
ENTIAT	D
ENUMCLAW	C
EPHRAIM	C
EPHRATA	B
EPOUFETTE	D
EPPING	D
EPSIE	D
ERA	B
ERAM	C
ERCK	C
ERIC	B
ERIN	C
ERNEST	C
ERRAMOUSPE	D
ESCABOSA	B
ESCALANTE	B
ESCAMBIA	C
ESCONDIDO	C

ESMOND	B
ESPARTO	B
ESPIL	D
ESPINAL	A
ESQUATZEL	B
ESS	B
ESSEN	C
ESSEX	C
ESSEXVILLE	D
ESTACADO	B
ESTELLINE	B
ESTER	D
ESTERBROOK	B
ESTHERVILLE	B
ESTO	C
ESTRELLA	B
ETHAN	B
ETHETE	B
ETHRIDGE	C
ETIL	A
ETNA	
ETCHEN	B
ETOWAH	B
ETTA	C
ETTER	B
ETTERSBURG	B
ETTRICK	D
EUBANKS	B
EUDORA	B
EUFAULA	A
EUREKA	D
EUSTIS	A
EUTAW	D
EVANGELINE	C
EVANS	B
EVANSTON	B
EVARD	B
EVART	D
EVENDALE	
EVERETT	B
EVERGLADES	A/D
EVERLY	B
EVERMAN	C
EVERSON	C
EVESBORO	A
EWA	B
EWALL	A
EWINGSVILLE	B
EXCHEQUER	D
EXETER	C
EXLINE	D
EXRAY	D
EXUM	C
EYERBOW	D
EYRE	B
FABIUS	B
FACEVILLE	B
FAHEY	B
FAIM	C
FAINES	A
FAIRBANKS	B
FAIRDALE	B
FAIRFAX	B
FAIRFIELD	B
FAIRHAVEN	B
FAIRMOUNT	D
FAIRPORT	C
FAJARDO	C
FALAYA	C
FALCON	C
FALFURRIAS	A
FALK	B
FALKNER	C
FALL	B
FALLBROOK	B
FALLON	C
FALLSBURG	
FALLSINGTON	D
FANCHER	C
FANG	C
FANNIN	B
FANNO	C
FANU	C
FARADAY	C
FARALLONE	B
FARAWAY	D
FARBER	B
FARISITA	B
FARLAND	B
FARMINGTON	C/D
FARNHAM	B

FARNUM	B
FARRAGUT	C
FARRAR	B
FARRELL	B
FARRENBURG	B
FARROT	C
FARSON	B
FARWELL	C
FATIMA	B
FATTIG	C
FAUNCE	A
FAUQUIER	C
FAWCETT	C
FAWN	B
FAXON	D
FAYAL	C
FAYETTE	B
FAYETTEVILLE	B
FAYWOOD	C
FE	D
FEDORA	B
FELDA	B/D
FELIDA	B
FELLOWSHIP	D
FELT	B
FELTA	C
FELTHAM	A
FELTON	B
FELTONIA	B
FENCE	B
FENDALL	C
FENWOOD	B
FERDELFORD	C
FERDIG	C
FERGUS	B
FERGUSON	B
FERNANDO	C
FERNDALE	B
FERNLEY	C
FERNOW	B
FERNPOINT	C
FERNWOOD	B
FERRIS	D
FERRON	B
FERTALINE	D
FESTINA	B
FETTIC	D
FIANDER	C
FIDEA	C
FIDALGO	C
FIDDLETOWN	C
FIDDYMENT	C
FIELDING	B
FIELDON	B
FIELDSON	A
FIFE	B
FIFER	D
FILLMORE	D
FINCASTLE	C
FINGAL	C
FINLEY	B
FIRESTEEL	B
FIRCHELL	B
FIRMAGE	B
FIRO	C
FIRTH	B
FISH CREEK	B
FISHERS	B
FISHHOOK	D
FISHKILL	
FITCH	A
FITCHVILLE	C
FITZGERALD	B
FITZHUGH	B
FIVE OUT	B
FIVEMILE	B
FIVES	B
FLACE	A
FLAGSTAFF	C
FLAK	B
FLAMING	B
FLAMINGO	D
FLANAGAN	B
FLANDREAU	B
FLASHER	A
FLATHEAD	A
FLAT HORN	B
FLATTOP	D
FLAXTON	A
FLEAK	A
FLECHADO	C
FLEETWOOD	

FLEISCHMANN	D
FLEMING	C
FLETCHER	B
FLOKE	D
FLOM	C
FLOMATON	A
FLORENCE	C
FLORIDANA	B/D
FLORISSANT	C
FLOWELL	C
FLOWEREE	B
FLOYD	B
FLUSHING	
FLUVANNA	C
FLYGARE	B
FLYNN	B
FOARD	D
FOGELSVILLE	B
FOLA	B
FOLEY	D
FONDA	D
FONDIS	C
FONTAL	C
FONTREEN	B
FORDANO	D
FORBES	B
FORD	D
FORDNEY	A
FORDVILLE	B
FORE	D
FORELAND	C
FORELLE	B
FORESMAN	B
FORESTDALE	D
FORESTER	C
FORGAY	A
FORMAN	B
FORNEY	D
FORREST	C
FORSEY	C
FORSGREN	C
FORT COLLINS	B
FORT DRUM	C
FORT LYON	B
FORT MEADE	A
FORT MOTT	A
FORT PIERCE	C
FORT ROCK	C
FORTUNA	D
FORTWINGATE	C
FORWARD	C
FOSHOME	B
FOSSUM	B
FOSTER	B/C
FOSTORIA	B
FOUNTAIN	D
FOURLOG	D
FOURMILE	B
FOUR STAR	B/C
FOUTS	B
FOX	B
FOXCREEK	C
FOXMOUNT	C
FOXOL	D
FOXPARK	B
FOXTON	C
FRAILEY	C
FRAM	B
FRANCIS	A
FRANK	D
FRANKFORT	D
FRANKIRK	C
FRANKLIN	B
FRANKSTOWN	B
FRANKTOWN	D
FRANKVILLE	B
FRATERNIDAD	D
FRAZER	C
FRED	C
FREDENSBORG	C
FREDERICK	B
FREDON	C
FREDONIA	C
FREDRICKSON	C
FREEBURG	C
FREECE	D
FREEHOLD	B
FREEL	B
FREEMAN	C
FREEMANVILLE	B
FREEON	B
FREER	C

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Table B.1--Continued

FREESTONE	C	GASCONADE	D	GLENFIELD	D	GRANGER	C	GUNN	B
FREEZENER	B	GAS CREEK	C	GLENFORD	C	GRANGEVILLE	B/C	GUNTER	A
FREMONT	C	GASKELL	C	GLENHALL	B	GRANILE	B	GURABO	D
FRENCH	C	GASS	D	GLENMAR	B	GRANO	D	GURNEY	C
FRENCHTOWN	D	GASSET	D	GLENMORA	C	GRANT	B	GUSTAVUS	B
FRENEAU		GATESBURG	A	GLENNALLEN	C	GRANTSBURG	C	GUSTIN	C
FRESNO	C	GATEVIEW	B	GLENOMA	B	GRANTSDALE	A	GUTHRIE	D
FRIANA	D	GATEWAY	C	GLENROSE	B	GRANVILLE	B	GUYTON	D
FRIANT	D	GATEWOOD	D	GLENSTED	D	GRAPEVINE	C	GWIN	D
FRIDLO	C	GAULDY	B	GLENTON	B	GRASMERE	B	GWINNETT	B
FRIEDMAN	B	GAVINS	C	GLENVIEW	B	GRASSNA	B	GYNER	C
FRIES	D	GAVIOTA	D	GLENVILLE	C	GRASSY BUTTE	A		
FRIO	B	GAY	D	GLIDE	B	GRATZ	C	HACCKE	C
FRIZZELL	C	GAYLORD	B	GLIKON	B	GRAVDEN	C	HACIENDA	C
FROBERG	D	GAYNOR	C	GLORIA	C	GRAVE	B	HACK	B
FROHMAN	C	GAYVILLE	B	GLOUCESTER	B	GRAVITY	C	HACKERS	B
FROHNHOFER	C	GAZELLE	D	GLOVER	C/D	GRAYCALM	A	HACKETTSTOWN	B
FRONTON	D	GAZOS	B	GLYNDON	B	GRAYFORD	B	HADLEY	B
FROST	D	GEARHART	A	GLYNN	C	GRAYLING	A	HADO	B
FRUITA	B	GEARY	B	GOBLE	C	GRAYLOCK	B	HAGEN	B
FRUITLAND	B	GEE	B	GODDARD	B	GRAYPOINT	B	HAGENBARTH	B
FRYE	D	GEEBURG	C	GODE	D	GRAYS	B	HAGENER	A
FUEGO	C	GEER	C	GODECKE	C	GREAT BEND	B	HAGER	C
FUERA	C	GEFO	A	GODFREY	C	GREELEY	B	HAGERMAN	B
FULDA	C	GELKIE	B	GODWIN	D	GREEN BLUFF	B	HAGERSTOWN	C
FULLERTON	B	GEM	C	GOEGLEIN	C	GREEN CANYON	D	HAGGA	B
FULMER	B/D	GEMID	C	GOESSEL	D	GREENCREEK	B	HAIG	C
FULSHEAR	C	GEMSON	C	GOFF	C	GREENDALE	B	HAIKU	B
FULTON	D	GENESEE	B	GOGEBIC	B	GREENFIELD	B	HAILMAN	B
FUQUAY	A	GENEVA	C	GOLBJIN	C	GREENHORN	D	HAINES	B/C
FURNISS	B/D	GENOA	D	GOLCONDA	D	GREENLEAF	B	HAIRE	C
FURY	B/D	GENOLA	B	GOLDENDALE	B	GREENOUGH	C	HALAWA	B
		GEORGEVILLE	B	GOLDFIELD	B	GREENPORT		HALDER	C
GAASTRA	C	GEORGIA	B	GOLDHILL	B	GREEN RIVER	B	HALE	B
GABALDON	C	GERALD	D	GOLDMAN	C	GREENSBORO		HALEIWA	B
GABICA	D	GERBER	D	GOLDRIDGE	B	GREENSON	C	HALEY	B
GACEY	D	GERIG	B	GOLDRUN	A	GREENTON	C	HALF MOON	B
GADDES	C	GERING	B	GOLDSBORO	C	GREENVILLE	B	HALFORD	A
GADES	C	GERLAND	C	GOLDSTON	C	GREENWATER	A	HALFWAY	D
GADSDEN	D	GERMANIA		GOLDSTREAM	D	GREENWICH	B	HALII	B
GAGE		GERMANY	B	GOLDVALE	C	GREENWOOD	D	HALIIMAILE	B
GAGEBY	B	GESTRIN	B	GOLDVEIN	C	GREER	C	HALIS	B
GAGETOWN	C	GETTA	C	GOLIAD	C	GREGORY	A	HALL	B
GAHEE	B	GETTYS	C	GOLLAHER	A	GRELL	C	HALLECK	B
GAINES	C	GEYSEN	D	GOMEZ	B	GRENADA	C	HALL RANCH	C
GAINESVILLE	A	GHENT	C	GONVICK	B	GRENVILLE	B	HALLVILLE	A
GALATA	D	GIBBLER	C	GOOCH	D	GRESHAM	C	HALSEY	D
GALE	B	GIBSON	B	GOODALE	C	GREWINGK	C	HAMAKUAPOKO	B
GALEN	B	GINOS	D	GOODING	C	GREYBACK	B	HAMAN	B
GALENA	C	GIBBSTOWN	A	GOODINGTON	C	GREYBULL	C	HAMAR	B
GALEPPI	C	GIFFIN	C	GOODLOW	B	GREYCLIFF	C	HAMBLEN	C
GALESTOWN	A	GIFFORD	C	GOODMAN	B	GRIFFY	B	HAMBRIGHT	D
GALEY	B	GILA	C	GOODRICH	B	GRIGSTON	B	HAMBURG	B
GALISTEO	D	GILBY	B	GOODSPRINGS	D	GRIPSTAD	B	HAMEL	C
GALLAGHER	B	GILCHRIST	B	GOOSE CREEK	B	GRISWOLD	B	HAMERLY	C
GALLATIN	A	GILCREST	B	GOOSE LAKE	C	GRIVER	C	HAMILTON	A
GALLEGOS	B	GILEAD	C	GOOSMUS	B	GRIZZLY	C	HAMLET	B
GALLINA	C	GILES	B	GORDO	C	GROGAN	B	HAMLIN	B
GALLION	B	GILFORD	C/D	GORE	D	GROSECLOSE	C	HAMPDEN	
GALVA	B	GILHOULY	B	GORGONIO	A	GROSS	C	HAMPSHIRE	C
GALVESTON	A	GILLESPIE	C	GORHAM	B	GROTON	A	HAMPTON	C
GALVIN	C	GILLIAM	C	GORIN	C	GROVE	A	HAMTAH	C
GAMBLER	A	GILLIGAN	B	GORING	C	GROVELAND	B	HANA	A
GANNETT	D	GILLS	C	GORMAN	B	GROVER	B	HANALEI	C
GANSNER	D	GILMORE	D	GORUS	A	GROVETON	B	HANAMAULU	A
GAPO	D	GILPIN	C	GORZELL	B	GRUBBS	D	HANCEVILLE	B
GAPPMAYER	B	GILROY	C	GOSHEN	B	GRULLA	D	HAND	B
GARA	B	GILSON	B	GOSHUTE	D	GRUMMIT	C	HANFORD	B
GARBER	B	GILT EDGE	C	GOSPORT	C	GRUNDY	C	HANEY	B
GARBUTT	B	GINAT	D	GOTHAM	A	GRUVER	C	HANGAARD	C
GARCENO	C	GINGER	C	GOTHARD	D	GRYGLA	C	HANGEN	B
GARDENA	B	GINI	B	GOULD	C	GUADALUPE	B	HANIPOE	B
GARDINER	A	GINSER	C	GOURD	C	GUAJE	A	HANKINS	C
GARDNERS FORK	B	GIRD	A	GOULDING	D	GUALALA	D	HANKS	B
GARDNERVILLE	D	GIVEN	C	GOVAN	C	GUAMANI	B	HANLY	A
GARDONE	A	GLADDEN	A	GOVE	B	GUANAJIBO	C	HANNA	C
GAREY	C	GLADSTONE	B	GOWEN	B	GUANICA	D	HANOVER	C
GARFIELD	C	GLADWIN	A	GRACE	B	GUAYABO	B	HANS	C
GARITA	C	GLAMIS	C	GRABLE	B	GUAYABOTA	D	HANSEL	C
GARLAND	C	GLANN	B/C	GRACEMONT	B	GUAYAMA	D	HANSKA	C
GARLET	A	GLASGOW	C	GRACEVILLE	B	GUBEN	B	HANSON	A
GARLOCK	C	GLEAN	B	GRADY	D	GUCKEEN	C	HANTHO	B
GARMON	C	GLEASON	C	GRAFTON	B	GUELPH	B	HANTZ	D
GARMORE	B	GLEN	B	GRAHAM	D	GUENOC	C	HAP	B
GARNER	D	GLENBERG	A	GRAIL	C	GUERNSEY	C	HAPGOOD	B
GARO	D	GLENBROOK	D	GRAMM	B	GUERNERO	C	HAPNEY	C
GARR	D	GLENCOE	D	GRANATH	B	GUEST	D	HARBORD	B
GARRARD	B	GLENHALE	B	GRANBY	A/D	GUIN	A	HARBOURTON	
GARRETSON	B	GLENDALE	B	GRANDE RONDE	D	GULLEN	B	HARCO	B
GARRETT	B	GLENDIVE	B	GRANDFIELD	B	GULKANA	B	HARDEMAN	B
GARRISON	B	GLENDORA	C	GRANDVIEW	C	GUNBCOT	C	HARDESTY	B
GARWIN	C	GLENELG	B	GRANER	C	GUNBARREL	A	HARDING	D

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Taken from Technical Release No. 55, "Urban Hydrology for Small Watersheds," USDA-SCS, October 1981

Table B .1--Continued

PEGLER	D	PIE CREEK	C	POE	B/C	PREBLE	C	QUINN	D
PEGHAM	B	PIEMIAN	A	POGANEAB	C	PRENTISS	C	QUINNEY	C
PEKIN	C	PIERPONT	C	POGUE	B	PRESQUE ISLE	B	QUINICH	
PELHAM	B/D	PIERRE	D	POHAKUPU	A	PRESTO	A	QUITMAN	C
PELIC	C	PIIHONUA	A	POINSETT	B	PRESTON	A	QUONSET	A
PELLA	D	PIKE	B	POINT	B	PREWITT	C		
PELONA	C	PILCHUCK	A	POINT ISABEL	C	PREY	D	RABER	C
PEMBERTON	A	PILGRIM	B	POJOAQUE	B	PRICE	C	RABEY	A
PEMBINA	C	PILOT	B	POKEGEMA	B	PRIDA	D	RABIDEUX	B
PEMBROKE	B	PILOT ROCK	C	POKER	B	PRIDHAM	D	RABUN	B
PENA	B	PIMA	B	POLAND		PRIETA	C	RACE	D
PENCE	B	PINAL	D	POLAR	B	PRIMEAUX	C	RACHERT	D
PENDEN	B	PINALENO	B	POLATIS	C	PRINCHAR	B	RACINE	B
PEND OREILLE	B	PINATA	C	POLE	A	PRINCETON	B	RACOON	D
PENDROY	D	PINAVETES	A	POLEBAR	C	PRINEVILLE	C	RAD	C
PENISTAJA	B	PINCHER	C	POLELINE	B	PRING	B	RADFORD	B
PENITENTE	B	PINCKNEY	C	POLEO	C	PRINS	C	RADLEY	C
PENN	C	PINCONNING	D	POLEY	C	PROCTOR	B	RADNOR	D
PENNANT	C	PINCUSHION	B	POLICH	C	PROGRESSO	C	RAFAEL	D
PENNINGTON	B	PINEDA	B/D	POLLARD	C	PROMISE	D	RAGLAN	D
PENNSUCO	C	PINEDALE	B	POLLASKY	C	[illegible]	D	RAGNAR	B
PENO	C	PINEGUEST	B	POLLY	B	PROMONTORY	B	RAGO	C
PENOYER	C	PINELLAS	B/D	POLO	B	PRONG	C	RAGSDALE	B/D
PENROSE	D	PINETOP	C	POLSON	C	PROSPECT	B	RAGTOWN	D
PENTHOUSE	D	PINEVILLE	B	POLVADERA	C	PROSPER	B	RAHM	C
PENTZ	D	PINEY	C	POMAT	C	PROSSER	C	RAIL	C/D
PENWOOD	A	PINICON	B	POMELLO	C	PROTIVIN	C	RAINBOW	C
PEOGA	C	PINKEL	C	POMPANO	A/D	PROUT	C	RAINEY	B
PEOH	C	PINKSTON	B	POMPACKIE	C	PROVIDENCE	C	RAINS	B/D
PEONE	B/C	PINNACLES	C	POMPTON	B	PROVO	C	RAINSBORO	C
PEOTONE	C	PINO	C	POMROY	B	PROVO BAY	D	RAKE	D
PEPOON	D	PINUCA	C	PONCENA	D	[illegible]	B	RALSEN	B/C
PEQUEA	C	PINOLE	B	PONCHA	A	PTARMIGAN	B	RAMADA	C
PERCHAS	D	PINON	C	POND	B/C	PUAULU	A	RAMADERO	B
PERCIVAL	C	[illegible]	D	POND CREEK	B	PUCHYAN	A	RAMBLER	B
PERELLA	C	PINTAS	C	PONDILLA	A	PUDDLE	D	RAMELLI	C
PERHAM	C	PINTLAR	B	PONIL	D	PUERCO	D	RAMIRES	D
PERICO	B	PINTO	C	PONTOTOC	B	PUETT	C	RAMMEL	C
PERKINS	C	PINTURA	A	PONZER	D	PUGET	C	RAMO	C
PERKS	A	PINTWATER	C	[illegible]	A	PUGSLEY	B	RAMONA	B
PERLA	C	PIOPOLIS	D	POOLE	B/D	PUHI	A	RAMPART	B
PERMA	A	PIPER	B/C	POOLER	D	PUHIMAU	D	RAMPARTAN	A
PERMANENTE	C	PIROUETTE	C	POOPAA	B	PULASKI	B	RAMSEY	D
PERRIN	B	PISGAH	C	POPE	B	PULEHU	B	RAMSHORN	B
PERRINE	D	PISHKUN	B	POPPLETON	A	PULLMAN	D	RANCE	C
PERRIOT	D	PISTAKEE	B	POQUONOCK	C	PULS	D	RANCHERIA	B
PERRY	D	PIT	C	PORRETT	B/D	PULSIPHER	D	RAND	B
PERRYVILLE	B	PITTMAN	C	PORT	B	PULTNEY	C	RANDADO	C
PERSAYO	D	PITTSFIELD	B	PORTAGEVILLE	D	PUMPER	C	RANDALL	D
PERSHING	C	PITTSTOWN	C	PORTALES	C	PUNA	A	RANDOLPH	D
PERSIS	B	PITTWOOD	B	PORT BYRON	B	PUNALUU	D	RANGE	C
PERT	D	PLACENTIA	D	PORTERS	B	PUNOHU	A	RANGER	D
PERU	C	PLACERITOS	C	PORTERVILLE	D	PURDAM	C	RANIER	C
PESCADERO	C/D	PLACID	B/D	PORTHILL	C	PURDY	D	RANKIN	C
PESET	C	PLACK	D	PORTING	C	PURGATORY	D	RANTOUL	D
PESHASTIN	B	PLAINFIELD	A	PORTLAND	D	PURNER	D	RANTHAN	B
PESO	C	PLAINVIEW	C	PORTNEUF	B	PURSLEY	B	RAPELJE	C
PETEETNEET	D	PLAISTED	C	PORTOLA	C	PURVES	D	RAPHO	B
PETERBORO	B	PLANO	B	PORTSMOUTH	D	PUSTOI	A	RAPIDAN	B
PETERS	D	PLATA	B	POSANT	C	PUTNAM	D	RARDEN	C
PETOSKEY		PLATEA	C	POSEY	B	PUUKALA	D	RARICK	B
PETRIE	D	PLATEAU	D	POSITAS	D	PUUONE	C	RARITAN	C
PETROLIA	D	PLATNER	C	POSKIN	C	PUU OO	A	RASBAND	B
PETTONS	C	PLATO	C	POSOS	C	PUU OPAE	D	RASSET	B
PEWAMO	B/D	PLATTE	C	POST	D	PUU PA	B	RATHBUN	C
PEYTON	B	PLATTEVILLE	B	POTAMO	C	PUYALLUP	B	RATLIFF	B
PHAGE	B	PLAZA	D/C	POTLATCH	C	PYLE	A	RATON	C
PHARO	D	PLEASANT	C	POTRATZ	C	PYLON	D	RATTLER	B
[illegible]	D	PLEASANT GROVE	B	POTSDAM	C	PYOTE	A	RAUB	B
PHEBA	C	PLEASANTON	B	[illegible]	C	PYRAMID	D	RAUVILLE	D
PHEENEY	B	PLEASANT VALE	B	POTTER	C	PYRMONT	D	RAUZI	B
PHELAN	B	PLEASANT VIEW	B	POTTS	B			RAVALLI	C
PHELPS	B	[illegible]	C	[illegible]	B	QUAKER	C	RAVENDALE	D
PHIFERSON	B	PLEER	C	POULTNEY	B	QUAKERTOWN	C	RAVENNA	C
PHILBON	B/D	PLEINE	C	POVERTY	A	QUAMBA	D	RAVOLA	B
PHILLIPS	C	PLEVNA	C	POWDER	B	QUANAH	B	RAWAH	B
PHILLIPSBURG	A	PLOME	C	POWDERHORN	C	QUANDAHL	B	RAWHIDE	D
PHILO	B	PLOVER	B	POWELL	C	QUARLES	C	RAWSON	B
[illegible]	D	PLUMAS	D	POWER	B	QUARTZBURG	C	RAY	B
PHIPPS	C	PLUMMER	B/D	POWHITE	C	QUATAMA	C	RAYADO	C
PHOEBE	B	PLUSH	B	POWLEY	D	QUAY	C	RAYENOUF	B
PHOENIX	D	PLUTH	B	POWWATKA	C	QUEBRADA	C	RAYMONDVILLE	D
PIASA	D	PLUTOS	C	POY	D	QUEETS	B	RAYNE	B
PICACHO	C	PLYMOUTH	A	POYGAN	D	QUEMADO	C	RAYNESFORD	B
PICAYUNE	B	POALL	C	POZO	C/D	QUENZER	D	RAYNHAM	C
PICKAWAY	C	POARCH	B	POZO BLANCO	B	QUICKSELL	C	RAYNOR	D
PICKENS	D	POCALLA	A	PRAG	C	QUIGLEY	B	RAZOR	C
PICKETT	B	POCATELLO	B	PRAIRIE	B	QUILCENE	C	REAKOR	B
PICKFORD	D	POCKER	D	PRATLEY	C	QUILLAYUTE	B	READING	C
PICKWICK	B	POCOMOKE	C	PRATT	A	QUIMBY	C	READINGTON	C
PICO	B	PODO	D	PREACHER	B	QUINCY	A	READLYN	B
PICTOU	B	PODUNK	B	[illegible]	D	QUINLAN	C	REAGAN	B

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Table B.1--Continued

WAHAMA	B/C
WAHEE	D
WAMPSVILLE	B
WANAKAH	C
WANBLEE	D
WANDO	A
WANETTA	A
WANN	A
WANA	A
WAPAL	B
WAPATO	C/D
WAPELLO	D
WAPINITIA	B
WAPPING	B
WAPSIE	B
WARBA	B
WARD	D
WARDBORO	A
WARDELL	D
WARDEN	B
WARDWELL	C
WARE	B
WAREHAM	C
WARMAN	D
WARM SPRINGS	C
WARNERS	A/D
WARREN	
WARRENTON	B/D
WARRIOR	
WARSAW	B
WARSING	B
WARWICK	A
WASATCH	A
WASEPI	B
WASHBURN	
WASHINGTON	B
WASHOE	C
WASHOUGAL	B
WASHTENAW	C/D
WASILLA	C
WASIOJA	C
WASSAIC	B
WATAB	C
WATAUGA	B
WATCHAUG	B
WATCHUNG	D
WATERBORO	
WATERBURY	D
WATERINO	C
WATERS	C
WATKINS	B
WATKINS RIDGE	B
WATOPA	B
WATROUS	B
WATSEKA	C
WATSON	C
WATSONIA	D
WATSONVILLE	D
WATT	D
WATTON	C
WAUBAY	B
WAUBEEK	B
WAUBONSIE	B
WAUCHULA	B/D
WAUCOMA	B
WAUCONDA	B
WAUKEE	B
WAUKEGAN	B
WAUKENA	B
WAUKON	B
WAUMBEK	B
WAUPIKA	
WAUSEON	B/D
WAVERLY	B/D
WAWAKA	C
WAYCUP	B
WAYDEN	D
WAYLAND	C/D
WAYNE	B
WAYNESBORO	B
WAYSIDE	
WEA	B
WEAVER	C
WEBB	C
WEBER	B
WEBSTER	C
WEDGE	A
WEDOWEE	B
WEEDING	A
WEEDMARK	B
WEEKSVILLE	B/D
WEHADKEE	D
WEIKERT	C/D
WEIMER	D
WEINBACH	C
WEIR	D
WEIRMAN	B
WEISER	C
WEISHAUPT	C
WEISS	A
WEITCHPEC	B
WELBY	B
WELCH	C
WELD	C
WELDA	C
WELDON	C
WELDONA	B
WELLER	C
WELLENHORN	C
WELLINGTON	C
WELLMAN	B
WELLNER	B
WELLSBORO	C
WELLSTON	B
WELLSVILLE	B
WEMPLE	B
WENAS	B/C
WENATCHEE	C
WENDEL	B/C
WENHAM	
WENONA	C
WENTWORTH	B
WEPHER	B
WESO	C
WESSEL	B
WESTBROOK	D
WESTBURY	C
WESTCREEK	B
WESTERVILLE	C
WESTFALL	C
WESTFIELD	
WESTFORD	
WESTLAND	B/D
WESTMINSTER	C/D
WESTMORE	B
WESTMORELAND	B
WESTON	D
WESTPHALIA	B
WESTPLAIN	C
WESTPORT	A
WESTVILLE	B
WETHERSFIELD	C
WETHEY	B/C
WETZEL	D
WEYMOUTH	B
WHALAN	B
WHARTON	C
WHATCOM	C
WHATELY	D
WHEATLEY	D
WHEATRIDGE	C
WHEATVILLE	B
WHEELER	B
WHEELING	B
WHEELOCK	C
WHEELON	B
WHELCHEL	B
WHETSTONE	B
WHIDBEY	C
WHIPPANY	C
WHIPSTOCK	C
WHIRLO	C
WHIT	B
WHITAKER	C
WHITCOMB	C
WHITE BIRD	C
WHITECAP	D
WHITEFISH	B
WHITEFORD	B
WHITEHORSE	B
WHITE HOUSE	C
WHITELAKE	B
WHITELAW	B
WHITEMAN	D
WHITEROCK	D
WHITESBURG	C
WHITE STORE	D
WHITE SWAN	C
WHITEWATER	B
WHITEWOOD	C
WHITLEY	B
WHITLOCK	B
WHITMAN	D
WHITNEY	B
WHITORE	B
WHITSOL	B
WHITSON	D
WHITWELL	C
WHOLAN	C
WIBAUX	D
WICHITA	C
WICHUP	D
WICKERSHAM	B
WICKETT	C
WICKHAM	B
WICKIUP	C
WICKLIFFE	
WICKSBURG	B
WIDTSOE	C
WIEHL	C
WIEN	D
WIGGLETON	B
WILBRAHAM	C
WILBUR	C
WILCO	C
WILCOX	D
WILCOXSON	C
WILDCAT	D
WILDER	B
WILDERNESS	C
WILDROSE	D
WILDWOOD	D
WILEY	C
WILKES	C
WILKESON	C
WILKINS	D
WILL	D
WILLACY	B
WILLAKENZIE	C
WILLAMAR	D
WILLAMETTE	B
WILLAPA	C
WILLARD	C
WILLETTE	A/D
WILLHAND	B
WILLIAMS	B
WILLIAMSBURG	B
WILLIAMSON	C
WILLIS	C
WILLITS	B
WILLOUGHBY	B
WILLOW CREEK	B
WILLOWDALE	B
WILLOWS	D
WILLWOOD	B
WILMER	C
WILPAR	D
WILSON	D
WILTSHIRE	C
WINANS	B/C
WINCHESTER	A
WINCHUCK	C
WINDER	B/D
WINDMILL	B
WINDOM	B
WIND RIVER	B
WINDSOR	A
WINDTHORST	C
WINDY	C
WINEG	B
WINEMA	C
WINETTI	B
WINFIELD	C
WING	D
WINGATE	B
WINGER	C
WINGVILLE	B/D
WINIFRED	C
WINK	B
WINKLEMAN	C
WINLO	D
WINLOCK	C
WINN	C
WINNEBAGO	B
WINNEMUCCA	B
WINNESHIEK	B
WINNETT	D
WINONA	D
WINOOSKI	B
WINSTON	A
WINTERS	C
WINTERSBURG	B
WINTERSET	C
WINTHROP	A
WINTONER	C
WINU	C
WINZ	C
WIOTA	B
WISHEYLU	C
WISKAH	C
WISNER	D
WITBECK	D
WITCH	D
WITHAM	C
WITHEE	C
WITT	C
WITZEL	D
WODEN	B
WODSKOW	B
WOLCOTTSBURG	
WOLDALE	C/D
WOLF	B
WOLFESEN	C
WOLFORD	B
WOLF POINT	D
WOLFTEVER	C
WOLVERINE	A
WOODBINE	B
WOODBRIDGE	C
WOODBURN	C
WOODBURY	D
WOODCOCK	B
WOODENVILLE	C
WOODGLEN	D
WOODHURST	A
WOODLY	B
WOODLYN	C
WOODMANSIE	B
WOODPERE	B
WOOD RIVER	C
WOODROCK	B
WOODROW	C
WOODS CROSS	D
WOODSFIELD	C
WOODSIDE	A
WOODSON	D
WOODSTOCK	C/D
WOODSTOWN	C
WOODWARD	B
WOOLMAN	B
WOOLPER	C
WOOLSEY	C
WOOSLEY	B
WOOSTER	C
WOOSTERN	B
WOOTEN	A
WORCESTER	B
WORF	C
WORK	C
WORLAND	B
WORLEY	C
WORMSER	C
WOROCK	B
WORSHAM	D
WORTH	C
WORTHEN	B
WORTHING	D
WORTHINGTON	C
WORTMAN	C
WRENTHAM	C
WRIGHT	C
WRIGHTSVILLE	D
WUNJEY	B
WURTSBORO	C
WYALUSING	D
WYARD	B
WYARNO	C
WYATT	C
WYEAST	C
WYEVILLE	C
WYGANT	B
WYKOFF	B
WYMAN	B
WYMORE	C
WYNN	B
WYNOOSE	D
WYO	B
WYOCENA	B
YACOLT	B
YAHARA	B
YAHOLA	B
YAKIMA	B
YAKUS	D
YALLANI	B
YALMER	B
YAMHILL	C
YAMPA	C
YAMSAY	D
YAKA	B
YAQUINA	B/D
YARDLEY	C
YATES	D
YAUCIN	C
YAWKEY	C
YAXON	B
YEATES HOLLOW	C
YEGEN	B
YELM	B
YENRAB	A
YEOMAN	B
YETULL	A
YODER	B
YOKOHL	C
YOLLABOLLY	D
YOLO	B
YOLCGO	D
YOMONT	B
YONCALLA	C
YONGES	D
YONNA	B/D
YORDY	B
YORK	C
YORKVILLE	D
YOST	C
YOUGA	B
YOUMAN	C
YOUNGSTON	B
YOURAME	A
YOVIMPA	D
YSIDORA	C
YTURBIDE	A
YUBA	D
YUKON	D
YUNES	D
YUNQUE	C
ZAAR	D
ZACA	C
ZACHARIAS	B
ZACHARY	D
ZAFRA	D
ZAHILL	B
ZAHL	B
ZALESKI	C
ZALLA	A
ZAMORA	B
ZANE	C
ZANEIS	B
ZANESVILLE	C
ZANONE	C
ZAPATA	C
ZAVALA	B
ZAVCO	C
ZEB	B
ZEESIX	C
ZELL	B
ZEN	C
ZENDA	C
ZENIA	B
ZENIFF	B
ZEONA	A
ZIEGLER	C
ZIGWEID	B
ZILLAH	B/C
ZIM	D
ZIMMERMAN	A
ZING	C
ZINZER	B
ZION	C
ZIPP	C/D
ZITA	B
ZOAR	C
ZOHNER	B/D
ZOOK	C
ZORRAVISTA	A
ZUFELT	B
ZUMBRO	B
ZUMWALT	C
ZUNDELL	B
ZUNHALL	B
ZUNI	C
ZURICH	B

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Table 2-2.--Runoff curve numbers for selected agricultural, suburban, and urban land use. (Antecedent moisture condition II, and $I_a = 0.2S$)

LAND USE DESCRIPTION	HYDROLOGIC SOIL GROUP			
	A	B	C	D
Cultivated land[1/]: without conservation treatment	72	81	88	91
: with conservation treatment	62	71	78	81
Pasture or range land: poor condition	68	79	86	89
good condition	39	61	74	80
Meadow: good condition	30	58	71	78
Wood or Forest land: thin stand, poor cover, no mulch	45	66	77	83
good cover[2/]	25	55	70	77
Open Spaces, lawns, parks, golf courses, cemeteries, etc.				
good condition: grass cover on 75% or more of the area	39	61	74	80
fair condition: grass cover on 50% to 75% of the area	49	69	79	84
Commercial and business areas (85% impervious)	89	92	94	95
Industrial districts (72% impervious)	81	88	91	93
Residential:[3/]				
Average lot size — Average % Impervious[4/]				
1/8 acre or less — 65	77	85	90	92
1/4 acre — 38	61	75	83	87
1/3 acre — 30	57	72	81	86
1/2 acre — 25	54	70	80	85
1 acre — 20	51	68	79	84
Paved parking lots, roofs, driveways, etc.[5/]	98	98	98	98
Streets and roads:				
paved with curbs and storm sewers[5/]	98	98	98	98
gravel	76	85	89	91
dirt	72	82	87	89

1/ For a more detailed description of agricultural land use curve numbers refer to National Engineering Handbook, Section 4, Hydrology, Chapter 9, Aug. 1972.

2/ Good cover is protected from grazing and litter and brush cover soil.

3/ Curve numbers are computed assuming the runoff from the house and driveway is directed towards the street with a minimum of roof water directed to lawns where additional infiltration could occur.

4/ The remaining pervious areas (lawn) are considered to be in good pasture condition for these curve numbers.

5/ In some warmer climates of the country a curve number of 95 may be used.

Taken from Technical Release No. 55, "Urban Hydrology for Small Watersheds," USDA-SCS, October 1981

Table 2-1.--Runoff depth in inches for selected CN's and rainfall amounts

Rainfall (inches)	Curve Number (CN)[1/]								
	60	65	70	75	80	85	90	95	98
1.0	0	0	0	0.03	0.08	0.17	0.32	.56	.79
1.2	0	0	0.03	0.07	0.15	0.28	0.46	.74	.99
1.4	0	0.02	0.06	0.13	0.24	0.39	0.61	.92	1.18
1.6	0.01	0.05	0.11	0.20	0.34	0.52	0.76	1.11	1.38
1.8	0.03	0.09	0.17	0.29	0.44	0.65	0.93	1.29	1.58
2.0	0.06	0.14	0.24	0.38	0.56	0.80	1.09	1.48	1.77
2.5	0.17	0.30	0.46	0.65	0.89	1.18	1.53	1.96	2.27
3.0	0.33	0.51	0.72	0.96	1.25	1.59	1.98	2.45	2.78
4.0	0.76	1.03	1.33	1.67	2.04	2.46	2.92	3.43	3.77
5.0	1.30	1.65	2.04	2.45	2.89	3.37	3.88	4.42	4.76
6.0	1.92	2.35	2.80	3.28	3.78	4.31	4.85	5.41	5.76
7.0	2.60	3.10	3.62	4.15	4.69	5.26	5.82	6.41	6.76
8.0	3.33	3.90	4.47	5.04	5.62	6.22	6.81	7.40	7.76
9.0	4.10	4.72	5.34	5.95	6.57	7.19	7.79	8.40	8.76
10.0	4.90	5.57	6.23	6.88	7.52	8.16	8.78	9.40	9.76
11.0	5.72	6.44	7.13	7.82	8.48	9.14	9.77	10.39	10.76
12.0	6.56	7.32	8.05	8.76	9.45	10.12	10.76	11.39	11.76

[1/] To obtain runoff depths for CN's and other rainfall amounts not shown in this table, use an arithmetic interpolation.

Taken from Technical Release No. 55, "Urban Hydrology for Small Watersheds," USDA-SCS, October 1981

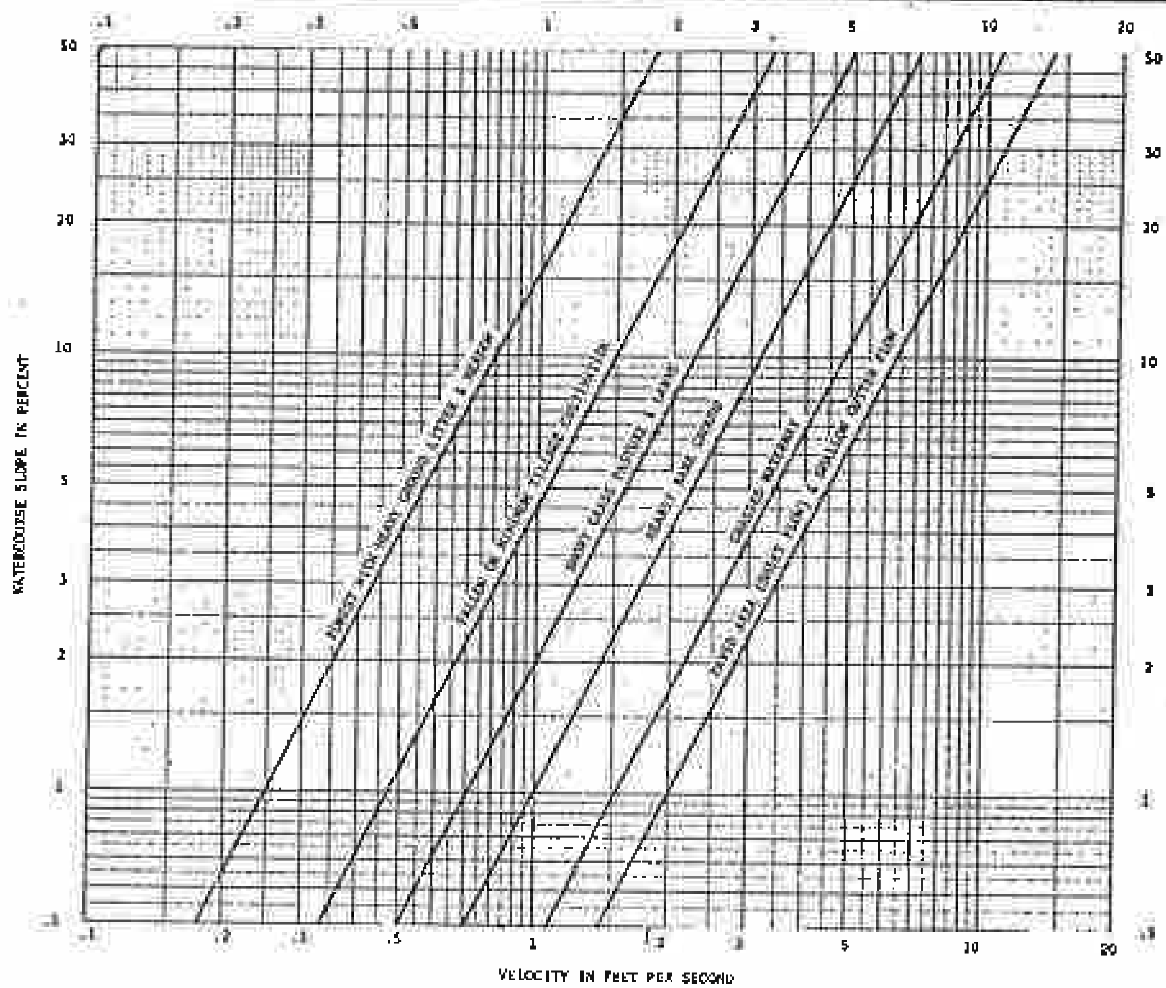

Figure 3-1.--Average velocities for estimating travel time for overland flow.

Taken from Technical Release No. 55, "Urban Hydrology for Small Watersheds," USDA-SCS, October 1981

Table 5-3.--Tabular discharges for type-II storm distribution (csm/in) Sheet 1 of 5

TIME OF CONCENTRATION = 0.1 hours
Hydrograph Time in Hours

T_t	11.0	11.5	11.7	11.8	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	12.9	13.0	13.2	13.5	14.0	14.5	15.0	16.0	18.0	20.0
0	24	51	299	991	746	477	233	152	132	121	111	85	74	70	68	65	52	48	39	33	29	24	18	14
0.25	20	38	66	140	327	626	686	546	364	236	169	137	117	97	83	75	66	52	41	35	30	24	18	14
0.50	15	27	36	43	67	133	288	482	580	543	429	310	222	168	134	110	81	63	47	38	32	26	19	15
0.75	12	20	25	29	34	42	65	125	245	392	496	515	452	360	273	206	127	80	53	42	35	27	19	15
1.00	9	15	19	21	24	28	32	41	63	115	209	328	427	470	451	389	245	121	64	47	38	29	20	16
1.50	6	10	12	13	14	16	17	19	22	25	29	38	56	92	154	236	410	360	133	66	47	33	21	16
2.00	3	6	7	8	9	10	11	12	13	14	16	18	20	23	27	34	74	244	371	142	68	38	23	17
2.50	2	4	4	5	5	6	7	7	8	9	10	11	12	13	15	16	21	41	243	343	150	48	26	19
3.00	1	2	2	3	3	4	4	4	5	5	6	7	7	8	9	10	12	17	50	239	321	74	29	20
3.50	0	1	1	1	1	2	2	2	3	3	4	4	4	5	6	6	7	10	17	59	304	159	33	21
4.00	0	0	0	0	0	1	1	1	1	2	2	2	2	3	3	4	5	6	10	18	67	290	39	23

TIME OF CONCENTRATION = 0.2 hours
Hydrograph Time in Hours

T_t	11.0	11.5	11.7	11.8	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	12.9	13.0	13.2	13.5	14.0	14.5	15.0	16.0	18.0	20.0
0	23	47	208	509	796	641	424	245	170	138	121	104	85	75	71	68	56	49	40	34	29	24	18	14
0.25	18	34	49	91	196	419	603	627	486	341	235	173	138	114	96	83	70	55	43	36	31	25	18	15
0.50	14	24	32	37	50	87	181	341	490	545	497	397	296	219	167	133	92	67	49	39	33	26	19	15
0.75	11	18	23	26	30	36	49	84	161	284	409	491	481	422	340	263	157	89	56	43	36	27	19	15
1.00	9	14	18	20	22	25	29	35	48	79	143	240	347	426	452	427	299	147	69	49	39	29	20	16
1.50	5	9	11	12	13	14	16	18	20	23	26	32	43	67	110	176	330	399	159	72	50	33	22	17
2.00	3	6	7	7	8	9	10	11	12	13	15	16	18	21	24	29	56	192	363	168	75	40	24	18
2.50	1	3	4	5	5	6	6	7	7	8	9	10	11	12	13	15	19	33	200	337	174	51	26	19
3.00	0	2	2	2	3	3	4	4	5	5	6	6	7	8	8	9	11	15	40	203	316	82	29	20
3.50	0	0	1	1	1	2	2	2	2	3	3	4	4	5	5	6	7	9	16	46	300	180	34	22
4.00	0	0	0	0	0	1	1	1	1	1	2	2	2	3	3	3	4	6	9	16	53	286	41	24

Taken from Technical Release No. 55, "Urban Hydrology for Small Watersheds," USDA-SCS, October 1981

Table 5-3.--Tabular discharges for type-II storm distribution (csm/in)--Continued   Sheet 5 of 5

TIME OF CONCENTRATION = 1.5 hours

Hydrograph Time in Hours

T_t	11.0	11.5	11.7	11.8	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	12.9	13.0	13.2	13.5	14.0	14.5	15.0	16.0	18.0	20.0
0	10	18	31	42	57	81	105	133	164	192	209	227	235	236	236	225	201	153	99	68	50	32	20	16
0.25	8	13	17	22	30	41	57	76	99	125	153	178	199	215	225	230	224	188	122	82	58	36	21	16
0.50	6	10	13	15	18	22	30	40	54	72	94	118	143	167	188	204	224	214	152	99	68	39	22	17
0.75	5	8	10	11	13	15	18	22	29	39	52	69	89	111	134	157	194	219	182	122	82	44	23	17
1.00	4	6	8	9	10	11	12	14	17	22	29	38	50	66	84	105	148	198	214	150	100	50	24	18
1.50	2	4	5	5	6	7	7	8	9	10	12	14	17	21	26	34	58	109	191	204	149	70	28	19
2.00	1	2	3	3	4	4	4	5	5	6	7	8	8	10	11	13	19	40	112	184	197	102	33	20
2.50	0	1	1	2	2	2	3	3	3	4	4	5	5	6	6	7	9	14	45	114	190	147	40	22
3.00	0	0	1	1	1	1	1	1	2	2	2	3	3	3	4	4	5	7	16	49	115	184	53	25
3.50	0	0	0	0	0	0	1	1	1	1	1	1	2	2	2	2	3	4	8	18	53	178	74	28
4.00	0	0	0	0	0	0	0	0	0	0	0	1	1	1	1	1	2	2	4	8	21	174	105	34

TIME OF CONCENTRATION = 2.0 hours

Hydrograph Time in Hours

T_t	11.0	11.5	11.7	11.8	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	12.9	13.0	13.2	13.5	14.0	14.5	15.0	16.0	18.0	20.0
0	7	14	22	30	38	49	64	80	95	114	133	152	165	175	184	192	190	176	129	93	68	41	23	17
0.25	6	10	13	17	22	28	37	47	61	75	91	108	126	143	157	168	185	189	153	109	79	46	24	17
0.50	5	8	10	11	13	17	21	27	35	45	57	71	86	103	119	135	162	186	172	129	92	52	26	18
0.75	4	6	8	8	10	11	13	16	21	26	34	43	55	67	82	97	129	166	183	149	109	59	27	18
1.00	3	5	6	7	7	8	9	11	13	16	20	26	33	42	52	64	92	136	180	167	127	68	29	19
1.50	1	3	3	4	4	5	5	6	7	8	9	10	12	15	18	23	37	68	135	175	163	93	34	21
2.00	1	1	2	2	3	3	3	4	4	5	5	6	6	7	8	10	14	26	71	133	170	127	42	23
2.50	0	1	1	1	1	1	2	2	2	3	3	3	4	4	5	5	7	11	29	74	132	166	53	26
3.00	0	0	0	0	1	1	1	1	1	1	2	2	2	2	3	3	4	5	12	32	76	162	71	30
3.50	0	0	0	0	0	0	0	0	1	1	1	1	1	1	1	2	2	3	6	13	35	158	95	35
4.00	0	0	0	0	0	0	0	0	0	0	0	0	0	1	1	1	1	2	3	6	14	80	155	43

Taken from Technical Release No. 55, "Urban Hydrology for Small Watersheds," USDA-SCS, October 1981

TABLE 2.10.13.1
ROUGHNESS COEFFICIENT "n" FOR MANNING'S EQUATION

Description	"n"
Concrete Pipe	.012
Annular Corrugated Steel and Alum. Alloy Pipe or Pipe Arch + (plain or coated)	.024
Vitrified Clay Pipe	.012
Cast Iron Pipe	.013
Brick Sewer	.015
Asphalt Pavement	.015
Concrete Pavement	.014
Grass Medians	.05
Earth	.02
Gravel	.02
Rock	.035
Cultivated Areas	.03 – .05
Dense Brush	.07 – .14
Heavy Timber—Little Undergrowth	.10 – .15
Streams	
a. some grass and weeds—little or no brush	.03 – .035
b. dense growth of weeds	.035 – .05
c. some weeds—heavy brush on banks	.05 – .07

Note: In considering each factor more critical judgment will be exercised if it is kept in mind that any condition that causes turbulence and retards flow results in a greater value of "n".

+ Roughness Coefficient (n) for Helical Corrugated Steel and Alum. Alloy Pipe

Corrugations	$2\frac{2}{3}'' \times \frac{1}{2}''$								$3'' \times 1''$
Diameters	18"	24"	36"	48"	60"	72"	84"	96"	ALL DIA
Plain or Coated	.014	.016	.019	.020	.021	.021	.021	.021	.024

Taken from "Pennsylvania Department of Transportation Design Manual, Part 2 - Highway Design," August 1981

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ENZ CO.

GAI CONSULTANTS, INC.

Engineers • Geologists • Planners
Environmental Specialists

570 Beatty Rd. • Pittsburgh,
Monroeville, Pa. 15146
412-856-6400

August 27, 1985

Project 85-205-7

Mr. Alex D. Lapinsky
H. F. Lenz Company
1732 Lyter Drive
Johnstown, PA 15905

Drainage Areas
Keystone Power Station

Dear Mr. Lapinsky:

This letter is to confirm the drainage areas provided to you by telephone by Ellen Kucharik on May 22, 1985. The drainage areas provided for your use were:

- Drainage area for the top surface of the East and West Valleys = 0.163 square mile

- Drainage area for the slope drain = 0.032 square mile

Very truly yours,
GAI Consultants, Inc.

Dana Burns

Dana Burns

DB/bws

cc: F. Straw, Penelec

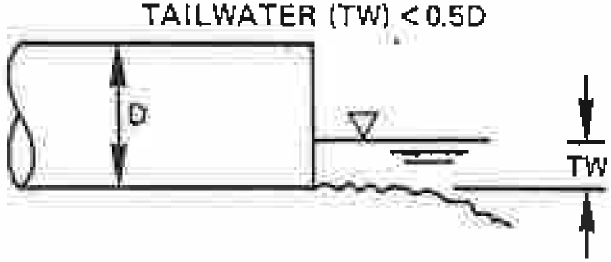
TABLE V–1 NATURAL CHANNEL SCOUR ESTIMATE

GENERAL EQUATION: DEPTH, WIDTH, LENGTH OR VOLUME = $\alpha (y_e)^{\gamma} \left(\frac{Q}{y_e^{5/2}} \right)^{\beta} t^{\theta}$

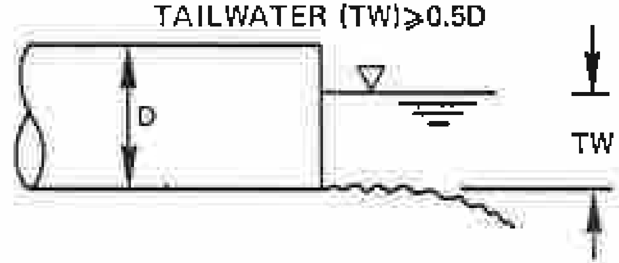
MAXIMUM SCOUR HOLE DIMENSION	COEFFICIENTS			
	α	β	θ	γ
DEPTH (hs)				
TW < 0.5D	0.82	0.375	0.10	1.0
TW ⩾ 0.5D	0.76	0.375	0.10	1.0
WIDTH (Ws)				
TW < 0.5D	0.55	0.915	0.15	1.0
TW ⩾ 0.5D	0.39	0.915	0.15	1.0
LENGTH (Ls)				
TW < 0.5D	1.67	0.71	0.125	1.0
TW ⩾ 0.5D	2.85	0.71	0.125	1.0
VOLUME (Vs)				
TW < 0.5D	0.29	2.0	0.375	3.0
TW ⩾ 0.5D	0.24	2.0	0.375	3.0

MAXIMUM DURATION OF PEAK DISCHARGE (t) IS 1440 MINUTES (24 HOURS)

TAILWATER (TW) < 0.5D



TAILWATER (TW) ⩾ 0.5D



Taken from "Hydraulic Design of Energy Dissipators for Culverts and Channels," U.S. Department of Transportation, Federal Highway Administration, December 1978

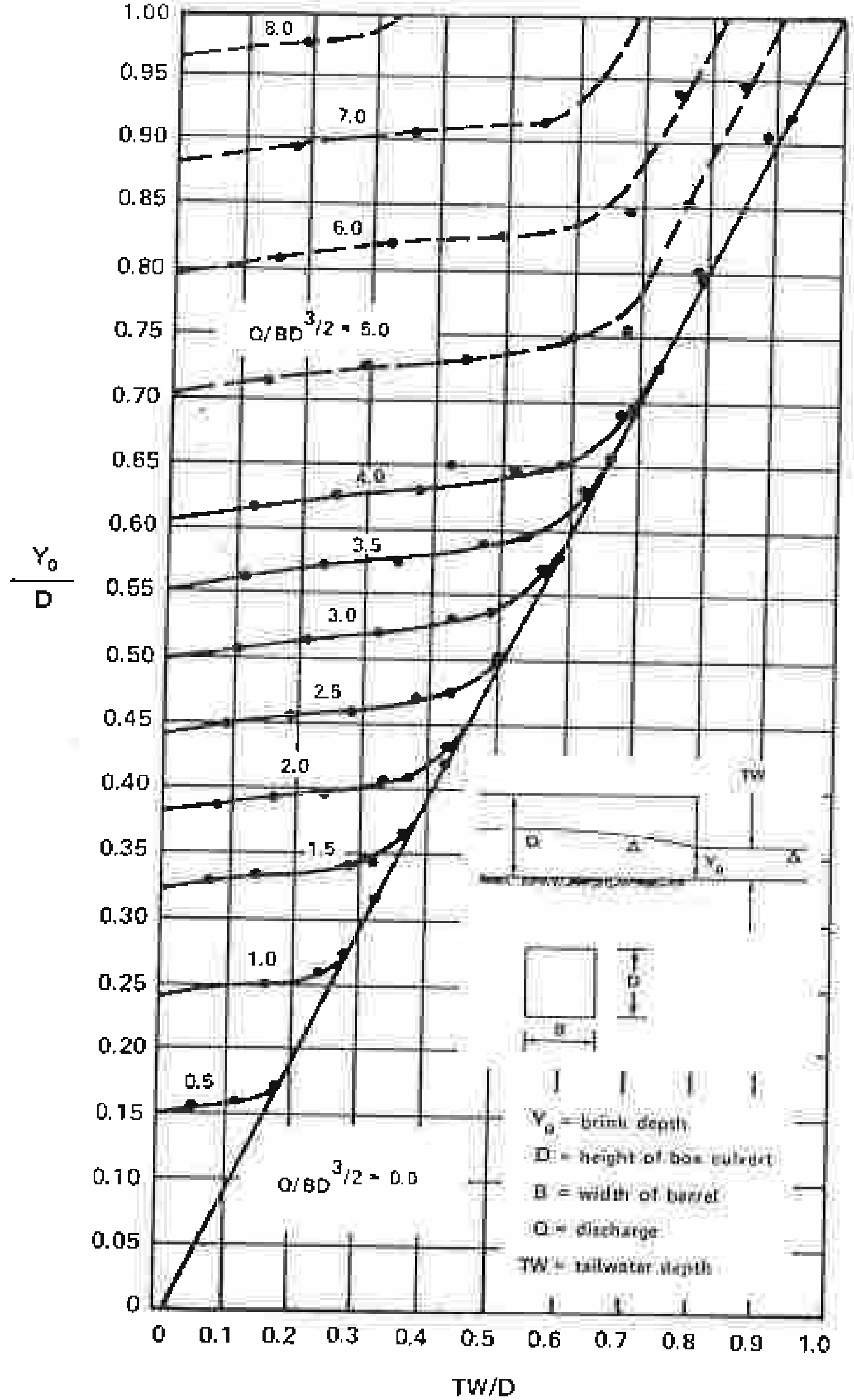


Figure III-9 Dimensionless Rating Curves for the Outlets of Rectangulars Culverts on Horizontal and Mild Slopes from Reference III-2

III-14

Taken from "Hydrualic Design of Energy Dissipators for Culverts and Channels," U.S. Department of Transportation, Federal Highway Administration, December 1978

Table III-2.—*Uniform flow in circular sections flowing partly full.* **From Reference III-3.**

d = depth of flow
D = diameter of pipe
A = area of flow
R = hydraulic radius

Q = discharge in cubic feet per second by Manning's formula
n = Manning's coefficient
S = slope of the channel bottom and of the water surface

$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{R}{D}$	$\frac{Qn}{D^{8/3}S^{1/2}}$	$\frac{Qn}{d^{8/3}S^{1/2}}$	$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{R}{D}$	$\frac{Qn}{D^{8/3}S^{1/2}}$	$\frac{Qn}{d^{8/3}S^{1/2}}$
0.01	0.0013	0.0066	0.00007	15.04	0.51	0.4027	0.2531	0.239	1.442
0.02	0.0037	0.0132	0.00031	10.57	0.52	0.4127	0.2562	0.247	1.415
0.03	0.0069	0.0197	0.00074	8.56	0.53	0.4227	0.2592	0.255	1.388
0.04	0.0105	0.0262	0.00138	7.38	0.54	0.4327	0.2621	0.263	1.362
0.05	0.0147	0.0325	0.00222	6.55	0.55	0.4426	0.2649	0.271	1.336
0.06	0.0192	0.0389	0.00328	5.95	0.56	0.4526	0.2676	0.279	1.311
0.07	0.0242	0.0451	0.00455	5.47	0.57	0.4625	0.2703	0.287	1.286
0.08	0.0294	0.0513	0.00604	5.09	0.58	0.4724	0.2728	0.295	1.262
0.09	0.0350	0.0575	0.00775	4.76	0.59	0.4822	0.2753	0.303	1.238
0.10	0.0409	0.0635	0.00967	4.49	0.60	0.4920	0.2776	0.311	1.215
0.11	0.0470	0.0695	0.01181	4.25	0.61	0.5018	0.2799	0.319	1.192
0.12	0.0534	0.0755	0.01417	4.04	0.62	0.5115	0.2821	0.327	1.170
0.13	0.0600	0.0813	0.01674	3.86	0.63	0.5212	0.2842	0.335	1.148
0.14	0.0668	0.0871	0.01952	3.69	0.64	0.5308	0.2862	0.343	1.126
0.15	0.0739	0.0929	0.0225	3.54	0.65	0.5404	0.2882	0.350	1.105
0.16	0.0811	0.0985	0.0257	3.41	0.66	0.5499	0.2900	0.358	1.084
0.17	0.0885	0.1042	0.0291	3.28	0.67	0.5594	0.2917	0.366	1.064
0.18	0.0961	0.1097	0.0327	3.17	0.68	0.5687	0.2933	0.373	1.044
0.19	0.1039	0.1152	0.0365	3.06	0.69	0.5780	0.2948	0.380	1.024
0.20	0.1118	0.1206	0.0406	2.96	0.70	0.5872	0.2962	0.388	1.004
0.21	0.1199	0.1259	0.0448	2.87	0.71	0.5964	0.2975	0.395	0.985
0.22	0.1281	0.1312	0.0492	2.79	0.72	0.6054	0.2987	0.402	0.965
0.23	0.1365	0.1364	0.0537	2.71	0.73	0.6143	0.2998	0.409	0.947
0.24	0.1449	0.1416	0.0585	2.63	0.74	0.6231	0.3008	0.416	0.929
0.25	0.1535	0.1466	0.0634	2.56	0.75	0.6319	0.3017	0.422	0.910
0.26	0.1623	0.1516	0.0686	2.49	0.76	0.6405	0.3024	0.429	0.891
0.27	0.1711	0.1566	0.0739	2.42	0.77	0.6489	0.3031	0.435	0.873
0.28	0.1800	0.1614	0.0793	2.36	0.78	0.6573	0.3036	0.441	0.856
0.29	0.1890	0.1662	0.0849	2.30	0.79	0.6655	0.3039	0.447	0.838
0.30	0.1982	0.1709	0.0907	2.25	0.80	0.6736	0.3042	0.453	0.821
0.31	0.2074	0.1756	0.0966	2.20	0.81	0.6815	0.3043	0.458	0.804
0.32	0.2167	0.1802	0.1027	2.14	0.82	0.6893	0.3043	0.463	0.787
0.33	0.2260	0.1847	0.1089	2.09	0.83	0.6969	0.3041	0.468	0.770
0.34	0.2355	0.1891	0.1153	2.05	0.84	0.7043	0.3038	0.473	0.753
0.35	0.2450	0.1935	0.1218	2.00	0.85	0.7115	0.3033	0.477	0.736
0.36	0.2546	0.1978	0.1284	1.958	0.86	0.7186	0.3026	0.481	0.720
0.37	0.2642	0.2020	0.1351	1.915	0.87	0.7254	0.3018	0.485	0.703
0.38	0.2739	0.2062	0.1420	1.875	0.88	0.7320	0.3007	0.488	0.687
0.39	0.2836	0.2102	0.1490	1.835	0.89	0.7384	0.2995	0.491	0.670
0.40	0.2934	0.2142	0.1561	1.797	0.90	0.7445	0.2980	0.494	0.654
0.41	0.3032	0.2182	0.1633	1.760	0.91	0.7504	0.2963	0.496	0.637
0.42	0.3130	0.2220	0.1705	1.724	0.92	0.7560	0.2944	0.497	0.621
0.43	0.3229	0.2258	0.1779	1.689	0.93	0.7612	0.2921	0.498	0.604
0.44	0.3328	0.2295	0.1854	1.655	0.94	0.7662	0.2895	0.498	0.588
0.45	0.3428	0.2331	0.1929	1.622	0.95	0.7707	0.2865	0.498	0.571
0.46	0.3527	0.2366	0.201	1.590	0.96	0.7749	0.2829	0.496	0.553
0.47	0.3627	0.2401	0.208	1.559	0.97	0.7785	0.2787	0.494	0.535
0.48	0.3727	0.2435	0.216	1.530	0.98	0.7817	0.2735	0.489	0.517
0.49	0.3827	0.2468	0.224	1.500	0.99	0.7841	0.2666	0.483	0.496
0.50	0.3927	0.2500	0.232	1.471	1.00	0.7854	0.2500	0.463	0.463

Taken from "Hydraulic Design of Energy Dissipators for Culverts and Channels," U.S. Department of Transportation, Federal Highway Administration, December 1978

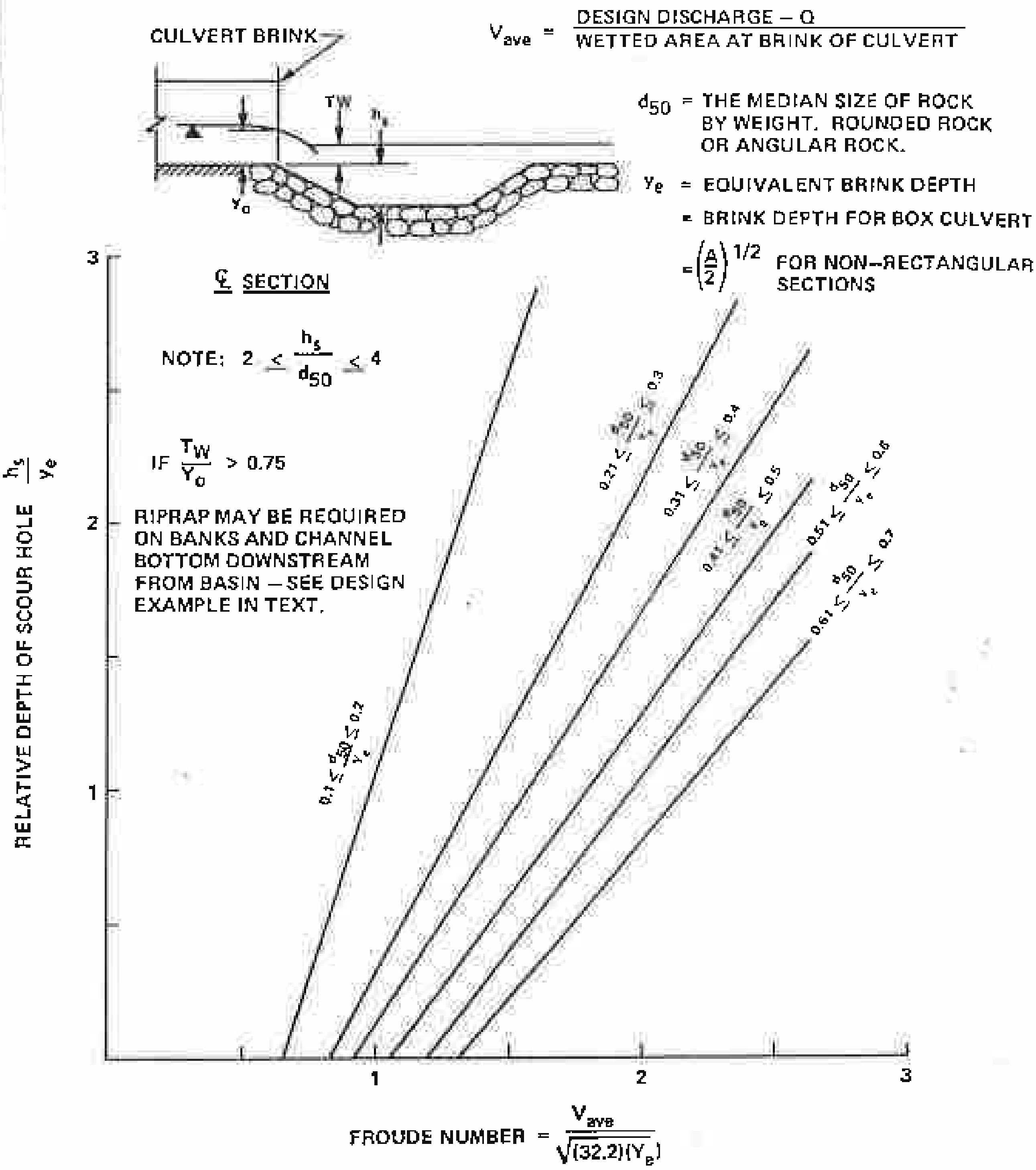
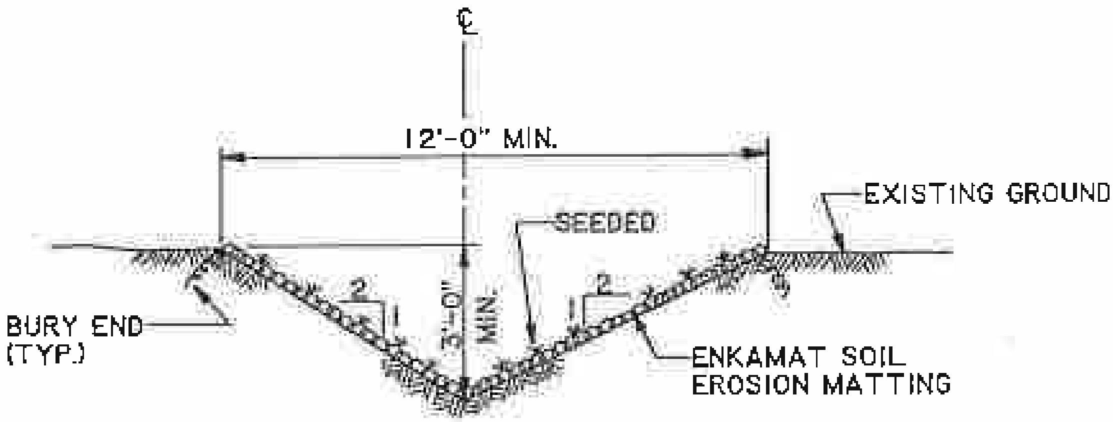


FIGURE X1–2. RELATIVE DEPTH OF SCOUR HOLE VERSUS FROUDE NUMBER AT BRINK OF CULVERT WITH RELATIVE SIZE OF RIPRAP AS A THIRD VARIABLE

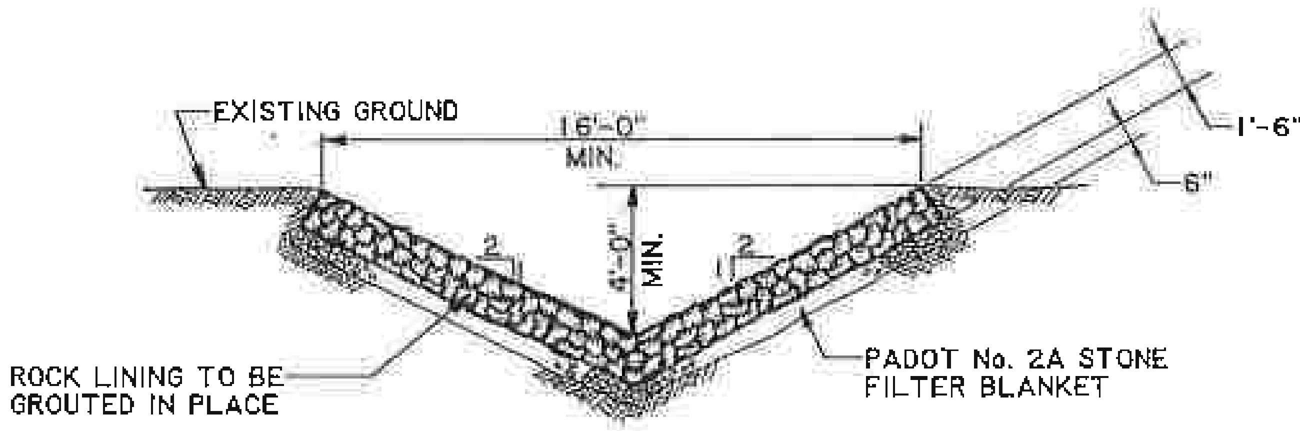
Taken from "Hydraulic Design of Energy Dissipators for Culverts and Channels," U.S. Department of Transportation, Federal Highway Administration, December 1978



TYPICAL SECTION

TYPE I DRAINAGE DITCH

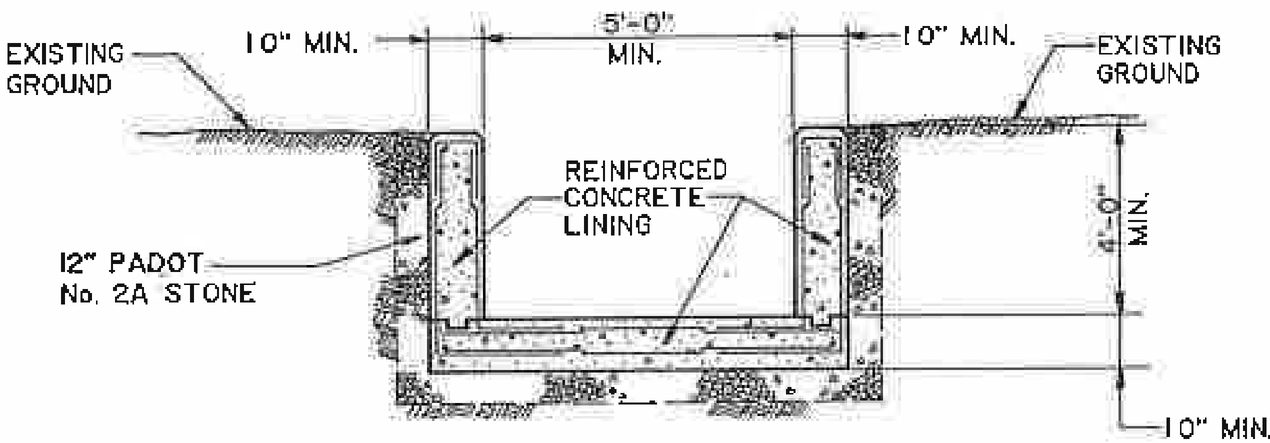
NOT TO SCALE



TYPICAL SECTION

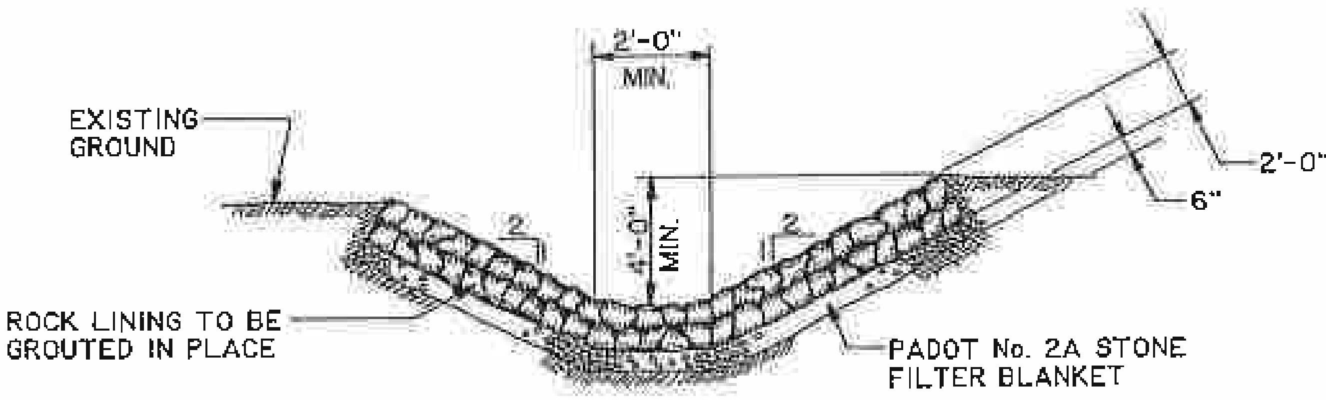
TYPE II DRAINAGE DITCH

NOT TO SCALE



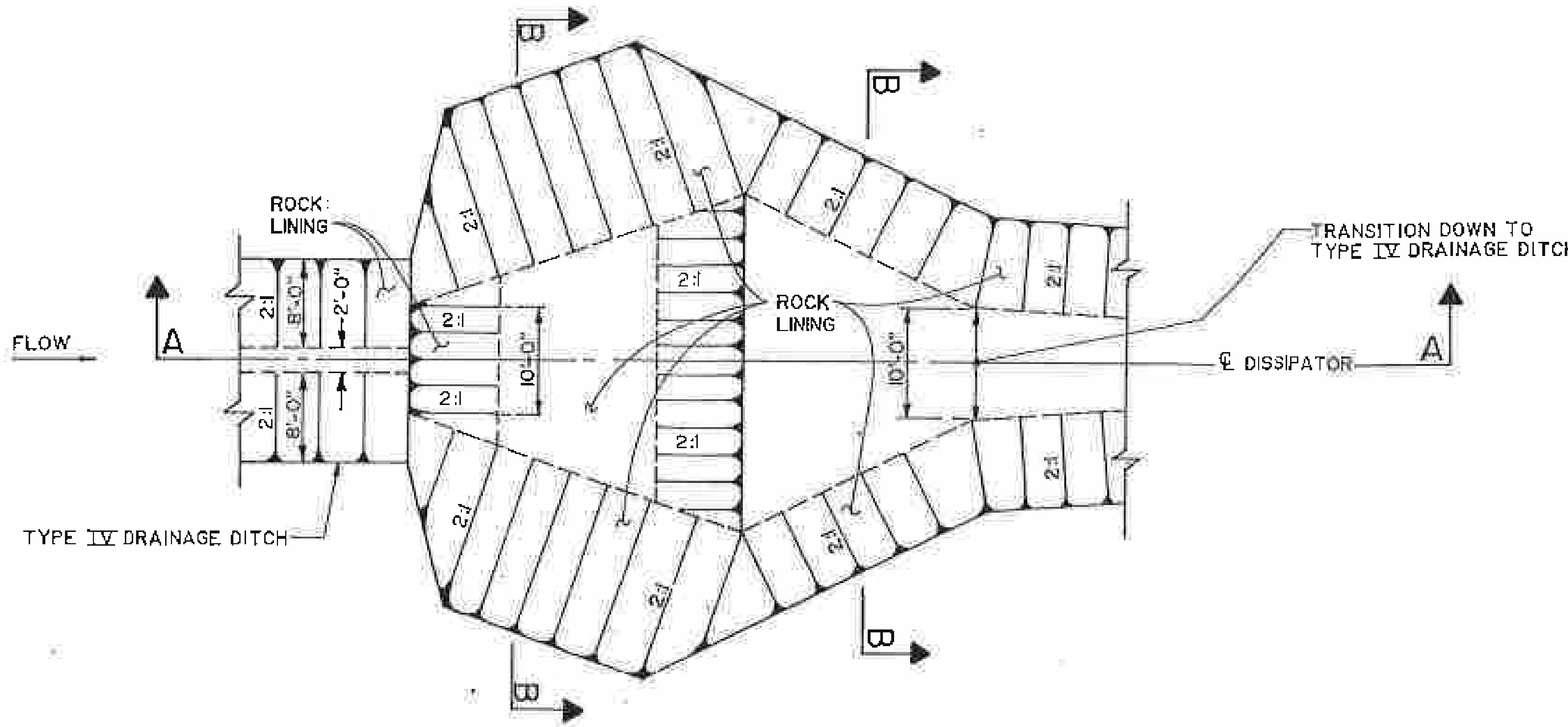
TYPICAL SECTION
TYPE III DRAINAGE DITCH

NOT TO SCALE



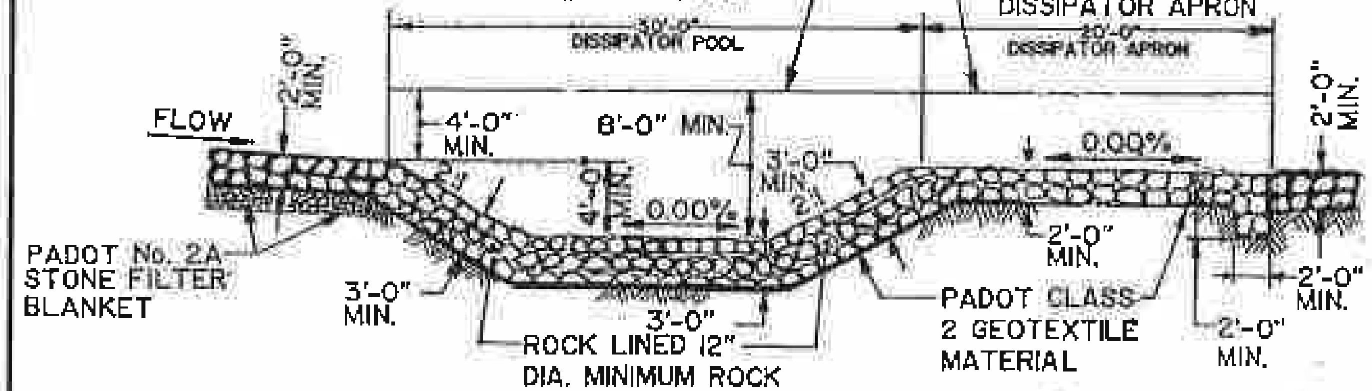
TYPICAL SECTION
TYPE IV DRAINAGE DITCH

NOT TO SCALE



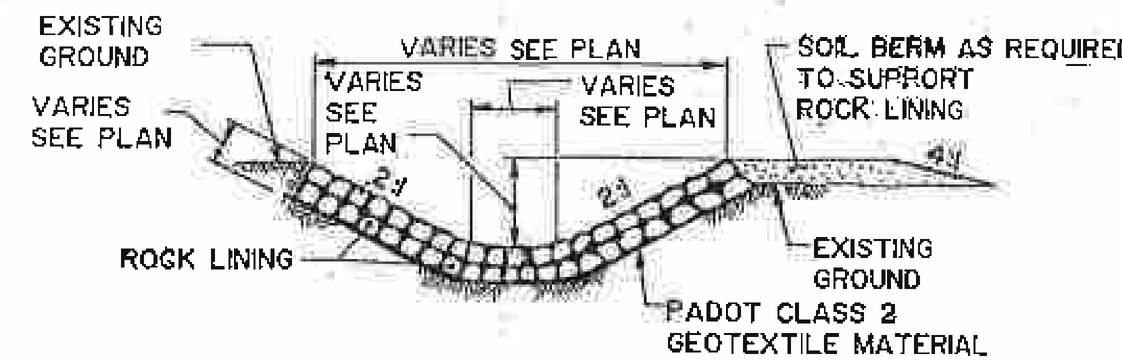
ROCK ENERGY DISSIPATOR - PLAN

NOT TO SCALE



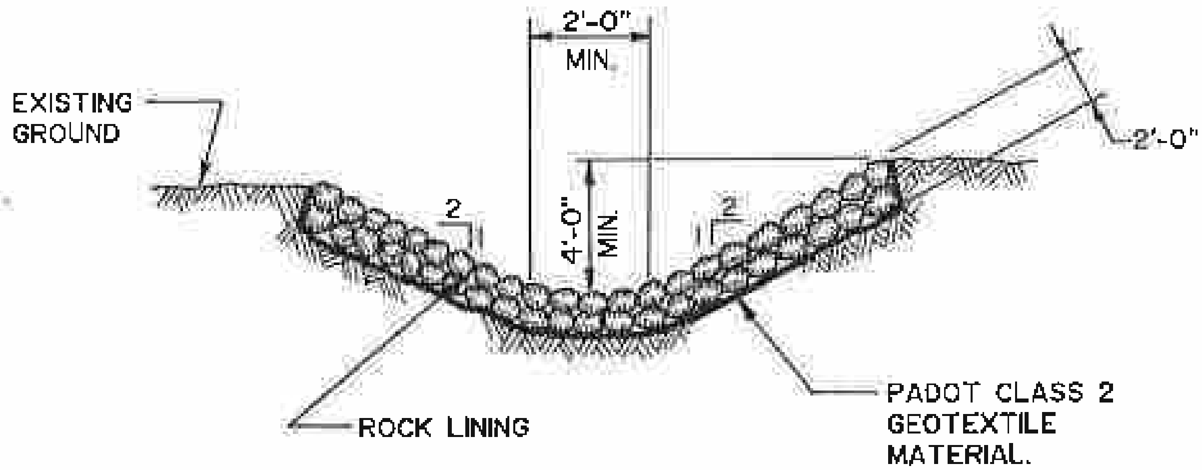
SECTION A-A

NOT TO SCALE



SECTION B-B

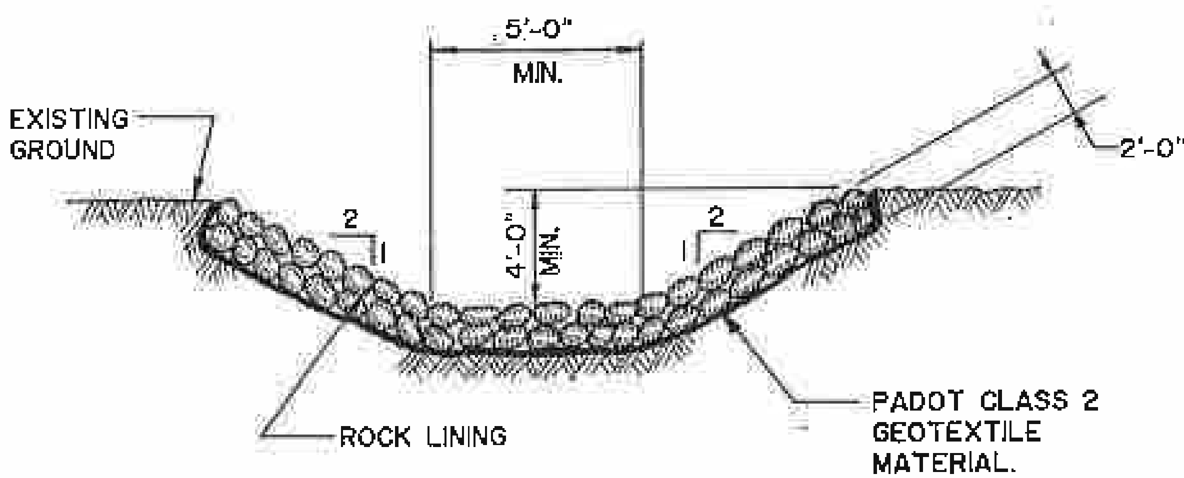
NOT TO SCALE



TYPICAL SECTION

TYPE V DRAINAGE DITCH

NOT TO SCALE



TYPICAL SECTION

TYPE VI DRAINAGE DITCH

NOT TO SCALE

REFERENCES

REFERENCES

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13. Technical Report H74-9, "Practical Guidance for Design of Lined Channel Expansions at Culvert Outlets," Hydraulic Model Investigation by B.P. Fletcher and J.L. Grace, Jr., Hydraulics Laboratory, U.S. Army Engineer Waterways Experiment Station, October 1974.

SUBJECT PENELEC - STAGE I HYDROLOGY

BY DMK DATE 3/15/85 PROJ. NO. 85-205-4

CHKD. BY EHK DATE 3/20/85 SHEET NO. 1 OF 10

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THE PEAK FLOWS WITH WHICH THE STAGE I COLLECTION DITCHES WILL BE OBTAINED USING THE TR-20 COMPUTER PROGRAM DEVELOPED BY THE SOIL CONSERVATION SERVICE. INPUT TO TR-20 CONSISTS OF DRAINAGE AREA, TIME OF CONCENTRATION, AND CURVE NUMBER FOR EACH SUBAREA FORMING THE STAGE I WATERSHED.

THE STAGE I HYDROLOGY WILL BE BROKEN DOWN INTO EAST AND WEST-SIDE DRAINAGE AS FOLLOWS.

EAST SIDE

EAST SIDE DRAINAGE IS LIMITED TO THE FRONT FACE OF STAGE I. THE DRAINAGE CHANNEL IS PARTIALLY BUILT UP TO THE FIRST BENCH (EL. 1148.2) AND WILL BE EXTENDED TO INTERCEPT FLOW FROM THE 1193.2 BENCH. THE ALIGNMENT OF THE CHANNEL AS IT FOLLOWS THE ACCESS ROAD SHOULD NOT POSE A PROBLEM DUE TO THE SMALL DRAINAGE AREA AND EXPECTED FLOW.

DITCH A

DRAINAGE AREA - 90,985 S.F. (OUTLINED IN ORANGE)

T_c : (SHOWN BY RED DASHED LINE)

a) 635 FT ALONG BENCH @ 1%
FROM SHEET 2, OBTAINED FROM SCS TR-55, FOR 1% SLOPE AND PILE IN PROCESS V = 0.71 FPS

$$t_c = \frac{635\ FT}{0.71\ FPS} = 894\ SEC.$$

b) 15 FT ALONG 2:1 SLOPE (S = 50%)
FROM SHEET 2, FOR 50% SLOPE, PILE IN PROCESS
V = 5 FPS.

then computed by dividing the total overland flow length by the average velocity.

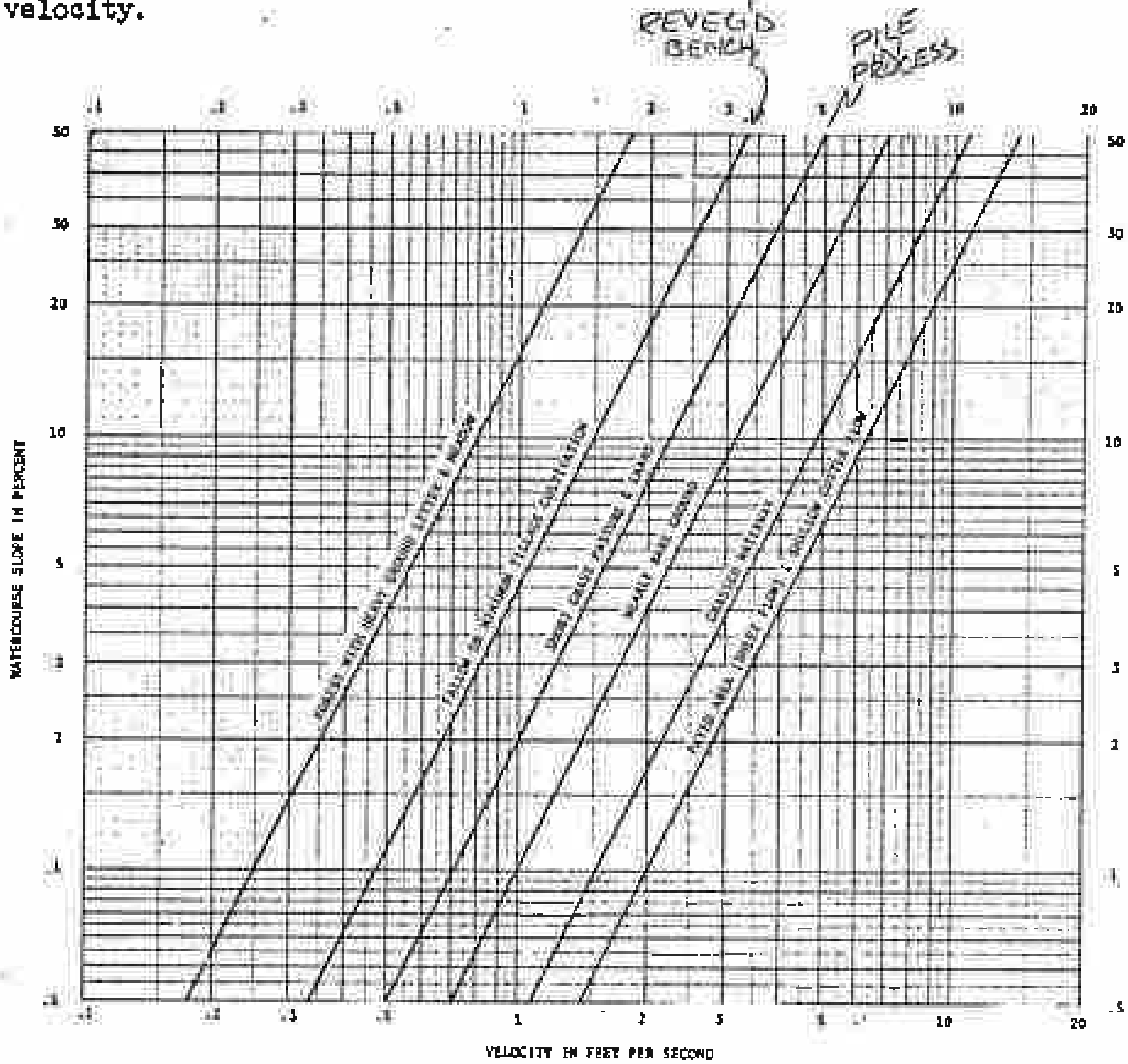

Figure 3-1.--Average velocities for estimating travel time for overland flow.

<u>Storm sewer or road gutter flow</u>
Travel time through the storm sewer or road gutter system to the main open channel is the sum of travel times in each individual component of the system between the uppermost inlet and the outlet. In most cases average velocities can be used without a significant loss of accuracy. During major storm events, the sewer system may be fully taxed and additional overland flow may occur, generally at a significantly lower velocity than the flow in the storm sewers. By using average conduit sizes and an average slope (excluding any vertical drops in the system), the average velocity can be estimated using Manning's formula.

Since the hydraulic radius of a pipe flowing half full is the same as when flowing full, the respective velocities are equal. Travel time may

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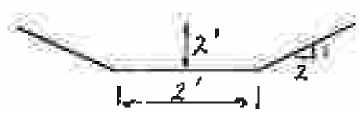
$t_c = \frac{15}{5} = 3$ SEC.

C) 300 FT ALONG CHANNEL

SLOPE (AVG) = $\frac{1182-1130}{300'} = 17.3\%$

ASSUME A CHANNEL IS TRAPEZOIDAL w/ SS=2:1, BASE=2F DEPTH = 2 FT. USE MANNING'S EQUATION TO COMPUTE THE VELOCITY FOR CHANNEL FLOWING FULL. USE GRASS FOR CHANNEL LINING w/ n=.045.

$V = \frac{1.49}{n} R^{2/3} S^{1/2}$ $\qquad R = A/P = \frac{1/2(2)(2+10)}{2+2(4.47)} = 1.096$



$= \frac{1.49}{.045} (1.096)^{2/3} (.173)^{1/2}$

$= 14.6$ FPS

$t_c = \frac{300 \text{ FT}}{14.6 \text{ FPS}} = 20.5$ SEC. ~ 21 SEC

$T_c = 21 + 3 + 894 = 918$ SEC.

CURVE NUMBER: CALCULATE A WEIGHTED CN ASSUMING THE UPPER SLOPE IS ACTIVE DISPOSAL WITH THE REST OF FACE IS IN PROCESS.

CN = 85 FOR ACTIVE DISPOSAL = 20,421 S.F.

CN = 82 FOR BENCHES IN PROCESS = 70,564 S.F.

$CN_w = \frac{85(20,421) + 82(70,564)}{90,985} = 82.7$

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SO, FOR DITCH A,

D.A. = 90,985 FT^2 = 0.0033 SQ. MI.
T_c = 918 SEC = 0.26 HR.
CN_w = 82.7

DITCH B

DRAINAGE AREA = 25,730 S.F. (SHOWN IN BLUE)
= 0.00092 SQ. MI.

Tc: (SHOWN BY ORANGE DASHED LINE)

300 FT, OVERLAND FLOW, SLOPED = $\frac{1153 - 1115}{300}$ = 12.7

FROM SHEET 2, FOR 12.7% SLOPE AND A GROUND COVER OF SHORT GRASS PASTURE & LAWNS, V=2.5 FF

$T_c = \frac{300 \text{ FT}}{2.5 \text{ FPS}} = 120$ SEC.

CURVE NUMBER - USE A CN = 80 CORRESPONDING TO AN OFFSITE PASTURE AREA.

SO, FOR DITCH B,

D.A. = 25,730 FT^2 = 0.00092 MI^2
T_c = 120 SEC = 0.033 HR.
CN = 80

SUBJECT KEYSTONE - STAGE I HYDROLOGY

BY DMK DATE 3/18/85 PROJ. NO. 85-205-4

CHKD. BY EHK DATE 3/20/85 SHEET NO. 5 OF 10

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WEST SIDE

THE WEST SIDE DRAINAGE SYSTEM WILL HANDLE THE ENTIRE FLOW FROM THE WORKING SURFACE OF THE PILE. APPROXIMATELY 300 FT. OF THE DITCH HAS BEEN CONSTRUCTED AND WILL BE EXTENDED AS SHOWN ON WORKSHEET WS-3.

DITCH C

DITCH C IS LOCATED AT THE SOUTHWEST CORNER OF STAGE I AS SHOWN BY THE RED LINE ON WS-3.

DRAINAGE AREA - THE MAXIMUM AREA CONTRIBUTING RUNOFF TO DITCH C IS THE SURFACE OF THE 1268 BENCH WHICH IS ALSO THE BENCH WITH THE MAXIMUM SURFACE AREA. FROM DMK's STAGE I VOLUME CALCS OF 2/20/85,

AREA OF 1268 BENCH = 1,123,100 FT^2 = 0.040 MI^2

26 ACRES

T_c: (SHOWN BY BLUE DASHED LINE)

a) 1700' OF OVERLAND FLOW ON ACTIVE WORKING SURFACE OF PILE AT ~ 1% SLOPE. ENTER FIGURE ON SHEET 2 WITH 1% SLOPE ON NEARLY BARE GROUND, V = 1 FPS.

$$t_c = \frac{1700'}{1\ FPS} = 1700\ SEC.$$

b) 100 FT ALONG CHANNEL: ASSUME THAT CHANNEL HAS CROSS-SECTION ON SHEET 3, AND IS FLOWING FULL. COMPUTE V.

SUBJECT KEYSTONE - STAGE I HYDROLOGY

BY DMK DATE 3/18/85 PROJ. NO. 85-205-4

CHKD. BY EHK DATE 3/20/85 SHEET NO. 6 OF 10

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$$V = \frac{1.49}{n} R^{2/3} S^{1/2}$$

$$S_{avg.} = \frac{1260 - 1243}{100} = 17.0\ \%$$

$n = 0.040$ FOR GROUTED RIP RAP

$$V = \frac{1.49}{0.04} (1.096)^{2/3} (.170)^{1/2}$$

$V = 16.3$ FPS.

$$t_c = \frac{100\ FT}{16.3\ FPS} = 6.1\ SEC. \sim 6\ SEC$$

T_c = 1700 SEC + 6 SEC = 1706 SEC = 0.47 HR.

CURVE NUMBER: USE CN = 85 CORRESPONDING TO ACTIVE DISPOSAL SURFACE.

SO, FOR DITCH C

D.A. = 1,123,000 FT^2 = 0.040 MI^2
T_c = 1706 SEC = 0.47 HR
CN = 85

DITCH D

DRAINAGE AREA - (SHOWN BY ORANGE LINE)
= 138,544 FT^2 = 0.0050 SQ.MI.

T_c: SHOWN BY GREEN DASHED LINE.

SUBJECT KEYSTONE - STAGE I HYDROLOGY

BY DMK DATE 3/13/85 PROJ. NO. 85-205-4

CHKD. BY EHK DATE 3/20/85 SHEET NO. 7 OF 10

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a) 1000 FT ALONG REVEGETATED BENCH SLOPED AT 1%. TO FIND OVERLAND VELOCITY ENTER FIGURE ON SHEET 2 WITH 1% SLOPE AND FALLOW CULTIVATION; V = 0.47 FPS.

$$t_c = \frac{1000 \text{ FT}}{0.47 \text{ FPS}} = 2128 \text{ SEC.}$$

b) 500 FT IN CHANNEL; ASSUME THAT CHANNEL HAS SAME CROSS SECTION AS THAT DESCRIBED ON SHEET 3 AND IS FLOWING FULL. COMPUTE V.

$$V = \frac{1.49}{n} R^{2/3} S^{1/2}$$

$$S_{avg} = \frac{1243 - 1210}{500} = 6.6\ \%$$

n = 0.04 FOR GROUTED RIP RAP

$$V = \frac{1.49}{0.04} (1.096)^{2/3} (.066)^{1/2}$$

V = 10.2 FPS

$$t_c = \frac{500 \text{ FT}}{10.2 \text{ FPS}} = 49.0 \text{ SEC.}$$

T_c = 2128 + 49 = 2177 SEC = 0.60 HRS.

CN: USE CURVE NUMBER = 78 FOR REVEGETATED BENCHES

SUBJECT KEYSTONE - STAGE I HYDROLOGY

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CHKD. BY EHK DATE 3/20/85 SHEET NO. 8 OF 10

SO, FOR DITCH D

D.A. = 0.0050 MI^2
T_c = 0.60 HRS
CN = 78

DITCH E

DRAINAGE AREA = 37,808 SQ.FT. = 0.0014 MI
(OUTLINED IN GREEN)

T_c: (SHOWN BY BLUE DASHED LINE)

a) 140 FT @ 10% ALONG HAUL ROAD
ENTER FIGURE ON SHEET 2 WITH 10% SLOPE AND NEARLY BARE GROUND. V = 3.1 FPS.

$t_c = \frac{140\ FT}{3.1\ FPS} = 45.2$ SEC. ~ 45 SEC

b) 170 FT ALONG BENCH SLOPED AT 1%.
ENTER FIGURE ON SHEET 2 WITH 1% SLOPE AND FALLOW CULTIVATION V = 0.47 FPS.

$t_c = \frac{170\ FT}{0.47\ FPS} = 361.7$ SEC ~ 362 SEC

c) 310 FT ALONG CHANNEL: ASSUME CHANNEL HAS SAME CROSS SECTION AS THAT DESCRIBED ON SHEET 3 AND IS FLOWING FULL. COMPUTE V

$$V = \frac{1.49}{n} R^{2/3} S^{1/2}$$

$$S_{avg} = \frac{1210 - 1155}{310'} = 17.7\ \%$$

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$n = 0.04$ FOR GROUTED RIP RAP

$$V = \frac{1.49}{.04} (1.096)^{2/3} (.177)^{1/2}$$

$V = 16.7$ FPS.

$$t_c = \frac{310 \text{ FT}}{16.7 \text{ FPS}} = 18.6 \text{ SEC.} \sim 19 \text{ SEC}$$

$T_c = 45 + 362 + 19 = 426$ SEC $= 0.12$ H

CN = 78 FOR REVEGETATED AREA.

SO, FOR DITCH E

DA = 0.0014

T_c = 0.12

CN = 78

DITCH F

SINCE DITCH F HAS VERY LITTLE DRAINAGE AREA, IT WILL BE SIZED FOR THE SAME PEAK FLOW AS DITCH E.

DITCH G

DITCH G MUST CARRY THE FLOWS FROM DITCHES B AND F.

SUBJECT KEYSTONE - STAGE I HYDROLOGY

BY DMK DATE 3/18/85 PROJ. NO. 85-205-4

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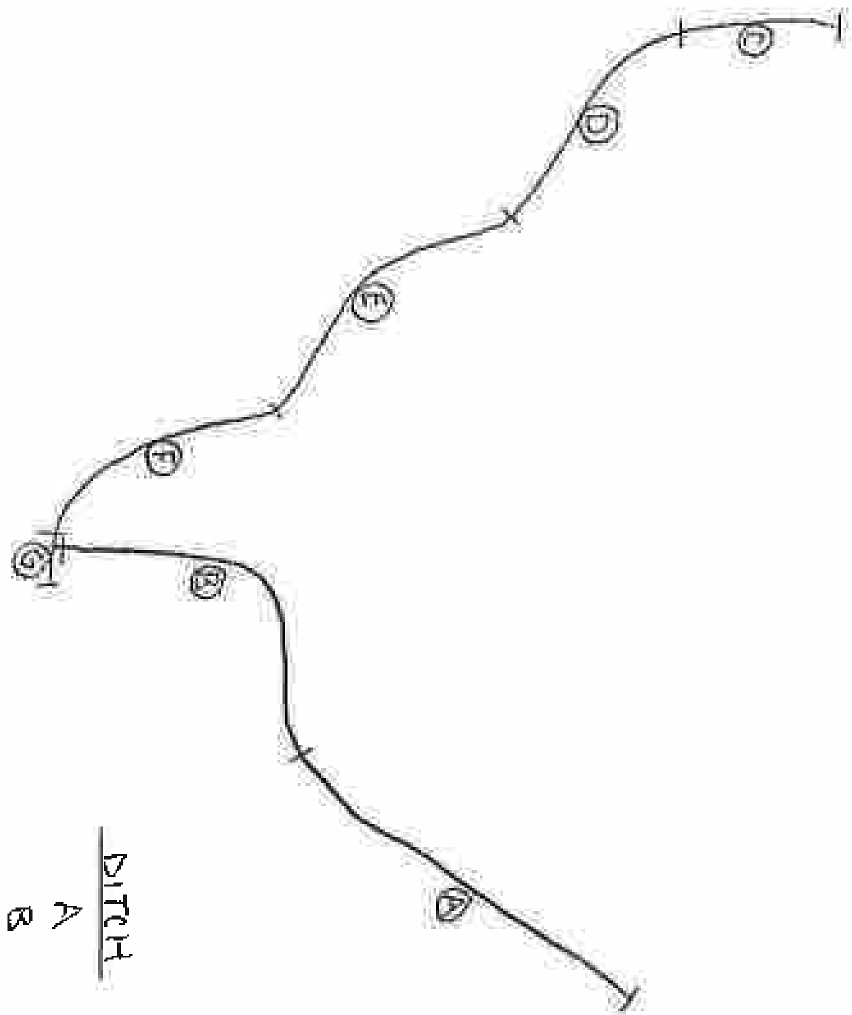
SUMMARY OF STAGE I HYDROLOGY



DITCH	DRAINAGE AREA	Tc	CN
A	0.0033 MI²	0.26 HR	82.7
B	0.00092	0.033	80
C	0.040	0.47	85
D	0.0050	0.60	78
E	0.0014	0.12	78
F	-	-	-
G	-	-	-

SUBJECT Hydrologic Parameters for Channel Design
Penelec-Keystone

BY EHK DATE 2/7/85 PROJ. NO. 85-205-7

CHKD. BY DMK DATE 3/27/85 SHEET NO. 1 OF 32

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The channel hydrology will be done by computer. Some background parameters will be needed for input into the program. This calculation summarizes those parameters.

The hydrology for all flows is based on the following:

Design Storm - 100-year, 24-hour storm — Permanent Channel = 5.4 i
10-year, 24-hour storm — Temporary Channel = 3.9 i

Drainage Areas for each channel section
Average Slopes for drainage areas
Surface Cover for determination of Curve Numbers
Curve Numbers
Time of Concentration
Assumed Channel Section
Distance to next channel

SUBJECT Hydrologic Parameters for Channel Design
Penelec-Keystone

BY EHK DATE 2/12/85 PROJ. NO. 85-205-7

CHKD. BY DMK DATE 3/27/85 SHEET NO. 2 OF 32

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Ditch	Area	Drainage Area (mi^2)	Time of Concentration (hrs)	Runoff Curve Number* 85	Distance To Next Channel	Runoff Curve Number 70
✓ A[1]	1	0.071	0.80	75		61.8
✓ B[2]	2	0.026	0.57	75		61.8
✓ C[3]	3	0.019	0.37	75		61.8
	A				680 ft	
✓ D[4]	4	0.016	0.50	75		61.8
	5	0.031				
	B				2200 ft	
	C				2200 ft	
✓ E[5]	6	0.0026	0.43	78, 70 → 73.7		60.7
	7	0.0030	0.089			
✓ F[6]	8	0.0044	0.37	70, 78 → 75.8		62.4
	9	0.012	0.60			
	D				1390 ft	
	E				1390 ft	

*See p. 32

SUBJECT Hydrologic Parameters for Channel Design
Penelec-Keystone

BY EHK DATE 2/12/85 PROJ. NO. 85-205-7
CHKD. BY DMK DATE 3/27/85 SHEET NO. 3 OF 32

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Ditch	Area	Drainage Area (mi²)	Time of Concentration (hrs)	Runoff Curve Number	Distance To Next Channel (ft)	
✓ G[7]	10	.0031	.47	70		57.6
	F				1010	
✓ H[8]	11	.016	.56	78		64.2
	12	.016	.89	78		64.2
✓ I[9]	13	.025	.77	78 74.2		61.1
	14	.022	.35	70		
	G				2320	
	H				2320	
✓ J[10]	~~15~~	~~.0039~~	~~.04~~	~~78~~		
	16	.0019	.076	70		57.6
✓ K[11]	15&17	.012	.66	78		64.2
	18	.013	.58	78		64.2
✓ L[12]	19	.014	.68	70		57.6

SUBJECT Hydrologic Parameters for Channel Design
Peneler-Keystone

BY EHK DATE 2/12/85 PROJ. NO. 85-205-7

CHKD. BY DMK DATE 3/27/85 SHEET NO. 4 OF 32

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Ditch	Area	Drainage Area (mi^2)	Time of Concentration (hrs)	Runoff Curve Number	Distance To Next Channel (ft)	
✓ L[12]	J				1970	
	K				1970	
✓ M[13]	20	.0045	.45	78		64.2
	21	.012	1.07	78		64.2
✓ N[14]	22	.0052	.54	78 75.1		61.8
	23	.0030	.16	70		
	M				660	
✓ O[15]	24	.023	.53	70		57.6
	N				2550	
	L					
✓ P[16]	25	.023	1.01	78		64.2
	O				270	
✓ Q[17]	26	.0031	.44	70		57.6
	P				1360	

SUBJECT Hydrologic Parameters for Channel Design
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CHKD. BY DMK DATE 3/27/85 SHEET NO. 5 OF 32

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Ditch	Area	Drainage Area (mi²)	Time of Concentration (hrs)	Runoff Curve Number	Distance To Next Channel (ft)	
✓ R18	27	.00024	.074	70		57.6
	Q				220	
1 S19	28	.0038	.39	78		63.1
	29	.00082	.13	70		
		.00462		76.6		

SUBJECT Hydrologic Parameters for Channel Design
Penelec-Keystone

BY EHK DATE 2/13/85 PROJ. NO. 85-205-7
CHKD. BY DMK DATE 3/27/85 SHEET NO. 6 OF 32

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A time of concentration must be determined for each area as described on pages 2-5 of these calculations.

Area 1

Overland:

1470 ft, 1.13% slope, Fallow or minimum tillage cultivation

From Figure 3.1 of Urban Hydrology for Small Watersheds, T.R. 55 (sheet 31)

$V = 0.52$ fps

$T = \frac{1470 \text{ ft}}{0.52 \text{ fps}} = 2827$ sec

Channel:

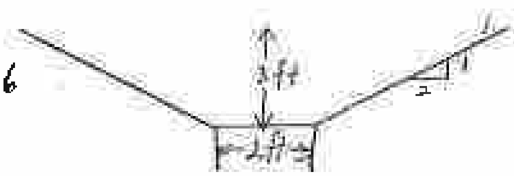
Use Manning's equation to compute bankfull velocity. Use this velocity as an average velocity

$V = \frac{1.49}{n} R_h^{2/3} S^{1/2}$

$n = .045$ for grass lined

$s = 0.01$

$R_h = \frac{A}{P_w} = \frac{[(2)(3) + 2(3)^2]}{2 + 2(3)\sqrt{1+2^2}} = 1.56$



$V = \frac{1.49}{.045}(1.56)^{2/3}(.01)^{1/2} = 4.45$ fps

$L = 240$ ft

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$T = \frac{240\ ft}{4.45\ fps} = 54\ sec$

$T_{Total} = 2827\ sec + 54\ sec = 2881\ sec = 48.0\ min = 0.80\ hr$

Area 2:

Overland:

1100 ft, 1.2% slope, Fallow or minimum tillage cultivation

From sheet 31

$V = 0.55\ fps$

$T = \frac{1100\ ft}{.55\ fps} = 2000\ sec$

Channel:

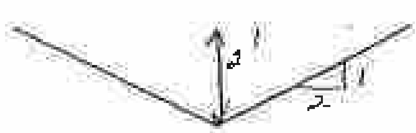
Compute an average velocity, assuming a channel and using bankfull conditions

$V = \frac{1.49}{n} R_h^{2/3} S^{1/2}$

$n = 0.045$

$S = 0.0098$

$R_h = \frac{2(2ft)}{2\sqrt{1+2^2}} = 0.89$



$V = \frac{1.49}{.045}(.89)^{2/3}(.0098)^{1/2}$

$V=3.03$ fps

$L=.210$ ft

$T=\frac{210 \text{ ft}}{3.03 \text{ fps}}=69$

$T_{Total}=2000$ sec + 69 sec = 2069 sec = 34.5 min = 0.57 hr.

Area 3:

Overland:

910 ft, 1.92%, Fallow or minimum tillage cultivation

From sheet 31

$V=.69$ fps

$T=\frac{910 \text{ ft}}{.69 \text{ fps}}=1319$ sec = 21.98 min = 0.37 hr.

Areas 4 & 5:

Overland:

T= 1319 sec from Area 3

Channel:

$V=\frac{1.49}{n}R_h^{2/3}S^{1/2}$

$n=.045$

$s=1\%$

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Penelec-Keystone

BY EHK DATE 2/14/85 PROJ. NO. 85-205-7

CHKD. BY DMK DATE 3/27/85 SHEET NO. 9 OF 32

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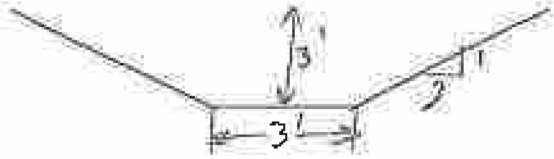

$$R_h = \frac{3(3)+2(3)^2}{3+2(3)\sqrt{1+2^2}} = 1.64$$

$$V = \frac{1.49}{.045}(1.64)^{2/3}(.01)^{1/2} = 4.60 \text{ fps}$$

$$T = \frac{2200 \text{ ft}}{4.60 \text{ fps}} = 478 \text{ sec}$$

$$T_{Total} = 1319 \text{ sec} + 478 \text{ sec} = 1797 \text{ sec} = 30.0 \text{ min} = .50 \text{ hr.}$$

SUBJECT Hydrologic Parameters for Channel Design
Penelec-Keystone

BY EHK DATE 2/13/85 PROJ. NO. 85-205-7

CHKD. BY DMK DATE 3/27/85 SHEET NO. 10 OF 32

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Area 6:

Overland:

L=30 ft at 50% slope

L= 770 ft at 1% slope

Fallow or minimum tillage cultivation

From sheet 31

$V_{50}=3.55$ fps, $V_1=0.5$ fps

$T=\frac{30 ft}{3.35 fps}+\frac{770 ft}{.5 fps}=1548 sec = 0.43 hr.$

SUBJECT Hydrologic Parameters for Channel Design
Penelec-Keystone

BY EHK DATE 2/13/85 PROJ. NO. 85-205-7
CHKD. BY DMK DATE 3/27/85 SHEET NO. 11 OF 32

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Area 7:

Overland:

65 ft, $S=4.62\%$, Forest with heavy ground litter and meado

From sheet 31

$V=.54$ fps

$T=\frac{65 \text{ ft}}{.54 \text{ fps}}=120.37 \text{ sec} \approx 120 \text{ sec}$

Channel:

$V=\frac{1.49}{n}R_h^{2/3}S^{1/2}$

$n=0.045$

$S=.0068$

$R_h=1.56$ (see sheet 6)

$V=\frac{1.49}{.045}(1.56)^{2/3}(.0068)^{1/2}=3.67$ fps

$L=730$ ft

$T=\frac{730 \text{ ft}}{3.67 \text{ fps}}=198.91 \text{ sec} \approx 199 \text{ sec}$

$T_{Total}=120 \text{ sec} + 199 \text{ sec} = 319 \text{ sec} \approx 5.32 \text{ min} = 0.089 \text{ hr}.$

Area 8:

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Overland:

650 ft, S=4.31%, Forest with heavy ground litter & meadow

From sheet 31

$V = .52$ fps

$T = \frac{650 \text{ ft}}{.52 \text{ fps}} = 1250$ sec

Channel:

$V = \frac{1.49}{n} R_h^{2/3} S^{1/2}$

$n = 0.045$

$S = 0.062$

$R_h = 1.56$ (see sheet 6)

$V = \frac{1.49}{.045}(1.56)^{2/3}(.062)^{1/2} = 11.09$ fps

$L = 950$ ft

$T = \frac{950 \text{ ft}}{11.09 \text{ fps}} = 85.66 \text{ sec} \approx 86$ sec

$T_{Total} = 1250 \text{ sec} + 86 \text{ sec} = 1336 \text{ sec} = 22.27 \text{ min} = 0.37$ hr

Area 9:

Overland:

$L = 25$ ft at 50%

$L = 1025$ ft at 1%

Fallow...or minimum tillage cultivation

From sheet 31

$V_{50} = 3.55$ fps, $V_1 = 0.5$ fps

$T = \frac{25 \text{ ft}}{3.55 \text{ fps}} + \frac{1025 \text{ ft}}{.5 \text{ fps}} = 2057$

Channel:

$V = 11.09$ fps (see sheet 12)

$L = 1200$ ft

$T = \frac{1200 \text{ ft}}{11.09 \text{ fps}} = 108.2 \text{ sec} \approx 108 \text{ sec}$

$T_{Total} = 2057 \text{ sec} + 108. \text{ sec} = 2165 \text{ sec} = 36.1 \text{ min} = .60 \text{ hr}$

Area 10:

Overland:

890 ft, $S = 6.52\%$, Forest with heavy ground litter & meadow

From sheet 31

$V = 0.64$ fps

SUBJECT Hydrologic Parameters for Channel Design
Penelec-Keystone

BY EHK DATE 2/13/85 PROJ. NO. 85-205-7

CHKD. BY DMK DATE 3/27/85 SHEET NO. 14 OF 32

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$T = \frac{890 \text{ ft}}{.64 \text{ fps}} = 1390.62 \text{ sec} \approx 1391 \text{ sec}$

Channel:

$V = \frac{1.49}{n} R_h^{2/3} S^{1/2}$

$n = 0.045$

$S = 0.005$

$R_h = 0.89$ (see sheet 7)

$V = \frac{1.49}{.045}(.89)^{2/3}(.005)^{1/2} = 2.17 \text{ fps}$

$L = 690 \text{ ft}$

$T = \frac{690 \text{ ft}}{2.17 \text{ fps}} = 317.97 \text{ sec} \approx 318 \text{ sec}$

$T_{Total} = 1391 \text{ sec} + 318 \text{ sec} = 1709 \text{ sec} = 28.48 \text{ min} = 0.47 \text{ hr.}$

Area 11:

Overland:

$L = 25$ ft at 50%

$L = 1000$ ft at 1%

Fallow or minimum tillage caltivation

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From sheet 31

$V_{60} = 3.55$ fps, $V_1 = 0.5$ fps

$T = \frac{25 \text{ ft}}{3.55 \text{ fps}} + \frac{1000 \text{ ft}}{.5 \text{ fps}} = 2007$ sec

Channel:

$V = \frac{1.49}{n} R_h^{2/3} S^{1/2}$

$n = 0.030$ for Fabriform

$S = 0.3$

$R_h = 0.89$ (See sheet 7)

$V = \frac{1.49}{.03} (.89)^{2/3} (.3)^{1/2} = 25.17$ fps

$L = 410$ ft

$T = \frac{410 \text{ ft}}{25.17 \text{ fps}} = 16.29$ sec ≈ 16 sec

$T_{Total} = 2007$ sec $+ 16$ sec $= 2023$ sec $= 33.7$ min $= 0.56$ hr.

Area 12:

Overland:

1595 ft, S=1%, Fallow or minimum tillage cultivation

From sheet 31

SUBJECT Hydrologic Parameters for Channel Design
Penelec-Keystone

BY EHK DATE 2/13/85 PROJ. NO. 85-205-7

CHKD. BY DMK DATE 3/27/85 SHEET NO. 16 OF 32

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$V = 0.50$ fps

$T = \frac{1595 \text{ ft}}{.50 \text{ fps}} = 3190$ sec

Channel:

$V = 25.17$ fps (see sheet 15)

$L = 60$ ft

$T = \frac{60 \text{ ft}}{25.17 \text{ fps}} = 2.38 \text{ sec} \approx 2 \text{ sec}$

$T_{Total} = 3190 \text{ sec} + 2 \text{ sec} = 3192 \text{ sec} = 53.2 \text{ min} = .89 \text{ hr.}$

Area 13:

Overland:

1315 ft, S=1%, Fallow or minimum tillage cultivation

$V = .50$ fps (see sheet 16)

$T = \frac{1315 \text{ ft}}{.50 \text{ fps}} = 2630$ sec.

Channel:

$V = \frac{1.49}{n} R_h^{2/3} S^{1/2}$

$n = 0.045$

$S = .0886$

SUBJECT Hydrologic Parameters for Channel Design
Penelec-Keystone

BY EHK DATE 2/13/85 PROJ. NO. 85-205-7
CHKD. BY DMK DATE 3/27/85 SHEET NO. 17 OF 32

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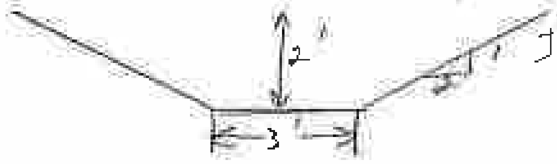

$$R_h = \frac{(3)(2)+2(2)^2}{3+2(2)\sqrt{1+2^2}} = 1.17$$

$$V = \frac{1.49}{.045}(1.17)^{2/3}(.0886)^{1/2} = 10.94 \text{ fps}$$

$$T = \frac{1700 \text{ ft}}{10.94 \text{ fps}} = 155 \text{ sec}$$

$T_{Total} = 2630$ sec + 155 sec = 2785. sec = 0.77 hr

Area 14:

Overland:

1400 ft, S=18.75%, Forest with heavy ground litter & meado

From sheet 31

V=1.1 fps

$$T = \frac{1400 \text{ ft}}{1.1 \text{ fps}} = 1273 \text{ sec}$$

$T_{Total} = 1273$ sec = 21.2 min = 0.35 hr

~~Area 15:~~

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BY EHK DATE 2/14/85 PROJ. NO. 85-205-7

CHKD. BY DMK DATE 3/27/85 SHEET NO. 20 OF 32

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$T = \frac{1180\ ft}{.50\ fps} = 2630\ sec$

Channel:

$V = \frac{1.49}{n} R_h^{2/3} S^{1/2}$

$n = 0.03$ for Fabriform

$S = 0.3$

$R_h = 1.56$ (see sheet 6)

$V = \frac{1.49}{.03}(1.56)^{2/3}(.3)^{1/2} = 36.59\ fps$

$L = 350\ ft$

$T = \frac{350\ ft}{36.59\ fps} = 10\ sec$

$T_{Total} = 2360\ sec + 10\ sec = 2370\ sec = 39.5\ min = 0.66\ hr$

Area 18:

Overland:

$L = 10\ ft$ at $S = 50\%$

$L = 1030\ ft$ at $S = 1\%$

Fallow or minimum tillage cultivation

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BY EHK DATE 2/14/85 PROJ. NO. 85-205-7

CHKD. BY DMK DATE 3/27/85 SHEET NO. 21 OF 32

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From sheet 31

$V_{50} \doteq 3.55$ fps, $V_1 = 0.5$ fps

$$T = \frac{10\ ft}{3.55\ fps} + \frac{1030\ ft}{0.5\ fps} = 2063\ sec$$

Channel:

T=10 sec (see sheet 20)

$T_{Total} = 2063\ sec + 10\ sec = 2073\ sec = 34.6\ min = .58\ hr$

Area 19

Overland:

900 ft, S=4.27%, Forest with heavy ground litter & meadow

From sheet 31

V=0.515 fps

$$T = \frac{900\ ft}{.515\ fps} = 1748\ sec$$

Channel:

$$V = \frac{1.49}{n} R_h^{2/3} S^{1/2}$$

n=0.045

S=.005

R_h=0.89 (see sheet 7)

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Penelec-Keystone

BY EHK DATE 2/14/85 PROJ. NO. 85-205-7

CHKD. BY DMK DATE 3/27/85 SHEET NO. 22 OF 32

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$V=\frac{1.49}{.045}(.89)^{2/3}(.005)^{1/2}=2.17$ fps

$L=1500$ ft

$T=\frac{1500 \text{ ft}}{2.17 \text{ fps}}=692$ sec

$T_{Total}=1748$ sec $+ 692$ sec $=2440$ sec $=40.7$ min $=0.68$ hr

Area 20:

Overland

$L=10$ ft at $S=50\%$

$L=800$ ft at $S=1\%$

From sheet 31 Fallow or minimum tillage cultivation

$V_{50}=3.55$ fps, $V_1=0.5$ fps

$T=\frac{10 \text{ ft}}{3.55 \text{ fps}}+\frac{800 \text{ ft}}{0.5 \text{ fps}}=1603$ sec

Channel

$V=\frac{1.49}{n}R_h^{2/3}S^{1/2}$

$n=0.03$ for Fabriform

$S=0.3$

SUBJECT Hydrologic Parameters for Channel Design Penelec-Keystone

BY EHK DATE 2/14/85 PROJ. NO. 85-205-7

CHKD. BY DMK DATE 3/27/85 SHEET NO. 23 OF 32

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$R_h = 1.56$ (see sheet 6)

$V = \frac{1.49}{.03}(1.56)^{2/3}(.3)^{1/2} = 36.59$ fps

$L = 190$ ft

$T = \frac{190 \text{ ft}}{36.59 \text{ fps}} = 5$ sec

$T_{Total} = 1603$ sec $+ 5$ sec $= 1608$ sec $= 26.8$ min $= .45$ hr

Area 21

Overland:

540 ft, 10%, Paved area & shallow gutter flow

1880 ft, 1%, Fallow or minimum tillage cultivation

From sheet 31

$V_{10\%} = 6.3$ fps

$V_{1\%} = .50$ fps

$T = \frac{540 \text{ ft}}{6.3 \text{ fps}} + \frac{1880 \text{ ft}}{.50 \text{ fps}} = 3846$ sec

Channel:

$V = 36.59$ fps (see sheet 23)

$L = 60$ ft

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$T = \frac{60 \text{ ft}}{36.59 \text{ fps}} = 2 \text{ sec}$

$T_{Total} = 3846 \text{ sec} + 2 \text{ sec} = 3848 \text{ sec} = 64.1 \text{ min} = 1.07 \text{ hr.}$

Area 22

Overland:

30 ft, s=50%, Fallow or minimum tillage caltivation

930 ft, s=1%, Fallow or minimum tillage cultivation

From sheet 31

$V_{50} = 3.55 \text{ fps}$

$V_1 = .50 \text{ fps}$

$T = \frac{30 \text{ ft}}{3.55 \text{ fps}} + \frac{930 \text{ ft}}{.50 \text{ fps}} = 1868 \text{ sec}$

Channel:

$V = \frac{1.49}{n} R_h^{2/3} S^{1/2}$

$n = .045$

$S = .0877$

$R_h = 0.89$ (see sheet 7)

$V = \frac{1.49}{.045} (.89)^{2/3} (.0877)^{1/2} = 9.07 \text{ fps}$

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$L = 650\ ft$

$T = \frac{650\ ft}{9.07\ fps} = 72\ sec$

$T_{Total} = 1868\ sec + 72\ sec = 1940\ sec = 32.3\ min = .54\ hr$

Area 23

Overland

390 ft, S=7.56%, Forest with heavy ground litter & meadow

From sheet 31

$V = .69\ fps$

$T = \frac{390\ ft}{.69\ fps} = 565\ sec$

Channel

$V = 9.07\ fps$ (see sheet 24)

$L = 260\ ft$

$T = \frac{260\ ft}{9.07\ fps} = 29\ sec$

$T_{Total} = 565\ sec + 29\ sec = 594\ sec = 9.9\ min = .16\ hr$

Area 24

Overland

590 ft, S=10.61%, Forest with heavy ground litter & meado

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Penelec-Keystone

BY EHK DATE 2/14/85 PROJ. NO. 85-205-7

CHKD. BY DMK DATE 3/27/85 SHEET NO. 26 OF 32

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From sheet 31

$V = .81$ fps

$T = \frac{590 \text{ ft}}{.81 \text{ fps}} = 728$ sec

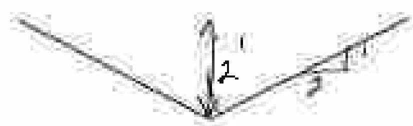
Channel

$V = \frac{1.49}{n} R_h^{2/3} S^{1/2}$

$n = .045$

$S = .005$

$R_h = \frac{2(2)}{2\sqrt{1+2^2}} = 0.89$



$V = \frac{1.49}{.045}(.89)^{2/3}(.005)^{1/2} = 2.17$ fps

$L = 2550$ ft

$T = \frac{2550 \text{ ft}}{2.17 \text{ fps}} = 1175$ sec

$T_{Total} = 728 \text{ sec} + 1175 \text{ sec} = 1903 \text{ sec} = 31.72 \text{ min} = .53 \text{ hr}$

Area 25

Overland

80 ft, 10%, Paved area & shallow gutter flow

1770 ft, 1%, Fallow or minimum tillage cultivation

SUBJECT Hydrologic Parameters for Channel Design
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BY EHK DATE 2/14/85 PROJ. NO. 85-205-7

CHKD. BY DMK DATE 3/27/85 SHEET NO. 27 OF 32

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$V_{10\%} = 6.3$ fps

$V_{1\%} = .50$ fps

$T = \frac{80 \text{ ft}}{6.3 \text{ fps}} + \frac{1770 \text{ ft}}{.50 \text{ fps}} = 3553$ sec

Channel

$V = \frac{1.49}{n} R_h^{2/3} S^{1/2}$

$n = .045$

$S = 11.8\%$

$R_h = 1.56$ (see sheet 6)

$V = \frac{1.49}{.045}(1.56)^{2/3}(.118)^{1/2} = 15.3$ fps

$L = 1040$ ft

$T = \frac{1040 \text{ ft}}{15.3 \text{ fps}} = 68$ sec

$T_{Total} = 3553$ sec $+ 68$ sec $= 3621$ sec $= 60.4$ min $= 1.01$ hr

Area 26

Overland

1250 ft, S=9.68%, Heavy forest with ground litter & meadow

From sheet 31

SUBJECT Hydrologic Parameters for Channel Design
Penelec-Keystone

BY EHK DATE 2/14/85 PROJ. NO. 85-205-7

CHKD. BY DMK DATE 3/27/85 SHEET NO. 28 OF 32

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$V = .79$ fps

$T = \frac{1250 \text{ ft}}{.79 \text{ fps}} = 1582$ sec

Channel

$V = \frac{1.49}{n} R_h^{2/3} s^{1/2}$

$n = .045$

$s = 7.48\%$

$R_h = 1.56$ (see sheet 6)

$V = \frac{1.49}{.045} (1.56)^{2/3} (.0748)^{1/2} = 12.18$ fps

$L = 310$ ft

$T = \frac{310 \text{ ft}}{12.18 \text{ fps}} = 25$ sec

$T_{Total} = 1582 \text{sec} + 25 \text{sec} = 1607$ sec $= 26.8$ min $= 0.44$ hr.

Area 27

Overland

250 ft, $s = 13.75\%$, Heavy forest with ground litter & meadow

From sheet 31

$V = .94$ fps

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Penelec-Keystone

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BY FHK DATE 2/14/85 PROJ. NO. 85-205-7
CHKD. BY DMK DATE 3/27/85 SHEET NO. 29 OF 32

$T = \frac{250\ ft}{.94\ fps} = 266\ sec = 4.43\ min = .074\ hr$

Area 28
Overland
30ft, S=50%, Fallow or minimum tillage cultivation

680 ft, S=1%, Fallow or minimum tillage cultivation

$V_{50\%} = 3.55\ fps$ (see sheet 24)

$V_{1\%} = .50\ fps$ (see sheet 24)

$T = \frac{30\ ft}{3.55\ fps} + \frac{680\ ft}{.50\ fps} = 1368\ sec$

Channel
$V = \frac{1.49}{n} R_h^{2/3} S^{1/2}$

$n = .045$

$S = 16.86\%$

$R_h = 1.10$ (see sheet 18)

$V = \frac{1.49}{.045}(1.10)^{2/3}(.1686)^{1/2} = 14.49\ fps$

$L = 350\ ft$

$T = \frac{350\ ft}{14.49\ fps} = 24\ sec$

$T_{Total} = 1368\ sec + 24\ sec = 1392\ sec = 23.2\ min = .39\ hr$

Area 29

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BY EHK DATE 2/14/85 PROJ. NO. 85-205-7

CHKD. BY DMK DATE 3/27/85 SHEET NO. 30 OF 32

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Over land

420 ft, S=15.96%, Heavy forest with ground litter + meado.

From sheet 31

$V \approx 1$ fps

$T = \frac{420 \text{ ft}}{1 \text{ fps}} = 420$ sec $= 7.83$ min $= .13$ hrs

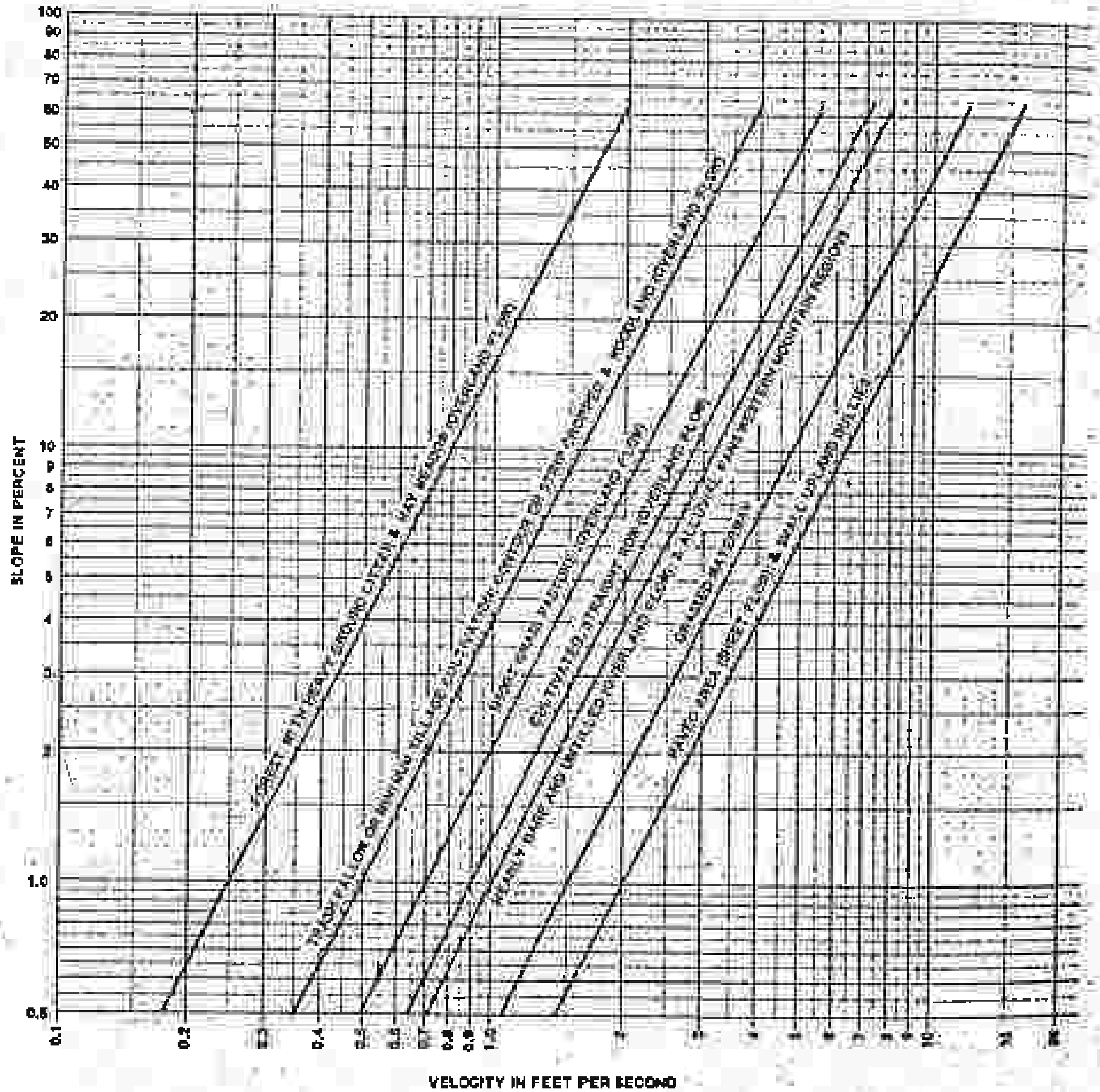

Figure 8. Velocities for upland method of estimating T_c

Table 9.1.--Runoff curve numbers for hydrologic soil-cover complexes

(Antecedent moisture condition II, and $I_a = 0.2\ S$)

Land use	Cover: Treatment or practice	Cover: Hydrologic condition	Hydrologic soil group: A	B	C	D
Fallow	Straight row	----	77	86	91	94
Row crops	"	Poor	72	81	88	91
	"	Good	67	78	85	89
	Contoured	Poor	70	79	84	88
	"	Good	65	75	82	86
	" and terraced	Poor	66	74	80	82
	" " "	Good	62	71	78	81
Small grain	Straight row	Poor	65	76	84	88
		Good	63	75	83	87
	Contoured	Poor	63	74	82	85
		Good	61	73	81	84
	" and terraced	Poor	61	72	79	82
		Good	59	70	78	81
Close-seeded legumes 1/ or rotation meadow	Straight row	Poor	66	77	85	89
	" "	Good	58	72	81	85
	Contoured	Poor	64	75	83	85
	"	Good	55	69	78	83
	" and terraced	Poor	63	73	80	83
	" and terraced	Good	51	67	76	80
Pasture or range		Poor	68	79	86	89
		Fair	49	69	79	84
		Good	39	61	74	80
	Contoured	Poor	47	67	81	88
	"	Fair	25	59	75	83
	"	Good	6	35	70	79
Meadow		Good	30	58	71	78
Woods		Poor	45	66	77	83
		Fair	36	60	73	79
		Good	25	55	70	77
Farmsteads		----	59	74	82	86
Roads (dirt) 2/		----	72	82	87	89
(hard surface) 2/		----	74	84	90	92

1/ Close-drilled or broadcast.
2/ Including right-of-way.

IBM FORTRAN Coding Form

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PROGRAMMER EHK
DATE 3/3/85
PUNCHING INSTRUCTIONS — GRAPHIC / PUNCH
PAGE 1 OF 9
CARD ELECTRO NUMBER 85-205-1

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3 STRUCT    01
8                   1321.0    0.0       50.0
9 ENDTBL
3 STRUCT    02
8                   1314.5    0.10      50.0
9 ENDTBL
3 STRUCT    03
8                   1314.5    0.0       50.0
9 ENDTBL
3 STRUCT    04
8                   1291.0    0.0       50.0
9 ENDTBL
```

*A standard card form, IBM electro 888157, is available for punching statements from this form

**Number of forms per pad may vary slightly

IBM FORTRAN Coding Form

GX28-7327-6 U/M 050**
Printed in U.S.A.

PROGRAM TRSD Keystone Closure Plan
PROGRAMMER EHK
DATE 3/3/85
PUNCHING INSTRUCTIONS GRAPHIC / PUNCH
PAGE 2 OF 9
CARD ELECTRO NUMBER* 85-205-7

```
 3 STRUCT      05
 8                      1291.0     0.0        50.0
 9 ENDTBL
 3 STRUCT      06
 8                      1206.0     0.0        50.0
 9 ENDTBL
 3 STRUCT      07
 8                      1204.0     0.0        50.0
 9 ENDTBL
 3 STRUCT      08
 8                      1204.0     0.0        50.0
 9 ENDTBL
```

*A standard card form, IBM electro 888157, is available for punching statements from this form.

**Number of forms per pad may vary slightly

IBM

FORTRAN Coding Form

GX28-7327-6 U/M 050**
Printed in U.S.A.

PROGRAM TR20 Keystone Closure Plan

PROGRAMMER EHK

DATE 3/3/85

PUNCHING INSTRUCTIONS | GRAPHIC | PUNCH

PAGE 3 OF 9

CARD ELECTRO NUMBER* 85-205-7

C FOR COMMENT	STATEMENT NUMBER	CONT.	FORTRAN STATEMENT	IDENTIFICATION SEQUENCE

```
 3   STRUCT   09

 8                      999.0       0.0        50.0

 9   ENDTBL

 3   STRUCT   10

 8                     1216.0       0.0        50.0

 9   ENDTBL

 3   STRUCT   11

 8                     1216.0       0.0        50.0

 9   ENDTBL

 3   STRUCT   12

 8                     1208.0       0.0        50.0

 9   ENDTBL
```

*A standard card form, IBM electro 888157, is available for punching statements from this form.

**Number of forms per pad may vary slightly.

IBM

FORTRAN Coding Form

GX28-7327-6 U/M 050**
Printed in U.S.A.

PROGRAM: TR20 Keystone Closure Plan
PROGRAMMER: EHK
DATE: 3/3/85
PUNCHING INSTRUCTIONS: GRAPHIC / PUNCH
PAGE 4 OF 9
CARD ELECTRO NUMBER*: X3 205 - 7

```
 3 STRUCT      13
 8                      1263.0     0.0         50.0
 9 ENDTBL
 3 STRUCT      14
 8                      1207.0     0.0         50.0
 9 ENDTBL
 3 STRUCT      15
 8                      1176.0     0.0         50.0
 9 ENDTBL
 3 STRUCT      16
 8                      1144.0     0.0         50.0
 9 ENDTBL
```

*A standard card form, IBM electro 888157, is available for punching statements from this form

**Number of forms per pad may vary slightly

IBM FORTRAN Coding Form GX28-7327-6 U/M 050** Printed in U.S.A.

PROGRAM TR20 Keystone Closure Plan | PUNCHING INSTRUCTIONS | GRAPHIC | PUNCH | PAGE 5 OF 9

PROGRAMMER EHK | DATE 3/3/85 | CARD ELECTRO NUMBER* 85-205-7

C FOR COMMENT	STATEMENT NUMBER	CONT.	FORTRAN STATEMENT	IDENTIFICATION SEQUENCE

```
 3 STRUCT        17
 8                       1043.0          0.0            50.0

 9 ENDTBL

 3 STRUCT        18
 8                       1010.0          0.0            50.0

 9 ENDTBL

 3 STRUCT        19
 8                       1010.0          0.0            50.0

 9 ENDTBL

 6 RUNOFF   1    01     5 0.071          75.0           0.80          1
 6 RUNOFF   1    02     6 0.026          75.0           0.57          1
 6 RUNOFF   1    03     7 0.01[illegible]     75.0           0.37          1
```

*A standard card form, IBM electro 888157, is available for punching statements from this form.

**Number of forms per pad may vary slightly

FORTRAN CODING FORM

SYSTEM

PROGRAM TR20 Keystone Closure Plan

DATE 3/4/85 |

PROGRAMMER EHK

```
6 ADDHYD 4    03 5 7 4                                              1
6 RUNOFF 1    04     5 0.047         75.0           0.50            1
6 ADDHYD 4    04 4 5 3                                              1
6 ADDHYD 4    04 3 6 4                                              1
6 RUNOFF 1    05     5 0.0056        73.7           0.43            1
6 RUNOFF 1    06     6 0.0164        75.8           0.60            1
6 ADDHYD 4    06 5 6 3                                              1
6 ADDHYD 4    06 3 4 2                                              1
6 RUNOFF 1    07     5 0.0031        70.0           0.47            1
6 ADDHYD 4    07 2 5 4                                              1
6 RUNOFF 1    08     5 0.032         78.0           0.89            1
6 RUNOFF 1    09     6 0.1047        74.2           0.47            1
```

STANDARD REGISTER CORPORATION

REORDER NO. FORTRAN 1465

FORTRAN CODING FORM

SYSTEM

PROGRAM TR20 Keystone Closure Plan

DATE 3/4/55

PAGE 7 OF 9

PROGRAMMER EHK

```
6 ADDHYD 4    09 4 6 3                                         1
6 ADDHYD 4    09 3 5 1                                         1
6 RUNOFF 1    10     5 0.0019      70.0        0.076           1
6 RUNOFF 1    11     6 0.025       78.0        0.66            1
6 RUNOFF 1    12     7 0.014       70.0        0.68            1
6 ADDHYD 4    12 5 7 4                                         1
6 ADDHYD 4    12 4 6 3                                         1
6 RUNOFF 1    13     5 0.0165      78.0        1.07            1
6 RUNOFF 1    14     6 0.0082      78.1        0.54            1
6 ADDHYD 4    14 5 6 4                                         1
6 RUNOFF 1    15     5 0.023       70.0        0.53            1
6 ADDHYD 4    15 4 5 2                                         1
```

FORTRAN CODING FORM

SYSTEM

PROGRAM TR20 Keystone Closure Fin.

DATE 3/14/80 PAGE 7 OF 9

PROGRAMMER EHK

STATEMENT LABEL | CONT | FORTRAN STATEMENT | STATEMENT SEQUENCE NO.

```
6 ADDHYD 4    15 2 3 4                                              1
6 RUNOFF 1    16     5 0.023        78.0         1.01               1
6 ADDHYD 4    16 4 5 3                                              1
6 RUNOFF 1    17     5 0.0031       70.0         0.44               1
6 ADDHYD 4    17 3 5 4                                              1
6 RUNOFF 1    18     5 0.00034      70.0         0.094              1
6 ADDHYD 4    18 4 5 2                                              1
6 RUNOFF 1    19     5 0.00462      76.6         0.39               1
  ENDATA
7 LIST
7 INCREM 6             0.2
7 COMPUT 7    01    09 0.0          5.3          1.0                3   2
```

COMPUTER SUPPLY CORPORATION

REORDER NO. FORTRAN 1465

FORTRAN CODING FORM

SYSTEM

PROGRAM TR20 Keystone Closure Plan

DATE 3/4/85 PAGE 9 OF 9

PROGRAMMER EHK

STATEMENT LABEL	CONT.	FORTRAN STATEMENT	STATEMENT SEQUENCE

```
   ENDCMP  1
17 COMPUT  7     10    18 0.0          5.3         1.0          3   2
   ENDCMP  1
7  COMPUT  7     19    19 0.0          5.3         1.0          3   2
   ENDCMP  1
   ENDJOB  2
```

JUPITER SUPPLY CORPORATION

REORDER NO. FORTRAN 1465

CONSULTANTS, INC.
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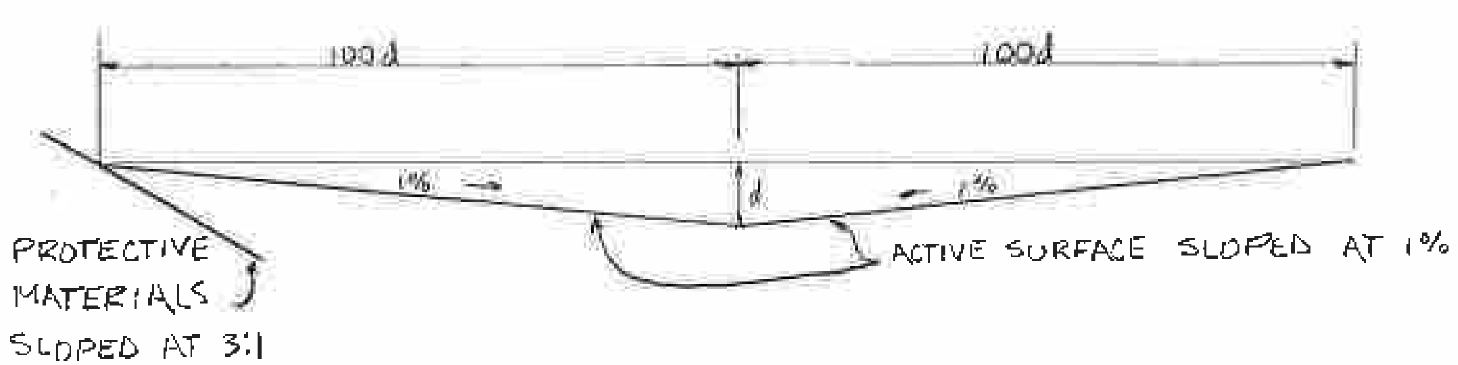
Environmental Specialists

THIS SET OF CALCULATIONS WAS PERFORMED TO INVESTIGATE THE POSSIBILITY OF USING SHEET FLOW TO DRAIN THE ACTIVE SURFACE OF STAGE I INTO THE WEST COLLECTION DITCH. THE PEAK FLOW USED FOR THE ANALYSIS WAS OBTAINED FROM THE TR-20, "COMPUTER PROGRAM FOR PROJECT FORMULATION -- HYDROLOGY" DISTRIBUTED BY THE SCS. THE FIRST SECTION OF THE CALCULATIONS EXAMINES FLOW OCCURING ON THE ACTIVE SURFACE OF THE DISPOSAL PILE AND THE SECOND SECTION DEALS WITH THE FLOW FROM THE ACTIVE SURFACE TO THE COLLECTION DITCH.

FLOW ON ACTIVE SURFACE

PEAK FLOW = 70 CFS · OBTAINED FROM THE TR-20 COMPUTER PROGRAM FOR THE WORST CASE FLOW, 100-YR STORM.

CROSS-SECTION OF FLOW - ASSUME FLOW OCCURS IN A SWALE FORMED BY THE ACTIVE SURFACE OF THE PILE SLOPED AT 1% FROM BOTH SIDES.



SUBJECT KEYSTONE - STAGE I HYDRAULICS
SHEET FLOW OFF OF ACTIVE SURFACE
BY DMK DATE 3/20/85 PROJ. NO. 85-205-4
CHKD. BY DB DATE 3/21/85 SHEET NO. 2 OF 4

GAI CONSULTANTS, INC.
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AREA OF FLOW = $\frac{1}{2}(200\,d)(d) = 100\,d^2$

WETTED PERIMETER = $2\sqrt{d^2 + (100d)^2} = 200.01\,d$

HYDRAULIC RADIUS = $A/P = \frac{100\,d^2}{200.01d} = 0.50\,d$

NOW, FIND DEPTH OF FLOW FROM MANNING'S EQUATION, THEN FIND VELOCITY OF FLOW.

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

ASSUME: S = 1% FOR ACTIVE SURFACE OF PILE IN ORDER TO MINIMIZE VELOCITY AND POTENTIAL FOR EROSION.

n = 0.018 FOR RECENTLY EXCAVATED EARTH OBTAINED FROM "DESIGN CHARTS FOR OPEN-CHANNEL FLOW" PUBLISHED BY THE FEDERAL HIGHWAY ADMINISTRATION, PAGE 100.

$$70 = \frac{1.49}{0.018}(100\,d^2)(0.50d)^{2/3}(0.01)^{1/2}$$

$d = 0.47$ FT

$A = 100(0.47)^2 = 22.09$ FT^2

$V = \frac{Q}{A} = \frac{70 \text{ CFS}}{22.09 \text{ FT}^2} = 3.2$ FPS

THE VELOCITY OF 3.2 FPS IS OK FOR FLOW ON THE ACTIVE SURFACE AND WILL NOT CAUSE EROSION PROBLEMS.

SUBJECT KEYSTONE - STAGE I HYDRAULICS
SHEET FLOW OFF OF ACTIVE SURFACE
BY DMK DATE 3/20/85 PROJ. NO. 85-205-4
CHKD. BY DB DATE 3/21/85 SHEET NO. 3 OF 4

GAI CONSULTANTS, INC.
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FLOW FROM ACTIVE SURFACE TO COLLECTION DITCH

ASSUME THE SAME CROSS-SECTIONAL SHAPE OF FLOW AS BEFORE WITH THE SLOPE OF THE SWALE EQUAL TO 50% CORRESPONDING TO THE 2:1 SIDE SLOPES WHICH THE RUNOFF MUST FLOW OVER TO GET INTO GROUTED ROCK CHANNEL.

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

$$70 = \frac{1.49}{0.018} (100\, d^2)(0.50\, d)^{2/3} (0.50)^{1/2}$$

$$d = 0.23 \text{ FT}$$

$$A = 100\,(0.23)^2 = 5.29 \text{ FT}^2$$

$$V = \frac{Q}{A} = \frac{70 \text{ CFS}}{5.29 \text{ FT}^2} = 13.2 \text{ FPS}$$

THIS VELOCITY OF 13.2 FPS IS THE THEORETICAL ULTIMATE VELOCITY THAT CAN BE ACHIEVED IN A CHANNEL WITH CROSS-SECTION AND SLOPE DESCRIBED ABOVE. THE MAXIMUM LENGTH OF THE FLOW PATH OVER THE 2:1 (50%) SLOPE IS LIMITED TO 33.5 FEET CORRESPONDING TO A HORIZONTAL DISTANCE OF 30 FEET AND A VERTICAL DISTANCE OF 10 FEET AND WILL BE SHORTER THAN THIS A GREAT DEAL OF THE TIME.

SUBJECT KEYSTONE - STAGE I HYDRAULICS
SHEET FLOW OFF OF ACTIVE SURFACE

BY DMK DATE 3/20/85 PROJ. NO. 85-205-4

CHKD. BY DB DATE 3/21/85 SHEET NO. 4 OF 4

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CONSIDERING THE SHORT LENGTH OF THE FLOW PATH, IT IS REASONABLE TO EXPECT THAT THE ULTIMATE VELOCITY OF 13.2 FPS WILL NOT BE REACHED AND THAT THE MAXIMUM VELOCITY ACHIEVED RANGE FROM 8 TO 10 FPS WHICH IS WITHIN THE ACCEPTABLE LIMITS OF ENKAMAT.

ALSO, THE PEAK FLOW OF 70 CFS WAS COMPUTED BASED ON THE 100-YEAR, 24-HOUR STORM AND THE MAXIMUM ACTIVE DISPOSAL AREA ENCOUNTERED DURING STAGE I. THE LIKELIHOOD OF THESE TWO EVENTS OCCURRING SIMULTANEOUSLY IS MINIMAL, THUS THE FLOW SHOULD NEVER REACH THIS MAXIMUM VALUE.

SUBJECT KEYSTONE - STAGE I HYDRAULICS

BY DMK DATE 3/21/85 PROJ. NO. 85-205-4

CHKD. BY DEM DATE 4/2/85 SHEET NO. 1 OF 22

GAI CONSULTANTS, INC.
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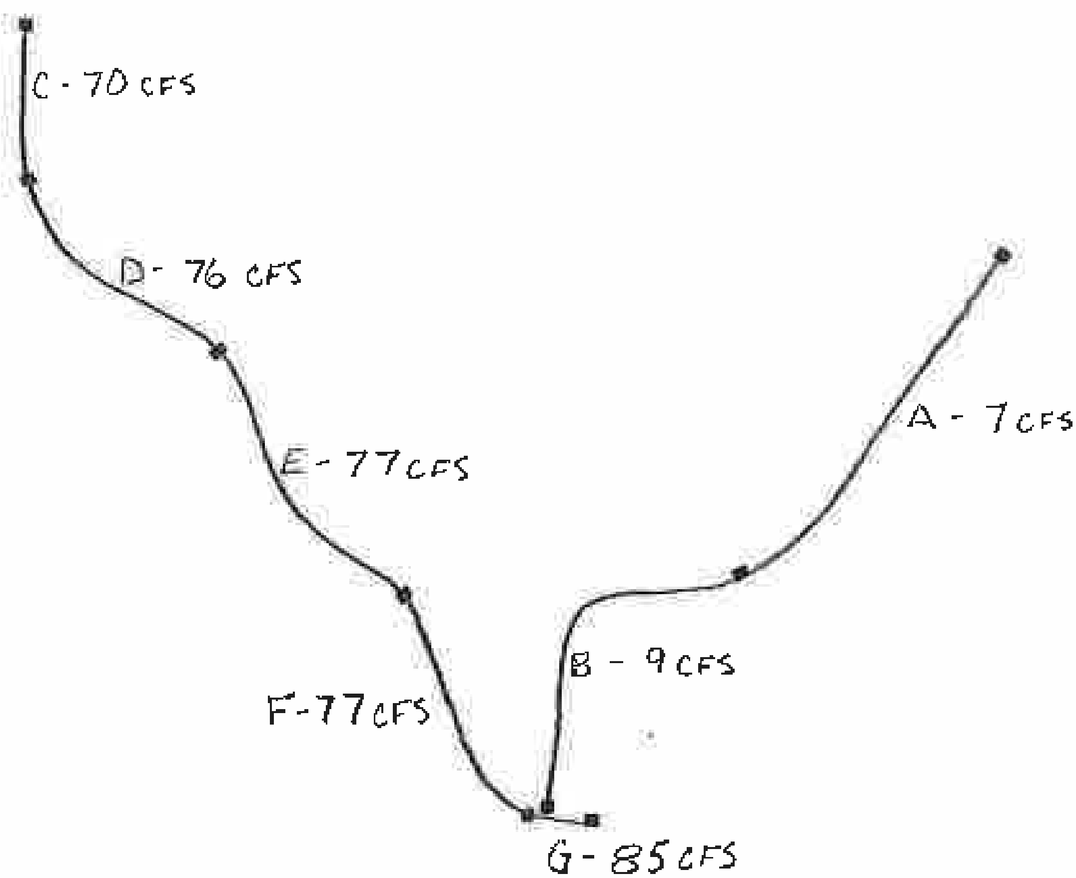
THIS SET OF CALCULATIONS WAS PERFORMED IN ORDER TO SIZE THE STAGE I COLLECTION DITCHES. THE PEAK FLOWS FOR WHICH THE DITCHES WERE SIZED MAY BE FOUND IN THE HYDROLOGY SECTION OF THE CALCULATIONS AND ARE TITLED STAGE I HYDROLOGY. THE PEAK FLOWS WERE DETERMINED BY USING THE TR-20 COMPUTER PROGRAM DEVELOPED BY THE SCS.

THE COLLECTION DITCHES WILL BE SIZED TO CARRY THE 100-YR, 24-HR STORM. THE DESIGN FLOWS ARE SHOWN BELOW ON A SKETCH OF THE GENERAL CONFIGURATION OF THE COLLECTION DITCHES AS PRESENTED IN THE HYDROLOGY CALCULATIONS.



SUBJECT KEYSTONE - STAGE I HYDRAULICS

BY DMK DATE 3/21/85 PROJ. NO. 85-205-4

CHKD. BY DEM DATE 4/2/85 SHEET NO. 2 OF 22

GAI CONSULTANTS, INC.
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THE DITCHES WILL BE SIZED USING MANNING'S EQUATION. ALL MAX. AND MIN. CHANNEL SLOPES OBTAINED FROM STAGE I HYDROLOGY WORKSHEET WS-3

WEST SIDE COLLECTION DITCHES

DITCH C

$Q = 70$ CFS

$n = 0.025$ FOR GROUTED RIP RAP LINING.

$S_{min} = 16.7\%$

$S_{MAX} = 20\%$

$b = 2$ FT.

SIDE SLOPES = 2:1

COMPUTE VALUES OF $\frac{Qn}{b^{8/3} S^{1/2}}$ TO ENTER TABLES ON

SHEETS 3, 4 AND 5 WITH.

FOR S_{max}: $\frac{Qn}{b^{8/3} S^{1/2}} = \frac{70(0.025)}{(2)^{8/3}(.20)^{1/2}} = 0.62$

FROM TABLE ON SHEET 4, $b/d \doteq 0.48$

$d = 0.48(2) = 0.96$ FT.

NOW, FIND AREA THEN COMPUTE VELOCITY

$A = db + zd^2 = 0.96(2) + 2(.96)^2 = 3.76$ FT^2

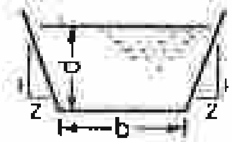
$V = \frac{Q}{A} = \frac{70 \text{cfs}}{3.76 \text{ft}^2} = 18.6$ FPS ∴ OK FOR GROUTED RIP RAP.

FOR S_{min}: $\frac{Qn}{b^{8/3} S^{1/2}} = \frac{70(0.025)}{(2)^{8/3}(.167)^{1/2}} = 0.67$

Hydraulic Design of Energy Dissipators for Culverts and Channels, H.E. Circular, #14 U.S. D.O.T., FHA

$A = \frac{1}{2}(b+b+2zd)d$
$= \frac{1}{2}(2b+2zd)d$
$= db+zd^2$

Table III-1.—*Uniform flow in trapezoidal channels by Manning's formula.* **From Reference III-3.**



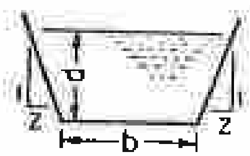
Values of $\frac{Qn}{b^{8/3}S^{1/2}}$

d/b[1]	z = 0	z = 1/4	z = 1/2	z = 3/4	z = 1	z = 1-1/4	z = 1-1/2	z = 1-3/4	z = 2	z = 3
.02	.00213	.00215	.00216	.00217	.00218	.00219	.00220	.00220	.00221	.00223
.03	.00414	.00419	.00423	.00426	.00429	.00431	.00433	.00434	.00437	.00443
.04	.00661	.00670	.00679	.00685	.00690	.00696	.00700	.00704	.00707	.00722
.05	.00947	.00964	.00980	.00991	.0100	.0101	.0102	.0103	.0103	.0106
.06	.0127	.0130	.0132	.0134	.0136	.0137	.0138	.0140	.0141	.0145
.07	.0162	.0166	.0170	.0173	.0176	.0177	.0180	.0182	.0183	.0190
.08	.0200	.0206	.0211	.0215	.0219	.0222	.0225	.0228	.0231	.0240
.09	.0240	.0249	.0256	.0262	.0267	.0271	.0275	.0279	.0282	.0296
.10	.0283	.0294	.0305	.0311	.0318	.0324	.0329	.0334	.0339	.0358
.11	.0329	.0342	.0354	.0364	.0373	.0380	.0387	.0394	.0400	.0424
.12	.0376	.0393	.0408	.0420	.0431	.0441	.0450	.0458	.0466	.0497
.13	.0425	.0446	.0464	.0480	.0493	.0505	.0516	.0527	.0537	.0575
.14	.0476	.0501	.0524	.0542	.0559	.0573	.0587	.0599	.0612	.0659
.15	.0528	.0559	.0585	.0608	.0628	.0645	.0662	.0677	.0692	.0749
.16	.0582	.0619	.0650	.0676	.0699	.0720	.0740	.0759	.0776	.0845
.17	.0638	.0680	.0717	.0748	.0775	.0800	.0823	.0845	.0867	.0947
.18	.0695	.0744	.0786	.0822	.0854	.0883	.0910	.0936	.0961	.105
.19	.0753	.0809	.0857	.0900	.0936	.0970	.100	.103	.106	.117
.20	.0813	.0875	.0932	.0979	.102	.106	.110	.113	.116	.129
.21	.0873	.0944	.101	.106	.111	.115	.120	.123	.127	.142
.22	.0935	.101	.109	.115	.120	.125	.130	.134	.139	.155
.23	.0997	.109	.117	.124	.130	.135	.141	.146	.151	.169
.24	.106	.116	.125	.133	.139	.146	.152	.157	.163	.184
.25	.113	.124	.133	.142	.150	.157	.163	.170	.176	.199
.26	.119	.131	.142	.152	.160	.168	.175	.182	.189	.215
.27	.126	.139	.151	.162	.171	.180	.188	.195	.203	.232
.28	.133	.147	.160	.172	.182	.192	.201	.209	.217	.249
.29	.139	.155	.170	.182	.193	.204	.214	.223	.232	.267
.30	.146	.163	.179	.193	.205	.217	.227	.238	.248	.286
.31	.153	.172	.189	.204	.217	.230	.242	.253	.264	.306
.32	.160	.180	.199	.215	.230	.243	.256	.269	.281	.327
.33	.167	.189	.209	.227	.243	.257	.271	.285	.298	.348
.34	.174	.198	.219	.238	.256	.272	.287	.301	.315	.369
.35	.181	.207	.230	.251	.270	.287	.303	.318	.334	.392
.36	.190	.216	.241	.263	.283	.302	.319	.336	.353	.416
.37	.196	.225	.251	.275	.297	.317	.336	.354	.372	.440
.38	.203	.234	.263	.289	.311	.333	.354	.373	.392	.465
.39	.210	.244	.274	.301	.326	.349	.371	.392	.412	.491
.40	.218	.254	.286	.314	.341	.366	.389	.412	.433	.518
.41	.225	.263	.297	.328	.357	.383	.408	.432	.455	.545
.42	.233	.279	.310	.342	.373	.401	.427	.453	.478	.574
.43	.241	.282	.321	.356	.389	.418	.447	.474	.501	.604
.44	.249	.292	.334	.371	.405	.437	.467	.496	.524	.634

[1] For d/b less than 0.04, use of the assumption R = d is more convenient and more accurate than interpolation in the table.

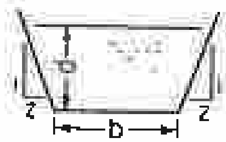
Table III-1.—*Uniform flow in trapezoidal channels by Manning's formula.*—Continued, from Reference III-3.

Values of $\frac{Qn}{b^{8/3} s^{1/2}}$



d/b	z = 0	z = 1/4	z = 1/2	z = 3/4	z = 1	z = 1-1/4	z = 1-1/2	z = 1-3/4	z = 2	z = 3
.45	.256	.303	.346	.385	.422	.455	.487	.519	.548	.665
.46	.263	.313	.359	.401	.439	.475	.509	.541	.574	.696
.47	.271	.323	.371	.417	.457	.494	.530	.565	.600	.729
.48	.279	.333	.384	.432	.475	.514	.552	.589	.626	.763
.49	.287	.345	.398	.448	.492	.534	.575	.614	.652	.797
.50	.295	.356	.411	.463	.512	.556	.599	.639	.679	.833
.52	.310	.377	.438	.496	.548	.599	.646	.692	.735	.906
.54	.327	.398	.468	.530	.590	.644	.696	.746	.795	.984
.56	.343	.421	.496	.567	.631	.690	.748	.803	.856	1.07
.58	.359	.444	.526	.601	.671	.739	.802	.863	.922	1.15
.60	.375	.468	.556	.640	.717	.789	.858	.924	.980	1.24
.62	.391	.492	.590	.679	.763	.841	.917	.989	1.06	1.33
.64	.408	.516	.620	.718	.809	.894	.976	1.05	1.13	1.43
.66	.424	.541	.653	.759	.858	.951	1.04	1.13	1.21	1.53
.68	.441	.566	.687	.801	.908	1.01	1.10	1.20	1.29	1.64
.70	.457	.591	.722	.842	.958	1.07	1.17	1.27	1.37	1.75
.72	.474	.617	.757	.887	1.01	1.13	1.24	1.35	1.45	1.87
.74	.491	.644	.793	.932	1.07	1.19	1.31	1.43	1.55	1.98
.76	.508	.670	.830	.981	1.12	1.26	1.39	1.51	1.64	2.11
.78	.525	.698	.868	1.03	1.18	1.32	1.46	1.60	1.73	2.24
.80	.542	.725	.906	1.08	1.24	1.40	1.54	1.69	1.83	2.37
.82	.559	.753	.945	1.13	1.30	1.47	1.63	1.78	1.93	2.51
.84	.576	.782	.985	1.18	1.36	1.54	1.71	1.87	2.03	2.65
.86	.593	.810	1.03	1.23	1.43	1.61	1.79	1.97	2.14	2.80
.88	.610	.839	1.07	1.29	1.49	1.68	1.88	2.07	2.25	2.95
.90	.627	.871	1.11	1.34	1.56	1.77	1.98	2.17	2.36	3.11
.92	.645	.898	1.15	1.40	1.63	1.86	2.07	2.28	2.48	3.27
.94	.662	.928	1.20	1.46	1.70	1.94	2.16	2.38	2.60	3.43
.96	.680	.960	1.25	1.52	1.78	2.03	2.27	2.50	2.73	3.61
.98	.697	.991	1.29	1.58	1.85	2.11	2.37	2.61	2.85	3.79
1.00	.714	1.02	1.33	1.64	1.93	2.21	2.47	2.73	2.99	3.97
1.05	.759	1.10	1.46	1.80	2.13	2.44	2.75	3.04	3.33	4.45
1.10	.802	1.19	1.58	1.97	2.34	2.69	3.04	3.37	3.70	4.96
1.15	.846	1.27	1.71	2.14	2.56	2.96	3.34	3.72	4.09	5.52
1.20	.891	1.36	1.85	2.33	2.79	3.24	3.68	4.09	4.50	6.11
1.25	.936	1.45	1.99	2.52	3.04	3.54	4.03	4.49	4.95	6.73
1.30	.980	1.54	2.14	2.73	3.30	3.85	4.39	4.90	5.42	7.39
1.35	1.02	1.64	2.29	2.94	3.57	4.18	4.70	5.34	5.90	8.10
1.40	1.07	1.74	2.45	3.16	3.85	4.52	5.18	5.80	6.43	8.83
1.45	1.11	1.84	2.61	3.39	4.15	4.88	5.60	6.29	6.98	9.62

Table III-1.—*Uniform flow in trapezoidal channels by Manning's formula.*—Continued, from Reference III-3.



d/b	Values of $\frac{Qn}{b^{8/3} S^{1/2}}$									
	z = 0	z = 1/4	z = 1/2	z = 3/4	z = 1	z = 1-1/4	z = 1-1/2	z = 1-3/4	z = 2	z = 3
1.50	1.16	1.94	2.78	3.63	4.46	5.26	6.04	6.81	7.55	10.4
1.55	1.20	2.05	2.96	3.88	4.78	5.65	6.50	7.33	8.14	11.3
1.60	1.25	2.15	3.14	4.14	5.12	6.06	6.99	7.89	8.79	12.2
1.65	1.30	2.27	3.33	4.41	5.47	6.49	7.50	8.47	9.42	13.2
1.70	1.34	2.38	3.52	4.69	5.83	6.94	8.02	9.08	10.1	14.2
1.75	1.39	2.50	3.73	4.98	6.21	7.41	8.57	9.72	10.9	15.2
1.80	1.43	2.62	3.93	5.28	6.60	7.89	9.13	10.4	11.6	16.3
1.85	1.48	2.74	4.15	5.59	7.01	8.40	9.75	11.1	12.4	17.4
1.90	1.52	2.86	4.36	5.91	7.43	8.91	10.4	12.4	13.2	18.7
1.95	1.57	2.99	4.59	6.24	7.87	9.46	11.0	12.5	14.0	19.9
2.00	1.61	3.12	4.83	6.58	8.32	10.0	11.7	13.3	14.9	21.1
2.10	1.71	3.39	5.31	7.30	9.27	11.2	13.1	15.0	16.8	23.9
2.20	1.79	3.67	5.82	8.06	10.3	12.5	14.6	16.7	18.7	26.8
2.30	1.89	3.96	6.36	8.86	11.3	13.8	16.2	18.6	20.9	30.0
2.40	1.98	4.26	6.93	9.72	12.5	15.3	17.9	20.6	23.1	33.4
2.50	2.07	4.58	7.52	10.6	13.7	16.8	19.8	22.7	25.6	37.0
2.60	7.16	4.90	8.14	11.6	15.0	18.4	21.7	25.0	28.2	40.8
2.70	2.26	5.24	8.80	12.6	16.3	20.1	23.8	27.4	31.0	44.8
2.80	2.35	5.59	9.49	13.6	17.8	21.9	25.9	29.9	33.8	49.1
2.90	2.44	5.95	10.2	14.7	19.3	23.8	28.2	32.6	36.9	53.7
3.00	2.53	6.33	11.0	15.9	20.9	25.8	30.6	35.4	40.1	58.4
3.20	2.72	7.12	12.5	18.3	24.2	30.1	35.8	41.5	47.1	68.9
3.40	2.90	7.97	14.7	21.0	27.9	34.8	41.5	48.2	54.6	80.2
3.60	3.09	8.86	16.1	24.0	32.0	39.9	47.8	55.5	63.0	92.8
3.80	3.28	9.81	18.1	27.1	36.3	45.5	54.6	63.5	72.4	107
4.00	3.46	10.8	20.2	30.5	41.1	51.6	61.9	72.1	82.2	122
4.50	3.92	13.5	26.2	40.1	54.5	68.8	82.9	96.9	111	164
6.00	4.39	16.7	33.1	51.5	70.3	89.2	108	126	145	216

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BY DMK DATE 3/21/85 PROJ. NO. 85-205-4

CHKD. BY DFM DATE 4/2/85 SHEET NO. 6 OF 22

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FROM TABLE ON SHEET 4, $b/d \simeq 0.51$

$d = 0.51(2) = 1.02$ FT ∴ USE CHANNEL DEPTH OF 2 FT.

SO, FOR CHANNEL C:
USE TRAPEZOIDAL SECTION WITH BASE = 2 FT, DEPTH = 2 FT, SIDE SLOPES = 2:1, LINED WITH 18 INCHES OF GROUTED NCSA R-4 ROCK.

DITCH D

$Q = 76$ CFS
$n = 0.025$ FOR GROUTED RIP RAP LINING
$S_{min} = 4.0\ \%$
$S_{MAX} = 14.3\ \%$
$b = 2$ FT.
SIDE SLOPES = 2:1

COMPUTE VALUES OF $\dfrac{Qn}{b^{8/3} S^{1/2}}$

FOR S_{MAX}: $\dfrac{Qn}{b^{8/3} S^{1/2}} = \dfrac{76(0.025)}{(2)^{8/3}(.143)^{1/2}} = 0.79$

FROM TABLE ON SHEET 4, $b/d \simeq 0.54$

$d = 0.54(2) = 1.08$

$A = db + zd^2 = 1.08(2) + 2(1.08)^2 = 4.49\ FT^2$

$V = \dfrac{Q}{A} = \dfrac{76\ CFS}{4.49\ FT^2} = 16.9$ FPS ∴ OK FOR GROUTED RIP RAP.

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CHKD. BY DEM DATE 4/2/85 SHEET NO. 7 OF 22

FOR S_{min}: $\frac{Qn}{b^{8/3} S^{1/2}} = \frac{76(0.025)}{(2)^{8/3}(.04)^{1/2}} = 1.50$

FROM SHEET 4, $b/d \simeq 0.73$

$d = 0.73(2) = 1.46$ FT. ∴ USE CHANNEL DEPTH OF 2 FT.

SO, FOR CHANNEL D:
USE SAME SECTION AS FOR CHANNEL C (SEE SHEET 6)

DITCH E

$Q = 77$ CFS
$n = 0.025$ FOR GROUTED RIP RAP LINING
$S_{min} = 9.1\%$
$S_{max} = 33.3\%$
$b = 2$ FT.
SIDE SLOPES = 2:1

COMPUTE VALUES OF $\frac{Qn}{b^{8/3} S^{1/2}}$

FOR S_{MAX}: $\frac{Qn}{b^{8/3} S^{1/2}} = \frac{77(0.025)}{(2)^{8/3}(0.333)^{1/2}} = 0.53$

FROM TABLE ON SHEET 3, $b/d \simeq 0.44$

$d = 0.44(2) = 0.88$ FT

$A = db + zd^2 = 0.88(2) + 2(0.88)^2 = 3.31$ FT^2

$V = \frac{Q}{A} = \frac{77 \text{ CFS}}{3.31 \text{ FT}^2} = 23.3$ FPS ∴ OK FOR GROUTED RIP RAP.

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BY DMK DATE 3/21/85 PROJ. NO. 85-205-4

CHKD. BY DEM DATE 4/2/85 SHEET NO. 8 OF 22

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FOR S_{min}: $\frac{Qn}{b^{8/3} S^{1/2}} = \frac{77(0.025)}{(2)^{8/3}(0.091)^{1/2}} = 1.00$

FROM TABLE ON SHEET 4, $b/d \approx 0.61$

$d = 0.61(2) = 1.22$ FT. ∴ USE DEPTH OF 2 FT.

SO, FOR CHANNEL E USE SAME SECTION AS FOR CHANNELS C AND D. (SEE SHEET 6).

DITCH F

$Q = 77$ CFS
$n = 0.025$ (GROUTED ROCK)
$S_{min} = 4.3\%$
$S_{max} = 38.5\%$
$b = 3$ FT
SIDE SLOPES = 2:1

} FROM EHK'S DESIGN OF DIVERSION DITCH CALCS DATED 10/31/84. SH. 80F

MUCH OF CHANNEL F HAS ALREADY BEEN BUILT AND THEREFORE, THE CALCULATIONS ARE TO PROVIDE A CHECK TO SEE IF THE CHANNEL WILL CARRY THE REVISED FLOWS. A SHORT PORTION (~50 FT) OF THE EXISTING CHANNEL WILL HAVE TO BE DESTROYED AND REPLACED TO TIE INTO THE NEXT DITCH DUE TO THE NEW ORIENTATION OF THE PILE.

COMPUTE VALUES OF $\frac{Qn}{b^{8/3} S^{1/2}}$

FOR S_{max}: $\frac{Qn}{b^{8/3} S^{1/2}} = \frac{77(0.025)}{(3)^{8/3}(.385)^{1/2}} = 0.166$

FROM TABLE ON SHEET 3, $d/b = 0.24$

$d = 0.24\,(3) = 0.72$ FT.

$A = db + zd^2 = 0.72(3) + 2(.72)^2 = 3.20$ FT^2

$V = \frac{Q}{A} = \frac{77 \text{ CFS}}{3.20 \text{ FT}^2} = 24.1$ FPS ∴ OK FOR GROUTED ROCK LINING.

FOR S_{min}: $\frac{Qn}{b^{8/3} S^{1/2}} = \frac{77(.025)}{(3)^{8/3}\,(0.043)^{1/2}} = 0.50$

FROM TABLE ON SHEET 4, $d/b = 0.43$

$d = 0.43\,(3) = 1.29$ FT.

FROM EHK'S "DESIGN OF DIVERSION DITCHES" CALCULATION DATED 10/31/84, SHT. 8 OF 16, THE CHANNEL HAS A DEPTH OF 2.5 FT, ∴ USE THIS DEPTH.

SO, FOR DITCH F, USE TRAPEZOIDAL SECTION WITH BASE = 3 FT, DEPTH = 2.5 FT, AND SIDE SLOPES OF 2:1. LINE WITH 18 IN. OF GROUTED NCSA R-4 ROCK.

DITCH G

$Q = 85$ CFS
$n = 0.025$ GROUTED ROCK
$S_{min} = 0.5\ \%$
$S_{max} = 33.3\ \%$
$b = 8$ FT.
SIDE SLOPES = 10:1

} FROM EHK'S "DESIGN OF DIVERSION DITCHES", DATE 10/31/84, SHT. 8 OF 16.

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CHANNEL (SWALE) G HAS ALREADY BEEN BUILT. THE FOLLOWING CALCS. ARE TO DETERMINE IF THE SECTION IS ADEQUATE.

$$Q = \frac{1.49}{n} AR^{2/3} S^{1/2}$$

$$Q = \frac{1.49}{0.025} [8d + 10d^2] \left[\frac{8d + 10d^2}{8 + 20.1d}\right]^{2/3} (S)^{1/2}$$

FOR S_{max}: $$85 = \frac{1.49}{.025} [8d + 10d^2] \left[\frac{8d + 10d^2}{8 + 20.1d}\right]^{2/3} (.333)^{1/2}$$

$$2.47 = AR^{2/3}$$

$d \simeq 0.43$ FT. BY TRIAL & ERROR.

$A = 8(.43) + 10(.43)^2 = 5.29$ FT^2

$V = \frac{Q}{A} = \frac{85 \text{ CFS}}{5.29 \text{ FT}^2} = 16.1$ FPS ∴ OK FOR GROUTED ROCK.

FOR S_{min}: $$85 = \frac{1.49}{.025} [8d + 10d^2] \left[\frac{8d + 10d^2}{8 + 20.1d}\right]^{2/3} (.005)^{1/2}$$

$$20.17 = AR^{2/3}$$

TRY $d = 1.5$ FT

$$AR^{2/3} = [(8)(1.5) + 10(1.5)^2] \left[\frac{8(1.5) + 10(1.5)^2}{8 + 20.1(1.5)}\right]^{2/3} = 32.3 \geq 20.17$$

d IS ACTUALLY ~1.22 FT.
∴ CHANNEL DEPTH OF 1.5 FT. IS ADEQUATE.

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EAST SIDE COLLECTION DITCH

DITCH A

$Q = 7$ CFS
$n = 0.045$ FOR GRASS LINED CHANNEL
$S_{min} = 16.7\%$
$S_{MAX} = 21.7\%$
$b = 2$ FT.
SIDE SLOPES = 2:1

COMPUTE VALUES OF $\dfrac{Qn}{b^{8/3} S^{1/2}}$

FOR S_{MAX}: $\dfrac{Qn}{b^{8/3} S^{1/2}} = \dfrac{7(0.045)}{(2)^{8/3}(.217)^{1/2}} = 0.106$

FROM TABLE ON SHEET 3, $d/b = .19$

$d = 0.19(2) = 0.38$ FT.

$A = db + zd^2 = 0.38(2) + 2(0.38)^2 = 1.05$ FT^2

$V = \dfrac{Q}{A} = \dfrac{7 \text{ CFS}}{1.05 \text{ FT}^2} = 6.7$ FPS. USE ENKAMAT + GRASS LINING.

FOR S_{MIN}: $\dfrac{Qn}{b^{8/3} S^{1/2}} = \dfrac{7(0.045)}{(2)^{8/3}(.167)^{1/2}} = 0.121$

FROM SHEET 3, $b/d = 0.205$

$d = 0.205(2) = 0.41$ FT

USE DEPTH = 1 FT.

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CHKD. BY DEM DATE 4/2/85 SHEET NO. 12 OF 22

SO, FOR DITCH A USE A TRAPEZOIDAL SECTION WITH A BASE OF 2 FT. AND DEPTH 1 FT AND 2:1 SIDE SLOPES. LINE WITH ENKAMAT AND GRASS.

DITCH B

DITCH B HAS ALREADY BEEN CONSTRUCTED AND WAS DESIGNED TO CARRY 32 CFS. DUE TO CHANGES IN THE WAY THE PILE WILL BE DRAINED, THE REVISED PEAK FLOW IS 9 CFS. THUS, THE CONSTRUCTED CHANNEL WILL BE ADEQUATE FOR FLOW OF 9 CFS. FOR THE ORIGINAL CHANNEL DESIGN, SEE EHK'S "EAST DRAINAGE CHANNEL LOCATION" CALCS DATED 11/13/84 SHTS. 1 THROUGH 7

NOW, CHECK THE FROUDE NUMBERS FOR MINIMUM AND MAXIMUM SLOPES AT EACH CHANNEL LOCATION TO DETERMINE ANY LOCATIONS OF HYDRAULIC JUMPS.

FROUDE NUMBER = $F = \left(\frac{Q^2 B}{g A^3} \right)^{1/2}$ WHERE Q = FLOW (CFS)

B = WIDTH OF LIQUID SURFACE (

g = 32.2 ft/s²

A = X-SECTIONAL AREA OF FLOW (FT²)

AT LOCATIONS WHERE THE FROUDE NUMBER DROPS FROM GREATER THAN 1 TO LESS THAN 1, A JUMP MAY OCCUR.

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CHKD. BY DEM DATE 4/2/85 SHEET NO. 13 OF 22

WEST SIDE

DITCH C = F_{C1}: SLOPE = 16.7%, d = 1.02 FT. (SHEET 6)

$B = b + 2zd = 2 + 2(2)(1.02) = 6.08$ FT.

$A = db + zd^2 = 2(1.02) + 2(1.02)^2 = 4.12$ FT^2

$$F_{C1} = \left(\frac{Q^2 B}{g A^3}\right)^{1/2} = \left(\frac{(70)^2 (6.08)}{(32.2)(4.12)^3}\right)^{1/2} = 3.6$$

F_{C2}: SLOPE = 20%, d = 0.96 FT., A = 3.76 FT^2 (SHEET 2)

$B = b + 2zd = 2 + 2(2)(.96) = 5.84$ FT.

$$F_{C2} = \left(\frac{Q^2 B}{g A^3}\right)^{1/2} = \left(\frac{(70)^2 (5.84)}{(32.2)(3.76)^3}\right)^{1/2} = 4.1$$

SINCE F_1 & $F_2 > 1$, NO JUMP FROM F_{C1} TO F_{C2}

DITCH D = F_{D1}: SLOPE = 14.3%, d = 1.08 FT, A = 4.49 FT (SHT. 6)

$B = b + 2zd = 2 + 2(2)(1.08) = 6.32$ FT.

$$F_{D1} = \left(\frac{Q^2 B}{g A^3}\right)^{1/2} = \left(\frac{(76)^2 (6.32)}{32.2 (4.49)^3}\right)^{1/2} = 3.5$$

∴ NO JUMP FROM F_{C2} TO F_{D1}

F_{D2}: SLOPE = 4%, d = 1.46 (SHEET 7)

$B = b + 2zd = 2 + 2(2)(1.46) = 7.84$ FT.

$A = db + zd^2 = 1.46(2) + 2(1.46)^2 = 7.18$ FT^2

$$F_{D2} = \left(\frac{Q^2 B}{g A^3}\right)^{1/2} = \left(\frac{(76)^2 (7.84)}{32.2 (7.18)^3}\right)^{1/2} = 1.9$$

∴ NO JUMP FROM F_{D1} TO F_{D2}

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CHKD. BY DEM DATE 4/2/85 SHEET NO. 14 OF 22

DITCH E - F_{E1}: SLOPE = 33.3%, d = 0.88 FT, A = 3.31 FT^2 (SHT. 7)

$B = b + 2zd = 2 + 2(2)(0.88) = 5.52$ FT.

$$F_{E1} = \left(\frac{Q^2 B}{gA^3}\right)^{1/2} = \left(\frac{(77)^2(5.52)}{32.2(3.31)^3}\right)^{1/2} = 5.3$$

∴ NO JUMP FROM F_{D2} TO F_{E1}

F_{E2}: SLOPE = 9.1%, d = 1.22 FT (SHEET 8)

$B = b + 2zd = 2 + 2(2)(1.22) = 6.88$ FT.

$A = db + zd^2 = 1.22(2) + 2(1.22)^2 = 5.42$ FT^2

$$F_{E2} = \left(\frac{Q^2 B}{g A^3}\right)^{1/2} = \left(\frac{(77)^2(6.88)}{32.2(5.42)^3}\right)^{1/2} = 2.8$$

∴ NO JUMP FROM F_{E1} TO F_{E2}

DITCH F - F_{F1}: SLOPE = 4.3%, d = 1.29 FT (SHEET 9)

$B = b + 2zd = 3 + 2(2)(1.29) = 8.16$ FT

$A = db + zd^2 = (1.29)(3) + 2(1.29)^2 = 7.20$ FT^2

$$F_{F1} = \left(\frac{Q^2 B}{g A^3}\right)^{1/2} = \left(\frac{(77)^2(8.16)}{32.2\,(7.20)^3}\right)^{1/2} = 2.0$$

∴ NO JUMP FROM F_{E2} TO F_{F1}

F_{F2}: SLOPE = 38.5%, d = 0.72 FT., A = 3.20 FT^2 (SHTS 8, 9)

$B = b + 2zd = 3 + 2(2)(0.72) = 5.88$ FT.

$$F_{F2} = \left(\frac{Q^2 B}{g A^3}\right)^{1/2} = \left(\frac{(77)^2(5.88)}{32.2(3.20)^3}\right)^{1/2} = 5.7$$

∴ NO JUMP FROM F_{F1} TO F_{F2}

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BY DMK DATE 3/21/85 PROJ. NO. 85-205-5
CHKD. BY DEM DATE 4/2/85 SHEET NO. 15 OF 22

DITCH G - F_{G1}: SLOPE = 0.5%, $d = 1.22$ FT (SHEET 10)

$B = b + 2zd = 8 + 2(10)(1.22) = 32.4$ FT

$A = db + zd^2 = (1.22)(8) + 10(1.22)^2 = 24.64$ FT^2

$$F_{G1} = \left(\frac{Q^2 B}{g A^3}\right)^{1/2} = \left(\frac{(85)^2(32.4)}{32.2\,(24.64)^3}\right)^{1/2} = 0.69$$

A HYDRAULIC JUMP WILL OCCUR IN SWALE AS IT CROSSES THE ROAD @ 0.5%.

TO COMPUTE THE DEPTH OF FLOW AFTER THE JUMP, ENTER FIGURE 5 ON SHEET 16 WITH THE FROUDE NUMBER PRIOR TO THE JUMP. FIRST, CONSIDER AN IDENTICAL CHANNEL CROSS-SECTION AS CHANNEL G (IN WHICH JUMP OCCURS) BUT WITH SLOPE OF PRIOR CHANNEL (CHANNEL F WITH SLOPE OF 38.5%).

BY MANNING'S EQUATION, USING DITCH G.

$$85 = \frac{1.49}{0.025}\left[8d + 10d^2\right]\left[\frac{8d + 10d^2}{8 + 20.1d}\right]^{2/3}(.385)^{1/2}$$

$$2.30 = AR^{2/3}$$

$d \sim 0.415$ BY TRIAL AND ERROR

$$A = db + zd^2 = 0.415(8) + 10(.415)^2 = 5.04 \text{ FT}^2$$

$$V = \frac{Q}{A} = \frac{85}{5.04} = 16.87 \text{ FPS}$$

$$F_1 = \frac{V_1}{\sqrt{g D_1}} = \frac{16.87}{\sqrt{(32.2)(.415)}} = 4.61$$

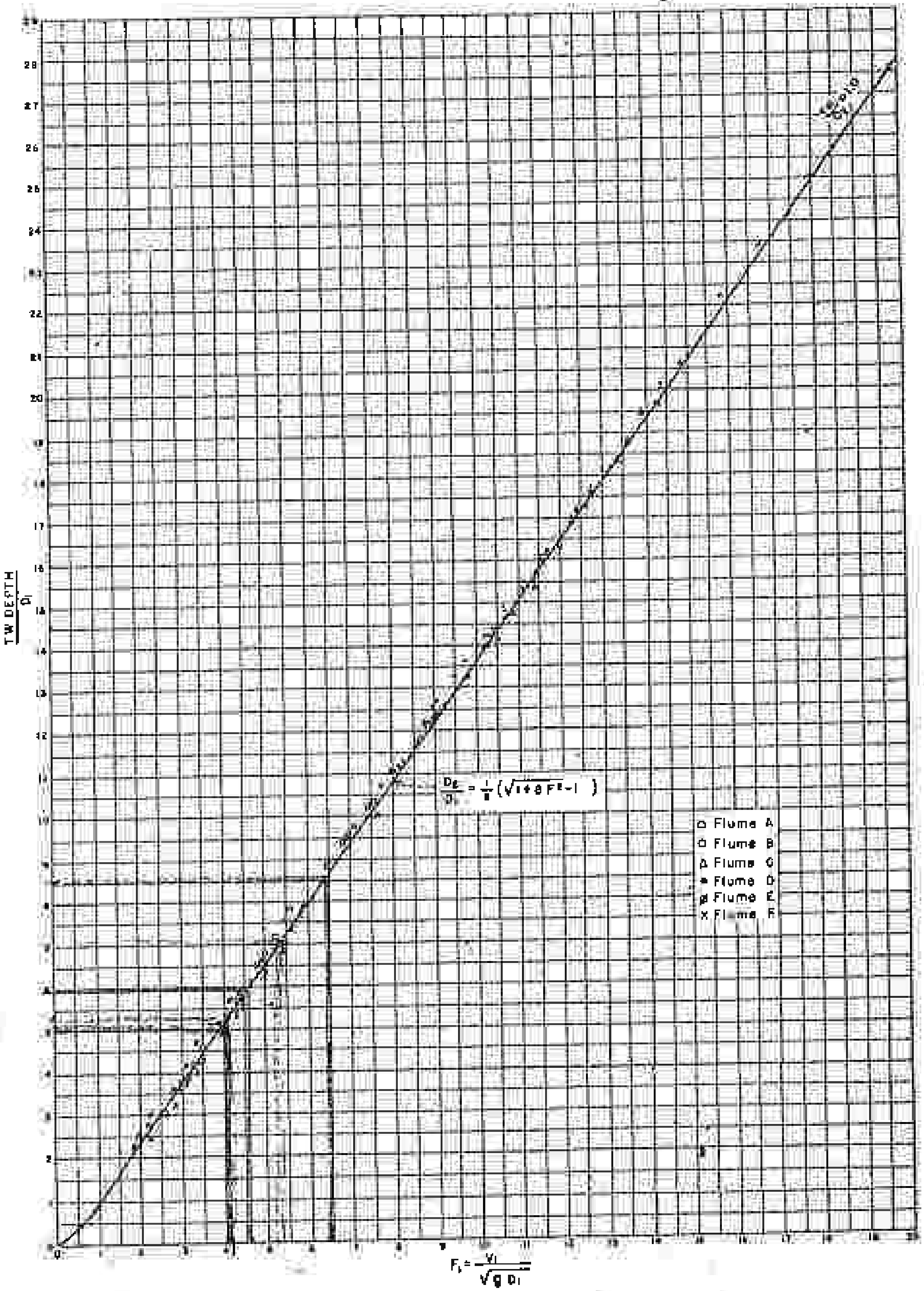

FIGURE 5.—*Ratio of tail water depth to* D_1 *(Basin I).*

opening. The extreme case involved a discharge of 0.14 c.f.s. and a value of D_1 of 0.032 foot, for $F_1=8.9$, which is much smaller than any discharge or value of D_1 used in the present experiments. Thus, it is reasoned that as the gate opening decreased, in the 6-inch-wide flume, frictional resistance in the channel downstream increased out of proportion to that which would have occurred in a larger flume or a prototype structure. Thus, the jump formed in a shorter length than it should. In laboratory language, this is known as "scale effect," and is construed to mean that prototype action is not faithfully reproduced. It is quite certain that this was the case for the major portion of curve 1. In fact, Bakhmeteff and Matzke were somewhat dubious concerning the small-scale experiments.

To confirm the above conclusion, it was found that results from Flume F, which was 1 foot wide, became erratic when the value of D_1 approached 0.10. Figures 6 and 7 show three points obtained with a value of D_1 of approximately 0.085. The three points are given the symbol ⊠ and fall short of the recommended curve.

The two remaining curves, labeled "3" and "4," on Figure 7, portray the same trend as the recommended curve. The criterion used by each experimenter for judging the length of the jump is undoubtedly responsible for the displacement. The curve labeled "3" was obtained at the Technical University of Berlin on a flume ½ meter wide by 10 meters long. The curve labeled "4" was determined from experiments performed at

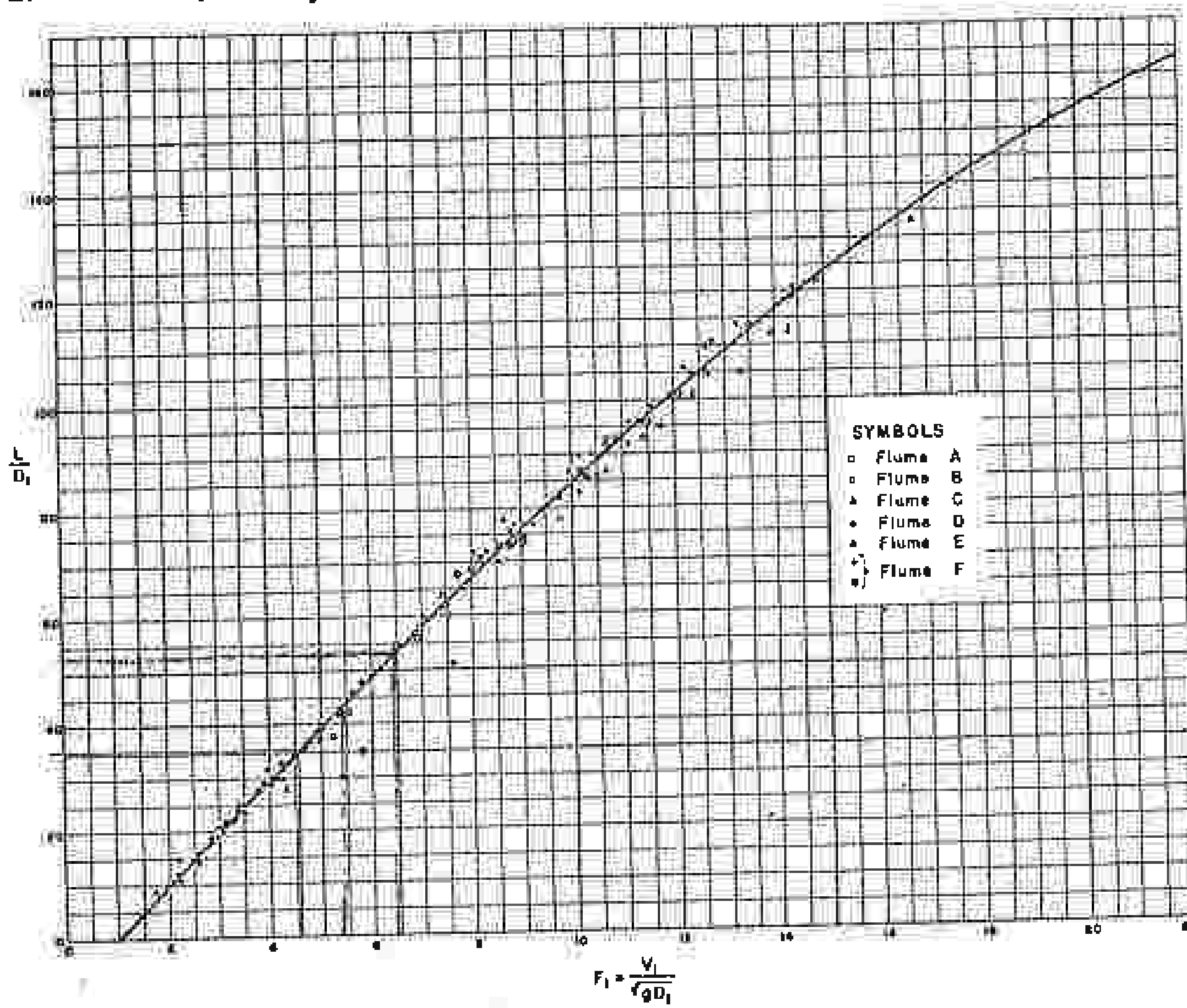

FIGURE 6.—*Length of jump in terms of D_1 (Basin I).*

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BY DMK DATE 3/21/85 PROJ. NO. 85-205-4

CHKD. BY DEM DATE 4/2/85 SHEET NO. 18 OF 22

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ENTERING FIG. 5 WITH $F_1 = 4.61$

$$\frac{\text{TW DEPTH}}{D_1} = 5.95$$

TW DEPTH = 5.95 (.415) = 2.47

∴ MAKE CHANNELG 2.5 FT DEEP.

INVESTIGATE LENGTH OF JUMP.

ENTER FIGURE 6 ON SHEET 17 WITH $F_1 = 4.61$ (SEE SHEET 15) GIVES $L/D_1 = 36$.

$L = 36 D_1 = 36(.415) \sim 15$ FT, THEREFORE, THE HYDRAULIC JUMP WILL OCCUR IN A LENGTH APPROXIMATELY EQUAL TO THE WIDTH OF THE ACCESS ROAD.

NOW, CHECK EAST SIDE FROUDE NUMBERS,

DITCH A – F_{A1}: SLOPE = 21.7 %, d = 0.38 FT, A = 1.05 FT² (SHT 11)
$B = b + 2zd = 2 + 2(2)(.38) = 3.52$ FT

$$F_{A1} = \left(\frac{Q^2 B}{g A^3}\right)^{1/2} = \left(\frac{(7)^2 (3.52)}{32.2 (1.05)^3}\right)^{1/2} = 2.2$$

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F_{A2}: SLOPE = 16.7%, $d = 0.41$ FT (SHEET 11)

$B = b + 2zd = 2 + 2(2)(.41) = 3.64$ FT.

$A = db + zd^2 = 0.41(2) + 2(.41)^2 = 1.16$ FT²

$$F_{A2} = \left(\frac{Q^2 B}{g A^3}\right)^{1/2} = \left(\frac{(7)^2(3.64)}{32.2(1.16)^3}\right)^{1/2} = 1.9$$ ∴ NO JUMP FROM F_{A1} TO F_{A2}

DITCH B - ASSUME MINIMUM SLOPE = 3% (EHK'S CALCS)

THEN, $\frac{Qn}{b^{8/3} S^{1/2}} = \left(\frac{9(0.025)}{(2)^{8/3}(.03)^{1/2}}\right) = 0.205$

THEN, $b/d \sim .27$ FT FROM SHEET 3.

$d = .27(2) = 0.54$

$B = b + 2zd = 2 + 2(2)(.54) = 4.16$ FT.

$A = db + zd^2 = .54(2) + 2(.54)^2 = 1.66$ FT²

$$F_B = \left(\frac{Q^2 B}{g A^3}\right)^{1/2} = \left(\frac{(9)^2(4.16)}{32.2(1.66)^3}\right)^{1/2} = 1.5$$ ∴ NO JUMP IN CHANNEL B

SIZE CULVERT TO BE LOCATED IN HAUL ROAD

FIRST CHECK INLET CONTROL - FROM THE INVERT OF CHANNEL E AT ELEV. 1193 WE HAVE APPROXIMATELY 7 FT. OF MATERIAL TO THE SURFACE OF THE HAUL ROAD AT ELEVATION 1200, IF WE KEEP 2 FT. OF FREEBOARD ON THE RAMP, A HEADWALL CAN BE CONSTRUCTED AND ALLOWS FOR A 5 FT. HEADWATER DEPTH.

ASSUME A 48 IN. CULVERT, DIA = 4 FT.
HW = 5 FT.

SUBJECT KEYSTONE - STAGE I HYDRAULICS

BY DMK DATE 3/22/85 PROJ. NO. 85-205-4

CHKD. BY DEM DATE 4/2/85 SHEET NO. 20 OF 22

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$\frac{HW}{D} = \frac{5}{4} = 1.25$

ENTER FIG. ON SHEET 21 WITH $HW/D = 1.25$, $Q = 77$ CFS, AND SCALE 1 CORRESPONDING TO A HEADWALL. ALSO, SCALE 3, CORRESPONDING TO A WORST CASE. A 48 IN. ϕ CULVERT IS SUFFICIENT.

CHECK OUTLET CONTROL.

$K_e = 0.50$ FOR HEADWALL

LENGTH ~ 95 FT.

$H = HW - h_o + LS_o$

h_o: DEPTH OF FLOW IN CHANNEL E AT $S = 9.1\%$
$= 1.22$ FT (SHT. 8)

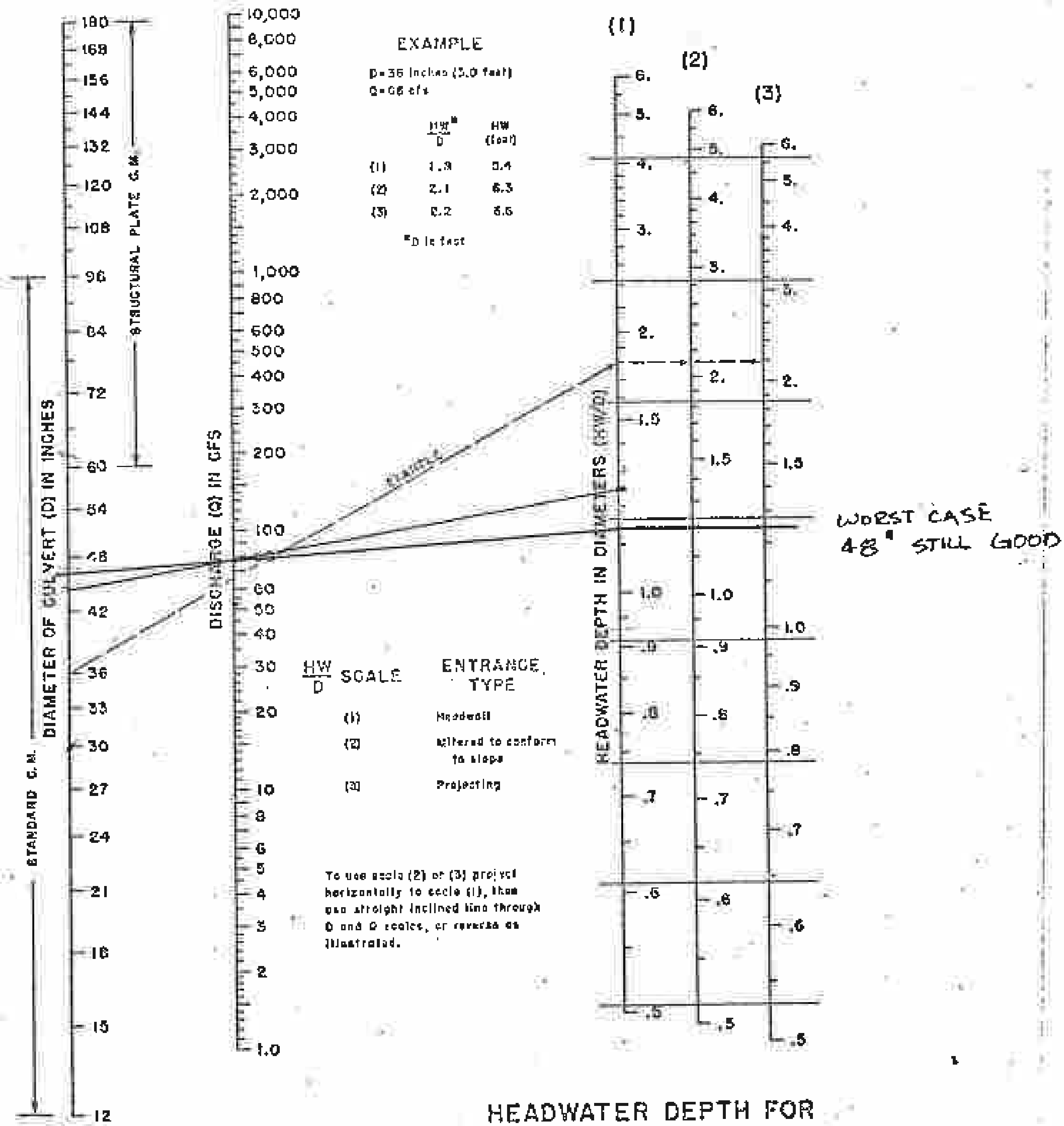
S_o: ASSUME CULVERT IS SLOPED SAME AS CHANNEL OR ~ 9.1%

$H = 5 - 1.22 + 95(.091)$
$= 12.4$

ENTER FIG. ON SHEET 22, CULVERT CAN PASS $Q = 190$ CFS. ∴ INLET CONDITIONS CONTROL.

USE 48 IN PIPE WITH OR WITHOUT HEADWALL

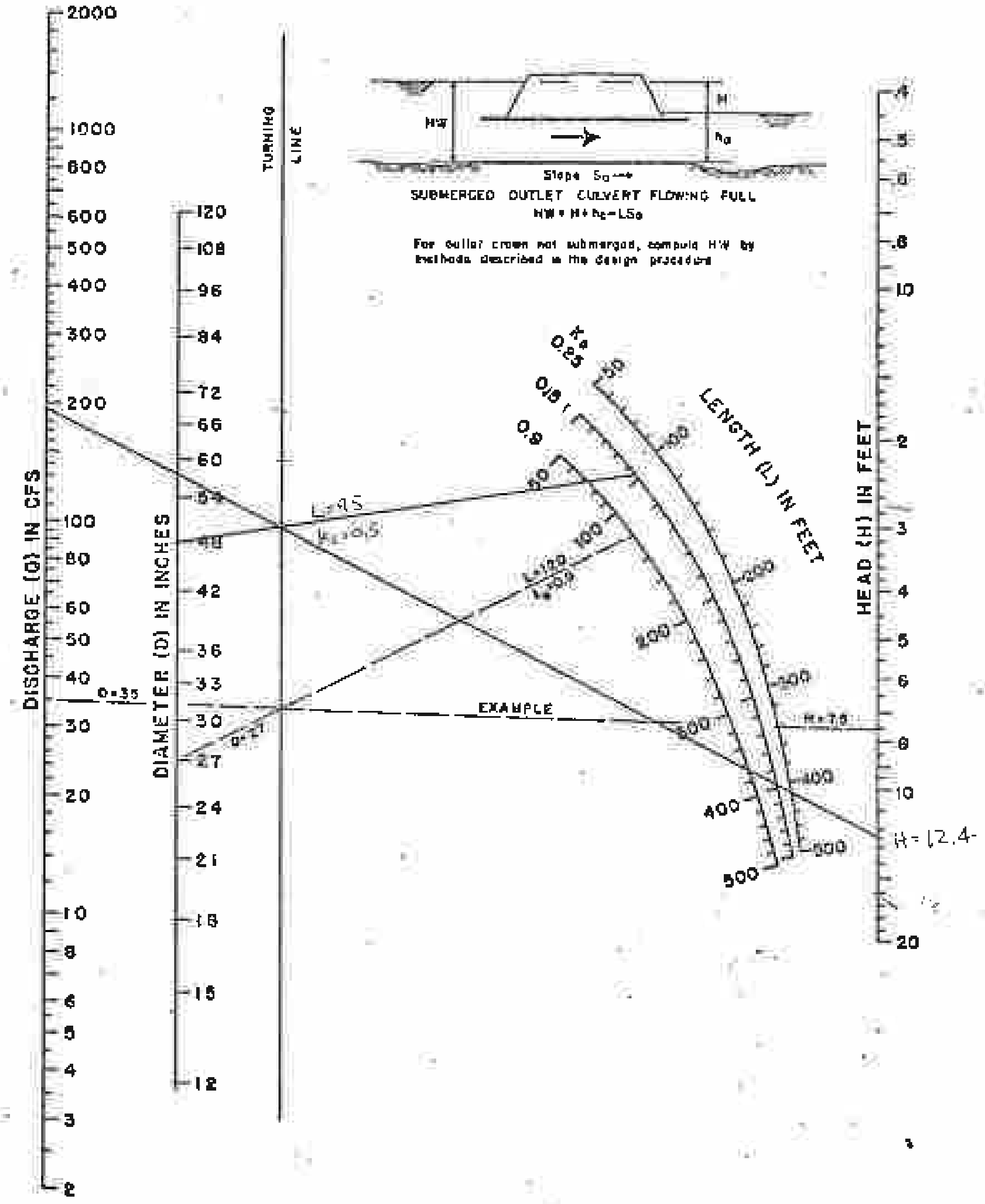
CHART 5



HEADWATER DEPTH FOR
C. M. PIPE CULVERTS
WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN. 1963

CHART 11



BUREAU OF PUBLIC ROADS JAN. 1963

HEAD FOR
STANDARD
C. M. PIPE CULVERTS
FLOWING FULL
n = 0.024

SUBJECT Penelec - Keystone
Emergency Spillway for Eq. Pond & Type Q Channel

BY DR DATE 7/25/85 PROJ. NO. 85-205-7

CHKD. BY EHK DATE 7/26/85 SHEET NO. 1 OF 4

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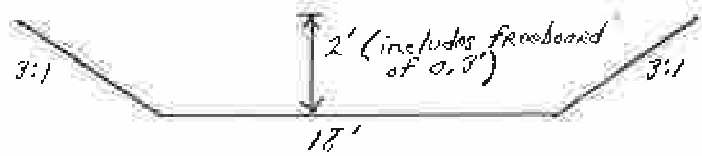
Emergency Spillway for Equalization Pond
and Type Q Channel

Determine if these channels are capable of carrying an additional flow of 190 cfs from the peripheral ditch by H. F. Lenz Co.

Emergency Spillway Below Equalization Pond



see Dwg. 41-E-0134

$A = (2 \times 18) + 2(3)(2)$
$= 48\ ft.^2$

$P = 18 + 2\sqrt{(2)^2 + (6)^2}$
$= 18 + 2(6.3)$
$= 30.6\ ft.$

$R = \frac{A}{P} = \frac{48}{30.6} = 1.6$

$n = 0.040$ Page 17/20 of "Equalization Basin Summary" by DB on 4/1/83 78-505

$S = 0.006$ (dwg. 41-E-0135) (min. slope)

$Q = \frac{1.49}{n} A R^{2/3} S^{1/2}$
$= \frac{1.49}{0.040}(48)(1.6)^{2/3}(0.006)^{1/2}$
$= 189.5\ cfs$

189.5 cfs < 300 cfs ← combined 100-yr, 24-hr, flow from Emergency spillway (110 cfs) & [illegible] channel (190 cfs)
∴ not ok

SUBJECT: Penske-Keystone
Emergency Spillway for Eq. Pond & Type of Channel

BY: DB DATE: 7/26/85 PROJ. NO.: 85-205-7

CHKD. BY: EHK DATE: 7/26/85 SHEET NO. 2 OF 4

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Determine the required bottom width of a 1.7' flow depth trapezoidal channel with 3:1 SS, $n = 0.040$, and a slope $= 0.006$.

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

$$300 = \frac{1.49}{0.040}(1.7Y + 8.7)\left(\frac{1.7Y + 8.7}{Y + 10.8}\right)^{2/3}(0.006)^{1/2}$$

$$104 = (1.7Y + 8.7)\left(\frac{1.7Y + 8.7}{Y + 10.8}\right)^{2/3}$$

Try $Y = 30 \Rightarrow 76.9 < 104$ ∴ not ok

Try $Y = 40 \Rightarrow 100.9 < 104$ ∴ not ok

Try $Y = 41 \Rightarrow 103.3 < 104$ ∴ not ok

Try $Y = 42 \Rightarrow 105.8 > 104$ ∴ ok

∴ Use a trapezoidal channel with a 42' bottom width, 3:1 SS, and a depth of 2' which allows for 0.3' of freeboard minimum.

SUBJECT: Penn Ice - Keystone
Emergency Spillway for G. Pond & Type Q Channel

BY: OB DATE: 7/23/85 PROJ. NO. 85-205-7

CHKD. BY: EHK DATE: 7/24/85 SHEET NO. 3 OF 7

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Type Q Channel (Connects Spillway with Plum Creek)

~78-565

From "Upgrading of Existing Channel from ... Spindle Outlet to Plum Creek" by ... on 11/11/82, the slope is 3.3%, n = 0.040, bottom width = 4', depth = 3.5' of which 0.43' is freeboard, and Q = 305.9 cfs. (2:1 SS)

New total Q = 305.9 + 190 = 495.9 cfs

a. With no freeboard $Q_{channel}$ =

$$A = (4 \times 3.5) + (2)(3.5)(3.5)$$
$$= 38.5\ ft.^2$$

$$P = 4 + 2\sqrt{(3.5)^2 + (7)^2}$$
$$= 19.7\ ft.$$

$$R = \frac{A}{P} = \frac{38.5}{19.7} = 2.0$$

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2}$$
$$= \frac{1.49}{0.040}(38.5)(2)^{2/3}(0.033)^{1/2}$$
$$= 413.6\ cfs$$

413.6 < 495.9
∴ not ok

b. Determine new width if keep 3.5' deep (flow to ≈3.1'), 2:1 SS, n = 0.040, & S = 0.033

$$A = (3.1Y + 19.22)$$
$$P = Y + 2\sqrt{3.1^2 + 6.2^2} = Y + 2(6.93) = Y + 13.86$$
$$R = \frac{A}{P}$$

SUBJECT [illegible]-Keystone

Emergency Spillway [illegible]

BY OB DATE 7/25/85 PROJ. NO. 85-205-7

CHKD. BY EHK DATE 7/26/85 SHEET NO. 4 OF 4

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$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

$$495.9 = \frac{1.49}{0.040} (3.1Y + 19.22) \left(\frac{3.1Y + 19.22}{Y + 13.86} \right)^{2/3} (0.033)^{1/2}$$

$$73.3 = (3.1Y + 19.22) \left(\frac{3.1Y + 19.22}{Y + 13.86} \right)^{2/3}$$

try $Y = 4 \Rightarrow 46.3 < 73.3 \Rightarrow$ not ok

try $Y = 4.5 \Rightarrow 49.2 < 73.3 \Rightarrow$ not ok

try $Y = 10 \Rightarrow 82.5 > 73.3 \Rightarrow$ too big

try $Y = 8. \Rightarrow 70.2 < 73.3 \Rightarrow$ too small

try $Y = 8.5 \Rightarrow 73.3 < 73.3 \Rightarrow$ ok

∴ Use a trapezoidal channel with an 4.5' bottom width, 3.5' deep (allows freeboard), 2:1 ss, riprap lined.

SUBJECT KEYSTONE - SLOPE DRAIN ON EAST SIDE - STAGE I

BY DMK DATE 4/29/85 PROJ. NO. 85-205-4

CHKD. BY DEM DATE 5/1/85 SHEET NO. 1 OF 5

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THIS SET OF CALCULATIONS WAS PERFORMED TO SIZE A SWALE TO CONVEY WATER FROM THE SLOPE DRAIN ON THE EAST SIDE OF STAGE I ACROSS THE ACCESS ROAD AND INTO THE CLEAN WATER DITCH.

Q = 32 CFS FROM TR-20 PROGRAM FOR KEYSTONE CLOSURE HYDROL

TRY A SWALE WITH THE FOLLOWING CHARACTERISTICS:
TRAPEZOIDAL CROSS-SECTION WITH BASE = 8 FT.
SIDE SLOPES = 10:1
CHANNEL BOTTOM SLOPED AT 1 % (MIN.)

BY MANNINGS EQUATION, FIND DEPTH OF FLOW
ASSUME $n = 0.025$ (GROUTED ROCK)

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

$$32 = \frac{1.49}{0.025} (8d + 10d^2) \left(\frac{8d + 10d^2}{8 + 20.1d}\right)^{2/3} (0.01)^{1/2}$$

$$5.37 = (8d + 10d^2) \left(\frac{8d + 10d^2}{8 + 20.1d}\right)^{2/3}$$

$d \simeq 0.64$ FT.

$A = 8(0.64) + 10(0.64)^2 = 9.216 \text{ FT}^2$

$V = Q/A = \frac{32 \text{ CFS}}{9.216 \text{ FT}^2} = 3.5 \text{ FPS}$ ∴ OK FOR GROUTED ROCK

CHECK FOR A POSSIBLE HYDRAULIC JUMP, THEN SIZE SWALE TO CONTAIN THIS JUMP.

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FROUDE NUMBER FOR SWALE CROSSING IS:

$$F = \left(\frac{Q^2 B}{g A^3}\right)^{1/2}$$

WHERE
Q = FLOW (CFS)
B = SURFACE WIDTH OF FLOW (FT)
g = 32.2 (FPS²)
A = AREA (FT²)

$$B = b + 2zd = 8 + 2(10)(0.64) = 20.8 \text{ FT.}$$

$$F_{1\%} = \left(\frac{(32)^2(20.8)}{(32.2)(9.216)^3}\right)^{1/2} = 0.92$$

NOW, CHECK CONDITIONS IN A SWALE SLOPED AT 15% CORRESPONDING TO THE SLOPE AT THE EAST TOE OF PILE. THE CHANNEL WILL BE DIFFERENT FROM THE SWALE DUE TO THE TRANSITION FROM THE SLOPE DRAIN TO THE SWALE BUT FOR THE PURPOSES OF COMPARING FROUDE NOS. AND DETERMINING THE CHARACTERISTICS OF THE JUMP, ASSUME THE SAME SECTION.

$$32 = \frac{1.49}{0.025}(8d + 10d^2)\left(\frac{8d + 10d^2}{8 + 20.1d}\right)^{2/3}(0.15)^{1/2}$$

$$1.39 = (8d + 10d^2)\left(\frac{8d + 10d^2}{8 + 20.1d}\right)^{2/3}$$

$d = 0.32$ FT

$$A = 8(0.32) + 10(0.32)^2 = 3.584 \text{ FT}^2$$

$$V = \frac{32 \text{ CFS}}{3.584 \text{ FT}^2} = 8.9 \text{ FPS} \therefore$$ OK FOR GROUTED ROCK

$$B = b + 2zd = 8 + 2(10)(0.32) = 14.4 \text{ FT.}$$

SUBJECT KEYSTONE - SLOPE DRAIN ON EAST SIDE - STAGE I

BY DMK DATE 4/29/85 PROJ. NO. 85-205-4

CHKD. BY DEM DATE 5/1/85 SHEET NO. 3 OF 5

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$$F = \left(\frac{Q^2 B}{g A^3}\right)^{1/2} = \left(\frac{(32)^2(14.4)}{32.2\,(3.584)^3}\right)^{1/2} = 3.2$$

∴ A JUMP WILL OCCUR IN GOING FROM A 15% SLOPE TO A 1% SLOPE.

SIZE CHANNEL/SWALE TO CONTAIN JUMP.
USE CHART ON SHEET 4, FROM HYDRAULIC DESIGN OF STILLING BASINS AND ENERGY DISSIPATORS BY A.J PETERKA, A WATER RESOURCES TECHNICAL PUBLICATION, ENGINEERING MONOGRAPH NO. 25.

$$F_1 = \frac{V_1}{\sqrt{g D_1}} = \frac{8.9}{\sqrt{32.2 \cdot 0.32}} = 2.8$$

ENTER FIGURE 5 ON SHT. 4 WITH $F_1 = 2.8$ WHICH YIELDS TW DEPTH$/D_1 = 3.5$

TW DEPTH = 3.5 (0.32) = 1.12 FT.

∴ SWALE MUST BE AT LEAST 1.12 FT IN DEPTH TO CONTAIN JUMP. MAKE SWALE 1.5 FT. DEEP IN THE ROAD.

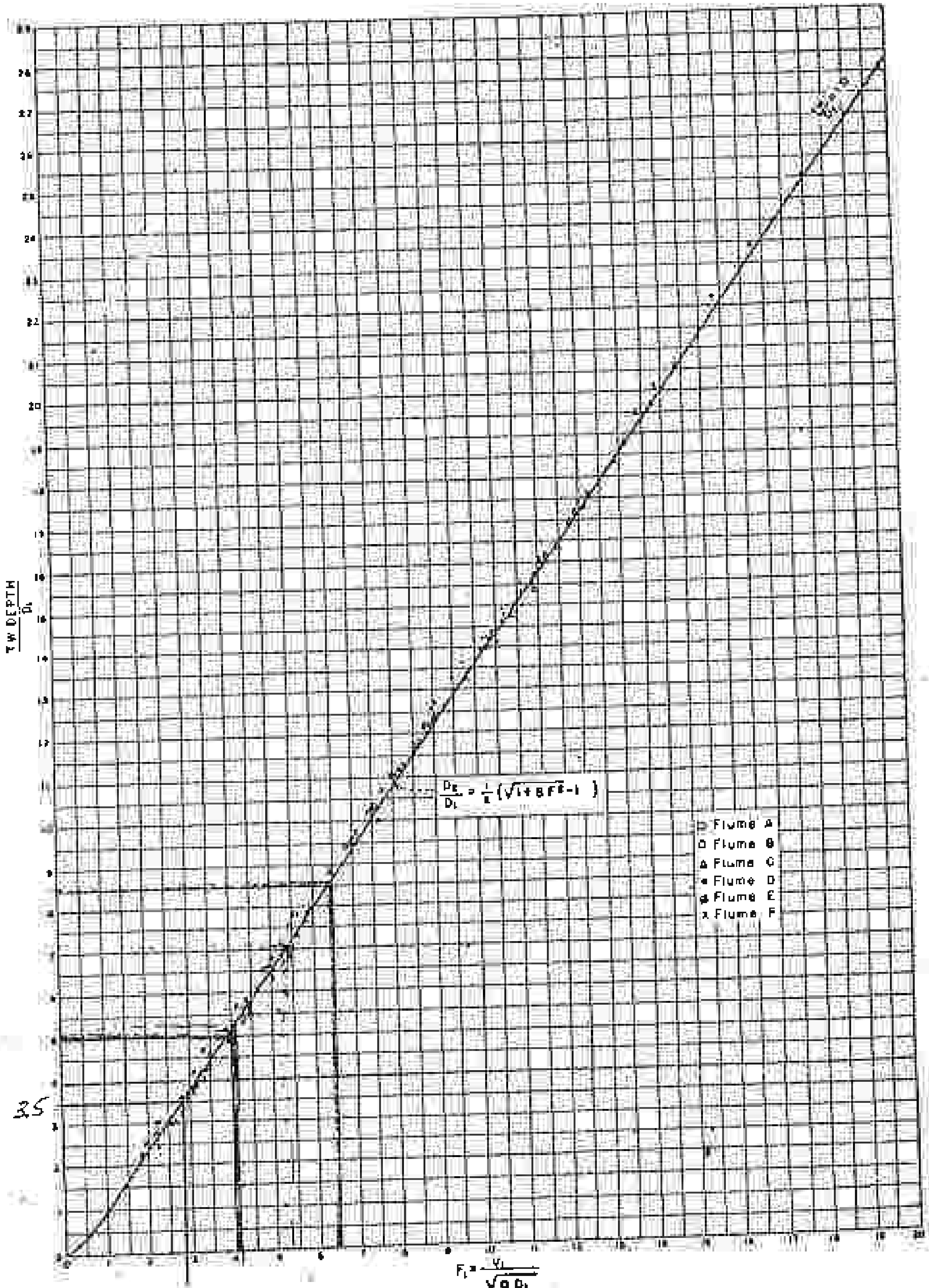

FIGURE 5.—*Ratio of tail water depth to D_1 (Basin I).*

SUBJECT KEYSTONE - SLOPE DRAIN ON EAST SIDE - STAGE I

BY DMK DATE 4/29/85 PROJ. NO. 85-205-4

CHKD. BY DEM DATE 5/1/85 SHEET NO. 5 OF 5

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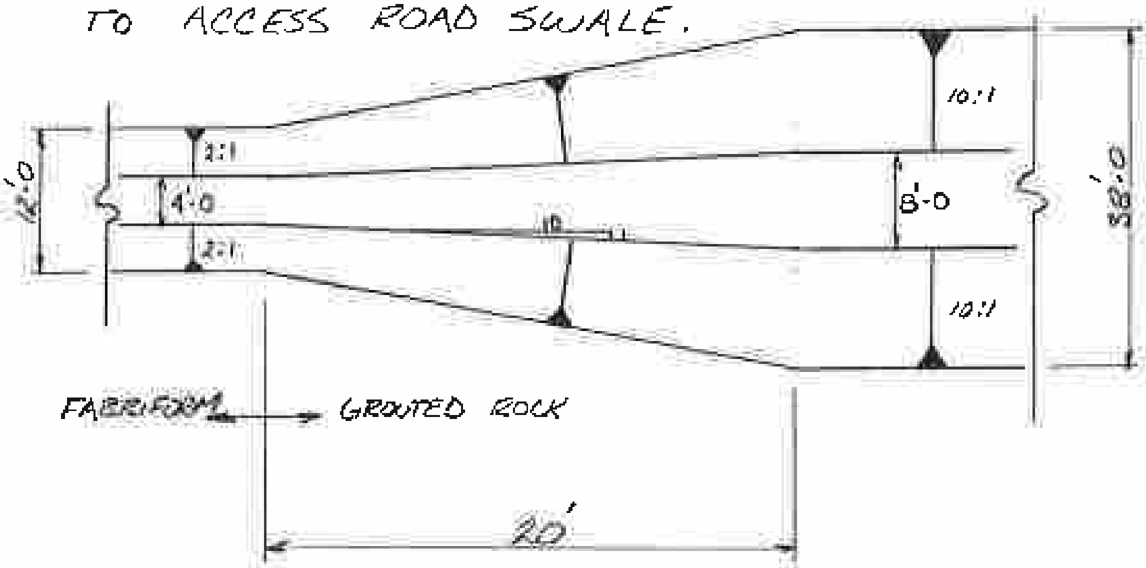
LENGTH OF TRANSITION FROM SLOPE DRAIN TO ACCESS ROAD SWALE.



APPENDIX I-1-D

FORM I

REVISIONS TO EXISTING FACILITIES - DESIGN CALCULATIONS

SUBJECT KEYSTONE STATION
PHASE II PERMITTING

BY SER DATE 4/16/96 PROJ. NO. 92-220-73-7
CHKD. BY KMB DATE 7/16/96 SHEET NO. 1 OF 15

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REVISIONS TO EXISTING FACILITIES

THE DESIGN FLOWS AND THE CAPACITY OF THE FOLLOWING EXISTING FACILITIES WILL BE ESTIMATED. THE DRAINAGE TO EACH OF THESE FACILITIES WILL BE CHANGED BY THE PROPOSED DESIGN. SEE SHEET 2 FOR LOCATION SKETCH

1) EAST VALLEY WEST SIDE COLLECTION CHANNEL (EVWSCC) PART 1 (ABOVE CULVERTS)
2) EVWSCC CULVERTS.
3) EVWSCC PART 2 (BELOW CULVERTS)
4) EV HAUL ROAD DITCH (ON THE PILE)
5) EV EAST PERIPHERAL DRAINAGE DITCH (EV EPDD)

REFERENCES

1) "ULTIMATE CONDITIONS - DRAINAGE FACILITIES" CALCS BY SER 3/19/96. SEE THIS REF. FOR DESIGN FLOWS AND DESIGN DATA. DESIGN EVENT IS THE 25YR 24HR STORM.
2) H.F. LENZ CO. "DRAINAGE DESIGN COMPUTATIONS FOR KEYSTONE STATION, EAST VALLEY ASH DISPOSAL SITE, EAST PERIPHERAL DRAINAGE DITCH", DESIGN REPORT, JUNE, 1985

EVWSCC PART 1

SHEET 5 SHOWS A CALCULATION OF FLOW DEPTH FOR MINIMUM SLOPE CONDITIONS FOR THE DESIGN FLOW OF 108 CFS.

d_{MAX} = 1.6 FEET AND TOTAL DEPTH = 2.5 FEET :

∴ FREEBOARD = 0.9 FEET WHICH IS ACCEPTABLE

∴ EVWSCC PART 1 HAS SUFFICIENT CAPACITY.

SUBJECT Keystone Station

BY SER DATE 6/9/96 PROJ. NO. 92-220-73-7

CHKD. BY KWB DATE 7/16/96 SHEET NO. 2 OF 15

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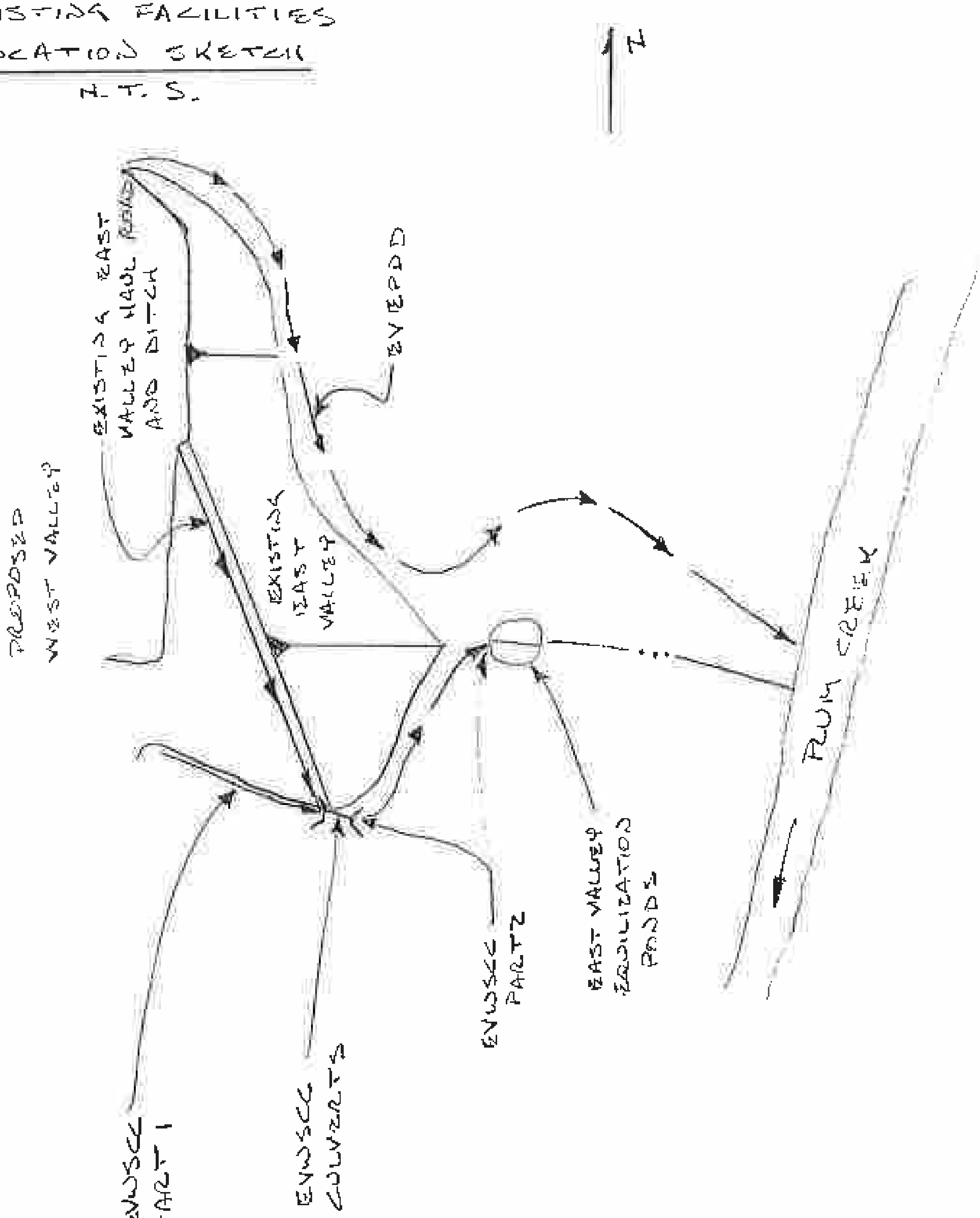
EXISTING FACILITIES
LOCATION SKETCH
N.T.S.



SUBJECT KEYSTONE STATION

BY SER DATE 4/16/96 PROJ. NO. 92-220-73-7

CHKD. BY KMB DATE 11/6/96 SHEET NO. 3 OF 15

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EV WSCC CULVERTS

SHEETS 6-9 SHOW THE CALCULATION OF THE CAPACITY OF THE CULVERTS. THE CAPACITY IS >160 CFS AND IS GREATER THAN THE DESIGN FLOW OF 158 CFS. (REF 1, SHEETS 24 & 25)

A BERM WILL BE CONSTRUCTED TO ALLOW FOR A HEADWATER POOL. THE CREST OF THE BERM WILL BE AT ELEVATION 1138 FEET NGVD.

EV WSCC PART 2

SHEET 10 SHOWS A CALCULATION OF FLOW DEPTH FOR MINIMUM SLOPE CONDITIONS FOR THE DESIGN FLOW OF 166 CFS.

d_{MAX} = 2.4 FEET AND TOTAL DEPTH = 2.5 FEET

∴ FREEBOARD = 0.1 FEET WHICH IS NOT ACCEPTABLE

SHEET 11 SHOWS A CALCULATION OF FLOW DEPTH FOR SLOPES OF 3%

d_{MAX} = 2.0 FEET AND TOTAL DEPTH = 2.5 FEET

∴ FREEBOARD = 0.5 FEET WHICH IS ACCEPTABLE

THE TOTAL DEPTH OF CHANNEL WILL BE INCREASED TO 3 FT FOR SLOPES LESS THAN 3%

EV HAUL ROAD DITCH

SHEET 12 SHOWS A CALCULATION OF FLOW DEPTH FOR MINIMUM SLOPE CONDITIONS FOR THE DESIGN FLOW OF 51 CFS

d_{MAX} = 1.0 FT AND THE TOTAL DEPTH = 2.0 FT

∴ FREEBOARD = 1.0 FT WHICH IS ACCEPTABLE

SUBJECT KEYSTONE STATION

BY SER DATE 7/8/96 PROJ. NO. 92-220-73-7

CHKD. BY [illegible] DATE 7/16/96 SHEET NO. 4 OF 15

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EVEPDD

THE LENZ DESIGN BROKE THE WATERSHED FOR THE EVEPDD INTO 11 INDIVIDUAL DRAINAGE AREAS. EACH DRAINAGE AREA WAS ADDED TO THE UPSTREAM TOTAL SUCH THAT A DESIGN FLOW WAS ESTIMATED AT THE OUTLET OF EACH OF THE 11 DRAINAGE AREAS. THE OUTLET OF THE LENZ DRAINAGE AREA 2 IS APPROXIMATELY THE SAME AS THE ULTIMATE CONDITIONS (PROPOSED) WATERSHED N3 (SEE REF 17). THE DESIGN PARAMETERS FOR ULTIMATE CONDITIONS AT THIS COMMON POINT ARE

COMPOSITES OF WATERSHEDS N1, N2, & N3

$t_c = 0.32$ HR MAX OF N1, N2, & N3

$$AREA = (0.0036 + 0.0072 + 0.0400)\,MI^2 = 0.0508\,MI^2$$

$$CN = \left(\frac{0.0036 \cdot 76 + 0.0072 \cdot 74 + 0.04 \cdot 75}{0.0508}\right) = 76$$

THE LENZ CUMULATIVE DESIGN PARAMETERS ARE SHOWN ON SHEET 13, COMBINE THE LENZ DATA WITH THE PROPOSED DATA TO ESTIMATE FLOWS DOWNSTREAM OF DRAINAGE AREA 2 SINCE NO CHANGE HAS BEEN TO THE PREVIOUS DESIGN DOWNSTREAM OF DRAINAGE AREA 2, THIS CALC IS SHOWN ON SHEET 13 AND THE TR-20 INPUT AND OUTPUT RUNS ARE INCLUDED ON SHEETS 14-15 THE DESIGN PEAK FLOWS ARE SUMMARIZED ON SHEET 13 AS WELL AS THE DITCH CAPACITIES ESTIMATED BY LENZ. THE CAPACITY ARE GREATER THAN THE DESIGN FLOWS IN ALL CASES.

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Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right)\cdot a \cdot r^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right)\cdot (r)^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$

Existing East Valley West Side Collection Channel - Part 1

Design Flow, $Q_d = 108 \cdot ft^3 \cdot sec^{-1}$ from sheet 25 of 45 of the "Ultimate Conditions - Drainage Facilities" calc by SER 3/19/96 (reference 1)

Bottom Width, $b = 3 \cdot ft$

Side Slopes, $z = 2$

Channel Lining is Grouted Rock with Manning's roughness coefficient, $n = 0.025$

Channel Minimum Slope, $S_{min} = \frac{5 \cdot ft}{160 \cdot ft}$ (from Sheet 26 of reference 1) or $S_{min} = 0.031 \cdot \frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 1.638 \cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 10.3 \cdot ft^2$

Minimum Velocity, $V_{min} = 10.5 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 9.6 \cdot ft$

Freeboard, $F_b = 0.9 \cdot ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990

Total depth, $D = 2.5 \cdot ft$ actual depth of existing channel REF GAI DRWG No 92-189-F8 AND DETAIL ON GAI DRWG. NO 92-189-F6

Top Width at Total Depth, $T_D = 13 \cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 265 \cdot ft^3 \cdot sec^{-1}$

Channel Maximum Slope, $S_{max} = \frac{45 \cdot ft}{255 \cdot ft}$ (from Sheet 26 of reference 1) or $S_{max} = 0.176 \cdot \frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 1.064 \cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 5.5 \cdot ft^2$

Maximum Velocity, $V_{max} = 19.8 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 7.3 \cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 630 \cdot ft^3 \cdot sec^{-1}$

SUBJECT Keystone Station

East Valley West Side Collection Channel Culvert

BY SER DATE 4/16/96 PROJ. NO. 92-220-73-07

CHKD. BY KMB DATE 7/16/96 SHEET NO. 6 OF 15

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PROJECT : ______

STATION : ______

SHEET ____ OF ____

CULVERT DESIGN FORM

DESIGNER / DATE : ______ / ______

REVIEWER / DATE : ______ / ______

HYDROLOGICAL DATA

SEE ADD'L SHTS.

☐ METHOD: ______

☐ DRAINAGE AREA: ______ ☐ STREAM SLOPE: ______

☐ CHANNEL SHAPE: ______

☐ ROUTING: ______ ☐ OTHER: ______

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW (cfs)	TW (ft)

ROADWAY ELEVATION : ______ (ft)

EL_{hd}: ______ (ft)

EL_{sf}: ______ (ft) S_o: ______

EL_i: 1134 (ft)

EL_o: 1133 (ft)

HW_i H TW FALL ORIGINAL STREAM BED

$S = S_o - FALL / L_a$

S = ______

L_a = 70 FT

CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE	TOTAL FLOW Q (cfs)	FLOW PER BARREL Q/N (1)	HEADWATER CALCULATIONS: INLET CONTROL HW_i/D (2)	HW_i	FALL (3)	EL_{hi} (4)	OUTLET CONTROL TW (5)	d_c	$\frac{d_c+D}{2}$	h_o (6)	k_e	H (7)	EL_{ho} (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS
DUAL 38"x60" (RxS) OVAL	158	[illegible]	1.15	3.6		1137.6	1.6	2.3	2.7	2.7	0.5	1.1**	1136.8	1137.6*		INLET CONTROL
CONCRETE WITH HEADWALL AND WINGWALLS																

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

(2) $HW_i/D = HW/D$ OR HW_i/D FROM DESIGN CHARTS

(3) $FALL = HW_i - (EL_{hd} - EL_{sf})$; FALL IS ZERO FOR CULVERTS ON GRADE

(4) $EL_{hi} = HW_i + EL_i$ (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL.

(6) $h_o = TW$ or $(d_c + D/2)$ (WHICHEVER IS GREATER)

(7) $H = \left[1 + k_e + (29 n^2 L) / R^{1.33}\right] V^2/2g$

(8) $EL_{ho} = EL_o + H + h_o$

SUBSCRIPT DEFINITIONS:

a. APPROXIMATE
f. CULVERT FACE
hd. DESIGN HEADWATER
hi. HEADWATER IN INLET CONTROL
ho. HEADWATER IN OUTLET CONTROL
i. INLET CONTROL SECTION
o. OUTLET
sf. STREAMBED AT CULVERT FACE
tw. TAILWATER

COMMENTS / DISCUSSION :

TW = 1.6 FEET SEE SHEET 5, MIN SLOPE CONDITION

CULVERT BARREL SELECTED :

SIZE: 38" x 60" (RxS)

SHAPE: OVAL

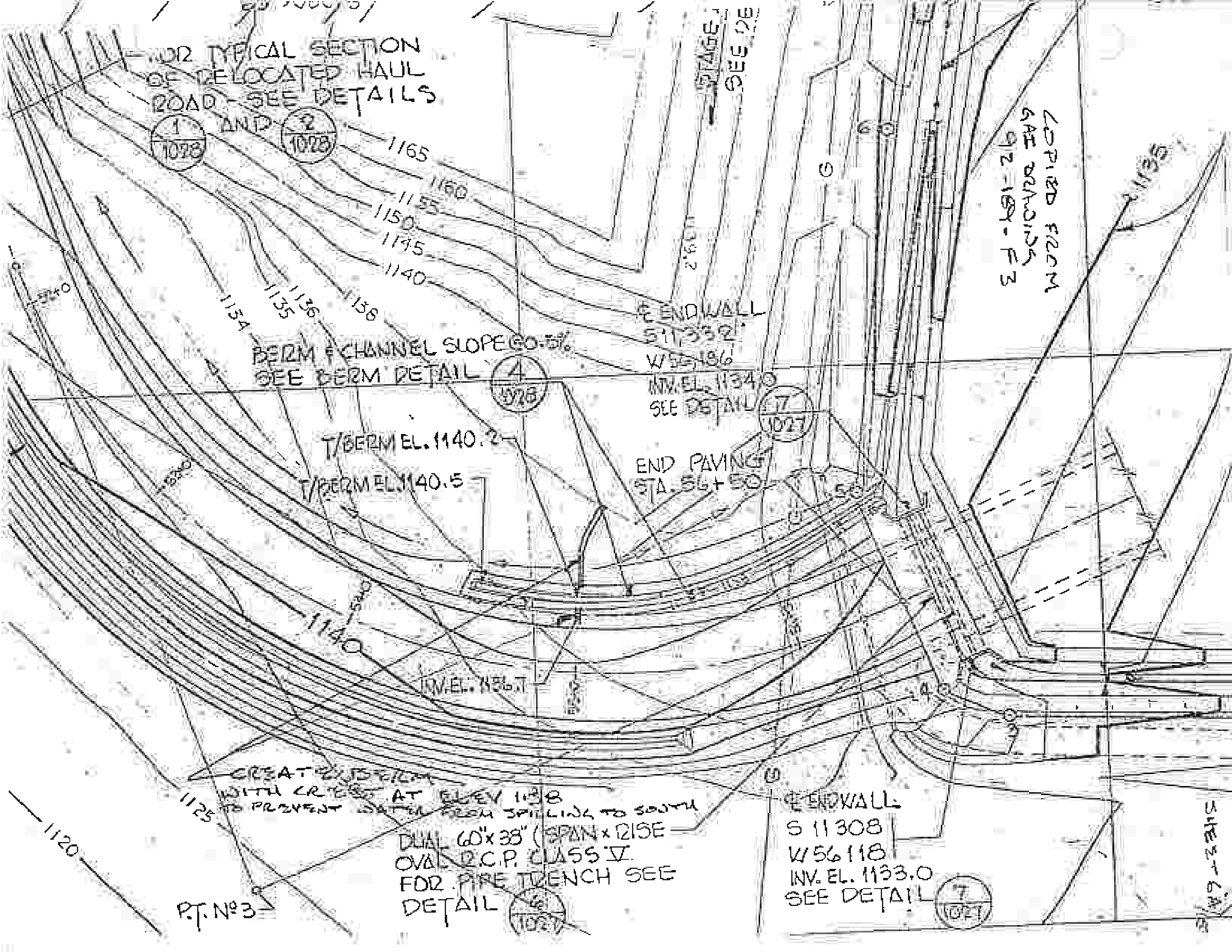
MATERIAL: CONCRETE

ENTRANCE: WINGWALLS + HW

REF DRAWING D-728-1023 FOR DIMENSIONS AND ELEVATIONS

* CREATE A BERM WITH TOP ELEV = 1138 TO CREATE A HEADWATER POOL TO PREVENT WATER SPILLING OVER CREST IN ADJACENT DITCHES. SEE SHEET 6A.

** SEE SHEET 8



FOR TYPICAL SECTION OF RELOCATED HAUL ROAD - SEE DETAILS
1/1028 AND 2/1028
1165
1160
1155
1150
1145
1140
1136
1135
1134
1139.2
STAGE
SEE
COPIED FROM GAI DRAWING 92-184-F3
1135
℄ ENDWALL S11332 W56186 INV. EL. 1134.0 SEE DETAIL 7/1027
BERM & CHANNEL SLOPE @ 0.5% SEE BERM DETAIL 4/1028
T/BERM EL. 1140.2
T/BERM EL. 1140.5
END PAVING STA. 56+50
1140
INV. EL. 1136.7
CREATE BERM WITH CREST AT ELEV 1138 TO PREVENT WATER FROM SPILLING TO SOUTH
1125
1120
DUAL 60"x38" (SPAN x RISE) OVAL R.C.P. CLASS V. FOR PIPE TRENCH SEE DETAIL 6/1027
℄ ENDWALL S 11308 W56118 INV. EL. 1133.0 SEE DETAIL 7/1027
P.T. No 3
SHEET 6A

SUBJECT Keystone Station
East Valley West Side Collection Channel Culvert

BY SFN DATE 4/16/96 PROJ. NO. 92-220-73-07

CHKD. BY KWB DATE 7/16/96 SHEET NO. 7 OF 15

CONSULTANTS, INC.

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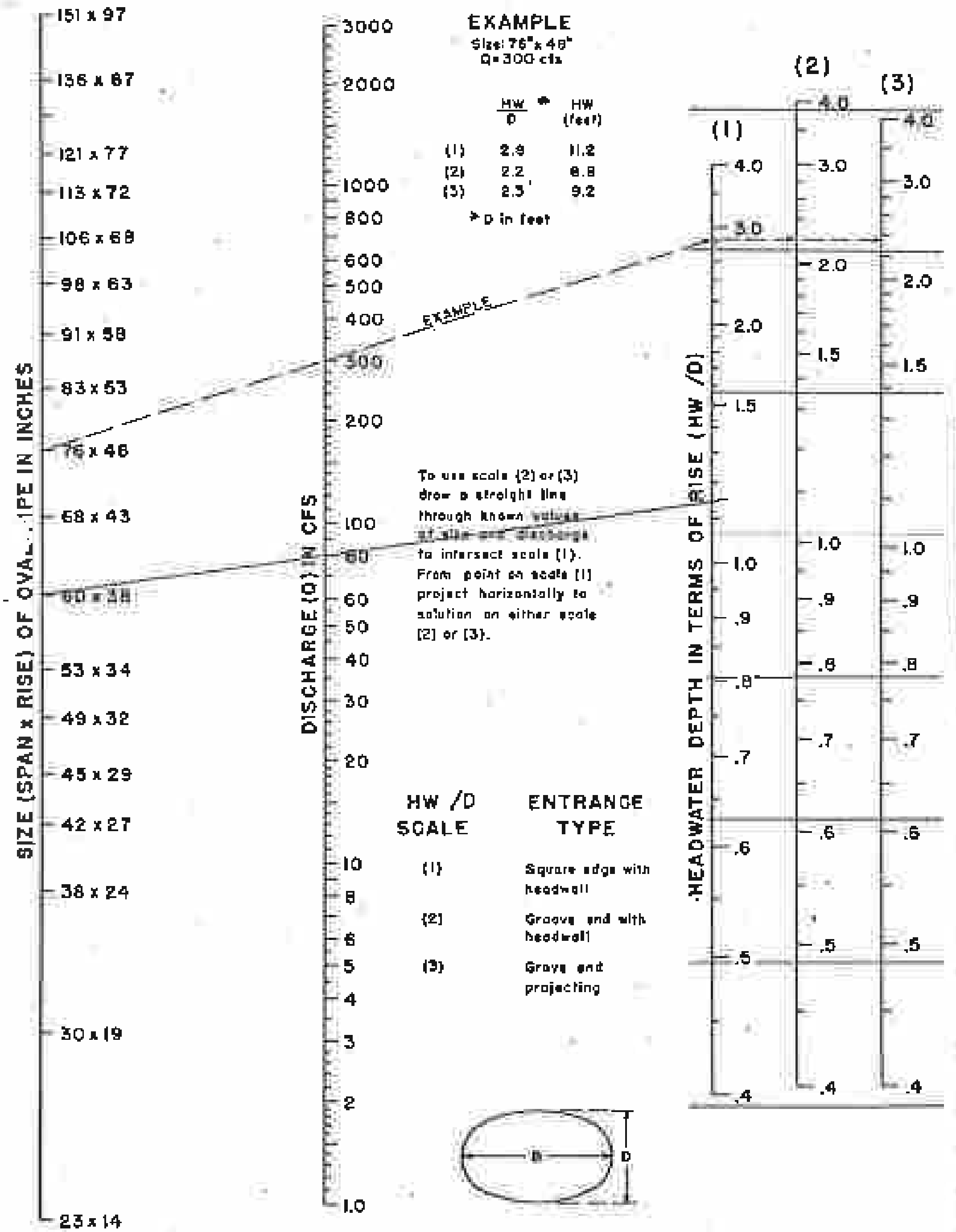
CHART 29



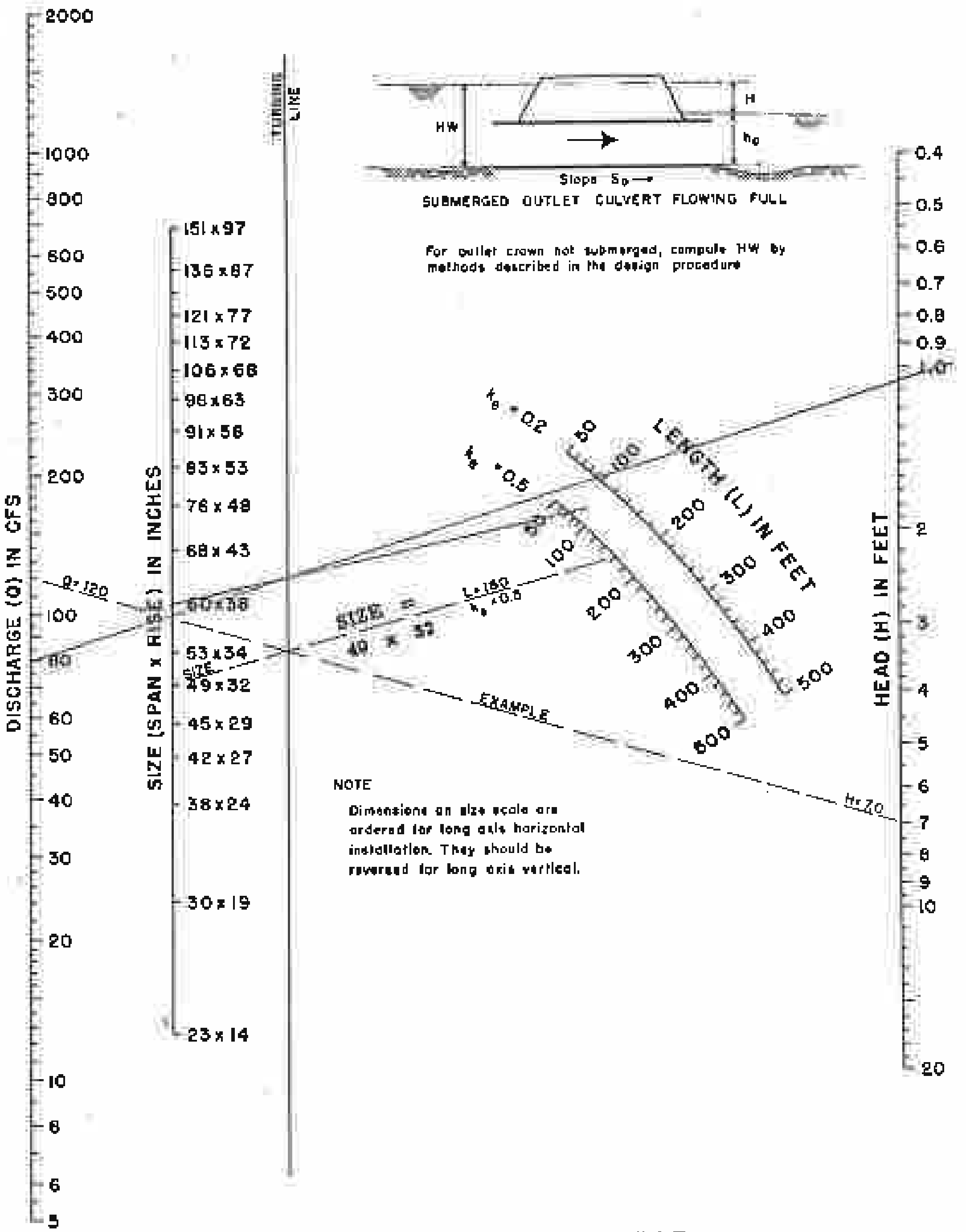
TAKEN FROM:
"HYDRAULIC DESIGN OF HIGHWAY CULVERTS",
HDS No 5, FHWA,
SEPTEMBER, 1985

HEADWATER DEPTH FOR
OVAL CONCRETE PIPE CULVERTS
LONG AXIS HORIZONTAL
WITH INLET CONTROL

Keystone Station - East Valley West Side
Collection Channel Culvert
Proj No 92-220-73-07
By SER 4/16/96
✓ KMB 7/16/96

SH 8/15

CHART 33



HEAD FOR OVAL CONCRETE PIPE CULVERTS LONG AXIS HORIZONTAL OR VERTICAL FLOWING FULL

$n = 0.012$

BUREAU OF PUBLIC ROADS JAN. 1963

SUBJECT Keystone Station

BY SER DATE 4/16/96 PROJ. NO. 92-220-93-07

CHKD. BY KMB DATE 7/16/96 SHEET NO. 7 OF 15

CONSULTANTS, INC.

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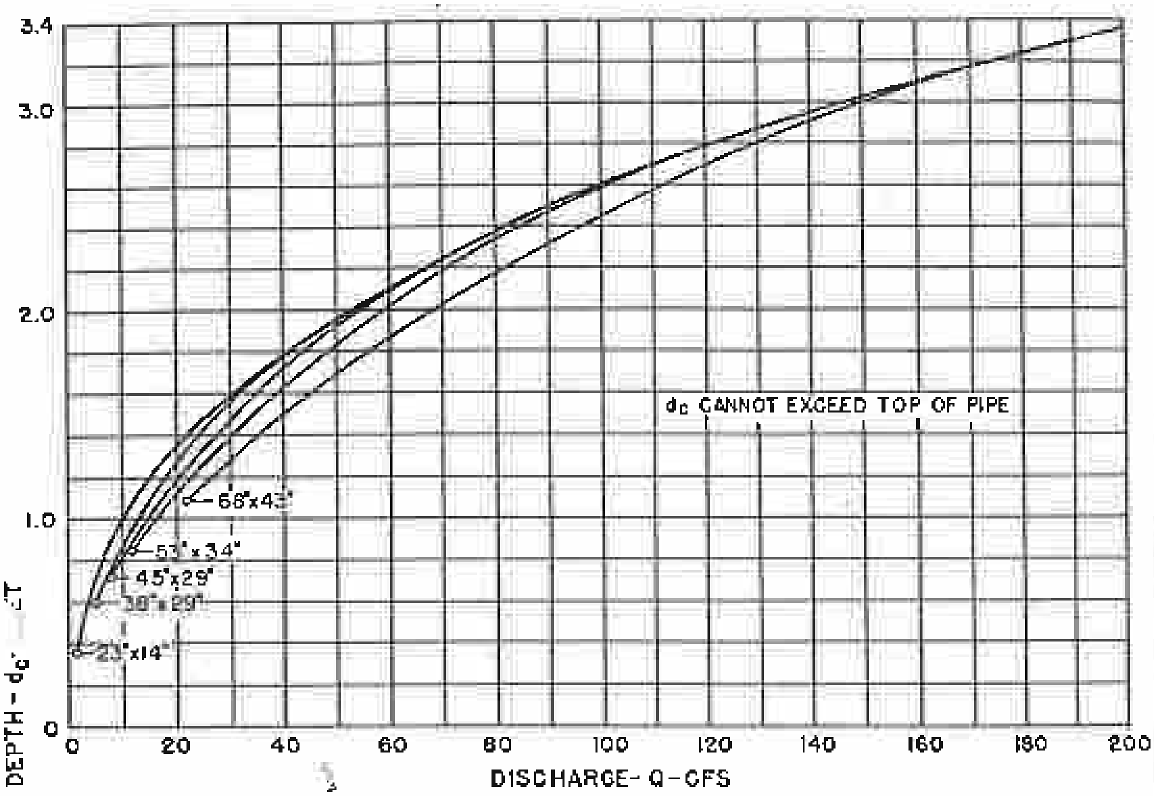
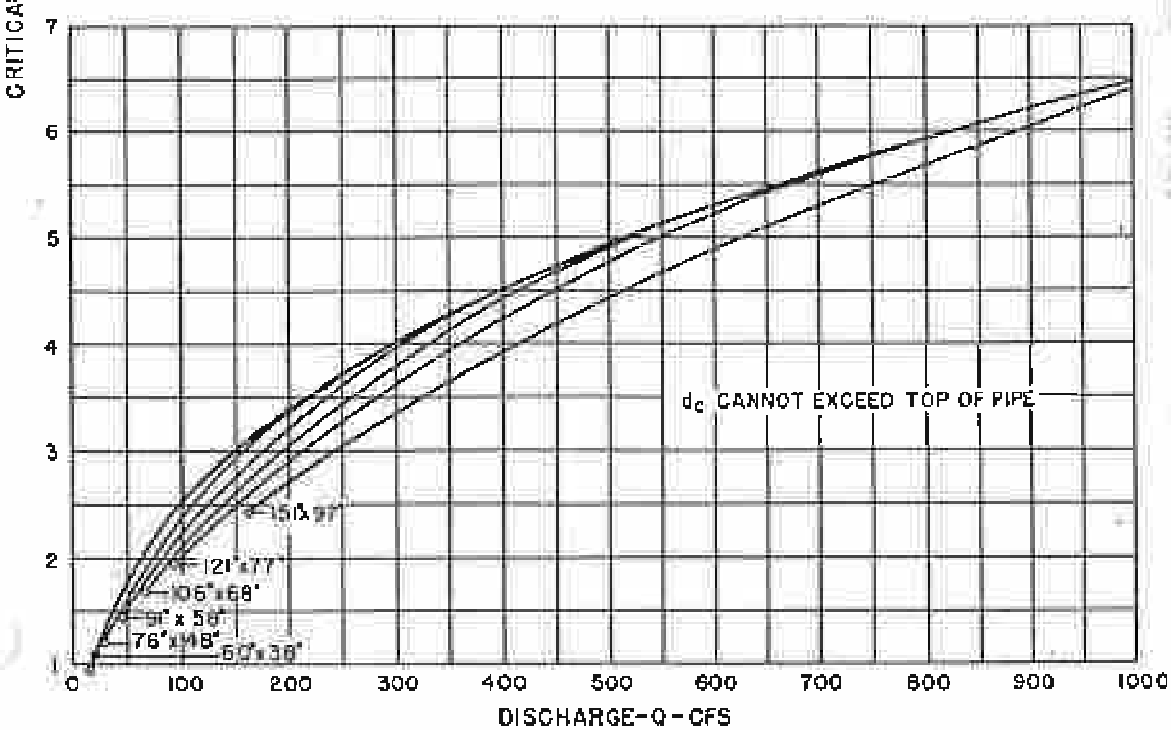
CHART 31



TAKEN FROM: "HYDRAULIC DESIGN OF HIGHWAY CULVERTS", HDS-No 5, FHWA, SEPTEMBER, 1965



BUREAU OF PUBLIC ROADS

JAN. 1964

CRITICAL DEPTH
OVAL CONCRETE PIPE
LONG AXIS HORIZONTAL

SUBJECT: Keystone Station
Phase II Permitting
BY: SER DATE: 6/8/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KMB DATE: 7/16/96 SHEET NO. 12 OF 15

GAI CONSULTANTS, INC.
Engineers Geologists Planners
Environmental Specialists

Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right) \cdot a \cdot r^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right) \cdot (r)^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$

Existing East Valley West Side Collection Channel - Part 2

Design Flow, $Q_d = 166 \cdot ft^3 \cdot sec^{-1}$ from sheet 25 of 45 of the "Ultimate Conditions - Drainage Facilities" calc by SER 3/19/96 (reference 1)

Bottom Width, $b = 3 \cdot ft$

Side Slopes, $z = 2$

Channel Lining is Grouted Rock with Manning's roughness coefficient, $n = 0.025$

Channel Minimum Slope, $S_{min} := 0.016 \cdot \frac{ft}{ft}$ (from calc by DMK 3/26/85 titled "Keystone - Closure Hydraulics")

Maximum Flow Depth, $d_{max} = 2.352 \cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 18.1 \cdot ft^2$

Minimum Velocity, $V_{min} = 9.2 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 12.4 \cdot ft$

Freeboard, $F_b = 0.6 \cdot ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990

Total depth, $D = 3 \cdot ft$

Top Width at Total Depth, $T_D = 15 \cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 284 \cdot ft^3 \cdot sec^{-1}$

Channel Maximum Slope, $S_{max} := 0.227 \cdot \frac{ft}{ft}$ (from calc by DMK 3/26/85 titled "Keystone - Closure Hydraulics")

Minimum Flow Depth, $d_{min} = 1.241 \cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 6.8 \cdot ft^2$

Maximum Velocity, $V_{max} = 24.4 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 8 \cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 1 \cdot 10^3 \cdot ft^3 \cdot sec^{-1}$

SUBJECT: Keystone Station
Phase II Permitting
BY: SER DATE: 6/8/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KWB DATE: 9/16/96 SHEET NO. 11 OF 15

GAI CONSULTANTS, INC.
Engineers Geologists Planners
Environmental Specialists

Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right) \cdot a \cdot r^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right) \cdot (r)^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$

Existing East Valley West Side Collection Channel - Part 2 - minimum allowable slope for existing depth

Design Flow, $Q_d = 166 \cdot ft^3 \cdot sec^{-1}$ from sheet 25 of 45 of the "Ultimate Conditions - Drainage Facilities" calc by SER 3/19/96 (reference 1)

Bottom Width, $b = 3 \cdot ft$

Side Slopes, $z = 2$

Channel Lining is Grouted Rock with Manning's roughness coefficient, $n = 0.025$

Channel Minimum Slope, $S_{min} := 0.03 \cdot \frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 2.031 \cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 14.3 \cdot ft^2$

Minimum Velocity, $V_{min} = 11.6 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 11.1 \cdot ft$

Freeboard, $F_b = 0.5 \cdot ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990

Total depth, $D = 2.5 \cdot ft$

Top Width at Total Depth, $T_D = 13 \cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 260 \cdot ft^3 \cdot sec^{-1}$

SUBJECT: Keystone Station
Phase II Permitting - Ultimate Conditions
BY: SER DATE: 4/12/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KWB DATE: 7/26/96 SHEET NO. 12 OF 15

GAI CONSULTANTS, INC.
Engineers Geologists Planners
Environmental Specialists

Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right) \cdot a \cdot r^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right) \cdot (r)^{\left(\frac{2}{3}\right)} \cdot s^{\left(\frac{1}{2}\right)}$

Existing East Valley Haul Road Ditch

Design Flow, $Q_d = 51 \cdot ft^3 \cdot sec^{-1}$ from sheet 25 of 45 of reference 1

Bottom Width, $b = 2 \cdot ft$

Side Slopes, $z = 2$

Channel Lining is Grouted Rock with Manning's roughness coefficient, $n = 0.025$

Channel Minimum Slope, $S_{min} := \frac{25 \cdot ft}{250 \cdot ft}$ (from Sheet 26 of reference 1) or $S_{min} = 0.1 \cdot \frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 0.966 \cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 3.8 \cdot ft^2$

Minimum Velocity, $V_{min} = 13.4 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 5.9 \cdot ft$

Freeboard, $F_b = 1 \cdot ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990

Total depth, $D = 2 \cdot ft$ Actual depth of existing channel

Top Width at Total Depth, $T_D = 10 \cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 240 \cdot ft^3 \cdot sec^{-1}$

Channel Maximum Slope, $S_{max} := \frac{25 \cdot ft}{250 \cdot ft}$ (from Sheet 26 of reference 1) or $S_{max} = 0.1 \cdot \frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 0.966 \cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 3.8 \cdot ft^2$

Maximum Velocity, $V_{max} = 13.4 \cdot ft \cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 5.9 \cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 240 \cdot ft^3 \cdot sec^{-1}$

SHEET 13/15

Keystone Station									
Phase II Permitting									
Project 92-220-73-07									
By SER 7/8/96									
Chkd By WPHS	7/16/96								
East Peripheral Drainage Ditch									
Lenz Drainage Area	Lenz Cumulative Area Sq. Mi.	Incremental Area Sq. Mi.	Lenz Cumulative t_c hours	Incremental t_c hours	Proposed t_c hours	Proposed Area Sq. Mi.	Proposed CN	Lenz Ditch Capacity cfs	Design Flow cfs
2	0.169		1.67		0.32	0.051	76	196	70
3	0.180	0.011	1.69	0.02	0.34	0.062	76	338	83
4	0.183	0.003	1.70	0.01	0.35	0.065	76	336	86
5	0.188	0.005	1.71	0.01	0.36	0.070	76	167	92
6	0.225	0.037	1.75	0.04	0.40	0.107	77	171	137
7	0.226	0.001	1.77	0.02	0.42	0.108	77	590	134
8	0.228	0.002	1.78	0.01	0.43	0.110	77	768	135
9	0.228	0.000	1.79	0.01	0.44	0.110	77	532	133
10	0.233	0.005	1.80	0.01	0.45	0.115	77	1004	139
11	0.250	0.017	1.80	0.00	0.45	0.132	77	315	160

Notes: All values are cumulative to the outlet of the drainage area unless noted otherwise.

SHEET
14/15

```
JOB TR-20                    FULLPRINT          SUMMARY   NOPLOTS
TITLE 111 KEYSTONE EAST PERIPHERAL DRAINAGE DITCH - 92-220-73-7
 6 RUNOFF 1 001        1 0.051        76.          0.32         1            AREA2
 6 RUNOFF 1 001        2 0.062        76.          0.34         1            AREA3
      OFF 1 001        3 0.065        76.          0.35         1            AREA4
 6 ..NOFF 1 001        4 0.070        76.          0.36         1            AREA5
 6 RUNOFF 1 001        5 0.107        77.          0.40         1            AREA6
 6 RUNOFF 1 001        6 0.108        77.          0.42         1            AREA7
 6 RUNOFF 1 001        7 0.110        77.          0.48         1            AREA8
 6 RUNOFF 1 001        1 0.110        77.          0.44         1            AREA9
 6 RUNOFF 1 001        2 0.115        77.          0.45         1            AREA10
 6 RUNOFF 1    01      3 0.132        77.          0.45         1            AREA11
   ENDATA
 7 LIST
 7 INCREM 6              0.05
 7 COMPUT 7 001       01 0.             4.4        1.           2 2          25 YR
   ENDCMP 1
   ENDJOB 2
```

✓KMB 7/16/96

SHEET 15/15

SUMMARY TABLE 1 - SELECTED RESULTS OF STANDARD AND EXECUTIVE CONTROL INSTRUCTIONS IN THE ORDER PERFORMED
(A STAR(*) AFTER THE PEAK DISCHARGE TIME AND RATE (CFS) VALUES INDICATES A FLAT TOP HYDROGRAPH
A QUESTION MARK(?) INDICATES A HYDROGRAPH WITH PEAK AS LAST POINT.)

S[illegible]ON/ S[illegible]URE ID	STANDARD CONTROL OPERATION	DRAINAGE AREA (SQ MI)	RAIN TABLE #	ANTEC MOIST COND	MAIN TIME INCREM (HR)	PRECIPITATION BEGIN (HR)	PRECIPITATION AMOUNT (IN)	PRECIPITATION DURATION (HR)	RUNOFF AMOUNT (IN)	PEAK DISCHARGE ELEVATION (FT)	PEAK DISCHARGE TIME (HR)	PEAK DISCHARGE RATE (CFS)	PEAK DISCHARGE RATE (CSM)
ALTERNATE 0	STORM 0												
XSECTION 1	RUNOFF	.05	2	2	.05	.0	4.40	24.00	1.53	---	12.10	69.99	1372.3
XSECTION 1	RUNOFF	.06	2	2	.05	.0	4.40	24.00	1.53	---	12.11	83.25	1342.7
XSECTION 1	RUNOFF	.06	2	2	.05	.0	4.40	24.00	1.53	---	12.11	85.65	1317.7
XSECTION 1	RUNOFF	.07	2	2	.05	.0	4.40	24.00	1.53	---	12.12	91.80	1311.4
XSECTION 1	RUNOFF	.11	2	2	.05	.0	4.40	24.00	1.60	---	12.14	137.31	1283.3
XSECTION 1	RUNOFF	.11	2	2	.05	.0	4.40	24.00	1.59	---	12.16	134.28	1243.3
XSECTION 1	RUNOFF	.11	2	2	.05	.0	4.40	24.00	1.59	---	12.16	135.08	1228.0
XSECTION 1	RUNOFF	.11	2	2	.05	.0	4.40	24.00	1.59	---	12.17	132.59	1205.4
XSECTION 1	RUNOFF	.12	2	2	.05	.0	4.40	24.00	1.59	---	12.17	139.08	1209.4
STRUCTURE 1	RUNOFF	.13	2	2	.05	.0	4.40	24.00	1.59	---	12.17	159.64	1209.4

1

APPENDIX I-1-E

FORM I

WEST VALLEY EQUALIZATION POND - DESIGN CALCULATIONS

SUBJECT Keystone West Valley Equalization Pond

BY MAG DATE 3/3/08 PROJ. NO. C060665.00.040

CHKD. BY DATE SHEET NO. 1 OF 1

gai consultants

Run off Areas

* West Valley Active Disposal Area, 2006 Aerial Survey
  48.9 ac.

* Proposed Borrow Area #1
  10.0 ac

Pond Design

* Sediment Storage
  3,000 cf per disturbed acre (1,800 cf req by DEP)

* Maximum Area of Active Disposal
  58 ac

* Maximum Drainage Area of Pond
  80 ac

* Stabilized Area unable to be diverted
  22 ac

* West Valley Liner Area
  108 ac

Calculations

* Sediment Storage Volume
  (Active Disposal Area) × (3,000 cf per acre)

* Pond Req. Volume
  (Sediment Volume) + (Leachate Volume)
  * Leachate Volume based on 108 ac liner area, 11 day storage
  - Design Volume = 7.0 MG
  - Actual Volume = 7.6 MG

SUBJECT Table of Contents Appendix I-1-E

BY ______ DATE ______ PROJ. NO. ______

CHKD. BY ______ DATE ______ SHEET NO. 1 OF 1

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Environmental Specialists

WEST VALLEY EQUALIZATION POND CALCULATIONS

TABLE OF CONTENTS

SUBJECT: KEYSTONE - PHASE II PERMITTING

BY: SEL DATE: 4/22/96 PROJ. NO. 92-220-73-07

CHKD. BY: JMJ DATE: 5/6/96 SHEET NO. 1 OF 7

[illegible] 4/14/98 / KMB 4/19/98 PLUS ATTACHED [illegible]

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LEACHATE / SURGE POND (WEST VALLEY EQUALIZATION POND)

PURPOSE: ESTIMATE TOTAL VOLUME OF STORAGE REQUIRED.

CRITERIA: 1) POND MUST STORE THE RUNOFF FROM A 10-YEAR, 24-HOUR EVENT WITH NO DISCHARGE TO STREAM, NPDES REQUIREMENT (AS PER JMJ AS PER DISCUSSIONS WITH PENN DEP)

2) POND WILL BE USED TO STORE ~~30~~ 10-DAY LEACHATE VOLUME AT A MINIMUM.

∴ SET THE PRINCIPAL SPILLWAY CREST ELEVATION AT THE 10-YR 24 HR STORAGE VOLUME + SEDIMENT STORAGE AND STORE ~~30~~ 10-DAY LEACHATE VOLUME BETWEEN PRINCIPAL AND EMERGENCY SPILLWAY CRESTS. ~~STORE 15-DAYS LEACHATE STORAGE ABOVE HEAD TO PASS 25-YEAR, 24-HOUR STORM PEAK FLOW THROUGH PRINCIPAL SPILLWAY (≈1.6 FT)~~

RUNOFF VOLUME

~~FROM EQUALIZATION [illegible]~~

ESTIMATE MAXIMUM VOLUME OF RUNOFF FOR 10-YR, 24-HOUR EVENT (AND 25-YR, 24-HOUR EVENT)

$P_{10,24} = 3.9$ IN. ($P_{25,24} = 4.4$ IN.) SEE "ULTIMATE CONDITIONS - DRAINAGE FACILITIES" CALC BY SEL 3/19/96, SHEET 7

CN = 85, ACTIVE DISPOSAL AREA SEE KEYSTONE STATION, PROJECT DESIGN PARAMETERS OUTLINE, EAST VALLEY DISPOSAL AREA, PROJ 85-376-04 SEPT. 1987

$$S = \frac{1000}{CN} - 10$$
$$Q = \frac{(P - 0.2 \cdot S)^2}{P + 0.8 \cdot S}$$
FROM TR-55

$S = \frac{1000}{85} - 10 = 1.8$ in.

$Q_{10} = 2.3$ in. ($Q_{25} = 2.8$ in.)

SUBJECT Keystone - Phase II Permitting

BY SER DATE 4/22/96 PROJ. NO. 92-220-73-07

CHKD. BY JMJ DATE 5/6/96 SHEET NO. 2 OF 7

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EVALUATE MAXIMUM AREA OF ACTIVE DISPOSAL

AT ANY ONE INSTANT THE ACTIVE DISPOSAL AREA SHOULD BE ≥ 30 AC. AS PER STATION REQUIREMENTS (AS PER JMJ).

ASSUME NEW LINER AREA ≐ ACTIVE DISPOSAL AREA.

AT ELEV 1375, STAGE 3 ACTIVE AREA ≐ 30 AC,

∴ STAGE 4A MUST BE CONSTRUCTED BEFORE THIS POINT IN TIME

STAGE 4A LINER AREA ≐ 20 AC

∴ TOTAL ACTIVE AREA = 50 AC AT TIME $t = t_1$.

THE FOLLOWING YEAR STAGE 4B LINER (17 AC) WILL BE CONSTRUCTED.

STAGE 4A AND 4B LINER AREA = 38 AC, ASSUME STAGE 3 ACTIVE AREA ≐ 20 AC, AND STAGE 4A REVEGATATED AREA IS VERY SMALL, SAY 0 AC.

∴ TOTAL ACTIVE AREA = 58 AC AT TIME $t = t_2$.

THE FOLLOWING YEAR STAGE 4C LINER (19 AC) WILL BE CONSTRUCTED.

STAGE 4 LINER AREA = 56 AC, ASSUME STAGE 3 ACTIVE AREA ≐ 0 AND VERY LITTLE STAGE 4A AREA IS REVEG. SAY 2 ACRES

∴ TOTAL ACTIVE AREA = 54 AC AT TIME $t = t_3$.

∴ THE MAXIMUM AREA OF ACTIVE DISPOSAL = 58 AC, OCCURS WHEN STAGE 3 IS STILL ACTIVE AND STAGE 4 HAS 2 YRS OF LINER (STAGE 4A & 4B) INSTALLED ($t = t_2$).

EVALUATE MAXIMUM DRAINAGE AREA OF LEACHATE/SURGE POND

THE MAX. AREA DRAINING TO THE POND WOULD EQUAL THE MAX. AREA OF ACTIVE DISPOSAL + STABILIZED AREAS WHICH CANNOT BE DIVERTED

ASSUME THAT THE MAX. DR. AREA OF POND = 80 AC

THIS ASSUMES THAT A MAX. OF (80 - 58) AC = 22 AC OF STABILIZED AREA COULD NOT BE DIVERTED. THIS IS CONSIDERED REASONABLE ALSO STABILIZED AREAS WOULD PRODUCE LESS RUNOFF.

SUBJECT KEYSTONE - PHASE II PERMITTING

BY SER DATE 4/22/96 PROJ. NO. 92-220-73-07

CHKD. BY JMJ DATE 5/6/96 SHEET NO. 3 OF 7

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∴ RUNOFF VOLUME 10 YR 24 HR EVENT = 80 AC · 2.3 IN · $\frac{FT}{12 IN}$
= 15.3 AC-FT · $\frac{43560 FT^2}{AC}$ · 7.48 GAL/FT^3
= 5.0 MILLION GALLONS = 5 MG

RUNOFF VOLUME 25 YR 24 HR EVENT = 80 AC · 2.8 IN · $\frac{FT}{12 IN}$
= 18.7 AC-FT · 43560 · 7.48 $\frac{MG}{AC FT}$
= 6.1 MG

LEACHATE VOLUME (Average Flowrate Method)

ESTIMATE LEACHATE FLOWRATE

USE FLOW CHARTS FOR EXISTING EAST VALLEY WEIR BOX (ATTACHED)

YEAR	AVERAGE FLOW *	LINER ACRES	FLOW RATE (GPM/AC)
1990	80,000 GPD	75	0.74
1991	72,000 GPD	87	0.57
1992	72,000 "	96	0.52
1993	79,000 "	105	0.52
1994	111,000 "	115	0.67
1995 (PARTIAL)	77,000 "	115	0.46
		AVG =	0.6 GPM/AC

* ESTIMATED FROM FLOW CHARTS ATTACHED. AVERAGE FLOW GIVEN ABOVE IS THE AREA BENEATH THE CURVE DIVIDED BY THE DURATION. THE AVERAGE FLOWS SHOWN ON THE CHARTS ARE VISUAL ESTIMATES WHICH YIELD THE SAME AVERAGE = 0.6 GPM/AC (AS PER PREVIOUS ESTIMATE BY JMJ)

SUBJECT KEYSTONE - PHASE II PERMITTING

BY SER DATE 4/22/96 PROJ. NO. 92-220-73-07

CHKD. BY JMJ DATE 5/6/96 SHEET NO. 4 OF 7

REVISED SER 1/9/98 ✓KMP 1/19/98

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EAST VALLEY HISTORICAL AVERAGE FLOWRATE = 0.6 GPM/AC

DISCUSSION

THE EAST VALLEY HAS HAD LARGE AREAS OF "OPEN" LINER OVER THE PERIOD OF RECORD OF THE WEIR BOX. "OPEN" LINER REFERS TO LINER WHICH HAD NOT YET RECIEVED WASTE AND HAD AN EXPOSED 24" BOTTOM ASH PROTECTIVE LAYER. THIS CONDITION CAUSED LARGE PEAK FLOWRATES TO OCCUR DUE TO HIGH INFILTRATION RATES ON THESE "OPEN" LINER AREAS.

THE WEST VALLEY WILL HAVE AN 18" BOTTOM ASH LEACHATE COLL. ZONE WITH A ~~AND~~ 12" FLYASH PROTECTIVE COVER. THIS WILL ALLOW A SUBSTANTIALLY HIGHER RUNOFF CONDITION, THEREFORE THE INFILTRATION AND LEACHATE RATES WILL BE LOWER. THEREFORE THE WEST VALLEY AVERAGE FLOW RATE IS EXPECTED TO BE LOWER THAN 0.6 GPM/AC.

NOTE THAT THE CN VALUE OF 85 FOR ACTIVE DISPOSAL IS FOR AVERAGE/NORMAL WASTE CONDITIONS NOT BOTTOM ASH.

AN ESTIMATE OF MAX 30 DAY LEACHATE FLOWRATE HAS BEEN PRODUCED BASED ON HELP MODELLING OF PROPOSED CONDITIONS, SEE CALC BY JMJ 9/18/97 "ESTIMATE OF LEACHATE QUANTITY" IN FORM 17R APPENDIX Z.

MAX. 30 DAY LEACHATE FLOWRATE = 0.68 GPM/AC

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LEACHATE VOLUMES AND FLOWRATE

WEST VALLEY LINER AREA = 108AC

WEST VALLEY LEACHATE FLOWRATE = 108AC · 0.68[1] GPM/AC = 73GPM

WEST VALLEY 10 DAY LEACHATE VOLUME = 73GPM · 10DAY · (1440MIN / DAY) = 1.1 MG

WEST VALLEY 30 DAY LEACHATE VOLUME = 73GPM · 30DAY · (1440MIN / DAY) = 3.2 MG

(1) WORST MONTHLY FLOWRATE

*SEDIMENT STORAGE VOLUME PROVIDE A MINIMUM OF 2000 CF OF SEDIMENT STORAGE VOLUME PER DISTURBED ACRE IN ADDITION TO THE RUNOFF AND LEACHATE VOLUMES. THE POND WILL BE CLEANED BEFORE SEDIMENT REACHES THIS ELEV.

∴ SEDIMENT STORAGE VOLUME = 58AC · 2000 CF/AC · 7.48 GAL/CF = 0.9 MG.

WEST VALLEY EQUALIZATION POND REQUIRED VOLUMES

SEDIMENT STORAGE VOLUME = 0.9 MG, USE 1.35 MG @ ELEV 1076.5

10-YEAR, 24-HOUR STORM RUNOFF VOLUME = 5.0 MG FROM ELEV 1076.5 TO 1086.1

**PASSING 25-YEAR, 24-HOUR STORM PEAK = 7.45 MG @ ELEV 1087.6

10-DAY LEACHATE VOLUME = 1.1 MG FROM ELEV 1086.1 TO 1087.6

TOTAL REQUIRED VOLUME = (SED + 10YR 24 HR + 10 DAY LEACH.) = 0.9 + 5.0 + 1.1 = 7.0 MG

ACTUAL VOLUME

THE STAGE-STORAGE RATING CURVE IS SHOWN ON SHEET 6

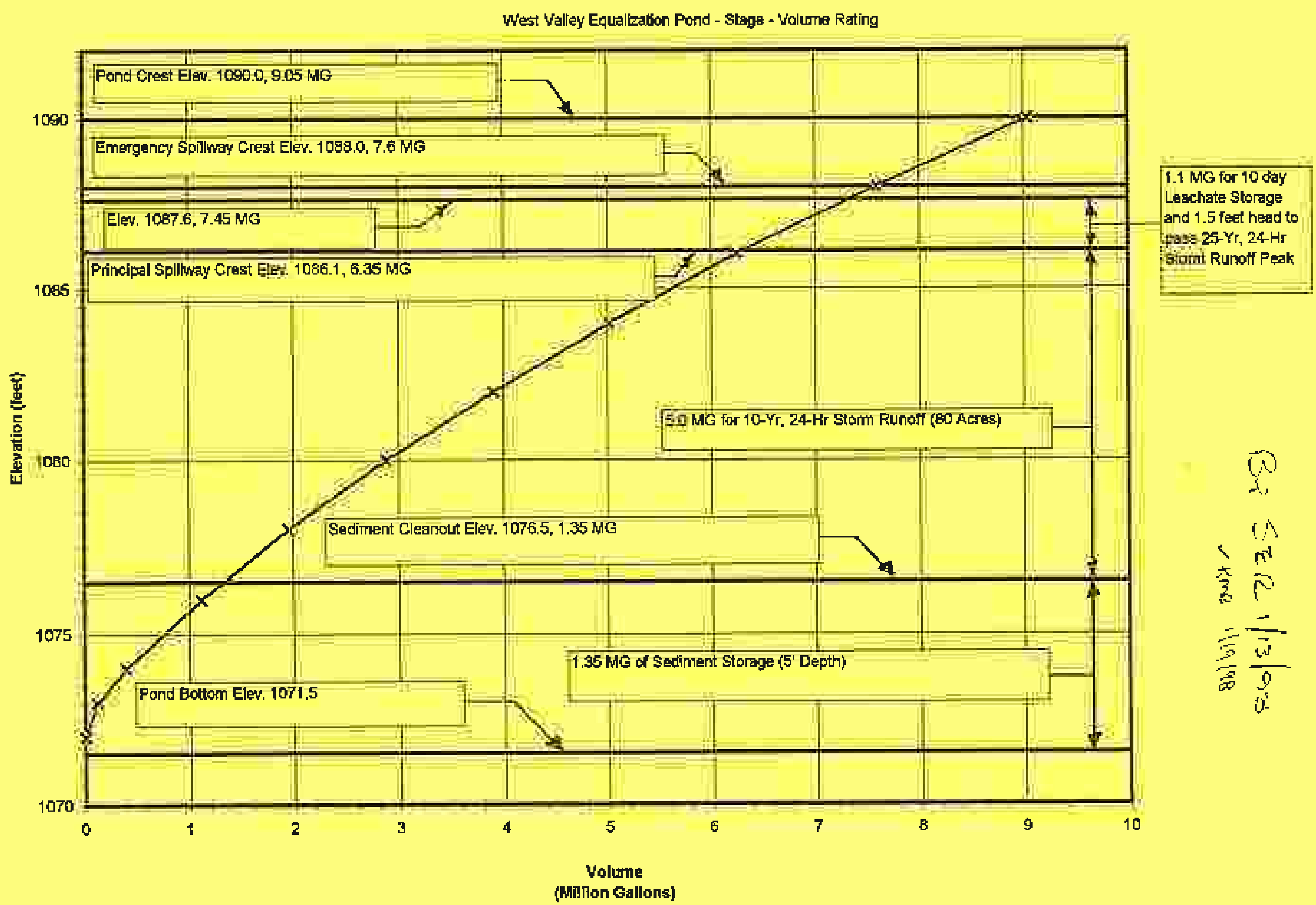
5.0 MG IS AVAILABLE BETWEEN THE SEDIMENT STORAGE ELEVATION OF 1076.5 AND THE PRINCIPAL SPILLWAY CREST ELEVATION OF 1086.1, WHICH IS ACCEPTABLE (THIS IS 10 YR 24 HR RUNOFF STORAGE)

1.25 MG IS AVAILABLE BETWEEN THE PRINCIPAL SPILLWAY CREST ELEV OF 1086.1 AND THE EMERGENCY SPILLWAY CREST ELEV OF 1088.0, WHICH IS ACCEPTABLE (THIS IS > 10-DAY LEACHATE STORAGE)

TOTAL VOLUME IS GREATER THAN 7.0 MG ∴ OK
≈ 7.6 MG

** POND DOES NOT STORE 25-YR STORM, BUT HANDLES IT THROUGH PRINCIPAL SPILLWAY.

* SEDIMENT SETTLING VOLUME AVAILABLE IS 5 MG WHICH IS GREATER THAN THE REQUIRED SETTLING VOLUME OF 80AC · 5000 CF/AC = 400,000 CF = 3.0 MG



By SER 1/13/98

✓ KMB 1/19/98

Sheet 6/7

1/13/98

Sheet 7/7

Keystone Station					
Project No. 92-220-73-7					
By: SER 1/12/98					
Chkd. By: KMS 1/13/98					
Revised West Valley Equalization Pond					
Volume estimate by average end area method					
		Average	Incremental	Cumulative	Cumulative
Elevation	Area	Area	Volume	Volume	Volume
	(acre)	(acre)	(acre ft)	(acre ft)	(million gallons)
1071.5	0.00				0
1072	0.07	0.035927	0.0179637	0.0	0.01
1073	0.63	0.352342	0.3523416	0.4	0.12
1074	1.06	0.846414	0.8464141	1.2	0.40
1076	1.20	1.12875	2.2575	3.5	1.13
1078	1.35	1.2725	2.545	6.0	1.96
1080	1.49	1.4175	2.835	8.9	2.88
1082	1.64	1.56125	3.1225	12.0	3.90
1084	1.81	1.72	3.44	15.4	5.02
1086	1.97	1.8875	3.775	19.2	6.25
1088	2.14	2.055	4.11	23.3	7.59
1090	2.32	2.23	4.46	27.8	9.05

SUBJECT Keystone
Phase II Permitting

BY STR DATE 7/23/96 PROJ. NO. 97-220-72-7

CHKD. BY DATE SHEET NO. 1 OF 7

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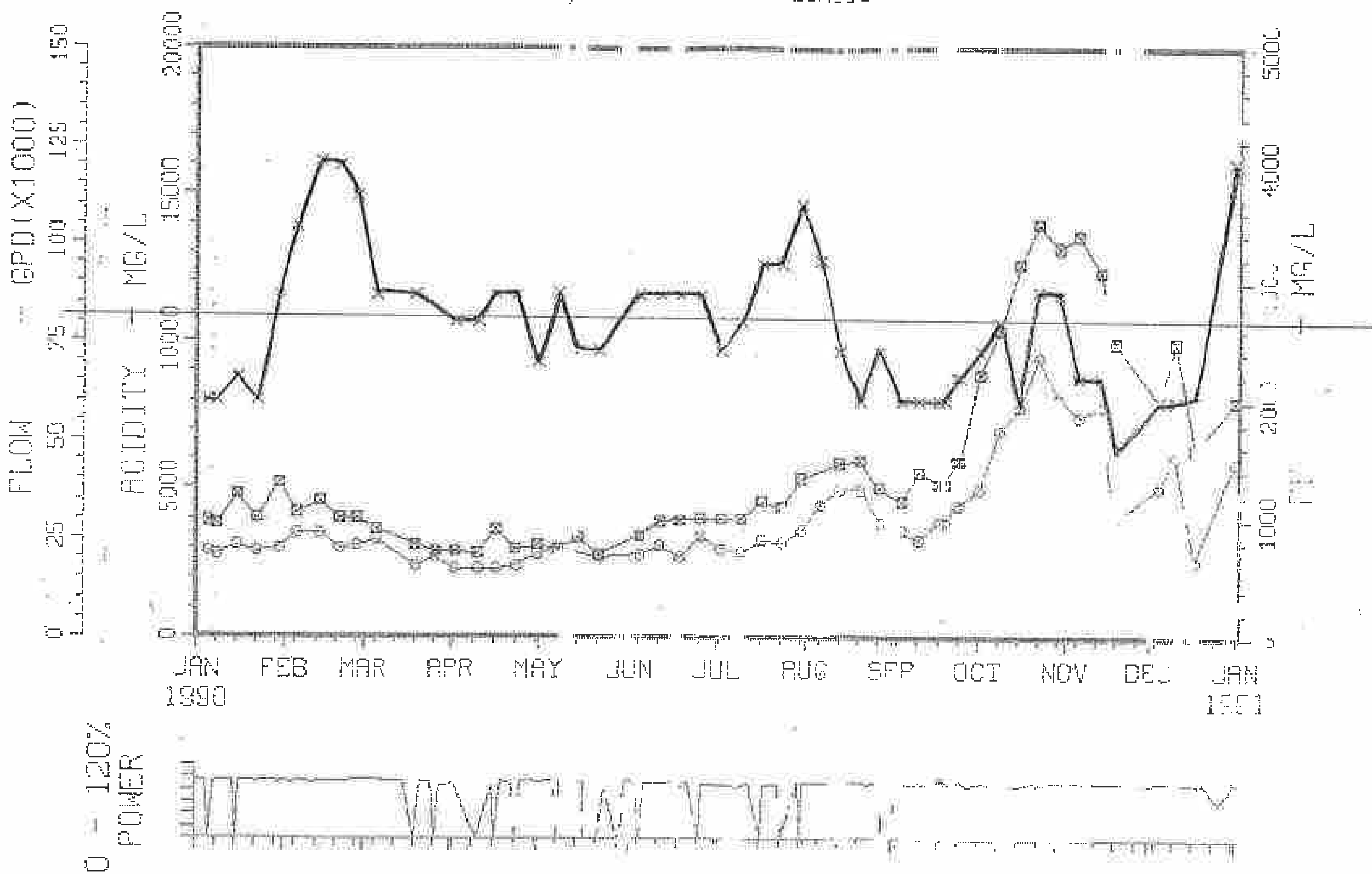
EXISTING EAST VALLEY LEACHATE WEIR BOX

FLOWCHARTS

KEYSTONE -# 01 E. VAL LEACHATE #2 (GRAB)

01/01/90 TO 01/01/91 ○ - ACIDITY ⊠ - FE X - FLOW

→ , ← = OPERATING LIMITS



Ave. 82,000 GPD

A = 21.8 m²

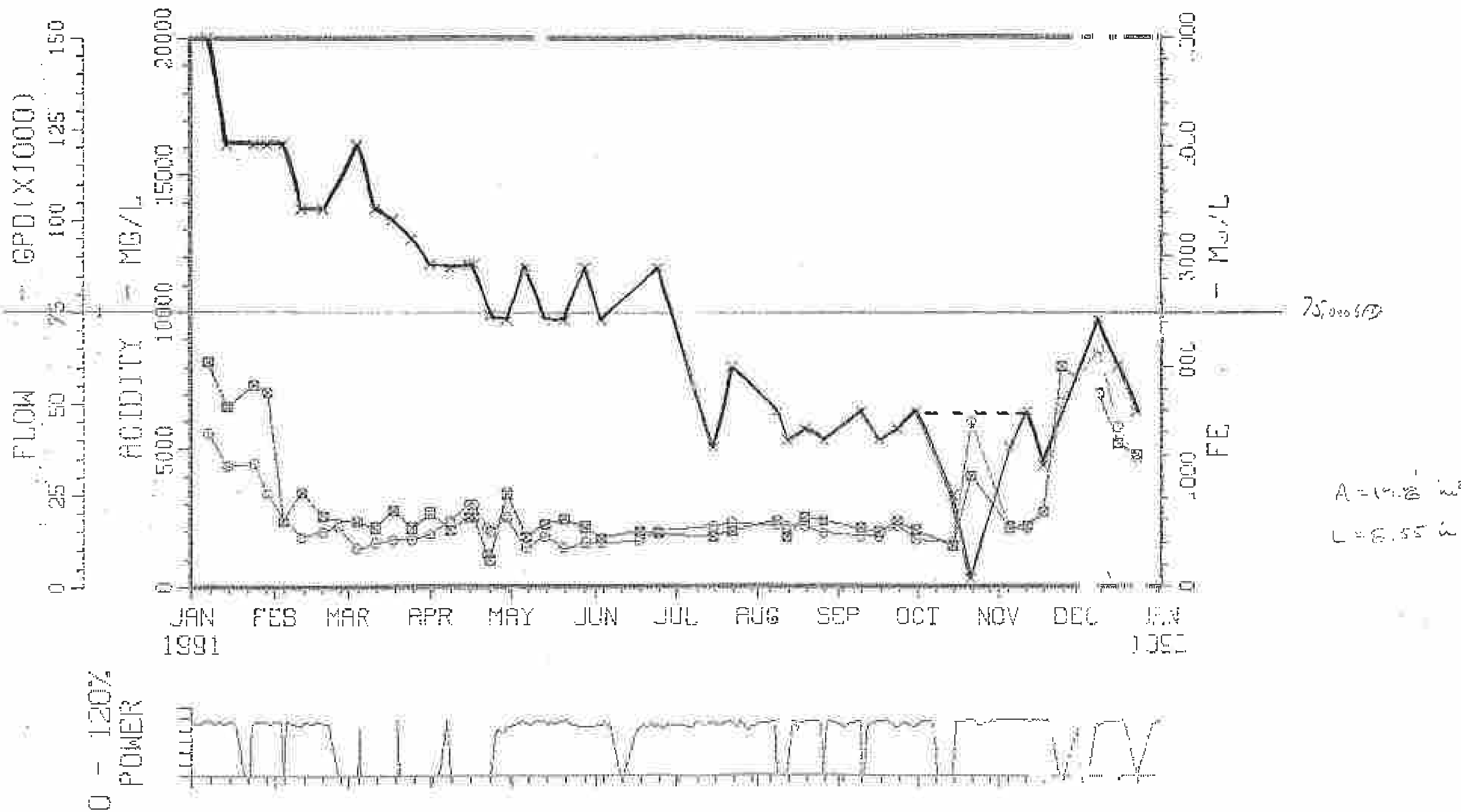
L = 8.55 m

SHEET 3/7

KEYSTONE -# 01 E. VAL LEACHATE #2 (GRAB)

01/01/91 TO 01/01/92 ○ - ACIDITY ⊠ - FE X - FLOW

→ , ← = OPERATING LIMITS

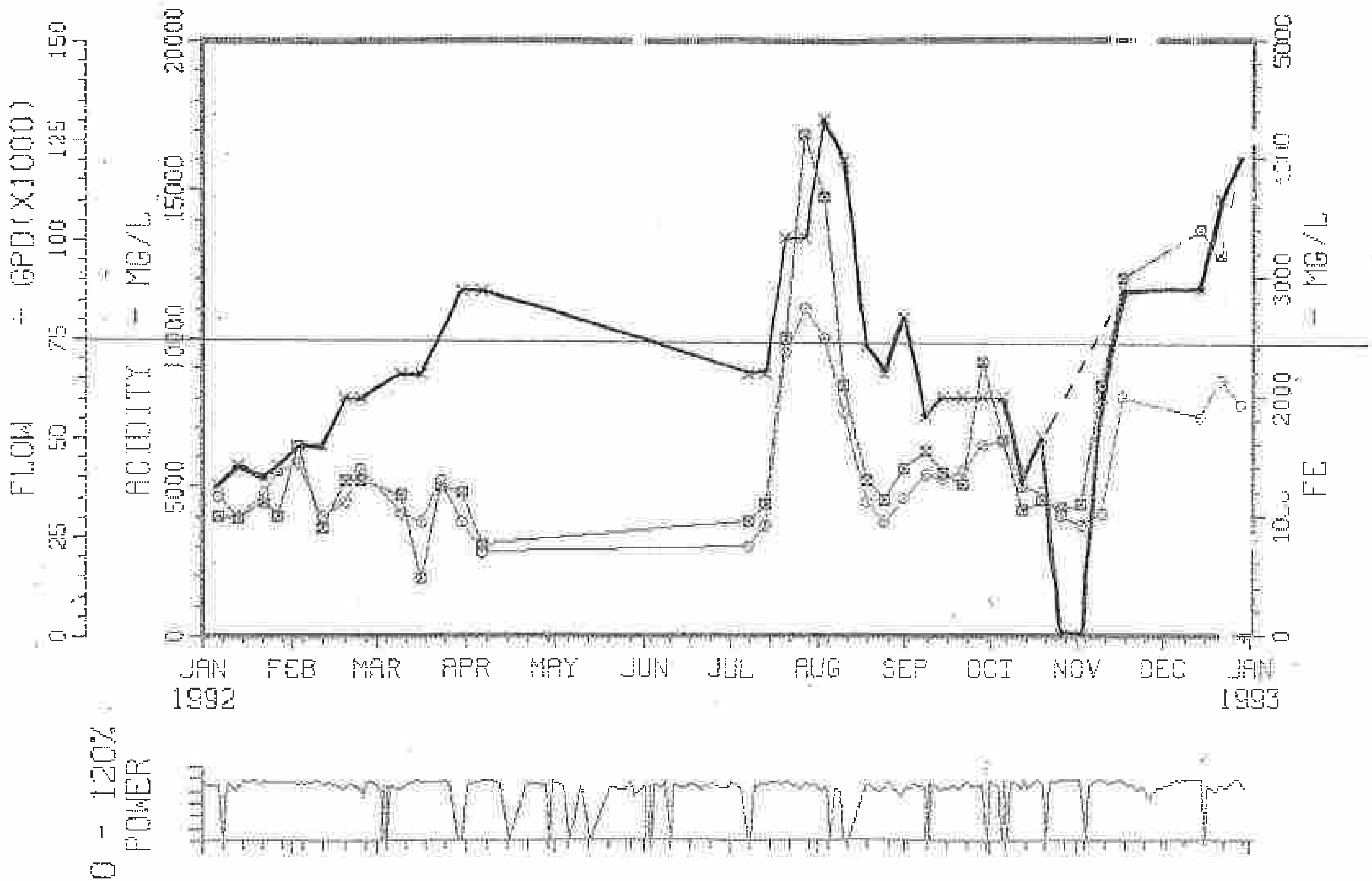


SHEET 4/7

KEYSTONE -# 01 E. VAL LEACHATE #2 (GRAB)

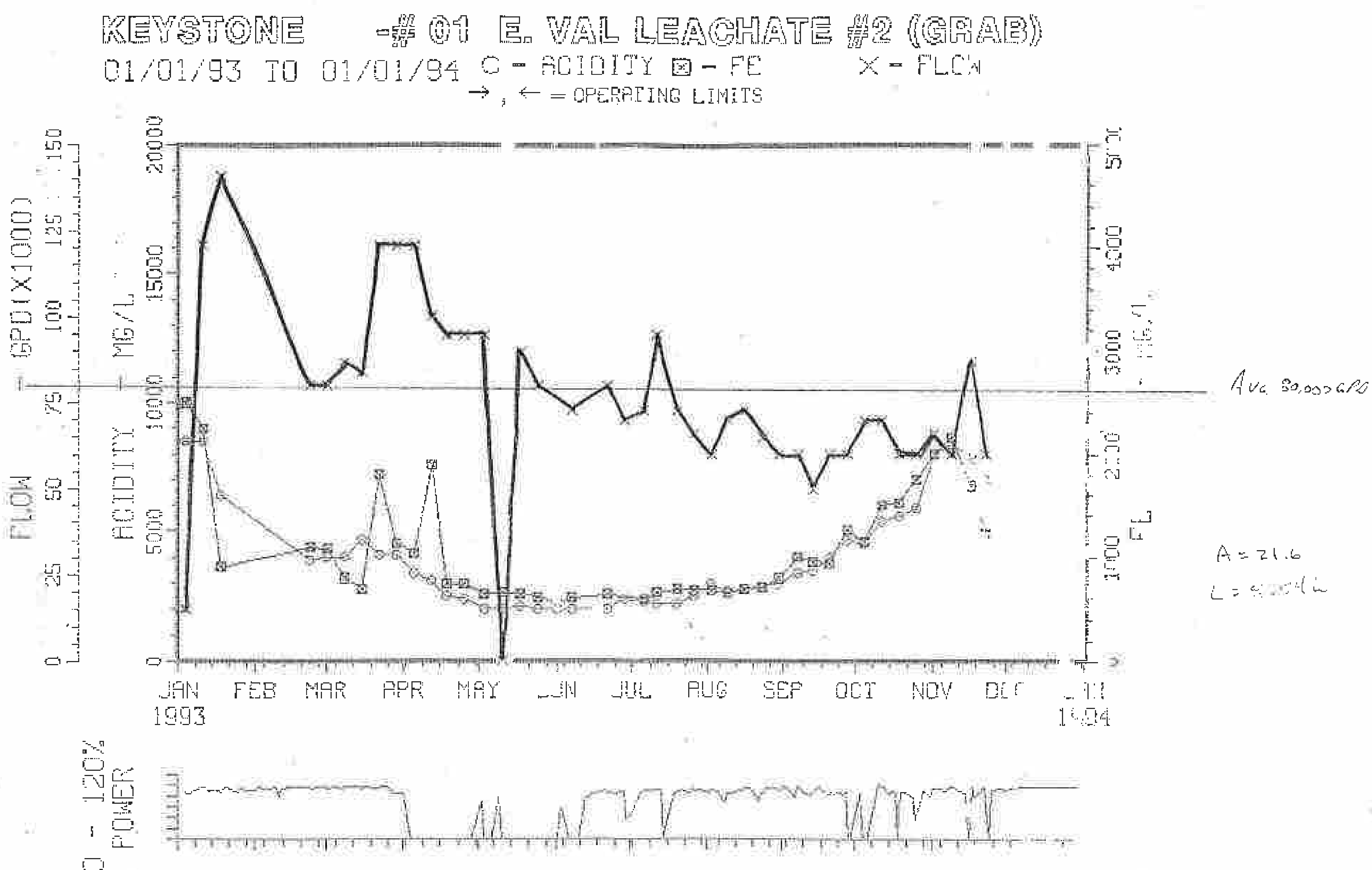
01/01/92 TO 01/01/93 ○ - ACIDITY ⊠ - FE X - FLOW

→ , ← = OPERATING LIMITS



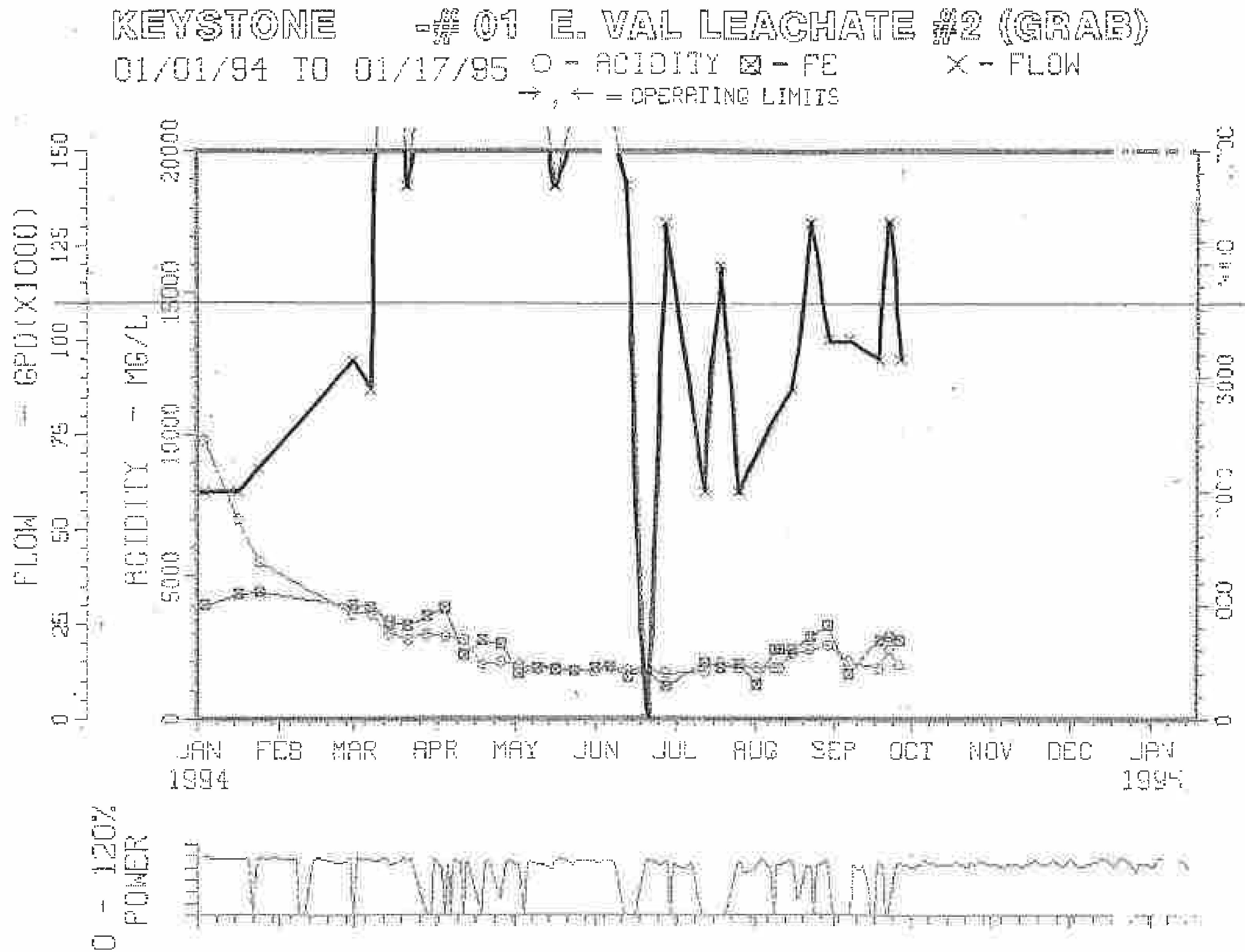
Avg 75,000 G/D

SHEET 5/7


KEYSTONE -# 01 E. VAL LEACHATE #2 (GRAB)
01/01/93 TO 01/01/94 ○ - ACIDITY ⊠ - FE X - FLOW
→ , ← = OPERATING LIMITS
FLOW GPD(X1000)
ACIDITY MG/L
FE MG/L
JAN 1993
FEB
MAR
APR
MAY
JUN
JUL
AUG
SEP
OCT
NOV
DEC
1994
0 - 120% POWER
AVG 80,000 GPD
A = 21.6

2

SHEET 6/7



Avg 129,000 GPD

A = 21.4 in²
L = 6.63 in

GRAPHIC LIMITS EXCEEDED

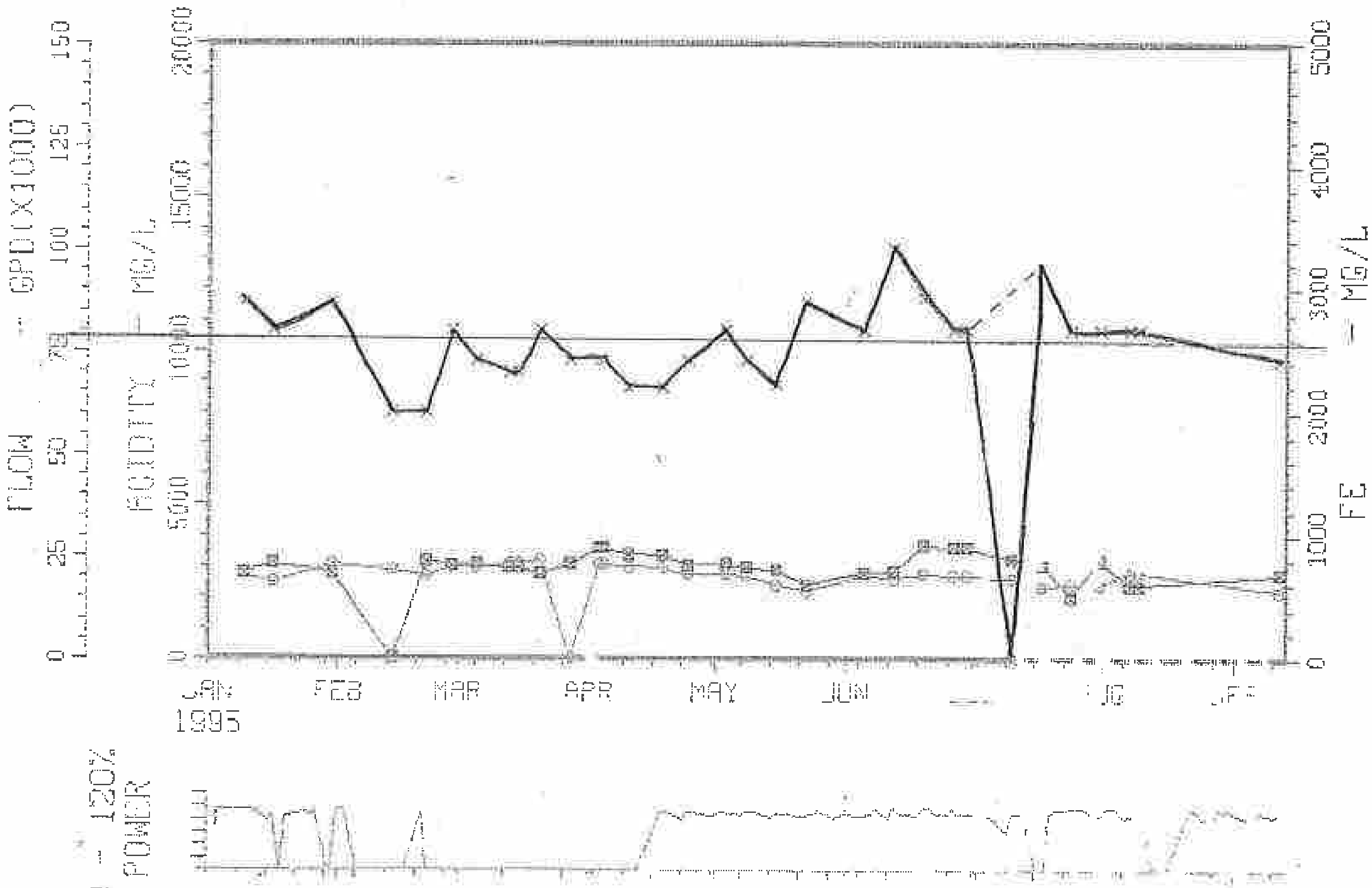
1

SHEET 7/7

KEYSTONE -# 01 E. VAL LEACHATE #2 (GRAB)

01/01/95 TO 09/12/95 O - ACIDITY ⊠ - FE X - FLOW

→ , ← = OPERATING LIMITS



Avg. 78,000 GPD

A=21.1

L=8.51

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SUBJECT: KEYSTONE WEST VALLEY
PHASE II PERMITTING

BY: KRB DATE: 6/3/96 PROJ. NO.: 92-220-73-07

CHKD. BY: MRL DATE: 7/3/96 SHEET NO. 1 OF 14

▲ Revised by MRL 7/17/96
CHKD BY SER 7/22/96

▼ REVISED BY SER 4/30/96

■ REVISED BY SER 1/13/99 / KRB 1/8/99

EQUALIZATION POND - OUTLET STRUCTURE

▲ DESIGN THE RISER AND THE BARREL OF THE PRINCIPAL SPILLWAY.

VARIOUS CRITERIA WILL BE USED TO DESIGN THE ▲ PRINCIPAL SPILLWAY:

1) PRINCIPAL SPILLWAY ▲ MUST ALLOW FOR SEDIMENT STORAGE AND STORAGE OF 10-YEAR 24-HOUR RUNOFF VOLUME ▲ BEFORE DISCHARGING.

2) DESIGN THE ▲ PRINCIPAL SPILLWAY TO PASS PEAK FLOW FROM 25-YEAR 24-HOUR STORM.

$Q_{p,25} \cong 100$ cfs (GRAVITY WATER DIVIDES CALCS BY SER, SHEET 14 OF 24)

▼ ~~*~~ 3) STORE ■ ~~30~~ 10-DAY LEACHATE VOLUME BETWEEN THE PRINCIPAL SPILLWAY ▲ RISER CREST AND THE EMERGENCY SPILLWAY CREST

■ ~~4) STORE 10-DAY LEACHATE VOLUME BETWEEN ▲ THE HEAD REQUIRED TO PASS THE $Q_{p,25}$ THROUGH THE PRINCIPAL SPILLWAY RISER ORIFICES AND THE EMERGENCY SPILLWAY CREST.~~

▼ ~~* THIS IS A DESIRED CRITERIA ONLY AND IS NOT ABSOLUTELY NECESSARY~~

SUBJECT KEYSTONE WEST VALLEY
PHASE II PERMITTING

BY [illegible] DATE 6/13/96 PROJ. NO. 92-220-73-07
CHKD. BY MRL DATE 7/3/96 SHEET NO. 2 OF 14

▲ Revised by MRL 7/17/96 ▼ REVISED BY SER 9/20/96
CHKD BY BBR 9/22/96 ■ REVISED BY SER 1/13/98 VRMG 1/17/98

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EQUALIZATION POND - OUTLET STRUCTURE CONTINUED

USE THE CRITERIA ON THE PREVIOUS PAGE TO DETERMINE ▲INVERT ELEVATIONS FOR THE PRINCIPAL SPILLWAY. STAGE - STORAGE DATA FOR THE POND IS DOCUMENTED

FROM CRITERION 1:

ALLOWING FOR 5.0 MILLION GALLONS (RUNOFF FROM 80 ACRES DURING 10-YR 24-HR STORM) + ■5.0' OF SEDIMENT STORAGE, MINIMUM PRINCIPLE SPILLWAY ▲CREST ELEVATION = ■1086.1'

(REF. ▼"LEACHATE/SURGE POND" CALCS BY SER 4/22/96) ■REV 1/9/98

FROM CRITERION 3:

EMERGENCY SPILLWAY CREST = 1088.

■10 DAYS OF LEACHATE STORAGE IS AVAILABLE BETWEEN EL. ■1036.1' AND THE EMERGENCY SPILLWAY CREST.

(REF. ▼"LEACHATE/SURGE POND" CALCS)

▼ ~~FROM CRITERION 4:~~

~~FROM EL. 1088, SUBTRACTING 10 DAYS OF LEACHATE STORAGE PRODUCES EL. 1086.1. THIS ALLOWS A HEAD ON THE PRINCIPAL SPILLWAY ▲CREST OF 1086.1 - 1084.6 = 1.5 ft.~~

~~(REF. JMN + SBE CALCS)~~

NOTE: 30-DAY LEACHATE VOLUME = ■3.2 MILLION GALLONS
■ ~~10~~ 10 - DAY LEACHATE VOLUME = ■1.1 MILLION GALLONS

(REF. ▼"LEACHATE/SURGE POND" CALCS)

SUBJECT KEYSTONE , WEST VALLEY
PHASE II PERMITTING

BY KMB DATE 6/19/96 PROJ. NO. 92-220-73-07
CHKD. BY MRL DATE 7/3/96 SHEET NO. 3 OF 14
▲ Revised MRL 7/17/96
CHKD BY SRG 7/26/96
▼ REVISED BY SRG 9/30/96
■ REVISED BY SRG 1/13/97 CHKD 1/19/98

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EQUALIZATION POND - OUTLET STRUCTURE CONTINUED:

PROPOSED: A SQUARE, CONCRETE RISER STRUCTURE WITH RECTANGULAR OPENINGS IN THREE OF THE FOUR FACES ▲ WITH INVERTS AT ELEVATION ■1066.1'. THE RISER WILL HAVE 1.5 FOOT THICK WALLS. SIZE THE RISER OPENINGS.

▼ SIZE THE RISER OPENINGS TO BE 1.5 FEET HIGH. FIND THE LENGTH.

THIS WILL PRODUCE AN ORIFICE WITH THE WATER SURFACE ~~AT~~ NEAR THE CROWN OF THE OPENING. THIS SITUATION IS DESCRIBED ON PG 4-5 OF BRATER & KING'S "HANDBOOK OF HYDRAULICS":

ORIFICES, GATES, AND TUBES 4-5

discharging under a head y, is

$$dQ_t = L\sqrt{2gy}\,dy$$

which, integrated between the limits h_2 and h_1, gives

$$Q_t = \tfrac{2}{3}L\sqrt{2g}\,(h_2^{3/2} - h_1^{3/2}) \quad (4\text{-}16)$$

When h_1 is zero

$$Q_t = \tfrac{2}{3}L\sqrt{2g}\,h_2^{3/2} \quad (4\text{-}17)$$

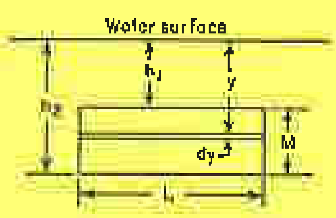

FIG. 4-4. Rectangular orifice.

which is the theoretical formula, without velocity-of-approach correction, for discharge over a weir.

▼ FROM SHEET 9 THE HW ELEV OF THE PRINCIPAL SPILLWAY BARREL WILL BE 1064.3' ∴ THE PRINCIPAL SPILLWAY CREST WILL HAVE ■(1066.1' - 1064.2') = ~~[illegible]~~ ■2.0 FT OF TAILWATER

ASSUME h_1 = ■0.0 FT

EQ. (4-17) IS APPLICABLE FOR THE CASE WHERE THE WATER LEVEL IS AT THE TOP OF THE OPENING. $h_1 = 0$, $h_2 = 1.5'$ AND h_2 = ■1.5' FT

$$Q = \frac{2}{3} L \sqrt{2g}\, h^{3/2}$$

$$Q = 100 \text{ cfs}$$
$$h = 1.5 \text{ ft}$$

$$L = \frac{Q}{\frac{2}{3}\sqrt{2g}\,h^{3/2}} = \frac{100}{\frac{2}{3}\sqrt{2\times 32.2}\times 1.5^{1.5}} = 10.2'$$

SUBJECT KEYSTONE WEST VALLEY
PHASE II PERMITTING

BY KMB DATE 6/19/96 PROJ. NO. 92-220-73 07

CHKD. BY MRL DATE 7/7/96 SHEET NO. 9 OF 14

■ REVISED BY SER 1/13/98 / KMB 1/19/98

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EQUALIZATION POND - OUTLET STRUCTURE CONTINUED

THE VALUE OF $L = 10.2$ CAN BE SEEN AS A TOTAL LENGTH FOR ALL THREE OPENINGS

$$\text{REQ'D OPENING WIDTH} = \frac{10.2}{3} = 3.4 \text{ ft}$$

ALSO, CHECK THE STANDARD WEIR AND ORIFICE EQUATIONS.

WEIR EQUATION : $Q = CLH^{3/2}$

USE $C = 3.0$ (FROM BRATER & KING pg 5-40, C=3.0 FOR H=1.5' & BREADTH = 1.5')

■ ASSUME $H = 1.5$ ft

$$Q = 100 = 3.0\, L\, 1.5^{1.5} \Rightarrow L = \frac{100}{3.0 \times 1.5^{1.5}} = 18.1 \text{ ft}$$

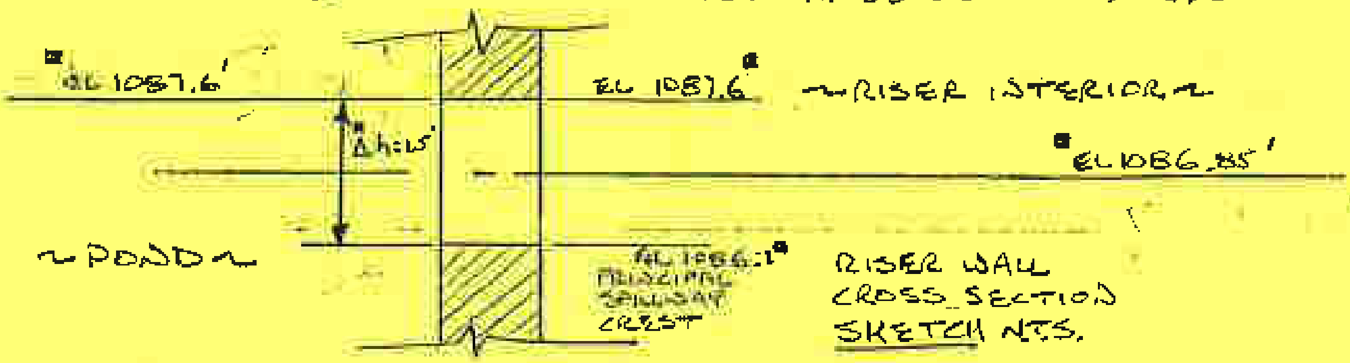
DIVIDED BY THREE SIDES, REQ'D OPENING WIDTH = 6.0 ft

ORIFICE EQUATION : $Q = CA\sqrt{2gh}$ $C = 0.6$, $A = 1.5' \times L$

■ h = HEAD ON ORIFICE CENTER = 0.75'

$$100 = 0.6 \times (1.5 \times L)\sqrt{2 \times 32.2 \times 0.75} = 6.25L$$

■ ∴ $L = 16.0$ FT OVER THREE SIDES → 5.3 FT



RISER WALL CROSS SECTION SKETCH NTS.

SUBJECT KEYSTONE WEST VALLEY
PHASE II PERMITTING

BY KMB DATE 6/19/96 PROJ. NO. 92-220-73-07

CHKD. BY MRL DATE 7/3/96 SHEET NO. 5 OF 14

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*REVISED BY SER 9/30/96 ■ REVISED BY SER 1/13/98 VKMB 1/19/98

EQUALIZATION POND - OUTLET STRUCTURE CONTINUED

OF THE THREE METHODS CONSIDERED, THE WEIR FLOW CONDITION PROVED TO BE THE MOST RESTRICTIVE. SIZE THE RISER OPENINGS BASED ON THIS CASE.

RISER OPENINGS ARE 6" WIDE BY 1.5" HIGH

THERE WILL BE THREE OPENINGS WITH INVERTS AT ELEVATION * ~~1084.2'~~

■ 1086.1'

SUBJECT Keystone West Valley
Phase II Permitting

BY MRL DATE 7/17/96 PROJ. NO. 92-220-73-07

CHKD. BY SER DATE 7/22/96 SHEET NO. 6 OF 14

▼ REVISED BY SER 9/30/96
■ REVISED BY SER 1/17/98 /KMB 1/22/98

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Equalization Pond – Outlet Structure (continued)

Principal Spillway Barrel :

~~First determine the highest barrel invert elevation for a 36" Φ HDPE (Spirolite) pipe (I.D. = 36") which can pass the 25-year, 24-hour peak discharge (≈ 100 cfs) at the interior riser headwater elevation of 1004.6.~~

▼ USE BARREL INLET ELEVATION ■1075.90 ~~1073.50~~ FT.

▼ USE MAXIMUM INTERIOR RISER WATER SURFACE ELEVATION ■1086.1 ~~1084.3~~ FEET

Inlet Control

For $Q = 100$ CFS
$D = 36''$

Using Chart No. 1, sheet 8 ...

$$\frac{HW}{D} = 3.4$$

$$HW = 3.4(D) = 3.4(3') = 10.2' \text{ minimum}$$

→ Set barrel inlet invert elevation = ■1086.1 − 10.2 = 1075.9

Outlet Control

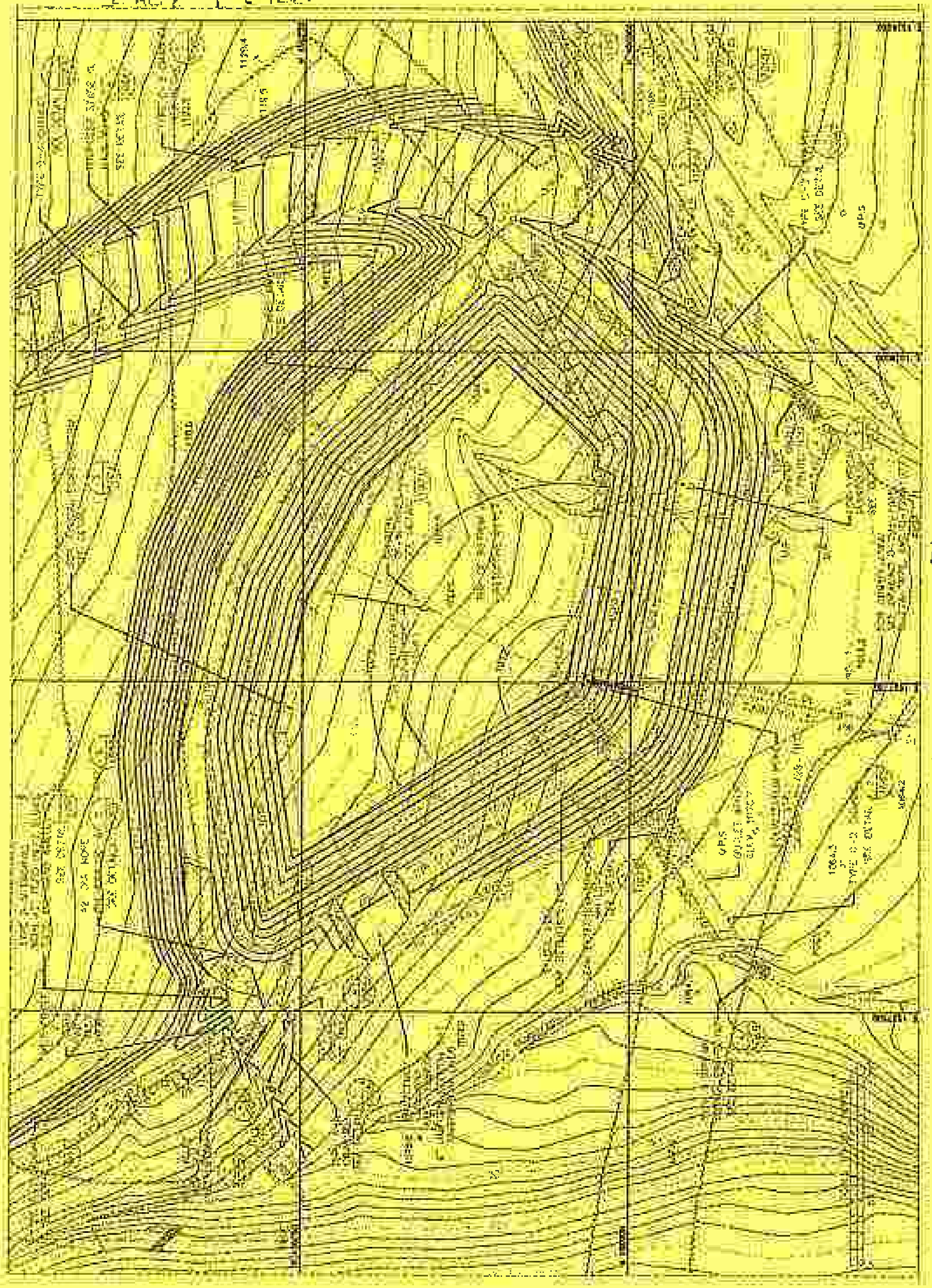
See pipe layout on plan view, sheet 7

■ ~~Use slope = 1%~~, length = ~~180'~~ ■100' ~~130'~~

Outlet invert elevation = ■1073.5' ~~1072.5'~~ –

KEYSTONE WEST VALLEY EQUALIZATION POND 92-220-75-037 7/14

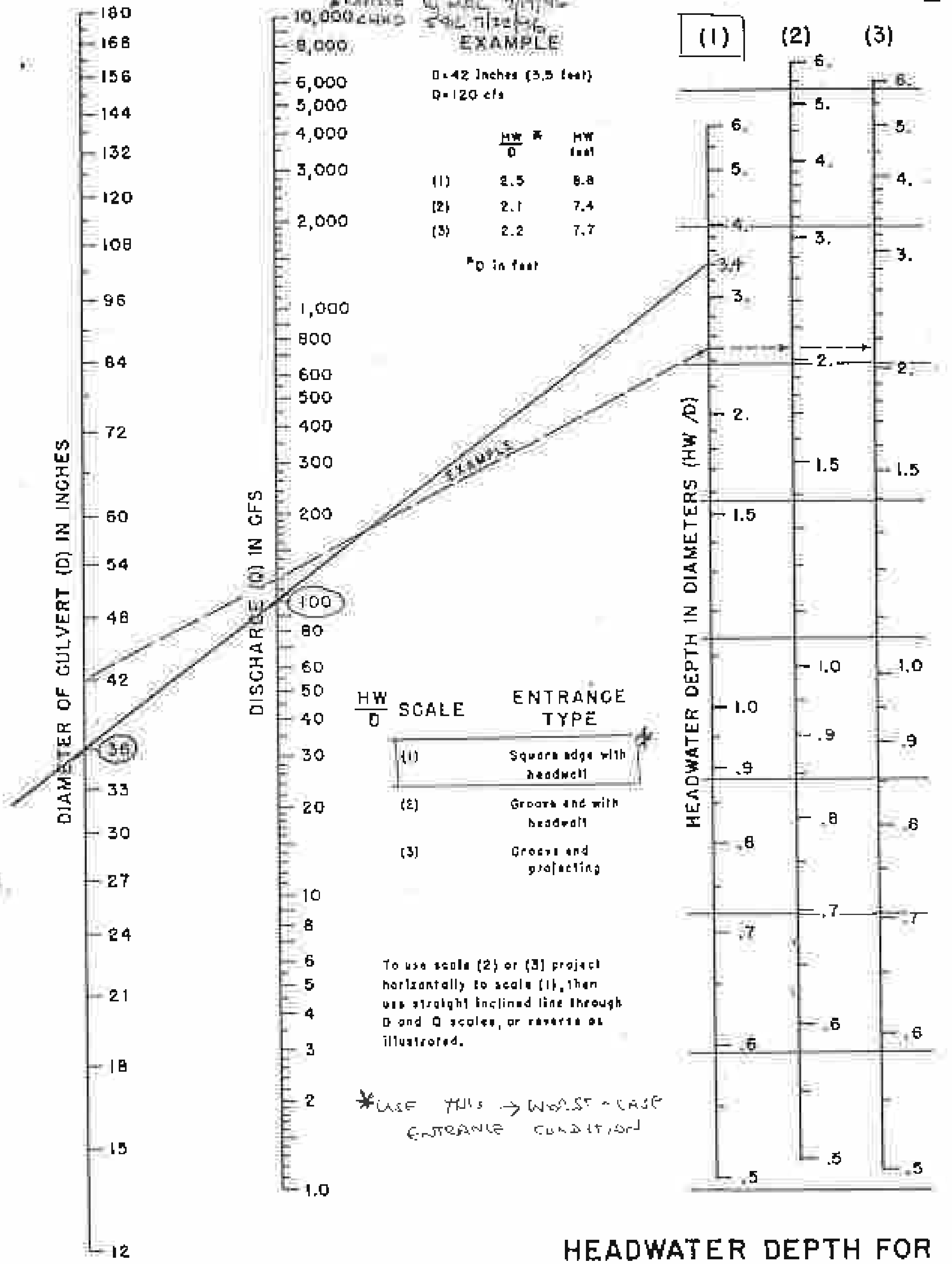
SCALE 1"=100'



SEE DRAWING D-725-1071

PENELEC KEYSTONE WEST VALLEY
PHASE II PERMITTING
32-220-93-80
KMS 6/11/96 ✓MRL 7/3/96
Adjusted by MRL 7/17/96
CHKD SAL 7/22/96

CHART 10 8/14



HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL

HEADWATER SCALES 2&3
REVISED MAY 1964

BUREAU OF PUBLIC ROADS JAN. 1963

FROM "HYDRAULIC DESIGN OF HIGHWAY CULVERTS"
HDS-5, FHWA-IP-85-15

SUBJECT KEYSTONE WEST VALLEY
PHASE II PERMITTING

BY KMB DATE 6/6/96 PROJ. NO. 92-220-73-01

CHKD. BY MRL DATE 7/3/96 SHEET NO. 9 OF 14

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▲ Revision by MRL 7/17/96
CHKD SSR 7/22/96
▼ REVISED BY SSR 9/30/96
■ REVISED BY SSR 1/13/98 ✓KMB 1/27/98

DISCHARGE PIPE CONTINUED

BARREL FLOW — OUTLET CONTROL

CHECK THE CAPACITY OF THE BARREL AT DESIGN HEAD IN THE RISER.

PIPE MATERIAL = HDPE — USE $n = 0.011$

▲ LENGTH = ~~180~~ ~~130'~~ ■100'

▼ LIMITING HEADWATER ELEV. = ▼~~1084.5'~~ ■1086.1'

▲ OUTLET INVERT = EL. ~~1070.7~~ ~~1071.2'~~ ■1073.5'

CHECK A 3' DIAMETER PIPE. USE A CRITICAL FLOW DEPTH $d_c = 3' \Rightarrow d_c = D$

FROM HDS-5, THE HEAD LOSS EQUATION FOR A CULVERT IS

$$H = \left[1 + K_e + K_b + (29 n^2 L)/R^{1.33}\right] V^2/2g$$

USE $K_e = 0.5$ (ENTRANCE CONDITION "PROJECTING, SQUARE CUT" FOR CONCRETE)

$K_b = 1.0$ (ASSUMED LOSS COEFFICIENT FOR BENDS)

R = HYDRAULIC RADIUS = $D/4$ FOR FULL FLOW = 0.75'

$$V = Q/A = Q/\pi D^2/4 = 0.141\,Q$$

$$H = \left[1 + 0.5 + 1.0 + \frac{(29 \times 0.011^2 \times 100)}{0.75^{1.33}}\right]\left[\frac{0.0199\,Q^2}{2 \times 32.2}\right]$$

■ $= [3.01][0.00031\,Q^2] = 0.000931\,Q^2$

■ @ Q = 100 CFS, $H = 0.000931(100^2) = \cancel{10.6'}$ 9.3'

■ HW = ~~1070.7'~~ 1073.5' + ~~10.6'~~ 9.3' + 3' = ~~1084.3'~~ 1085.8'

■ ▲▼ ~~1084.3'~~ 1085.8' < 1086.1 ~~< 1084.3'~~ ∴ O.K ∴ INLET CONDITIONS CONTROL

TABLE 12 - ENTRANCE LOSS COEFFICIENTS

Outlet Control, Full or Partly Full Entrance head loss

$$H_e = k_e \left(\frac{V^2}{2g} \right)$$

Type of Structure and Design of Entrance	Coefficient k_e
Pipe, Concrete	
Projecting from fill, socket end (groove-end)	0.2
Projecting from fill, sq. cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square-edge	0.5
Rounded (radius = 1/12D)	0.2
Mitered to conform to fill slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side-or slope-tapered inlet	0.2
Pipe, or Pipe-Arch, Corrugated Metal	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side-or slope-tapered inlet	0.2
Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of 1/12 barrel dimension, or beveled edges on 3 sides	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of 1/12 barrel dimension, or beveled top edge	0.2
Wingwall at 10° to 25° to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side-or slope-tapered inlet	0.2

*Note: "End Section conforming to fill slope," made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections, incorporating a closed taper in their design have a superior hydraulic performance. These latter sections can be

FROM "HYDRAULIC DESIGN OF HIGHWAY CULVERTS"

PROJECT : ______________________

STATION : ______________________

SHEET ______ OF ______

CULVERT DESIGN FORM

DESIGNER / DATE : ____________ / ________

REVIEWER / DATE : ____________ / ________

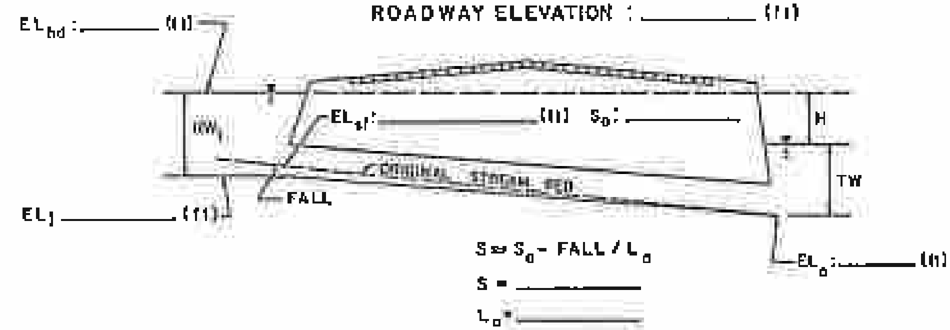
HYDROLOGICAL DATA

SEE ADD'L SHTS.

- ☐ METHOD: ______________________
- ☐ DRAINAGE AREA: ________ ☐ STREAM SLOPE: ________
- ☐ CHANNEL SHAPE: ______________________
- ☐ ROUTING: ________ ☐ OTHER: ________

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW(cfs)	TW (ft)



$S \approx S_o - FALL / L_o$

S = ________

L_o = ________

CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE	TOTAL FLOW Q (cfs)	FLOW PER BARREL Q/N (1)	HEADWATER CALCULATIONS: INLET CONTROL HW_i/D (2)	HW_i	FALL (3)	EL_{hi} (4)	OUTLET CONTROL TW (5)	d_c	$\frac{d_c+D}{2}$	h_o (6)	k_e	H (7)	EL_{ho} (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

(2) $HW_i / D = HW / D$ OR HW_i / D FROM DESIGN CHARTS

(3) $FALL = HW_i - (EL_{hd} - EL_{sf})$; FALL IS ZERO FOR CULVERTS ON GRADE

(4) $EL_{hi} = HW_i + EL_i$ (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL.

(6) $h_o = TW$ or $(d_c + D/2)$ (WHICHEVER IS GREATER)

(7) $H = \left[1 + k_e + (29 n^2 L) / R^{1.33}\right] V^2 / 2g$

(8) $EL_{ho} = EL_o + H + h_o$

SUBSCRIPT DEFINITIONS :

a. APPROXIMATE
f. CULVERT FACE
hd. DESIGN HEADWATER
hi. HEADWATER IN INLET CONTROL
ho. HEADWATER IN OUTLET CONTROL
i. INLET CONTROL SECTION
o. OUTLET
sf. STREAMBED AT CULVERT FACE
tw. TAILWATER

COMMENTS / DISCUSSION :

CULVERT BARREL SELECTED :

SIZE : ______________

SHAPE : ______________

MATERIAL : ______________ n ______

ENTRANCE : ______________

FROM "HYDRAULIC DESIGN OF HIGHWAY CULVERTS"

11/14

SUBJECT Keystone West Valley
Phase II Permitting

BY MRL DATE 7/19/96 PROJ. NO. 92-220-73-07

CHKD. BY SER DATE 7/22/96 SHEET NO. 13 OF 14

* REVISED BY SER 1/13/97

SHEET 12 HAS BEEN OMITTED

GAI CONSULTANTS, INC.
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Environmental Specialists

STRENGTH CALCULATIONS

OBJECTIVE : A 36" ϕ HDPE Spirolite pipe is to be used for the riser principal spillway pipe. Determine acceptable class of pipe.

- Depth of cover * < 18 FEET
- Because the pond is lined, assume water table is at or below pipe.
- E' = soil modulus = 2000 psi minimum
  (Specs. are written to achieve this modulus)

Using attached Table No. 1, Class 40 pipe is acceptable.

USE CLASS 40 SPIROLITE PIPE

PIPE SELECTION

Spirolite pipe is manufactured in four standard ring stiffness classes. In preparing a specification, the designer selects a class of pipe appropriate for the application. The following tables may be used to assist the designer in making that selection. It is important that the designer perform all necessary calculations to verify the adequacy of a given class of pipe and be acquainted with all assumptions and installation requirements. Other design methods may be applicable.

The design of HDPE pipe for subsurface applications is typically based on the following performance limits: (1) wall crush strength, (2) constrained buckling resistance, and (3) deflection. Equations for these performance limits are given in the Appendix and were used to produce Table 1 and Table 2. The suitability of a class of pipe for installation at a given depth depends on the installation achieving the design E' and on the pipe being installed in accordance with ASTM D-2321 and the Spirolite Installation Guide. The designer is advised to review the applicability of these equations to each use of Spirolite.

The classes and depths shown in the tables are based on a design soil weight (dry or saturated) of 120 lbs/ft³ and an applied H-20 live load. (Where live load is present, Spirolite pipe normally requires a minimum depth of cover of one pipe diameter or three feet whichever is greater. Where this condition cannot be met, please consult Plexco/Spirolite.) The earth load for calculating crush resistance was found using the arching coefficients given in Figure 10. The prism load was used for buckling and deflection calculations. Deflection was calculated using 75% of the E' value given at the top of the respective column, a deflection lag factor of 1.5, and a deflection limit of 5 percent. Buckling was calculated using the E' value listed and a long-term pipe modulus value of 28,250 psi. Buckling resistance was considered only for pipe subjected to ground water, as buckling is normally not a controlling factor for dry ground installations in the range of depths given in the tables. A safety factor of two was applied to the crush and buckling values.

BURIAL ABOVE GROUND WATER LEVEL

Table 1 is based on calculations made assuming the ground water level is always below pipe grade elevation. For other sizes, and burial depths or conditions not listed, consult with Plexco/Spirolite.

Table 1: SPIROLITE PIPE CLASS SELECTION FOR BURIAL *ABOVE* THE GROUND WATER LEVEL

[illegible]	[illegible]			[illegible]			[illegible]			[illegible]			[illegible]			[illegible]			[illegible]			[illegible]		
E'	1000	2000	3000	1000	2000	3000	1000	2000	3000	1000	2000	3000	1000	2000	3000	1000	2000	3000	1000	2000	3000	1000	2000	3000
Depth of Cover (ft.) 2	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40
4	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40
6	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40
8	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40
10	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40
12	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40
14	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40
16	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40
18	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40	40
20	63	40	40	63	40	40	63	40	40	100	40	40	100	40	40	100	63	63	100	63	63	160	63	63
22	160	40	40	160	40	40	160	40	40		40	40		40	40		63	63		63	63		63	63
24		40	40		40	40		40	40		63	63		63	63		63	63		100	100		100	100
26		40	40		40	40		63	63		63	63		100	100		100	100		100	100		100	100
28		40	40		40	40		63	63		63	63		100	100		100	100		100	100		160	160
30		40	40		63	63		100	100		100	100		100	100		100	100		100	100		160	160
32		40	40		100	100		100	100		160	160		160	160		160	160		160	160		160	160
34		40	40		100	100		100	100		160	160		160	160		160	160		160	160		160	160
36		40	40		100	100		100	100		160	160		160	160		160	160		160	160		160	160
38		100	100		100	100		100	100		160	160		160	160									

Note: See text regarding live load.

REFERENCE: Spirolite Bulletin No. 910

SUBJECT KEYSTONE
PHASE II PERMITTING

BY SER DATE 7/10/96 PROJ. NO. 92-220-73-7

CHKD. BY MRL DATE 7/10/96 SHEET NO. 1 OF 6

CONSULTANTS, INC.
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WEST VALLEY EQUALIZATION POND
IWT PIPE

DESIGN THE IWT PIPE FROM THE WEST VALLEY EQUALIZATION POND'S RISER TO THE PLANT'S IWT FOR STRENGTH.

THE PIPE WILL BE A 12"ϕ HDPE PIPE.

ESTIMATE THE SDR REQUIRED.

THE WORST CASE LOADING WILL BE BELOW THE EQ. POND CREST. THE PIPE WILL BE CONCRETE ENCASED BELOW ALL ROADS (AS PER JMJ)

SEE SHEET 2 FOR COMPUTER OUTPUT FROM DRISCOPIPE PROGRAM

CONCLUSION: USE SDR 21 - 12"ϕ HDPE PIPE

SHEET 2/6

DRISCOPIPE DIRECT BURIAL - 07-10-1996

PROJECT: WEST VALLEY EQUAL. POND INT PIPE

WORST CASE CONDITION

REMARKS:

[Calcu]lated By SIER 7/10/96 CHKD BY: MRL 7/17/96

DRISCOPIPE 1000 Product Series

Dimension Ratio	(DR) =	32.50	
Burial Depth	=	21 Feet	12"φ PIPE INVERT AT ELEV 1068' POND CREST ELEV = 1090' BURIAL DEPTH = 1090' - 1068' - 1'
Soil Density	=	120 Pounds/Cu Ft	ASSUMED, ENGINEERING JUDGEMENT
Water Table	=	0 Feet Above Pipe	ASSUMED (WORST CASE)
Other Loads	=	144 Pounds/Sq Ft	H-20 LIVE LOAD = 1#/SQ.IN. FROM FIGURE 2, SHEET 5
Soil Modulus	=	2000 psi	SEE FIGURE 1, SHEET 5 SPECS WILL BE MADE TO MATCH CONDITIONS FOR THIS SOIL MODULUS
Pipe Modulus	=	35000 psi	
S(A) (Stress in Pipe Wall)	=	291.4 psi	
P(T) (Pressure @ Pipe Crown)	=	18.5 psi	
P(CB)(Critical Buckling Pressure)	=	55.0 psi	
Maximum Ring Deflection	=	8.13 %	
CRUSHING SAFETY FACTOR	=	5.1 to 1	
WALL BUCKLING SAFETY FACTOR	=	3.0 to 1	
CALCULATED RING DEFLECTION	=	0.93 %	

CALCULATED RING DEFLECTION IS ACCEPTABLE.

WARNING!

THE USE OF THIS PROGRAM TO DESIGN POLYETHYLENE PIPING SYSTEMS USING PRODUCTS NOT MANUFACTURED BY PHILLIPS DRISCOPIPE MAY RESULT IN SERIOUS DESIGN ERRORS.

USE SDR 21 WHICH WILL BE STRONGER THAN THE (S)DR 32.50 SHOWN ABOVE

A SDR 21 12"φ PIPE WILL HAVE AN ID = 11.536"
SAY ID = 11.5"

10" (10.750 OD)

SDR 7	267 psi	19.32 lbs./ft.	7.678 ID	1.536 wall
SDR 9	200 psi	15.61	8.362	1.194
SDR 11 ●	**160 psi**	**13.09**	**8.796**	**.977**
SDR 13.5	128 psi	10.87	9.158	.796
SDR 15.5	110 psi	9.58	9.362	.694
SDR 17 ●	**100 psi**	**8.78**	**9.486**	**.632**
SDR 19	89 psi	7.92	9.618	.566
SDR 21 ●	**80 psi**	**7.21**	**9.726**	**.512**
SDR 26 ●	**64 psi**	**5.87**	**9.924**	**.413**
SDR 32.5 ●	**51 psi**	**4.75**	**10.088**	**.331**

12" (12.750 OD)

SDR 7	267 psi	27.16 lbs./ft.	9.108 ID	1.821 wall
SDR 9	200 psi	21.97	9.916	1.417
SDR 11 ●	**160 psi**	**18.41**	**10.432**	**1.159**
SDR 13.5	128 psi	15.29	10.862	.944
SDR 15.5 ●	**110 psi**	**13.48**	**11.104**	**.823**
SDR 17 ●	**100 psi**	**12.36**	**11.250**	**.750**
SDR 19	89 psi	11.14	11.408	.671
SDR 21 ●	**80 psi**	**10.13**	**11.536**	**.607**
SDR 26 ●	**64 psi**	**8.26**	**11.770**	**.490**
SDR 32.5 ●	**51 psi**	**6.67**	**11.966**	**.392**

13" (13.386 OD)

SDR 7	267 psi	29.24 lbs./ft.	9.562 ID	1.912 wall
SDR 9	200 psi	23.62	10.412	1.487
SDR 11	160 psi	20.30	10.952	1.217
SDR 13.5	128 psi	16.87	11.402	.992
SDR 15.5	110 psi	14.85	11.658	.864
SDR 17	100 psi	13.62	11.812	.787
SDR 19	89 psi	12.28	11.976	.705
SDR 21	80 psi	11.16	12.112	.637
SDR 26	64 psi	9.12	12.356	.515
SDR 32.5	51 psi	7.36	12.562	.412

14.000 OD

SDR 7	267 psi	32.76 lbs./ft.	10.00 ID	2.000 wall
SDR 9	200 psi	26.50	10.888	1.556
SDR 11 ●	**160 psi**	**22.20**	**11.454**	**1.273**
SDR 13.5	128 psi	18.44	11.926	1.037
SDR 15.5	110 psi	16.24	12.194	.903
SDR 17 ●	**100 psi**	**14.91**	**12.352**	**.824**
SDR 19	89 psi	13.43	12.526	.737
SDR 21	80 psi	12.22	12.666	.667
SDR 26 ●	**64 psi**	**9.96**	**12.924**	**.538**
SDR 32.5	51 psi	8.05	13.138	.431

16.000 OD

SDR 9	200 psi	34.60 lbs./ft.	12.444 ID	1.778 wall
SDR 11 ●	**160 psi**	**29.00**	**13.090**	**1.455**
SDR 13.5	129 psi	24.09	13.630	1.185
SDR 15.5	110 psi	21.21	13.936	1.032
SDR 17 ●	**100 psi**	**19.46**	**14.118**	**.941**
SDR 19	89 psi	17.54	14.316	.842
SDR 21 ●	**80 psi**	**15.96**	**14.476**	**.762**
SDR 26 ●	**64 psi**	**13.01**	**14.770**	**.615**
SDR 32.5	51 psi	10.50	15.016	.492

18.000 OD

SDR 9	200 psi	43.79 lbs./ft.	14.000 ID	2.000 wall
SDR 11 ●	**160 psi**	**36.69**	**14.728**	**1.636**
SDR 13.5	128 psi	30.48	15.334	1.333
SDR 15.5 ●	**110 psi**	**26.84**	**15.678**	**1.161**
SDR 17 ●	**100 psi**	**24.64**	**15.882**	**1.059**
SDR 19	89 psi	22.19	16.106	.947
SDR 21	80 psi	20.19	16.286	.857
SDR 26 ●	**64 psi**	**16.47**	**16.616**	**.692**
SDR 32.5	51 psi	13.30	16.892	.554

20.000 OD

SDR 9	200 psi	54.05 lbs./ft.	15.556 ID	2.222 wall
SDR 11 ●	**160 psi**	**45.30**	**16.364**	**1.818**
SDR 13.5	128 psi	37.63	17.038	1.481
SDR 15.5	110 psi	33.14	17.420	1.290
SDR 17 ●	**100 psi**	**30.41**	**17.648**	**1.176**
SDR 19	89 psi	27.42	17.894	1.053
SDR 21	80 psi	24.93	18.096	.952
SDR 26 ●	**64 psi**	**20.34**	**18.462**	**.769**
SDR 32.5 ●	**51 psi**	**16.41**	**18.770**	**.615**

21.500 OD

SDR 9	200 psi	62.47 lbs./ft.	16.722 ID	2.389 wall
SDR 11	160 psi	52.37	17.590	1.955
SDR 13.5	128 psi	43.51	18.314	1.593
SDR 15.5	110 psi	38.30	18.726	1.387
SDR 17	100 psi	35.16	18.970	1.265
SDR 19	89 psi	31.68	19.236	1.132
SDR 21	80 psi	28.82	19.452	1.024
SDR 26	64 psi	23.51	19.846	.827
SDR 32.5	51 psi	18.98	20.176	.662

● denotes standard sizes

SHEET 4/6

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Application Note No. 1

Pipe Behavior Under Earth Loading

Polyethylene pipe is flexible conduit - it can deform without cracking or failing to the extent that the soil surrounding the pipe will provide support against further deformation. With rigid pipes, the predominant source of support must come from the pipe itself. The strength and stability of flexible pipe/soil systems has been well documented by extensive experience and laboratory testing, not just with polyethylene but also with other equally flexible materials.

Because of the interaction of flexible pipe with the surrounding soil, the nature of the embedment materials and the quality of their placement are important to the development of a satisfactory pipe/soil system. During this development, some pipe deflection is a natural and essential response that produces balanced soil support through the entire pipe circumference. However, to safeguard the performance capabilities of the pipe, it is necessary to conduct the installation so that the initial and ultimate deflections will not produce excessive wall stressing (or straining) that results in loss of volumetric flow capacity, endangers structural stability, or affects joint performance.

Designers should also be sure that the pipe, as installed, has a suitable margin of safety against buckling (see discussion of buckling behavior for constrained pipe in PLEXCO Application Note No. 2) and excessive loading (the safe stress under continuous compression may conservatively be assumed to equal the hydrodynamic design stress for the end use conditions). However, for pipes that are installed per recommended practices, it is rare that either of these two criteria will control design. Maximum permissible deflection will generally be the only criterion. Consequently, the key objective and primary consideration in the selection and installation buried polyethylene pipe is deflection control. Such deflection control is not unduly demanding nor difficult to attain.

The toughness and flexibility of polyethylene piping make it ideal for underground construction. It can easily follow a tortuous course with minimal need of fittings for changes in direction. Its ability to undergo deflection without material damage permits it to shed off earth loading and superimposed loadings that would damage rigid, brittle pipes.

A recent Plastic Pipe Institute Technical Report, TR-31, on "Underground Installation of Polyolefin Piping" provides general information and guidance on the underground installation of polyethylene piping for both pressure and non-pressure applications. Additionally, for pressure pipe installations, the recommendations outlined in ASTM D2774, "Underground Installation of Thermoplastic Pressure Piping," should be followed. For non-pressure pipe burial, the recommendations of ASTM D2321, "Underground Installation of Flexible Thermoplastic Sewer Pipe," should be followed so as to ensure proper development of the soil support and thereby prevent excessive pipe deformation. Proper installation, under these or equivalent conditions approved by the design engineer, presents no particular change or significant variation from techniques mandated by traditional materials. Properly installed, polyethylene pipe can be buried under considerable earth cover and/or traffic loading as shown on the next page.

Estimating Loads on Pipe

The total load impressed upon a buried pipe is the sum of the embedment load plus superimposed loads. Embedment loads per lineal dimension of pipe can be estimated by knowing the type of backfill, the trench dimensions, and the pipe diameter. For non-cohesive, granular materials the load may be reduced by 10%. The load may be increased by 30% for dry clay and by 40% for wet clay.

For pipe buried below a water table in soil, the actual load on the pipe can be reduced because of the buoyant effect of the water. However, whatever reduction exists should not be considered in pipeline design because the height of the water table could drop below the pipe level. Even if the height does not vary, a more conservative design will result from assuming the higher load on the pipe.

Superimposed loads, such as those due to vehicle traffic must also be considered in estimating the total load upon a buried pipe. Standard loads have been calculated for the effect of highway (see Figure 2) and railroad traffic (see Figure 3). These can be obtained and added to the earthload to arrive at the total load, when necessary.

When using either the H-20 or E-80 live loading charts, simply check the depth of cover and then determine the soil pressure. Multiply the soil pressure by the O.D. of the pipe in order to obtain the weight per inch, which can then be added to the weight in the deflection equations. At cover depths greater than listed on the charts, the effect of the live load becomes negligible.

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Figure 1
Bureau of Reclamation Values of E′ for Iowa Formula
(For Initial Flexible Pipe Deflection)

	E′ for degree of compaction of bedding (lb/in^2) [5/]			
Soil type-pipe bedding material (Unified Classification System[2/])	Dumped	Slight <85% Proctor <40% rel. den.	Moderate 85-95% Proctor 40-70% rel. den.	High >95% Proctor >70% rel. den.
Fine-grained Soils (LL>50) [3/] Soils with medium to high plasticity CH, MH, CH-MH	No data available; consult a competent Soils Engineer; Otherwise use E′ = 0			
Fine-grained Soils (LL<50) Soils with medium to no plasticity CL, ML, ML-CL, with less than 25% coarse-grained particles	50	200	400	1,000
Fine-grained Soils (LL<50) Soils with medium to no plasticity CL, ML, ML-CL, with more than 25% coarse-grained particles Coarse-grained Soils with Fines GM, GC, SM, SC[4/] contains more than 12% fines	100	400	1,000	2,000
Coarse-grained Soils with Little or No Fines GW, GP, SW, SP[4/] contains less than 12% fines	200	1,000	2,000	3,000
Crushed Rock	1,000	3,000		

[2/] ASTM Designation D2487, USBR Designation E-3.

[3/] LL = Liquid limit.

[4/] Or any borderline soil beginning with one of these symbols (i.e., GM-GC, GC-SC.).

[5/] 1 lb/in^2 = 0.07 kg/cm^2.

FIGURE 2
Variation In Soil Stress With Cover Depth Below H-20 Wheel Load

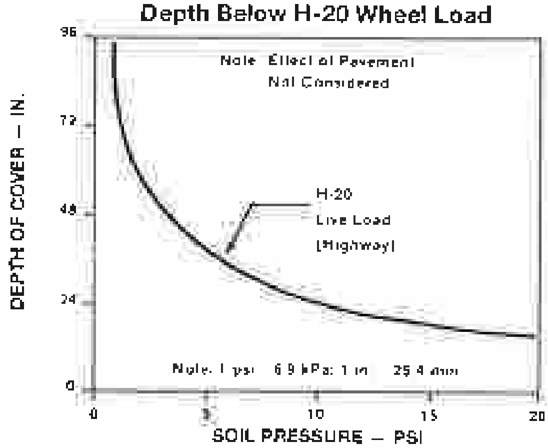
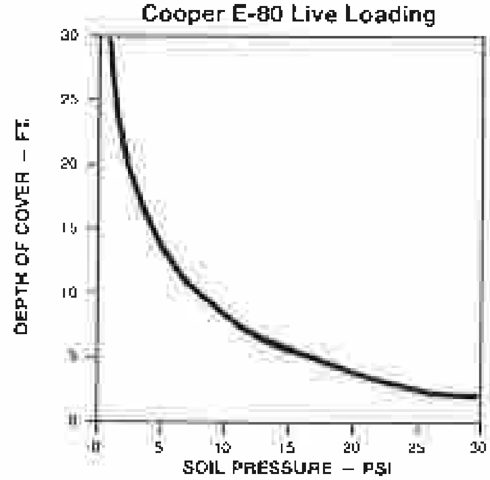


FIGURE 3
Cooper E-80 Live Loading



SHEET 6/6

ESTIMATED DEFLECTION

$$\% \text{ deflection} = \frac{\text{Calculated deflection} \times 100}{\text{Outside diameter of pipe}}$$

x = calculated deflection

$$X = \frac{K \times D \times W}{(0.149 \times PS + 0.061 \times E')}$$

$$W = \frac{DC \times OD \times SD}{144} + (\text{Soil Pressure}^{//} \times O.D.)$$

$$PS = \frac{E \times I}{0.149 \times R^3}$$

Where	OD	=	Outside diameter of pipe (inch)
	K	=	Bedding Factor = 0.1
	D	=	Deflection Lag Factor = 1.5
	W	=	Weight per lineal inch (#/inch)
	PS	=	Pipe Stiffness (PSI)
	E'	=	Soil Modulus (see figure 1)
	E	=	Flexural Modulus (PSI) (133,000 psi for PE3408) (100,000 psi for PE2406)
	DC	=	Depth of cover (feet)
	SD	=	Soil Density ($\#/ft^3$)
	SDR	=	Standard Dimensional Ratio
	t	=	Average Wall Thickness (inch))

$$t = \frac{OD \times 1.06}{SDR}$$

I = Moment of Inertia = $t^3/12$

R = Mean Radii of the pipe (inch)

$$R = \frac{(OD - t)}{2}$$

// Soil Pressure due to live loading. See Figures 2 and 3.

Safe design limits for the allowable deflection of polyethylene pipe of different dimension ratios have been determined[1] and are given below:

Dimension Ratio (SDR)	Safe Deflection as % of Diameter
32.5	8.5
26.0	7.0
21.0	6.0
17.0	5.0
11.0	3.0
9.0	2.5

EXAMPLE

As an example, assume a PLEXCO 12" SDR 11 PE3408 polyethylene pipe is to be buried 25 feet in the ground. This pipe is to be buried in a coarse grained soil with little or no fines and compacted to a proctor density of 90%. From Figure 1 a value of 2000 psi is obtained for E1. Soil density is assumed to be $120\#/ft^3$.

OD = 12.75 inches

t = (12.75 x 1.06)/11 = 1.229

I = $(1.229)^3/12 = 0.1547$

R = (12.75 - 1.229)/2 = 5.761

PS = $(133{,}000 \times 0.1547)/(0.149 \times 5.761^3)$ = 722.4

W = (25 x 12.75 x 120)/144 = 265.6

$$X = \frac{1.5 \times 0.1 \times 265.6}{(0.149 \times 722.4) + (0.061 \times 2000)}$$

X = 0.173 inch deflection

$$\% \text{ deflection} = \frac{0.173 \times 100}{12.75} = 1.36\%$$

Since 1.36% is less than the allowable 3.0%, this would be an adequate burial situation.

The method presented for calculating deflection represents one of many methods and should be adequate in most cases. However, when special installation conditions exist, other methods of calculating deflection may need to be used. The final design is left to the discretion of the responsible engineer.

CONSULTANTS, INC.
Engineers Geologists Planners
Environmental Specialists

Equalization Pond Emergency Spillway

Purpose: Design the Equalization Pond Emergency Spillway.

Methodology: "Earth Spillways", TR-2, US Soil Conservation Service, October 1, 1956.

Design Flow: Design for the 25-year, 24-hour peak flow of 100 cfs, reference "Dirty Water and Related Facilities" calc by SER 5/24/96.

Reference sheet 2 for plan view of the proposed emergency spillway.

Exit Channel

The exit of the proposed spillway is proposed to be to natural ground. This is considered acceptable since the principal spillway will be sized to pass the 25-year, 24-hour peak flow and flow over the emergency spillway will occur only in the most extreme, emergency circumstances.

Analyze conditions at the exit of the control section. The proposed inlet channel section is a 40 foot wide, concrete lined (with concrete filled geoweb or uniform section mat concrete revetment) with 5:1 side slopes and a 2 foot depth. This cross section will exit at an angle to the natural ground and the effective bottom width is 42 feet. Use the channel section bottom width of

$b := 40 \cdot ft$ to be conservative

Use the side slopes of the inlet channel section to analyze conditions in the area downstream of the control section. Actual condition is flow over an infinitely wide slope.

$z := 5$

Find critical depth for this cross section.

Critical depth occurs when the Froude number, F = 1

Flow $Q := 100 \cdot \frac{ft^3}{sec}$

Area $a(d_c) := (b + z \cdot d_c) \cdot d_c$

Velocity $v(d_c) := \frac{Q}{(b + z \cdot d_c) \cdot d_c}$

Celerity $c(d_m) := \sqrt{g \cdot d_m}$

Mean Depth $d_m(d_c) := \frac{(b - z \cdot d_c) \cdot d_c}{b - 2 \cdot z \cdot d_c}$ Area divided by top width

Define a function f(d) as velocity - celerity and find its root

$$f(d_c) := \frac{Q}{(b + z \cdot d_c) \cdot d_c} - \sqrt{g \cdot \frac{(b + z \cdot d_c) \cdot d_c}{b + 2 \cdot z \cdot d_c}}$$

Trial depth $d_c := 0.5 \cdot ft$

$solution := root(f(d_c), d_c)$

$d_c := solution$

$d_c = 0.565 \cdot ft$

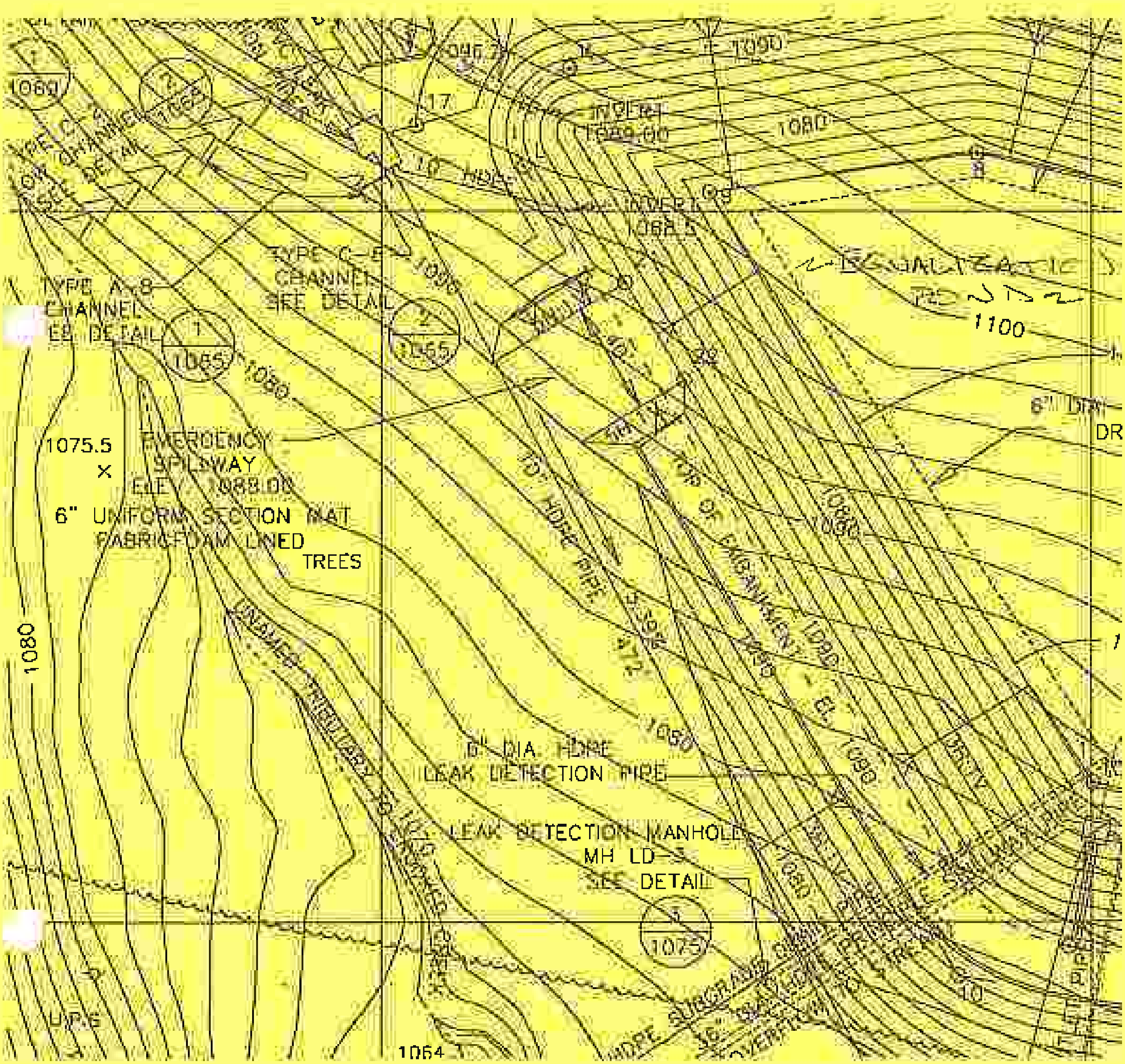
SHEET 3/4

PROJ 92-220-73-7

KEYSTONE WEST VALLEY
EQUALIZATION POND

N

SCALE 1" = 50'



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Proof

$$v(d_c) = 4.131 \cdot ft \cdot sec^{-1}$$

$$d_m(d_c) = 0.53 \cdot ft$$

$$\sqrt{g \cdot d_m(d_c)} = 4.131 \cdot ft \cdot sec^{-1}$$

$$F := \frac{v(d_c)}{\sqrt{g \cdot d_m(d_c)}} \qquad F = 1$$

Therefore $d_c = 0.565 \cdot ft$ is the critical depth at the control section. The actual critical depth will be slightly lower because the actual conditions at the control section are an infinitely wide channel.

Flow downstream of the control section must be supercritical. This allows the designer to assume critical flow at the Control Section. Use the inlet channel section conditions with the natural ground slope. Natural ground slope is

$$S := \frac{2 \cdot ft}{14 \cdot ft} \qquad S = 0.143 \cdot \frac{ft}{ft}$$

Find flow depth on natural slope.

Manning's $n := 0.045$ for grass.

Define function $F(d_d)$ as capacity (by manning's equation) minus the design flow (Q).

$$F(d_d) := \frac{1.49 \cdot \frac{ft^{\frac{1}{3}}}{sec}}{n} \cdot \left[(b + z \cdot d_d) \cdot d_d\right]^{\frac{5}{3}} \cdot \left(b + 2 \cdot d_d \cdot \sqrt{1 + z^2}\right)^{-\frac{2}{3}} \cdot S^{\frac{1}{2}} - Q$$

Trial depth $d_d := 0.5 \cdot ft$

$$solution := root(F(d_d), d_d)$$

$$d_d := solution$$

$$d_d = 0.377 \cdot ft$$

Therefore the downstream depth is $d_d = 0.377 \cdot ft$

The actual depth is smaller. Regardless, the downstream depth is less than the critical depth, therefore the flow is supercritical and flow at the control section will not be affected by the downstream conditions.

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Backwater Calculation

Critical depth at the control section is $d_c = 0.565 \cdot ft$

The length of the inlet channel is $L := 48 \cdot ft$ along the centerline of the channel.

The bottom width of the inlet channel is $b := 40 \cdot ft$

The side slopes of the inlet channel are $z = 5$

The inlet channel is level at elevation $EL_{control} := 1088 \cdot ft$

Find H_{ec} at the control section

$$a(d_c) = 24.209 \cdot ft^2$$

$$v(d_c) = 4.131 \cdot ft \cdot sec^{-1}$$

$$H_{ec} := d_c + \frac{(v(d_c))^2}{2 \cdot g} \qquad H_{ec} = 0.83 \cdot ft$$

Find α

$n := 0.015$ for concrete filled geoweb or uniform section mat concrete revetment

$$\alpha := \frac{4.315 \cdot ft^{\frac{1}{3}} \cdot n^2}{H_{ec}^{\frac{4}{3}}} \qquad \alpha = 0.001244 \cdot ft^{-1}$$

The head on the emergency spillway crest is

$$H_p := H_{ec} \cdot (1 + \alpha \cdot L)$$

$$H_p = 0.88 \cdot ft$$

The elevation of the water in the pond is

$$EL_{pond} := EL_{control} + H_p \qquad EL_{pond} = 1088.88 \cdot ft$$

The pond crest elevation is 1090, therefore the freeboard is

$$F_b := 1090 \cdot ft - EL_{pond} \qquad F_b = 1.12 \cdot ft$$ which is considered acceptable.

SUBJECT: Keystone Station
Phase II Permitting
BY: SER DATE: 6/17/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KMS DATE: 7/24/96 SHEET NO. 1 OF 2

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CULVERT DESIGN - DIRTY WATER INLET TO EQUALIZATION POND

Purpose: Design the culvert which will carry dirty water into the equalization pond from the West Dirty Water Ditch (DWD).

Methodology: "Hydraulic Design of Highway Culverts", HDS No. 5, Federal Highway Administration, September 1985

Data Input Section

Design Flow, $Q := 91 \cdot \frac{ft^3}{sec}$ 25-year, 24-hour peak flow for West DWD from "Dirty Water Ditch and Related Facilities" calc by SER 5/24/96

Inlet invert elevation, $EL_i := 1091.0 \cdot ft$

Outlet invert elevation, $EL_o := 1089.0 \cdot ft$

Limiting headwater elevation, $EL_1 := 1098.0 \cdot ft$

Pipe Length, $L := 52 \cdot ft$

Pipe Slope, $S := \frac{EL_i - EL_o}{L}$ $S = 0.038$

Pipe diameter, $D := \frac{42 \cdot in}{12 \cdot \frac{in}{ft}}$ $D = 3.5 \cdot ft$

Pipe material is HDPE with headwall and a sluice gate mounted on inlet of pipe.

Flow Area, $A := \frac{D^2 \cdot \pi}{4}$ $A = 9.621 \cdot ft^2$

Flow Velocity, $V := \frac{Q}{A}$ $V = 9.458 \cdot ft \cdot sec^{-1}$ assuming full flow

Hydraulic Radius, $R := \frac{D}{4}$ $R = 0.875 \cdot ft$ assuming full flow

Entrance Loss Coefficient, $k_e := 0.5$ from HDS No. 5 for concrete pipe with square edged headwall. Use this for best match with proposed pipe configuration.

Manning's loss Coefficient $n := 0.011$

Parameters for use in Equation 28 of HDS No. 5, for Submerged Conditions Inlet Control

$c := 0.0398 \cdot \frac{sec^2}{ft}$ from HDS No. 5 for concrete pipe with square edged headwall, units by dimensional analysis of Equation (28) below.

$Y := 0.67$ from HDS No. 5, table 9, for given pipe material and entrance type

Use these values for best match with proposed pipe configuration.

SUBJECT: Keystone Station
Phase II Permitting
BY: SER DATE: 6/17/96 PROJ. NO.: 92-220-73-07
CHKD. BY: KWB DATE: 7/24/96 SHEET NO. 2 OF 2

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Inlet Control Calculation Section

Submerged Equation (28) from HDS No. 5,

$$HW_i := D \cdot \left[c \cdot \left(\left(\frac{Q}{A \cdot D^{0.5}} \right) \right)^2 + Y - 0.5 \cdot S \right] \qquad HW_i = 5.8 \cdot ft$$

Inlet Control Headwater Elevation,

$$EL_{hi} := EL_i + HW_i \qquad EL_{hi} = 1096.8 \cdot ft$$

Outlet Control Calculation Section

Pipe Head Loss Equation from HDS No. 5,

$$H := \left(1 + k_e + \frac{29 \cdot n^2 \cdot L}{R^{1.33}} \cdot ft^{0.33} \right) \cdot \frac{V^2}{2 \cdot g} \qquad H = 2.4 \cdot ft$$

Critical Depth, $d_c := 2.9 \cdot ft$ from chart 4 in HDS-5

$$h_0 := \frac{D + d_c}{2}$$

Tailwater in pond will be well below the pipe invert, therefore the pipe outlet conditions govern.

$h_0 = 3.2 \cdot ft$

Outlet Control Headwater Elevation,

$$EL_{ho} := EL_o + H + h_0 \qquad EL_{ho} = 1094.6 \cdot ft$$

Controlling Headwater Elevation

$$EL_{hc} := \max \left(\begin{pmatrix} EL_{hi} \\ EL_{ho} \end{pmatrix} \right) \qquad EL_{hc} = 1096.8 \cdot ft$$

Compare to the limiting headwater elevation,

$EL_1 = 1098.0 \cdot ft$

$EL_{hc} < EL_1$, Therefore Pipe design is OK

SUBJECT KEYSTONE STATION

BY SER DATE 1/21/98 PROJ. NO. 92-220-71-7

CHKD. BY KRB DATE 1/22/98 SHEET NO. 1 OF 7

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WEST VALLEY EQUALIZATION POND
NORMAL POOL PIPE

THE FLOW INTO THE WEST VALLEY EQ. POND MUST BE LIMITED TO THE DESIGN CAPACITY FROM THE POND TO THE IWT.

DESIGN CAPACITY TO IWT FROM WEST VALLEY = 575 GPM

PORTION DIRECTLY FROM LEACHATE WEIR = 73.4 GPM
(SEE ROM 17R) → SAY 75 GPM

∴ DESIGN CAPACITY FROM POND TO IWT = 500 GPM

THE PIPE TO THE IWT IS PROPOSED TO BE A 12" φ HDPE SDR 21 ID ≐ 11.5"

THE FIRST LENGTH DOWNSTREAM OF THE POND IS 193 FT LONG WITH A SLOPE OF 2.05% OUTLET INV 1064.05 INLET INV. 1068.00 (DRAWING 92-220-F4031)

ESTIMATE HW FOR THIS PIPE

INLET CONTROL

USE CHART 1 NEXT SHEET

$Q = 500 GPM = 1.1 cfs$

$HW/D = 0.66$
$HW = 0.66 \cdot 11.5" = 0.6 ft$

SUBJECT Keystone Station

BY SBR DATE 1/21/98 PROJ. NO. 92-220-73-3

CHKD. BY KMH DATE 1/22/98 SHEET NO. 2 OF 7

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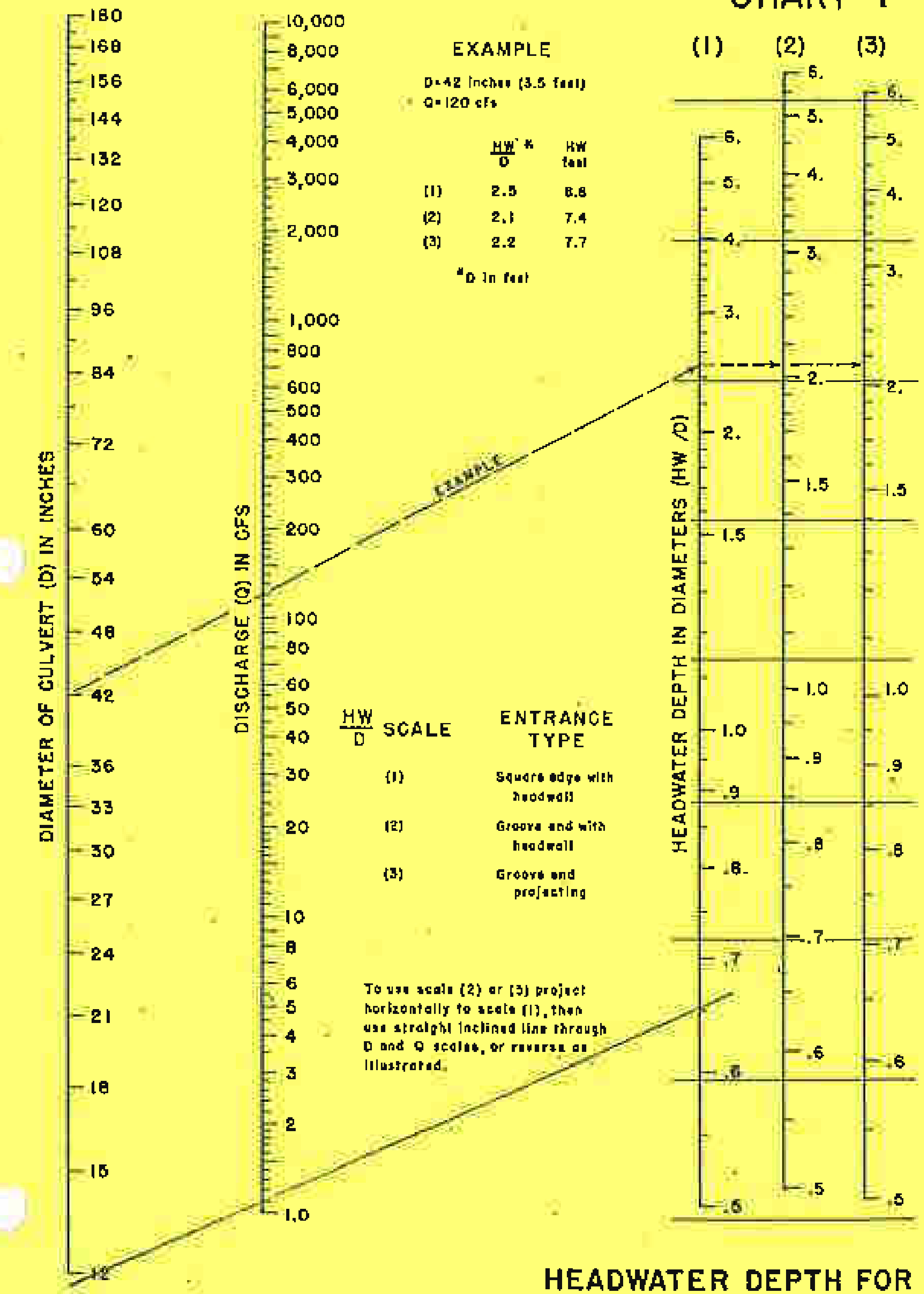
CHART 1



TAKEN FROM: "HYDRAULIC DESIGN OF HIGHWAY CULVERTS", HDS NO 5, FHWA SEPTEMBER 1965

HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL

HEADWATER SCALES 2&3 REVISED MAY 1964

BUREAU OF PUBLIC ROADS JAN. 1963

SUBJECT Keystone Station

BY SER DATE 1/21/98 PROJ. NO. 92-220-73-7

CHKD. BY [illegible] DATE 1/22/98 SHEET NO. 3 OF 7

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OUTLET CONTROL

THE SMALLEST SLOPE ALONG THIS PIPE IS 0.5%
(DWGS 92-220-F4031 TO 4033)

IF FULL FLOW CAPACITY IS > 1.1 CFS THEN PIPE ACTS UNDER INLET CONTROL

ESTIMATE FULL FLOW CAPACITY

USE MANNING'S EQUATION

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

$n = 0.011$ HDPE
$D = 11.5'' = 0.958'$
$A = 0.721\ FT^2$ FULLFLOW
$R = 0.24\ FT$ FULL FLOW
$S = 0.5\%$

$$\text{FULL FLOW } Q = \frac{1.49}{0.011}(0.721)\,0.24^{2/3} \cdot (0.005)^{1/2} = 2.7\ CFS$$

$\therefore$ INLET CONDITIONS CONTROL

HW ELEV ON INT PIPE = 1068 + 0.6 = 1068.6

SUBJECT: KEYSTONE STATION

BY: SER DATE: 1/21/98 PROJ. NO. 92-220-73-7

CHKD. BY: [signature] DATE: 1/22/98 SHEET NO. 4 OF 7

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NORMAL POOL PIPE

THE PROPOSED LAYOUT OF THE NORMAL POOL PIPE IS SHOWN ON SHEET 5

THE PEAK DESIGN WATER SURFACE ELEVATIONS INSIDE AND OUTSIDE OF THE RISER DURING A 10 YEAR, 24-HOUR STORM EVENT ARE:

INTERIOR → 1068.6 HW ON INLET PIPE @ 500 GPM
EXTERIOR → 1086.1 10 YR 24 HR PEAK

DETERMINE IF AN ORIFICE PLATE IS REQUIRED ON THE 4" ⌀ PIPE'S OUTLET. THE 10" ⌀ VALVE WILL BE CLOSED AT ALL TIMES UNLESS AN OPERATOR IS PRESENT

INLET CONTROL (AT POND WATER SURFACE)

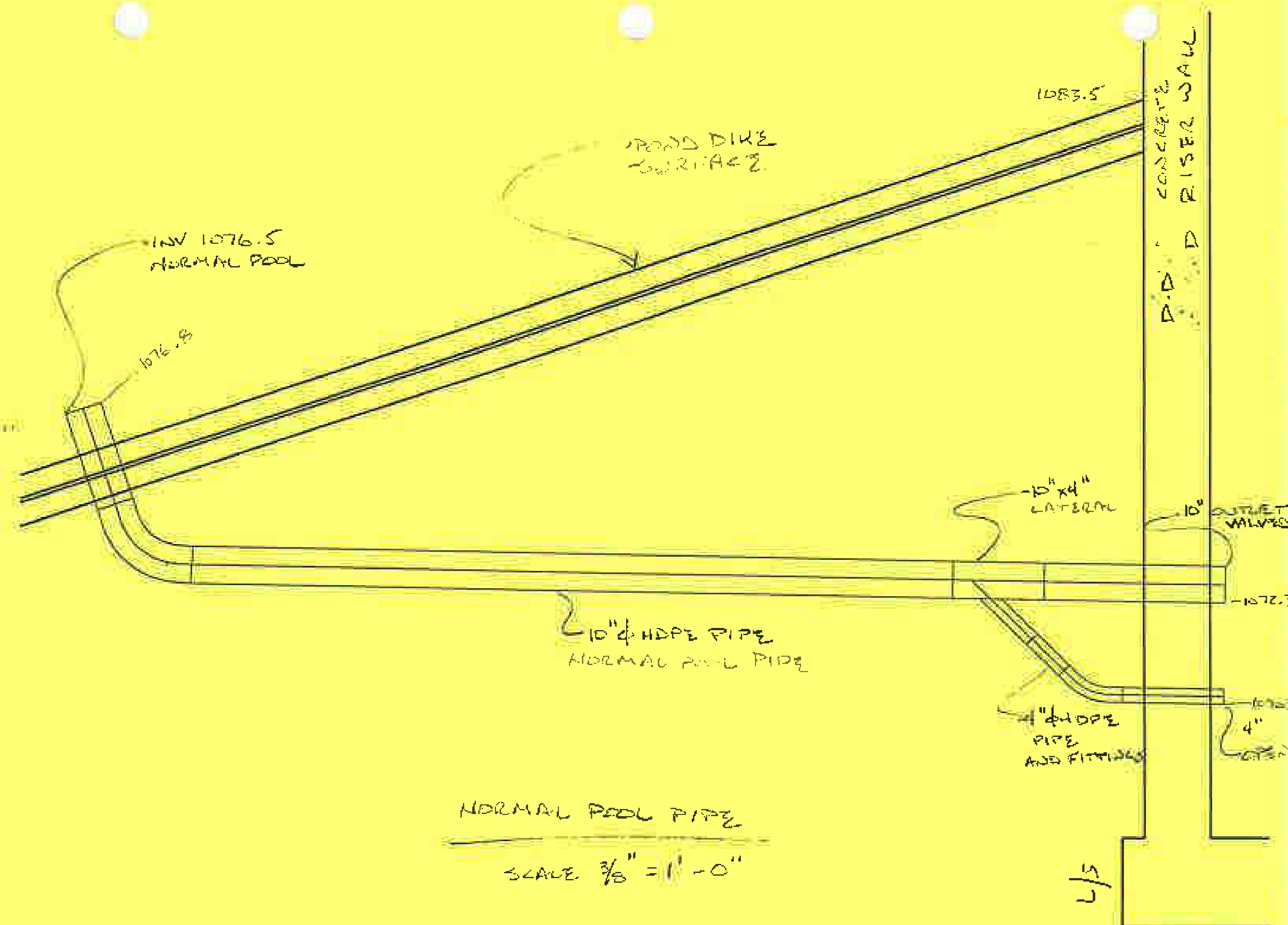
USE ORIFICE EQUATION, (ASSUME WEIR WILL BE SUBMERGED)

$$Q = CA\sqrt{2gh}$$

USE $C = 0.55$
$D = 10"$ NOMINAL $= 9.7\pm"$ ID $= 0.81'$ ID SDR-21 ALSD
$A = 0.52\ FT^2$
$h = 1086.1 - \left(\frac{1076.5 + 1076.[illegible]}{2}\right) = 9.5'$

$$Q = 0.55(0.52)\sqrt{2 \cdot g \cdot 9.5} = 7.1\ CFS$$

∴ INLET WILL NOT LIMIT FLOW TO 1.1 CFS



NORMAL POOL PIPE

SCALE 3/8" = 1'-0"

SUBJECT Keystone Station

BY SER DATE 1/21/98 PROJ. NO. 92-220-73-7

CHKD. BY KMB DATE 1/23/98 SHEET NO. 6 OF 7

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OUTLET CONTROL

USE HEAD LOSS EQ $h_L = \left(K_o + K_e + K_b + K_f + \frac{29 n^2 L}{R^{1.33}}\right)\frac{Q^2}{A^2}\frac{1}{2g}$

FOR EACH PIPE SECTION

FROM HDS-5, FHWA "HYDRAULIC DESIGN OF HIGHWAY CULVERTS" SEPT 1985

FOR 10" ϕ PIPE

$L = 23' \pm$ $D = 9.7" = 0.81'$

$K_o = 0$ $R = 0.20'$ $A = 0.52 ft^2$

$K_e = 0.5$; SQUARE EDGED ENTRANCE

$K_b = 0.2$; ~90° BEND WITH r/D = 2±

FOR 4" ϕ PIPE $D = 4.07" = 0.34'$

$L = 7' \pm$ $R = 0.085'$

$K_o = 1.0$ $A = 0.091 ft^2$

$K_e = 0$

$K_b = 0.1$, ~45° BEND

$K_f = 0.5$; ENGINEERING JUDGEMENT (INCLUDE A CONTRACTION AND A DIRECTION CHANGE LOSS)

FITTING LOSS

USE n = 0.011 HDPE

$$h_L = \left(0 + 0.5 + 0.2 + 0 + \frac{29(0.011)^2 23}{0.2^{1.33}}\right)\frac{Q^2}{0.52^2}\frac{1}{2g}$$

$$+ \left(1.0 + 0 + 0.1 + 0.5 + \frac{29(0.011)^2 7}{0.085^{1.33}}\right)\frac{Q^2}{0.091^2}\frac{1}{2g} = (0.080 + 4.23)Q^2$$

$$= 4.3 Q^2$$

ASSUME TW = 1070 + 4" = 1070.3

$\therefore h_L = 1086.1 - 1070.3 = 15.8' = 4.3Q^2 \therefore Q = 1.92$ cfs

$\therefore$ OUTLET CONDITIONS WILL NOT LIMIT FLOW TO 1.1 cfs AND AN ORIFICE PLATE IS REQUIRED ON OUTLET

SUBJECT KEYSTONE STATION

BY SER DATE 1/21/98 PROJ. NO. 92-220-73-7

CHKD. BY KMB DATE 1/22/98 SHEET NO. 7 OF 7

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ORIFICE PLATE (AT 4" PIPE OUTLET)

INVERT ELEV OF ORIFICE = 1070

∴ HEAD ON ORIFICE = 1086.1 − 1070 = 16.1 FT − 1/2 D

ASSUME 16 FT

USE ORIFICE EQUATION

$$Q = CA\sqrt{2gh}$$

FIND A

USE $C = 0.55$

$Q = 1.1$ CFS

$h = 16.$ FT

$$1.1 = 0.55 \cdot A\sqrt{2g \cdot 16.}$$

$$A = 0.062 \text{ FT}^2$$

USE A CIRCULAR ORIFICE

$$A = \frac{D^2\pi}{4} = 0.062$$

$D = 0.28$ FT $= 3.4$ IN ∴ $h = 16$ FT± OK

APPENDIX I-1-F

FORM I

EXISTING EAST VALLEY EQUALIZATION PONDS - DESIGN CALCULATIONS

SUBJECT

BY | DATE | PROJ. NO.

CHKD. BY | DATE | SHEET NO. | OF

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EXISTING EAST VALLEY EQUALIZATION PONDS - DESIGN CALCULATIONS

DESCRIPTION	NO. OF SHEETS
EQUALIZATION POND DESIGN SUMMARY SHEET	1
EQUALIZATION BASIN DESIGN	20

APPENDIX D

EQUALIZATION POND DESIGN SUMMARY SHEET

Required Capacity

10-Year 24-Hour Storm: 7.23 ac.-ft.

7000 cf/acre: 7.6 ac.-ft.

Actual Capacity: 9.13 ac.-ft.

Capacity of Each Chamber (2 Chambers): 4.565 ac.-ft.

Design Elevations

Top of Dike: 995.7
Bottom of Pond: 983
Emergency Spillway: 992
Design Pool (10-Year 24-Hour Storm): 990.6
Maximum Water Surface (100-Year 24-Hour Storm): 993.7
Maximum Sediment Accumulation: 985.7

Peak Inflows

10-Year 24-Hour Storm: 56.6 cfs

100-Year 24-Hour Storm: 108.4 cfs

Peak Discharge

Pumps: 500 GPM

Emergency Spillway: 110 cfs

Drainage Area: 99.28 acres (maximum)

Head on Emergency Spillway: 1.7 feet

Freeboard: 2' (over emergency spillway head)

SUBJECT [illegible] - Keystone

Equalization Basin Design

BY DB DATE 7/21/87 PROJ. NO. 78-505-41

CHKD. BY KLE DATE 10/1/87 SHEET NO. 1 OF 20

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Equalization Basin Design

The calculations provided on the following sheets are the basis of design for the equalization basin which is located below the Coal Valley disposal site. Runoff determinations, hydrographs, etc. are based on procedures outlined in the Soil Conservation Service Engineering Field Manual, 1969. Other sources are referenced where used.

The design is based on the site development occurring in two stages with Stage I completed and fully revegetated before Stage II is begun. The equalization basin will be sized for the unlikely event of two 10-year 24-hour storms occurring in a 72-hour period. Runoff from the disposal site will reach the basin within a relatively short time period, less than one hour after the beginning of the first 10-year 24-hour storm at which time the pumping station will begin to draw down the water elevation in the pond. After the end of the first storm a 24-hour lag time is provided for during which time pumping will continue. At this point the beginning of the second 10-year 24-hour storm will occur with pumping continuing until the pond is emptied.

The drainage area of 99.8 acres shown on sheet 2 is the maximum contributing drainage area which will occur only toward the end of Stage II. In the earlier phases of Stage II runoff from the undisturbed and undeveloped portions of the site will be diverted around the equalization pond thereby providing a contributing area far less than 99.8 acres. During the entire site development, areas will be revegetated upon completion and will be diverted from the equalization pond when possible upon attaining adequate vegetative cover.

SUBJECT Penelec-Keystone
Equalization Drain Design

BY DR DATE 9/21/82 PROJ. NO. 78-505-41

CHKD. BY WF DATE 10/6/82 SHEET NO. 2 OF 20

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Maximum Contributing Area

41.88 acres of disturbed coal ash/refuse
48.56 acres of undisturbed hillside (wooded or vegetated)
8.84 acres of revegetated bench
source: "Peak Discharge in CFS" by JMJ on 4/30/82 sheet 5/27
total area = 99.28 ac, ≈ 0.15 mi²

Runoff Curve Numbers

disturbed coal ash/refuse → CN=70 (flat)
vegetated hillside → CN=60 (steep)
revegetated bench → CN=60 (flat)
source: "Peak Discharge in CFS" by JMJ on 4/30/82 sheet 5/27

Weighted Curve Number

$$= \frac{41.88(70) + 48.56(60) + 8.84(60)}{41.88 + 48.56 + 8.84}$$

$= 64.2$

∴ use $CN_{weighted} = 65$ to be conservative

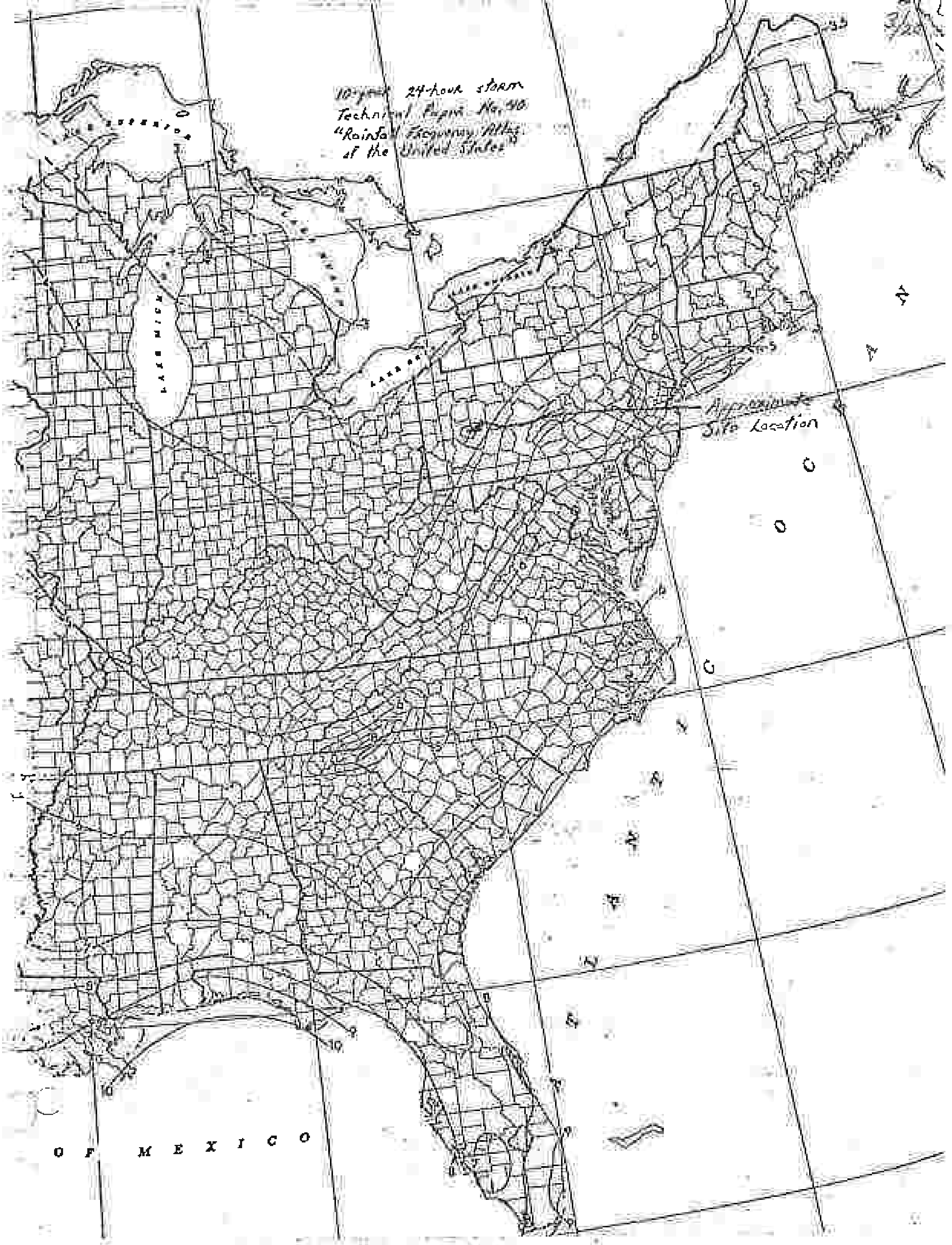
10-Year 24-Hour Rainfall

P = 3.9 in. for the 10-year 24-hour rainfall

source: see sheet 3 (attached)

Determine Rainfall-Runoff Depths for 10-Yr. 24-Hr. Rainfall

Entering the chart on attached sheet 4 with CN=65 and P=3.9 in. yields 0.97 in. of runoff.



10-year 24-hour storm
Technical Paper No. 40
"Rainfall Frequency Atlas of the United States"
Approximate Site Location
OF MEXICO

RAINFALL-RUNOFF DEPTHS FOR SELECTED RUNOFF CURVE NUMBERS

CURVE 65

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0										
1	0.00	0.00	0.00	0.01	0.02	0.03	0.04	0.06	0.08	0.10
2	0.13	0.16	0.19	0.23	0.26	0.30	0.33	0.37	0.42	0.46
3	0.50	0.55	0.60	0.65	0.70	0.75	0.80	0.86	0.91	0.97
4	1.03	1.09	1.15	1.21	1.27	1.33	1.39	1.45	1.52	1.58
5	1.65	1.72	1.78	1.85	1.92	1.99	2.06	2.13	2.20	2.28
6	2.35	2.42	2.50	2.57	2.64	2.72	2.80	2.87	2.94	3.02
7	3.10	3.18	3.25	3.33	3.41	3.49	3.57	3.64	3.73	3.81
8	3.89	3.97	4.05	4.13	4.22	4.30	4.38	4.46	4.54	4.62
9	4.71	4.80	4.88	4.96	5.05	5.13	5.22	5.30	5.39	5.47
10	5.56	5.65	5.73	5.82	5.90	5.99	6.08	6.17	6.26	6.34
11	6.43	6.52	6.60	6.69	6.78	6.87	6.96	7.05	7.14	7.22
12	7.31	7.40	7.49	7.58	7.67	7.75	7.85	7.94	8.03	8.12

CURVE 66

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
1	0.00	0.00	0.01	0.01	0.02	0.04	0.06	0.08	0.10	0.12
2	0.15	0.18	0.21	0.25	0.29	0.33	0.37	0.41	0.45	0.50
3	0.55	0.60	0.65	0.70	0.75	0.80	0.85	0.91	0.97	1.03
4	1.09	1.15	1.21	1.27	1.33	1.39	1.46	1.53	1.59	1.66
5	1.73	1.80	1.87	1.94	2.01	2.08	2.15	2.22	2.29	2.36
6	2.44	2.51	2.59	2.67	2.74	2.82	2.89	2.97	3.05	3.12
7	3.20	3.28	3.36	3.44	3.52	3.60	3.68	3.76	3.84	3.93
8	4.01	4.09	4.17	4.25	4.34	4.43	4.51	4.59	4.67	4.76
9	4.84	4.93	5.01	5.10	5.18	5.27	5.35	5.43	5.50	5.61
10	5.70	5.78	5.87	5.96	6.05	6.13	6.22	6.31	6.40	6.49
11	6.57	6.66	6.75	6.84	6.93	7.02	7.11	7.20	7.29	7.38
12	7.46	7.55	7.64	7.73	7.82	7.92	8.01	8.10	8.19	8.28

CURVE 67

Inches \ Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
1	0.00	0.00	0.01	0.02	0.03	0.05	0.07	0.09	0.12	0.15
2	0.18	0.21	0.24	0.28	0.32	0.36	0.40	0.44	0.49	0.54
3	0.59	0.64	0.69	0.74	0.79	0.85	0.91	0.97	1.03	1.09
4	1.15	1.21	1.27	1.34	1.40	1.47	1.53	1.60	1.67	1.74
5	1.81	1.88	1.95	2.02	2.09	2.16	2.23	2.31	2.39	2.46
6	2.54	2.61	2.69	2.76	2.84	2.92	3.00	3.08	3.15	3.23
7	3.31	3.39	3.47	3.55	3.64	3.72	3.80	3.88	3.96	4.04
8	4.13	4.21	4.29	4.38	4.46	4.55	4.63	4.71	4.80	4.89
9	4.97	5.06	5.14	5.23	5.31	5.40	5.49	5.58	5.66	5.75
10	5.84	5.92	6.01	6.10	6.19	6.28	6.36	6.45	6.54	6.63
11	6.72	6.81	6.90	6.99	7.08	7.17	7.26	7.35	7.44	7.53
12	7.62	7.71	7.80	7.89	7.98	8.07	8.16	8.25	8.35	8.44

Exhibit 2-7A

SUBJECT Hamlin - Keystone
Equalization Basin Design

BY DL DATE 9/27/82 PROJ. NO. 78-505-41

CHKD. BY KLE DATE 10/-/82 SHEET NO. 5 OF 20

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Determine Time of Concentration

Stage II will be constructed in phases. The drainage from the undisturbed areas will be channeled around the equalization pond in all phases. The worst case concerning flow to the equalization pond will be at practically completion of Stage II since there will be no large diversion of runoff as occurs earlier.

The flow paths for the time of concentration are shown on the attached dwg. (78-505-F4) (41-F-0023)

Lengths & slopes of flow path

1. A-B along the (ash) → 1560' (1%)
B-C (overland) → 464' $\frac{1250'-1205'}{464'} \times 100\% = 9.7\%$
C-D (channel) → 2900' $\frac{1205'-993'}{2900'} \times 100\% = 7.3\%$

2. G-E (ash) → 1550' (1%)
E-F (overland) → 370' $\frac{1318'-1286'}{370'} \times 100\% = 8.6\%$
F-C (channel) → 2864' $\frac{1218'-1205'}{2864'} \times 100\% = 0.5\%$
C-D (channel) → 2900' $\frac{1205'-993'}{2900'} \times 100\% = 7.3\%$

Entering Figure 3-1 in TR-55 "Urban Hydrology for Small Watersheds" by SCS with ash (use nearly bare ground) and 1% slope yields v = 1 fps; with natural ground surface (forest with heavy ground litter & meadow) and 9.7% slope yields v = 0.78 fps, and with natural ground surface (forest with heavy ground litter & meadow) and 8.6% slope yields v = 0.73 fps.

The channel flow (F-C & C-D) will be in a Type II ditch which is a V-notch ditch 3' deep with 2:1 side slopes of grouted rock.

SUBJECT	Hamilton - Keystone Equilization Basin Design
BY	AR
DATE	9/29/82
PROJ. NO.	78-505-41
CHKD. BY	WLK
DATE	10/6/82
SHEET NO.	6 OF 20

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$A = 3' \times 6' = 18\ ft.^2$

$P = 2\sqrt{3^2 + 6^2} = 13.42\ ft.$

$R = \frac{A}{P} = \frac{18\ ft.^2}{13.42\ ft.} = 1.34\ ft.$

$n = 0.026$

C-D

$v = \frac{1.49}{n} R^{2/3} S^{1/2}$

$= \frac{1.49}{0.026}(1.34)^{2/3}(0.075)^{1/2}$

$= 19.1\ fps$

F-C

$v = \frac{1.49}{n} R^{2/3} S^{1/2}$

$= \frac{1.49}{0.026}(1.34)^{2/3}(0.005)^{1/2}$

$= 4.9\ fps$

Time of concentration for A-B, B-C, & C-D

A-B → sod → $\frac{1560\ ft.}{1\ fps}$ = 1560 sec.

B-C → overland → $\frac{245\ ft.}{0.41\ fps}$ = 598 sec.

C-D → channel → $\frac{2970\ ft.}{19.1\ fps}$ = 155 sec.

Total = 2310 sec. = 38.5 min.

Time of concentration for G-E, E-F, F-C, & C-D

G-E → sod → $\frac{1550'}{1\ fps}$ = 1550 sec.

E-F → overland → $\frac{270'}{0.533\ fps}$ = 507 sec.

F-C → channel → $\frac{2865\ ft.}{4.9\ fps}$ = 585 sec.

C-D → channel → $\frac{2970\ ft.}{19.1\ fps}$ = 155 sec.

Total = 2797 sec. = 46.6 min. = 0.78 h.

∴ Use $T_c = 0.78\ hr.$ Note: T_c will be 0.

SUBJECT Paradise Keystone
Equalization Basin Design

BY DR DATE 9/27/82 PROJ. NO. 78-540-41
CHKD. BY KLF DATE 10/6/82 SHEET NO. 7 OF 20

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Determination of Inflow Hydrograph and Peak Discharge for 10-Yr. 24-Hr. Storm

Use the method in Chapter 5 of TR-55 "Urban Hydrology for Small Watersheds."

Since the time of concentration is 0.78 hr. interpolate between the values given for $t_c = 0.75$ hr. & $t_c = 1.00$ hr. on page 5-8 of TR-55.

Use $T_c = 0.78$ hr. & $T_t = 0.0$ hr.

Use the following equation to compute discharge from the values interpolated from the charts on page 5-8 of TR-55:

$$q = q_p (DA)(Q)$$

where q = hydrograph coordinate discharge in cfs
q_p = csm/in. (cubic feet per second per square mile per inch of runoff)
DA = discharge area in sq. mi.
Q = runoff in inches

hr.	q_p csm/in.	q cfs
11.0	14.8	2.2
11.5	28.4	4.3
11.7	55.6	8.3
11.8	94.2	14.1
11.9	156.3	23.4
12.0	236.8	35.5
12.1	314.8	47.2
12.2	361.0	54.2
12.3	377.6	56.6

$q = q_p (DA)(Q)$
$= q_p (0.15)(0.99)$
$= 0.15 (q_p)$

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SUBJECT [illegible] - Keystone
Equalization Basin Design

BY DR DATE PROJ. NO. 78-505-41

CHKD. BY KLF DATE 10/6/82 SHEET NO. 8 OF 20

hr.	qc (csm/in.)	q (cfs)
12.4	362.3	54.3
12.5	323.9	48.6
12.6	279.0	41.9
12.7	237.4	35.6
12.8	201.2	30.2
12.9	171.2	25.7
13.0	147.3	22.1
13.2	111.7	16.8
13.5	79.1	11.9
14.0	52.6	7.9
14.5	39.8	6.0
15.0	33.4	5.0
16.0	26.1	3.9
18.0	19	2.9
20.0	15	2.3

For plot of 10-Yr. 24-Hr. Hydrograph see sheet 12.

100-Year 24-Hour Rainfall

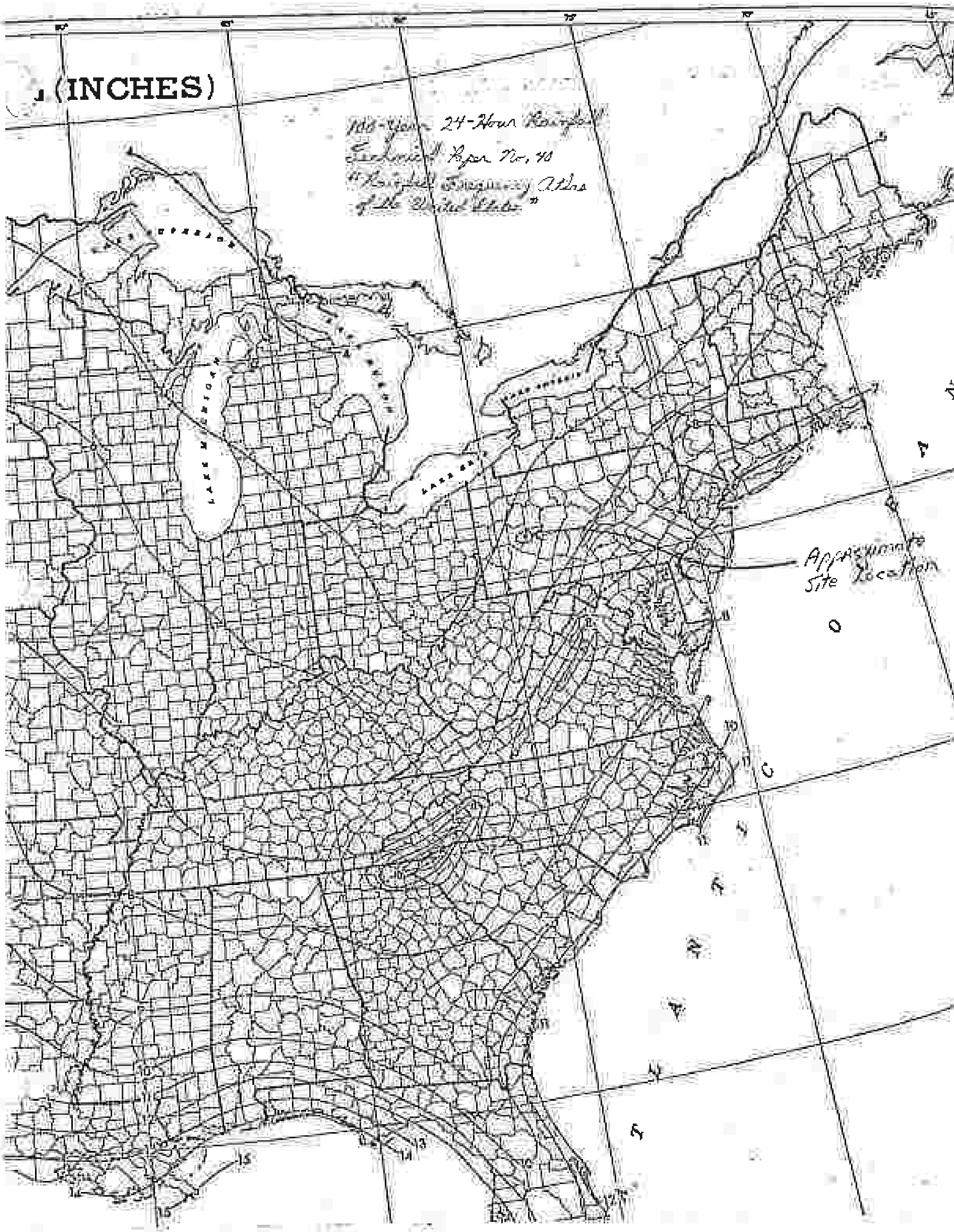
P=5.3 inches for the 100-Year 24-Hour rainfall

source: see attached sheet 9

Determine Rainfall-Runoff Depth for 100-Year 24-Hour Rainfall

Entering the chart on attached sheet 4 with CN=65 and P=5.3 in. yields 1.85 inches of runoff.

9/20


(INCHES)
100-Year 24-Hour Rainfall
Technical Paper No. 40
"Rainfall Frequency Atlas
of the United States"
LAKE SUPERIOR
LAKE MICHIGAN
LAKE HURON
LAKE ERIE
LAKE ONTARIO
Approximate
Site Location
OCEAN
ATLANTIC

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Determination of Inflow Hydrograph and Peak Discharge for 100-yr., 24-hr. Storm

Use the method in chapter 5 of TR-55.

T_c = 0.78 hr. ∴ interpolate between the values given for t_c = 0.78 hr. & t_c = 1.00 hr. on page 5-8 of TR-55

Use T_c = 0.78 hr. and T_t = 0.0 hr.

Use the following equation to compute discharge interpolated from the charts on page 5-8 of TR-55:

$$q = q_p\,(DA)(Q)$$

(see sheet 7 for description of terms)

$$q = q_p\,(DA)(Q) = q_p\,(0.155)(1.85) = 0.287\,(q_p)$$

hr.	q_p csm/in.	q cfs
11.0	14.8	4.2
11.5	28.4	8.2
11.7	55.6	16.0
11.8	94.2	27.0
11.9	156.3	44.9
12.0	236.8	68.0
12.1	314.8	90.3
12.2	361.0	103.6
12.3	377.6	108.4
12.4	362.3	104.0
12.5	323.9	93.0
12.6	279.0	80.1
12.7	237.4	68.1
12.8	201.2	57.7
12.9	171.2	49.1
13.0	147.5	42.3

SUBJECT [illegible]-Keystone Equalization Basin Design

BY OB DATE 9/20/82 PROJ. NO. 78-505-41

CHKD. BY KLF DATE 10/6/82 SHEET NO. 11 OF 20

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hr.	qp csm/in.	q cfs
13.2	111.7	32.1
13.5	79.1	22.7
14.0	52.6	15.1
14.5	39.8	11.4
15.0	33.4	9.6
16.0	26.1	7.5
18.0	19	5.5
20.0	15	4.3

For plot of 100-Year 24-Hour hydrograph see sheet 12.

Determine Size of Pond Required to Hold 10-yr. 24-Hr. Storm

Planimeter the area under the curve for the 10-yr. 24-hr. hydrograph to determine the size of pond required to hold the 10-yr. 24-hr. storm.

Area by planimetering $= 4.37\ in.^2$

Scale: 1" = 20 cfs vert.
1" = 1 hr. hor.

$\therefore 1\ in.^2 = 20\ ft.^3/sec. \times 1\ hr. \times \frac{3600\ sec.}{1\ hr.} = 72{,}000\ ft.^3$

Volume required to hold 10-yr. 24-hr. storm

$= 4.37\ in.^2 \times 72{,}000\ ft.^3/in.^2$

$= 314{,}640\ ft.^3$

$= 7.23\ ac.\text{-}ft.$

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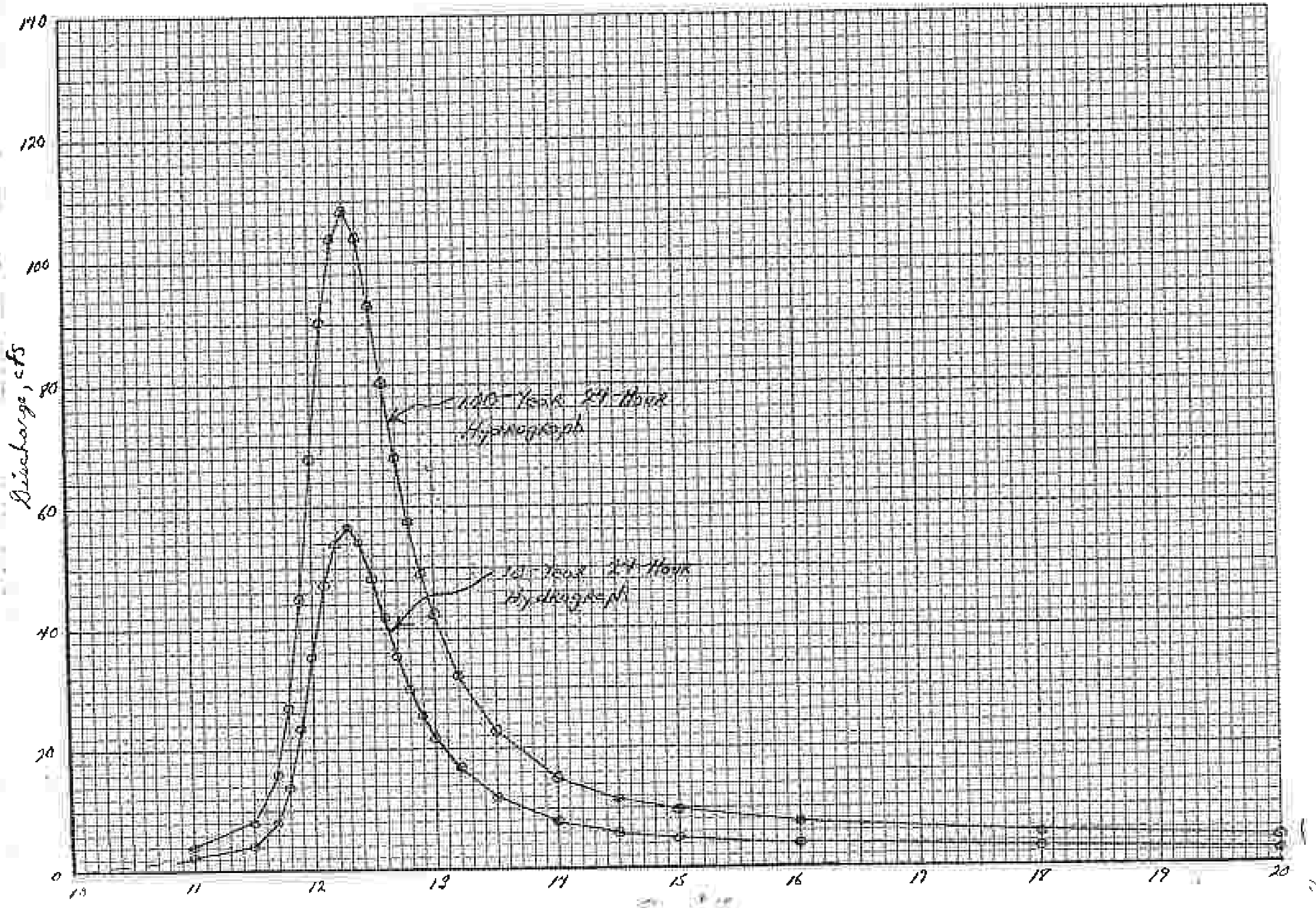

SUBJECT [illegible]
"Equalization" Basin Design

BY DR DATE [illegible] PROJ. NO. 78-505-41

CHKD. BY [illegible] DATE [illegible] SHEET NO. 13 OF 20

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Determine Size of Pond Required to Hold Two 10-Year 24-Hour Storms in 72 Hours with Pumping at the Rate of 500 GPM

$$500 \text{ gal./min.} \times \frac{60 \text{ min.}}{1 \text{ hr.}} \times 72 \text{ hrs.} \times \frac{1 \text{ ft.}^3}{7.48 \text{ gal.}} = 288,770 \text{ ft.}^3$$

$$\begin{aligned} \text{Volume of pond} &= 314,640 \text{ ft.}^3(2) - 288,770 \text{ ft.}^3 \\ &= 340,510 \text{ ft.}^3 \\ &= 7.82 \text{ ac.-ft.} \end{aligned}$$

Check of Pond Size Based on Soil Erosion and Sedimentation Regs

From dwgs. 78-505-F3 & F4 the largest exposed bench is the 1250 bench in Stage II. This area was planimetered to be 47.4 acres.

Soil Erosion & Sedimentation Control Reg ⇒ 7000 cf/acre as pond size requirement.

$$\begin{aligned} \therefore \text{pond size} &= 47.4 \text{ ac.} \times 7000 \text{ cf/ac.} \\ &= 7.6 \text{ ac.-ft.} \end{aligned}$$

$7.6 \text{ ac.-ft.} < 9.13 \text{ ac.-ft.}$ ← actual pond size see sheet 14

$\therefore$ ok

Elevation of Max Allowable Sediment Accumulation

Total Storage Capacity	9.13	ac-ft
Req'd Capacity for 10yr-24 hr event	7.23	
Available for Sediment Storage	1.90	ac-ft

From Storage-Elevation Curve, Max Sed. Stor El = 985.7

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Determine Dimensions of Equalization Basin

The equalization basin will consist of 2 separate chambers each capable of holding $\frac{1}{2}$ of the required volume.

required volume for equalization basin = 7.82 ac.-ft. = 340,639 ft.^{3}

required volume for each chamber = 340,639 ft.^{3} ÷ 2 = 170,320 ft.^{3}

assume a 9' depth for the pond and 3:1 internal side slopes

based on the equalization basin layout on the drawings a pond with the following dimensions will be used:

Bottom dimensions = 45' wide by 280' long
Top dimensions (@ 9' level) = 99' wide by 334' long

∴ ave. dimensions are:

$$\text{width} = \frac{45' + 99'}{2} = 72'$$

$$\text{length} = \frac{280' + 334'}{2} = 307'$$

Capacity of each chamber = 9' × 72' × 307' = 198,936 ft.^{3}

Required capacity per chamber = 170,320 ft.^{3}

∴ chambers are oversized ∴ ok

Actual total pond capacity = 2 (198,936 ft.^{3})
= 397,872 ft.^{3}
= 9.13 ac.-ft.

SUBJECT Penelec - Keystone
Equalization Basin Design

BY DL   DATE 9/22/82   PROJ. NO. 78-508-41

CHKD. BY KLF   DATE 10/4/82   SHEET NO. 15 OF 20

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Calculate Overflow Rate

source: Water Resources Engineering by Linsley & Franzini, 1964, page 427.

Overflow rate = $\frac{Q}{BL}$

where Q = overflow
B = width
L = length

$Q = 500 \text{ gal./min.} \times \frac{60 \text{ min.}}{1 \text{ hr.}} \times \frac{24 \text{ hr.}}{1 \text{ day}} = 720,000 \text{ gal./day}$

overflow rate $= \frac{Q}{BL} = \frac{720,000 \text{ gal/day}}{77 \text{ ft.} \times 101 \text{ ft.}} = 92.6 \text{ gpd/ft.}^2$

ave. dimensions see sheet 14

From Water Resources Engineering (ref. above) typical overflow rates for sedimentation basins are 800 - 1000 gpd/ft.^{2}.

From Water Supply and Pollution Control by Clark, Viessman, & Hammer, 1977, p. 399 overflow rates as low as 100 gpd/ft.^{2} are not unusual when silt & clay are to be removed by plain sedimentation.

∴ Since fly ash particles are approximately silt size the overflow rate of 92.6 gpd/ft.^{2} should be on the conservative side.

SUBJECT [illegible] - [illegible]
Equalization Basin Design

BY [illegible] DATE 9/28/82 PROJ. NO. 78-505-41

CHKD. BY KLF DATE 10/11/82 SHEET NO. 16 OF 20

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Determine Size of Emergency Spillway

The emergency spillway will be sized to pass the peak discharge from the 100-yr, 24-hr. storm which is 108.4 cfs (see sheet 10).

From page 11-19 of SCS Chapter 11 "Ponds and Reservoirs" from the Engineering Field Manual, the spillway side slopes should be no steeper than 3:1 unless excavated in rock. The soil in the area of the equalization basin and emergency spillway consist of clay with a permeability of 1×10⁻⁸ cm/sec. Therefore, use 3:1 spillway side slopes. From page 11-19 it is stated that exhibit 11-2 should be used where the spillway is to be excavated in cohesive soils with a high clay content. On exhibit 11-2 it is noted that the narrowest bottom width should be used where possible to cut down on meandering and that the chart computations are based on a roughness coefficient of n = 0.040 and a maximum velocity of 5.10 fps.

From attached sheet 18 (exhibit 11-2) entering with Q = 100 cfs yields a min. bottom width of 16', a stage of 1.71', and a slope range of 2.5 - 2.8% (we are excavating 1 [illegible] [illegible] slope). Entering with Q = 120 cfs yields a min. bottom width of 20', a stage of 1.71', and a slope range of 2.5 - 2.9%. The design Q is 108.4 cfs and based on interpolation a min. bottom width of 18' should be used yielding a stage of 1.71'. Use a 2.5% min. slope for the entrance and exit channels.

To provide a check on this enter exhibit 11.3 (sheet 19) with a stage of 1.7' and Q = 110 cfs yields bottom width of 18', slope = 2.5%, vel. = 5.3 fps, and X = 30' (X = min. length of channel below control section). ∴ [illegible] ok

17/2

Side slopes - 3 Horiz. to 1 Vert.

Discharge Q CFS	Slope Range Minimum Percent	Slope Range Maximum Percent	Bottom Width Feet	Stage Feet
15	3.3	12.2	8	.83
	3.5	18.2	12	.69
20	3.1	6.9	8	.97
	3.2	13.0	12	.81
	3.3	17.3	16	.70
25	2.9	7.1	8	1.09
	3.2	9.8	12	.91
	3.3	13.2	16	.79
	3.3	17.2	20	.70
30	2.9	6.0	8	1.20
	3.0	8.2	12	1.01
	3.0	10.7	16	.88
	3.3	13.8	20	.78
35	2.8	5.1	8	1.30
	2.9	6.9	12	1.10
	3.1	9.0	16	.94
	3.1	11.3	20	.85
	3.2	14.1	24	.77
40	2.7	4.5	8	1.40
	2.9	6.0	12	1.18
	2.9	7.6	16	1.03
	3.1	9.7	20	.91
	3.1	11.9	24	.83
45	2.6	4.1	8	1.49
	2.8	5.3	12	1.25
	2.9	6.7	16	1.09
	3.0	8.4	20	.98
	3.0	10.4	24	.89
50	2.7	3.7	8	1.57
	2.8	4.7	12	1.33
	2.8	6.0	16	1.16
	2.9	7.3	20	1.03
	3.1	9.0	24	.94
60	2.6	3.1	8	1.73
	2.7	3.9	12	1.47
	2.7	4.8	16	1.28
	2.9	5.9	20	1.15
	2.9	7.3	24	1.05
	3.0	8.6	28	.97
70	2.5	2.8	8	1.88
	2.6	3.3	12	1.60
	2.6	4.1	16	1.40
	2.7	5.0	20	1.28
	2.8	6.1	24	1.15
	2.9	7.0	28	1.05
80	2.5	2.9	12	1.72
	2.6	3.6	16	1.51
	2.7	4.3	20	1.38
80	2.8	5.2	24	1.24
	2.8	5.9	28	1.14
	2.9	7.0	32	1.06
90	2.5	2.6	12	1.84
	2.5	3.1	16	1.61
	2.6	3.8	20	1.45
	2.7	4.5	24	1.32
	2.8	5.3	28	1.22
	2.8	6.1	32	1.14
100	2.5	2.8	16	1.71
	2.6	3.3	20	1.54
	2.6	4.0	24	1.41
	2.7	4.8	28	1.30
	2.7	5.3	32	1.21
	2.8	6.1	36	1.13
120	2.5	2.8	20	1.71
	2.6	3.2	24	1.56
	2.7	3.8	28	1.44
	2.7	4.2	32	1.34
	2.7	4.8	36	1.26
140	2.5	2.7	24	1.71
	2.5	3.2	28	1.58
	2.6	3.6	32	1.47
	2.6	4.0	36	1.38
	2.7	4.5	40	1.30
160	2.5	2.7	28	1.70
	2.5	3.1	32	1.58
	2.6	3.4	36	1.49
	2.6	3.8	40	1.40
	2.7	4.3	44	1.33
180	2.4	2.7	32	1.72
	2.4	3.0	36	1.60
	2.5	3.4	40	1.51
	2.6	3.7	44	1.43
200	2.5	2.7	36	1.70
	2.5	2.9	40	1.60
	2.5	3.3	44	1.52
	2.6	3.6	48	1.45
220	2.4	2.6	40	1.70
	2.5	2.9	44	1.61
	2.5	3.2	48	1.53
240	2.5	2.6	44	1.70
	2.5	2.9	48	1.62
	2.6	3.2	52	1.54
260	2.4	2.6	48	1.70
	2.5	2.9	52	1.62
280	2.4	2.6	52	1.70
300	2.5	2.6	56	1.69

Example of Use

Given: Discharge, Q=87 c.f.s. Spillway Slope, Exit section (from profile)= 4%.

Find: Bottom Width and Stage in Reservoir.

Procedure: Enter table from left at 90 c.f.s. Note that spillway slope (4%) falls within slope ranges corresponding to bottom widths of 24, 28, and 32 feet. Use narrower bottom width, 24 feet, to minimize meandering. Stage in Reservoir will be 1.32 feet.

Note: Computations based on: Roughness coefficient, n=.040.
Maximum velocity=5.50 ft. per sec.

Exhibit 11-2 Design table for vegetated spillways excavated in erosion resistent soils.

4-28407 Rev. 12-68

DESIGN DATA FOR EARTH SPILLWAYS

SIDE SLOPE 3:1
VEGETATED n=0.040

STAGE (Hp) IN FEET	VARIABLES	BOTTOM WIDTH (b) IN FEET: 8	10	12	14	16	18	20	22	24	26	28	30	32	34	36	38	40
0.5	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
0.6	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
0.7	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
0.8	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
0.9	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1.0	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1.1	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1.2	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1.3	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1.4	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1.5	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1.6	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1.7	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1.8	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1.9	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
2.0	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
2.1	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
2.2	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
2.3	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
2.4	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

Exhibit 11-3.1

GAI CONSULTANTS, INC.
Engineers • Geologists • Planners
Environmental Specialists

SUBJECT [illegible] - Keystone
Equalization Basin Design

BY [illegible] DATE 9/28/82 PROJ. NO. [illegible]

CHKD. BY [illegible] DATE 10/1/82 SHEET NO. 19 OF 20

Therefore, use an emergency spillway with the following characteristics:

Q_{design} = 108.4 cfs
$Q_{capacity}$ = 110 cfs
bottom width = 18' { Increase this by 50% (27' width) at entrance and taper to 18' at level section
min. slope = 2.5% } for entrance & exit channels
max. slope = 2.8% } (use 2.5%)
stage = 1.71'
80' min. length of channel below control section

See sheet 21 for sketch of emergency spillway layout.

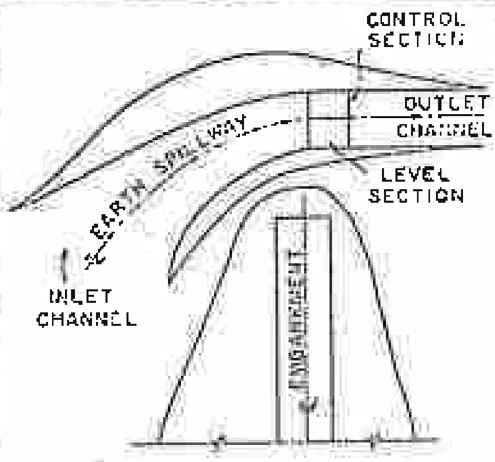
The spillway will be located at the northwest corner of the basin so that it is cut into natural material instead of fill material.

Total Height of Equalization Basin

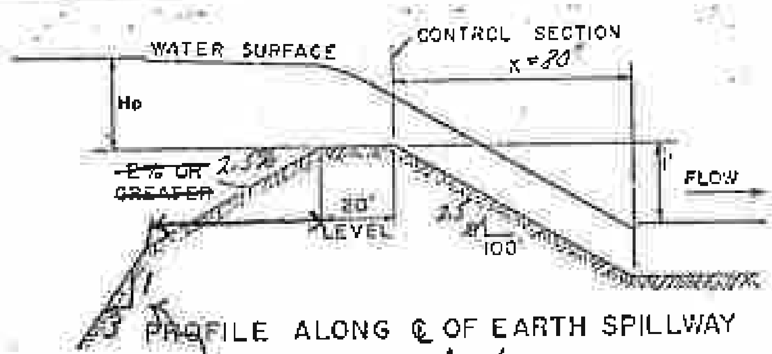
bottom of pond ≈ elev. 983'
pond depth = 9' (elev. 992)
emergency spillway head = 1.71' (elev. 993.71')
freeboard = 2' over emergency spillway head (elev. 995.71')

∴ total height of embankment = 995.71' - 983' = 12.71'

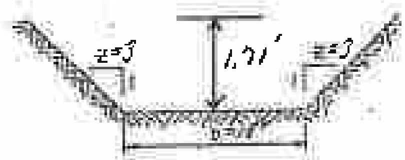
DESIGN DATA FOR EARTH SPILLWAYS



PLAN OF EARTH SPILLWAY



PROFILE ALONG ℄ OF EARTH SPILLWAY
Regular pond slope



CROSS SECTION OF EARTH SPILLWAY AT CONTROL SECTION

LEGEND

n Manning's Coefficient of Roughness
Hp Difference in Elevation between Crest of Earth Spillway at the Control Section and Water Surface in Reservoir, in Feet.
b Bottom Width of Earth Spillway at the Control Section, in Feet.
Q Total Discharge, in cfs.
V Velocity, in Feet Per Second, that will exist in Channel below Control Section, at Design Q, if Constructed to Slope (S) that is shown.
S Flattest Slope (S), in %, allowable for Channel below Control Section.
X Minimum Length of Channel below Control Section, in Feet.
Z Side Slope Ratio

INDEX			
SIDE SLOPE RATIO (Z)	COVER	COEFFICIENT OF ROUGHNESS	SHEET
4:1	VEGETATED	n = 0.040	2
3:1	VEGETATED	n = 0.040	3
2:1	VEGETATED	n = 0.040	4
4:1 with 2:1	VEGETATED	n = 0.040	5
1:1	VEGETATED	n = 0.040	6
4:1	BARE EARTH	n = 0.025	7
3:1	BARE EARTH	n = 0.025	8
2:1	BARE EARTH	n = 0.025	9
4:1 with 2:1	BARE EARTH	n = 0.025	10
1:1	BARE EARTH	n = 0.025	11

NOTE: DATA TO RIGHT OF HEAVY VERTICAL LINES ON DRAWINGS SHOULD BE USED WITH CAUTION, AS THE RESULTING SECTIONS WILL BE EITHER POORLY PROPORTIONED OR HAVE VELOCITIES IN EXCESS OF 6FT. / SEC.

Exhibit 11-3.1

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	RTSC-NE-ENG.
ENGINEERING HANDBOOK, SCS SECTION 5, HYDRAULICS; HANDBOOK OF HYDRAULICS BY KING FOURTH EDITION	SOIL CONSERVATION SERVICE; ENGINEERING & WATERSHED PLANNING UNIT UPPER DARBY, PENNSYLVANIA	1110; SHEET 1 OF 11

SUBJECT Re-In-Keystone
Storage-Elevation Curve for Equalization Basin

BY OR DATE 7/30/82 PROJ. NO. 78-305-41

CHKD. BY GB DATE 2/22/83 SHEET NO. 1 OF 2

GAI CONSULTANTS, INC.
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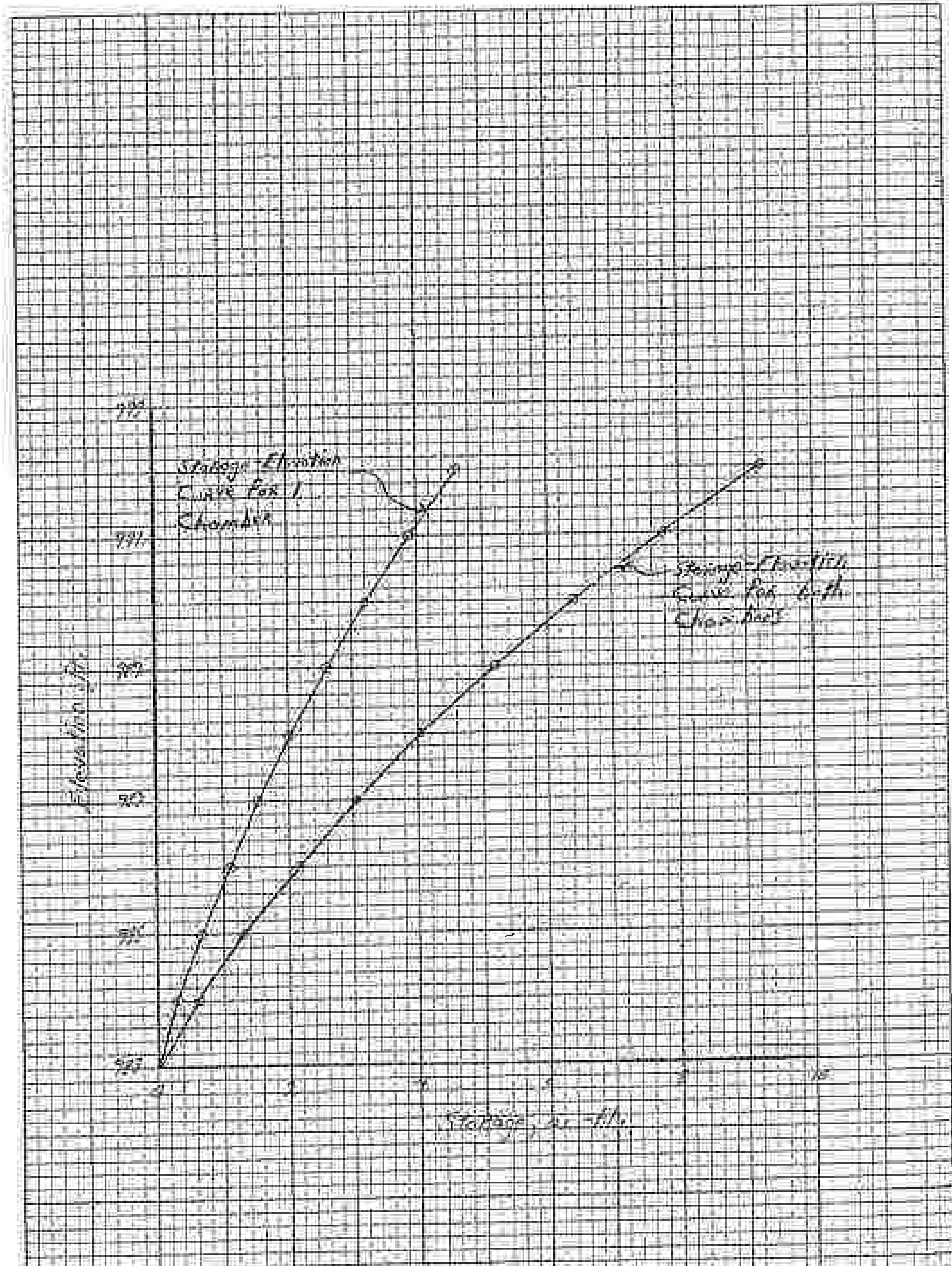
Storage-Elevation Curve
for Equalization Basin

From "Equalization Basin Design" by OB on 7/27/82 sheet 14, the bottom dimensions of each of the two chambers of the equalization pond are as follows:

45' wide
280' long
3:1 side slopes
bottom elev. = 983

Data for storage-elevation curve for 1 chamber

elev. (ft.)	area (ft.^{2})	area (acres)	ave. area (acres)	Δ vol. (ac.-ft.)	cum. vol. (ac.-ft.)	cum. v. for 2 chamb. (ac.-ft.)
983	12,600	0.29			0	0
			0.31	0.31		
984	14,596	0.33			0.31	0.62
			0.355	0.355		
985	16,644	0.38			0.665	1.33
			0.405	0.405		
986	18,774	0.43			1.07	2.14
			0.455	0.455		
987	20,976	0.48			1.525	3.05
			0.505	0.505		
988	23,250	0.53			2.03	4.06
			0.56	0.56		
989	25,596	0.59			2.59	5.18
			0.615	0.615		
990	28,014	0.64			3.205	6.41
			0.67	0.67		
991	30,504	0.70			3.875	7.75
			0.73	0.73		
992	33,066	0.76			4.605	9.21


Storage Elevation Curve for 1 Chamber
Storage Elevation Curve for both Chambers
Elevation, ft.
Storage, ac-ft.
0
1
2
3
4
5

APPENDIX I-1-H

FORM I

STAGE IIC DRAINAGE FACILITIES - DESIGN AND ANALYSIS CALCULATIONS

SUBJECT Keystone [illegible] STATION - FORM I

STAGE II C DRAINAGE

BY SER DATE 3/27/97 PROJ. NO. 92-220-73

CHKD. BY PWC DATE 4/8/97 SHEET NO. i OF i

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APPENDIX I-1-H

TABLE OF CONTENTS

STAGE II C DRAINAGE 8 SHEETS

REF: STAGE II C WORK SHEET 92-220-73-SK II C (IS DRAWING D-728-1055)

CHANNEL FOR STAGE II C 10 SHEETS

CULVERT FOR STAGE II C 11 SHEETS

SUBJECT Keystone Station

BY SER DATE 3/27/97 PROJ. NO. 92-220-73
CHKD. BY PWC DATE 4/3/97 SHEET NO. 1 OF 8

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STAGE II C DRAINAGE

STAGE II C INVOLVES A VERTICAL EXPANSION OF THE CURRENTLY ACTIVE STAGE II B AND TEMPORARILY COVERED STAGE II A, ABOVE THE CURRENTLY PERMITTED ELEVATION(S).

EVALUATE THE DRAINAGE FACILITIES IMPACTED BY THIS DEVELOPMENT AND DESIGN NEW FACILITIES AS NECESSARY, DESIGN EVENT IS 25 YR-24 HR STORM

REFERENCE WORKSHEET 92-220-73-SER IIC
(Drawing D-728-1055)

EXISTING EAST VALLEY EQUALIZATION PONDS

DESIGN PARAMETERS FROM REF. 1
TOTAL AREA = 99.28 AC
ACTIVE AREA = 41.58 AC

48 AC SHOWN ON WORKSH -SER IIC
+2.5 AC FROM REF 2, DRAINAGE
NEAR PONDS

PROPOSED CONDITIONS
TOTAL AREA = 51 TOTAL < 99.28 AC
ACTIVE AREA = 28 AC < 41.88 AC

THE DRAINAGE PATTERNS ARE SIMILAR THEREFORE THE t_c's WILL BE SIMILAR

THEREFORE THE PEAK FLOWS, RUNOFF VOLUMES, AND SEDIMENT VOLUMES WILL NOT BE INCREASED.

SUBJECT KEYSTONE STATION

BY SER DATE 3/27/97 PROJ. NO. 92-220-73

CHKD. BY PWC DATE 4/8/97 SHEET NO. 7 OF 8

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EAST VALLEY HAUL ROAD DIRTY WATER DITCH

ASSUME NORTH PORTION OF ACTIVE SURFACE DRAINS TO HAUL ROAD DWD AND SOUTH PORTION TO SLOPE DRAIN AT SW CORNER (OF STAGE II B)

ASSUME DESIGN ACTIVE SURFACE IS AT ELEV 1320 WHICH IS MAX, ACTIVE SURFACE AREA FOR STAGE II C

DRAINAGE AREA AND CN

SEE REF 2 FOR CN VALUES

	AREA	CN
ACTIVE SURFACE	9.2 AC	85
HAUL ROAD	2.1 AC	85
REV. BENCHES	3.8 AC	78
	15.1 AC = 0.024 MI²	$\overline{CN}$ = 83

$S = \frac{1000}{83} - 10 = 2.0$ IN

ASSUME $t_c = 0.1$ HR, MINIMUM VALUES ON TR-55 UNIT PEAK DISCHARGE GRAPHS, CONSERVATIVE ASSUMPTION SEE REF 3

$P_{25,24} = 4.4$ IN

$I_a/P = \frac{0.2 \cdot S}{P} = \frac{0.2 \cdot 2}{4.4} = 0.09 \cong 0.1$

$q_u = 1000$ CSM/IN, SEE REF 3

SUBJECT Keystone Station

BY SER DATE 3/27/97 PROJ. NO. 92-220-73

CHKD. BY PWC DATE 4/8/97 SHEET NO. 3. OF 8

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$$Q_{25,24} = \frac{(P-0.2\cdot S)^2}{P+0.8\cdot S} = \frac{(4.4-0.2\cdot 2)^2}{4.4+0.8\cdot 2} = 2.7 \text{ IN.}$$

$$\text{PEAK FLOW} = A\cdot Q\cdot q_u = 0.024\cdot 2.7\cdot 1000 = 65 \text{ CFS}$$

ACTUAL FLOW IS LESS
SINCE ACTUAL $t_c > 0.1$ HR

THE CHANNEL IS ANALYZED ON SHEET 4 AND IT HAS SUFFICIENT CAPACITY.

SUBJECT: Keystone Station
Phase II Permitting - Ultimate Conditions
BY: SER DATE: 3/27/97 PROJ. NO.: 92-220-73-07
CHKD. BY: PJC DATE: 4/8/97 SHEET NO. 4 OF 8

GAI CONSULTANTS, INC.
Engineers Geologists Planners
Environmental Specialists

Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right)\cdot a\cdot r^{\left(\frac{2}{3}\right)}\cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right)\cdot (r)^{\left(\frac{2}{3}\right)}\cdot s^{\left(\frac{1}{2}\right)}$

Existing East Valley Haul Road Ditch

Design Flow, $Q_d = 65\cdot ft^3\cdot sec^{-1}$ from sheet 3 of 8

Bottom Width, $b = 2\cdot ft$

Side Slopes, $z = 2$

Channel Lining is Grouted Rock with Manning's roughness coefficient, $n = 0.025$

Channel Minimum Slope, $S_{min} := \frac{25\cdot ft}{250\cdot ft}$ (from Ref. 2) or $S_{min} = 0.1\cdot\frac{ft}{ft}$

Maximum Flow Depth, $d_{max} = 1.088\cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 4.5\cdot ft^2$

Minimum Velocity, $V_{min} = 14.3\cdot ft\cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 6.4\cdot ft$

Freeboard, $F_b = 0.9\cdot ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990

Total depth, $D = 2\cdot ft$ Actual depth of existing channel

Top Width at Total Depth, $T_D = 10\cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 240\cdot ft^3\cdot sec^{-1}$

Channel Maximum Slope, $S_{max} := \frac{25\cdot ft}{250\cdot ft}$ (from Ref. 2) or $S_{max} = 0.1\cdot\frac{ft}{ft}$

Minimum Flow Depth, $d_{min} = 1.088\cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 4.5\cdot ft^2$

Maximum Velocity, $V_{max} = 14.3\cdot ft\cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 6.4\cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 240\cdot ft^3\cdot sec^{-1}$

SUBJECT KEYSTONE STATION

BY SER DATE 3/27/97 PROJ. NO. 92-220-73

CHKD. BY PWC DATE 4/8/97 SHEET NO. 5 OF 8

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SLOPE DRAINS

THE SLOPE DRAIN AT THE SOUTHWEST CORNER OF STAGE II WILL HAVE THE LARGEST DRAINAGE AREA AND THE MOST ACTIVE AREA DRAINING TO IT UNDER STAGE IIC CONDITIONS, THEREFORE IT WILL HAVE THE HIGHEST PEAK RUNOFF FLOWS

THE DESIGN CONDITION IS SHOWN ON WORKSHEET 92-220-73-SERIIC. USE 25-YR, 24 HR STORM EVENT FOR DESIGN

DRAINAGE AREA AND CN

	AREA	CN
ACTIVE SURFACE	16.5 AC	85
REV. BENCHES	3.1 AC	78

TOTAL = 19.6 AC $\overline{CN} = 84$
= 0.031 MI²

$P_{25,24} = 4.4''$

$S = \frac{1000}{84} - 10 = 1.9''$

$Q_{25,24} = \frac{(4.4 - 0.2 \cdot 1.9)^2}{4.4 + 0.8 \cdot 1.9} = 2.7''$

$t_c = 1.1$ HR SEE SHEET 6

$I_a/P = \frac{0.2 \cdot 1.9}{4.4} = 0.1$ HR

$q_u = 340$ csm/in

PEAK FLOW = 0.031 · 2.7 · 340 = 29 CFS

CHANNEL DESIGN/ANALYSIS ON SHEET 7

SUBJECT: Genco - Keystone ~~West Valley~~
Phase II Permitting
BY: SER DATE: 3/28/97 PROJ. NO.: 92-220-73-07
CHKD. BY: PWC DATE: 4/8/97 SHEET NO. 6 OF 8

Time of Concentration Worksheet - SCS Methods
Watershed - Stage 3 West Dirty Water Ditch
Postdevelopment Conditions

Reference: "Urban Hydrology for Small Watersheds", TR-55, Soil Conservation Service, June 1986

SHEET FLOW — Flowpath: a-b — units

1. Surface description (table 3-1) — Fallow
2. Manning's roughness coeff., n_{st} (table 3-1) — $n_{st} := 0.05$
3. Flow length, L_{st} (total $L_{st} \le 150$ feet) — $L_{st} := 300$ feet
4. Two-year, 24-hour rainfall, P_2 — $P_2 := 2.6$ inches
5. Land Slope, $S_{st} := 0.001$ — $S_{st} = 1 \cdot 10^{-3}$

Assume active ash area has a sheet flow n value = 0.05 which is the value for fallow ground.
Assume active ash area slope = 0.1% at head of flowpath and on working surface.
Assume sheet flow length can be maximum of 300 feet on active ash surface.

6. Sheet Flow Time, $T_{st} := \frac{0.007 \cdot (n_{st} \cdot L_{st})^{0.8}}{P_2^{0.5} \cdot S_{st}^{0.4}}$ — $T_{st} = 0.6$ hours

SHALLOW CONCENTRATED FLOW — Flowpath: b-c

7. Surface description (paved or unpaved) — unpaved
8. Flow length, L_{sc} — $L_{sc} := 920$ feet
9. Watercourse Slope, $S_{sc} := 0.001$ — $S_{sc} = 1 \cdot 10^{-3}$
10. Average Velocity, $V_{sc} := 16.1345 \cdot S_{sc}^{0.5}$ — $V_{sc} = 0.51$ fps
11. Shallow Conc. Flow time, $T_{sc} := \left(\frac{L_{sc}}{3600 \cdot V_{sc}}\right)$ — $T_{sc} = 0.5009$ hour

neglect time of flow in slope drain

Total Watershed Time-of-Concentration, $T_c := T_{st} + T_{sc}$

$T_c = 1.101$ hour

TCSWSD.MCD, 3/28/97, 10:26 AM, 1

SUBJECT: Keystone Station
Phase II Permitting
BY: SER DATE: 3/27/97 PROJ. NO.: 92-220-73-07
CHKD. BY: AWC DATE: 4/8/97 SHEET NO. 7 OF 8

GAI CONSULTANTS, INC.
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Purpose: Ditch Design

Methodology: Manning's Equation, $Q := \left(\frac{1.49}{n}\right)\cdot a\cdot r^{\left(\frac{2}{3}\right)}\cdot s^{\left(\frac{1}{2}\right)}$ or $V := \left(\frac{1.49}{n}\right)\cdot (r)^{\left(\frac{2}{3}\right)}\cdot s^{\left(\frac{1}{2}\right)}$

Existing East Valley Slope Drains

Design Flow, $Q_d = 29\cdot ft^3\cdot sec^{-1}$ from sheet 5 of 8

Bottom Width, $b = 4\cdot ft$ *

Side Slopes, $z = 2$ *

Channel Lining is Fabric Formed Grout, USM* with Manning's roughness coefficient, $n = 0.015$

Channel Minimum Slope, $S_{min} := \frac{1\cdot ft}{100\cdot ft}$ or $S_{min} = 0.01\cdot\frac{ft}{ft}$ MINIMUM SLOPE ACROSS BENCHES

Maximum Flow Depth, $d_{max} = 0.767\cdot ft$ from solution of Manning's Equation

Flow Area at Maximum Flow Depth, $a_{max} = 4.2\cdot ft^2$

Minimum Velocity, $V_{min} = 6.8\cdot ft\cdot sec^{-1}$ from Manning's Equation

Top Width at Maximum Flow Depth, $T_{max} = 7.1\cdot ft$

Freeboard, $F_b = 1.2\cdot ft$ by the method recommended in the PaDER Erosion and Sediment Pollution Control Program Manual, April 1990

Total depth, $D = 2\cdot ft$ * Actual depth of existing channel

Top Width at Total Depth, $T_D = 12\cdot ft$

Capacity at Total Depth and Minimum Slope, $Q_{tmin} = 183\cdot ft^3\cdot sec^{-1}$

Channel Maximum Slope, $S_{max} := \frac{1\cdot ft}{2\cdot ft}$ or $S_{max} = 0.5\cdot\frac{ft}{ft}$ EXISTING E.V. SLOPES BETWEEN BENCHES

Minimum Flow Depth, $d_{min} = 0.251\cdot ft$ from solution of Manning's Equation

Flow Area at Minimum Flow Depth, $a_{min} = 1.1\cdot ft^2$

Maximum Velocity, $V_{max} = 25.7\cdot ft\cdot sec^{-1}$ from Manning's Equation

Top Width at Minimum Flow Depth, $T_{min} = 5\cdot ft$

Capacity at Total Depth and Maximum Slope, $Q_{tmax} = 1\cdot 10^3\cdot ft^3\cdot sec^{-1}$

* CHANNEL SECTION AS PER DRAWING 41-D-0263
MATERIAL USED IS UNIFORM SECTION MAT AS PER KHK

SUBJECT: KEYSTONE STATION

BY: SER DATE: 3/26/97 PROJ. NO. 92-220-73-07

CHKD. BY: PWC DATE: 4/8/97 SHEET NO. 8 OF 8

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REFERENCES

1) "EQUALIZATION BASIN DESIGN" CALC BY DB 9/21/82, 78-505-411

2) "ULTIMATE CONDITIONS - DRAINAGE FACILITIES" CALC BY SER 3/19/96, 92-220-73-07

3) TR-55, "URBAN HYDROLOGY FOR SMALL WATERSHEDS", US SCS, JUNE 1986

4) HDS-5, "HYDRAULIC DESIGN OF HIGHWAY CULVERTS", FHWA SEPT. 1985

5) "DIRTY WATER DITCHES AND RELATED FACILITIES" CALC BY SER 5/21/96

SUBJECT KEYSTONE STATION - FORM I
ENLARGED STAGE II C - DRAINAGE
BY KNB/SRR DATE 12/24/97 / 2/4/98 PROJ. NO. 92-220-73-07
CHKD. BY JAN DATE 2/19/98 SHEET NO. 1 OF 10

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Channel for Stage II C

ESTIMATE THE REQUIRED SIZE OF REVERSED* BENCH CHANNEL NEEDED TO CONVEY RUNOFF FROM THE ENLARGED STAGE II C AREA TO THE EXISTING EAST VALLEY HAUL ROAD DITCH.

THE ACTIVE DISPOSAL AREA WILL BE AT ITS LARGEST AT THE LOWEST ELEVATION (NEXT PAGE). AREA IS ≈ 17 acres

ALSO, CONSIDER DRAINAGE ABOVE THE BENCH USED TO CONVEY WATER. THE BENCH WILL BEGIN AT THE NORTH EDGE OF ENLARGED STAGE II C, TRAVEL PAST THE SLOPE DRAIN, AND OUTLET INTO THE HAUL ROAD DITCH (NEXT PAGE)

APPROXIMATE BENCH LENGTH = 3200 FT

APPROXIMATELY 50' HORIZONTAL SEPARATES TYPICAL BENCHES. TO GET AN ESTIMATE OF BENCH DRAINAGE AREA, MULTIPLY THE BENCH LENGTH BY THIS 50'.

AREA = 3200' × 50' = 160,000 ft² = 3.7 acre

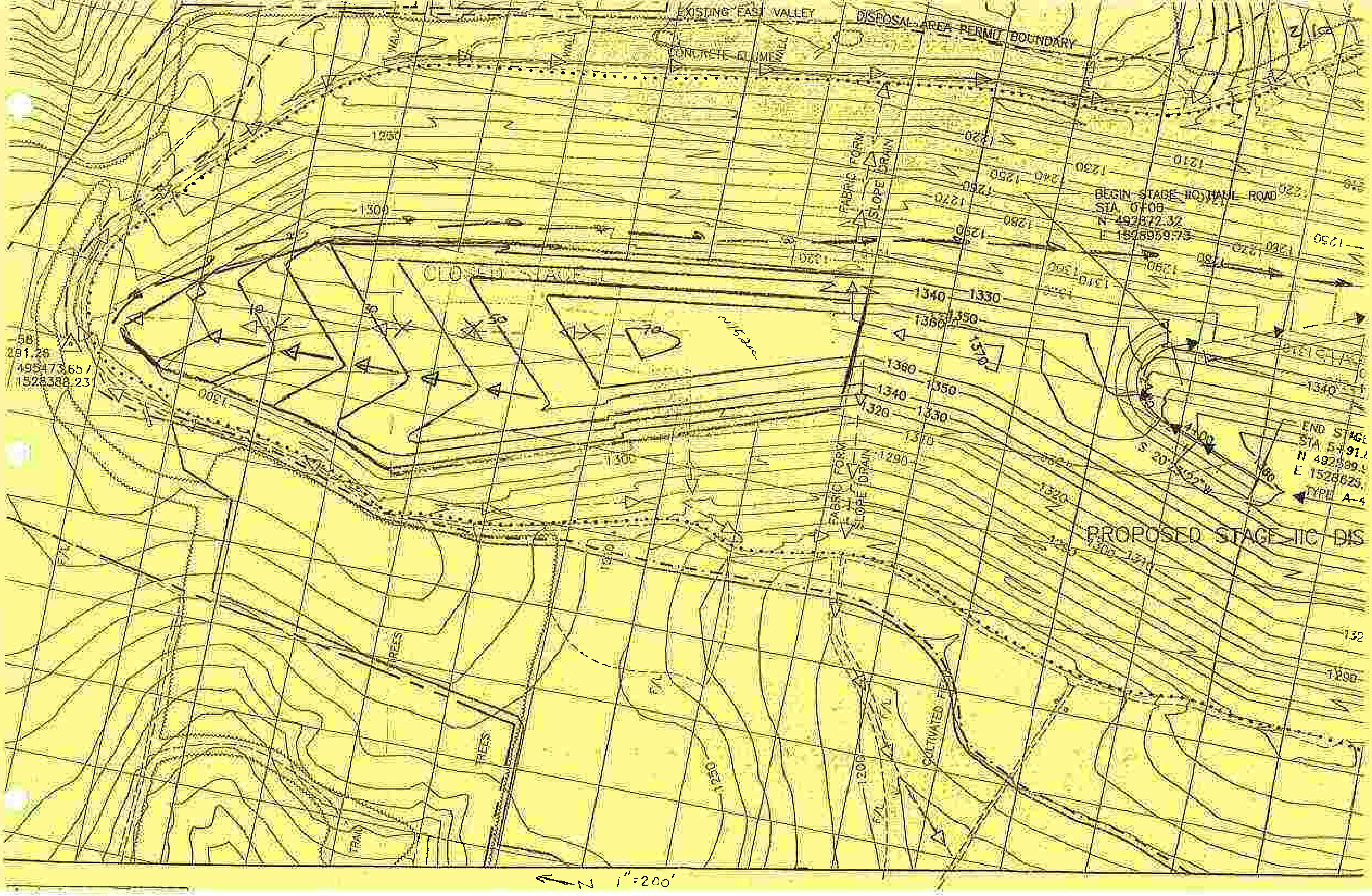
SAY 21 TOTAL ACRES OF WATERSHED.

ASSUME THE 17 ACRES ARE ACTIVE DISPOSAL (CN = 85) AND THE 4 ACRES ARE REVEGETATED (CN = 78)

$$\text{COMPOSITE CN} = \frac{(17 \times 85) + (4 \times 78)}{21} = 83.7$$

THE 25-YEAR STORM WILL BE THE DESIGN STORM. THE 25-YR PRECIPITATION IS 4.4 INCHES (24-hour)

* THE SLOPE OF AN EXISTING PERMITTED BENCH WILL BE REVERSED TO DRAIN TO THE EAST VALLEY HAUL ROAD DITCH.



EXISTING EAST VALLEY DISPOSAL AREA PERMIT BOUNDARY
CONCRETE FLUME
WALL
CLOSED STAGE I
FABRIC FORM SLOPE DRAIN
BEGIN STAGE IIC HAUL ROAD
STA. 0+00
N 492372.32
E 1528959.73
END STAGE
TYPE A-4
PROPOSED STAGE IIC DIS
CULTIVATED
TREES
1"=200'

SUBJECT KEYSTONE STATION – FORM I, Appendix I-1-H
Enlarged Stage II C – Drainage

BY KMB/KGC DATE 12/26/97 / 11/4/98 PROJ. NO. 92-220-73-07

CHKD. BY JMJ DATE 2/17/98 / 11/5/98 SHEET NO. 3 OF 10

REVISED BY KGC 11/3/98

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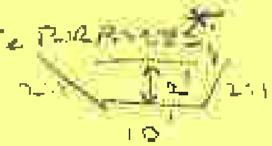
APPROXIMATE THE TIME-OF-CONCENTRATION FOR ENLARGED STAGE II C.
BECAUSE THE DISPOSAL SURFACE IS NOT SET, AND WILL BE ACTIVE, ASSUME 0.1 hour FOR t_c FROM THE DISPOSAL AREA. (MIN. t_c VALUE IN TR-55)

ALSO ALLOW FOR TRAVEL ALONG THE BENCH. ASSUME:

ASSUME FOR t_c DRAINAGE* CHANNEL →



SLOPE 1%
$n = 0.04$

$$A = 10 \times 2 + 2 \times 2 \times 2 = 28 \text{ ft}^2$$
$$P = 10 + 2\left[\sqrt{2^2 + 4^2}\right] = 18.9 \text{ ft}$$
$$R = \frac{A}{P} = 1.5 \text{ ft}$$

$$V = \frac{1.49}{n} R^{2/3} S^{1/2} = \frac{1.49}{0.04} 1.5^{2/3} 0.01^{1/2} = 4.9 \text{ ft/s}$$

$$\text{TRAVEL TIME} = \frac{3200 \text{ ft}}{4.9 \text{ ft/s}} = 653 \text{ sec} = 10.9 \text{ min} = 0.2 \text{ hr}$$

Use t_c = 0.3 hour

PEAK FLOW = 60 cfs (from next page)

* FINAL DESIGN DOES NOT INCLUDE A DRAINAGE CHANNEL AT BACK OF BENCH (SEE SHEET 5 & 7), HOWEVER ASSUME t_c ESTIMATE ON SHEET IS VALID REGARDLESS OF FINAL DESIGN (T_c WOULD NOT GET SHORTER).

C:\>b:

B:\>cd psuhm

B:\PSUHM>uhm

B:\PSUHM>ECHO OFF

SHEET 4 OF 10

****

* PSUHM: MODULE <3-B> - SCS TR-55 TABULAR METHOD *

****

WATERSHED TITLE: Keystone Stage IC

25 YR. TYPE II STORM: PRECIPITATION = 4.4 in.

===

SUMMARY OF INPUT PARAMETERS

SUBAREA	AREA (acre)	CURVE NUMBER	IA/P	RUNOFF (in)	TC (hrs)	ADJ. TC (hrs)	TT (hrs)	ADJ. TT (hrs)
1	21.000	84	0.100	2.70	0.300	0.300	0.000	0.000
COMPOSITE	21.000	84		2.70				

INDIVIDUAL SUBAREA & COMPOSITE HYDROGRAPHS

SUBAREA	TIME (hrs) 11.0	11.9	12.2	12.5	12.8	13.2	13.6	14.0	15.0	17.0	20.0	26.0
1	1.8	10.5	59.9	25.1	10.1	5.8	4.5	3.7	2.7	1.8	1.2	0.0
COMPOS.	1.8	10.5	59.9	25.1	10.1	5.8	4.5	3.7	2.7	1.8	1.2	0.0

THE PEAK FLOW IS 59.9 cfs - OCCURS AT 12.2 hrs

===

SUBJECT KEYSTONE STATION - FORM I Appendix I-H
Stage II C Drainage

BY KNB DATE 12/26/97 PROJ. NO. 92-220-73-7
CHKD. BY JMJ DATE 2/17/98/4-5-98 SHEET NO. 5 OF 10
REVISED SEN 11/5/98

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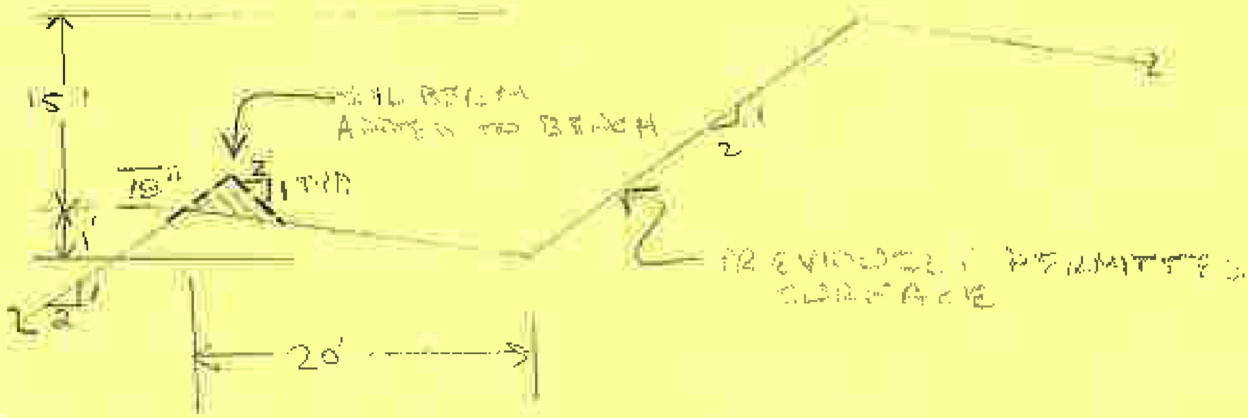
SIZE A BERM TO BE LOCATED ON A BENCH SUCH THAT THE BENCH CAN PASS THE PEAK FLOW (60 cfs) WITH 0.5' FREEBOARD. THE BENCH LAYOUT IS:



BENCH GEOMETRY
N.T.S.

THE BENCH IN QUESTION WILL BE SLOPED AT ≈ 0.8% FROM THE HIGH POINT OF EL. 1298 TO EL 1275 AT THE HAUL ROAD.

DETERMINE IF AN 18" HIGH BERM WILL PASS THE FLOW.

SUBJECT Keystone

BY [illegible] DATE 11/4/98 PROJ. NO. [illegible]

CHKD. BY [illegible] DATE 11/5/98 SHEET NO. 6 OF 10

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THIS SHEET IS INTENTIONALLY LEFT BLANK

SUBJECT Keystone [illegible] - [illegible]
Stage II C, Bench Drainage

BY [illegible] DATE [illegible] PROJ. NO. 92-220-73-7

CHKD. BY JMN DATE [illegible] SHEET NO. 7 OF 10

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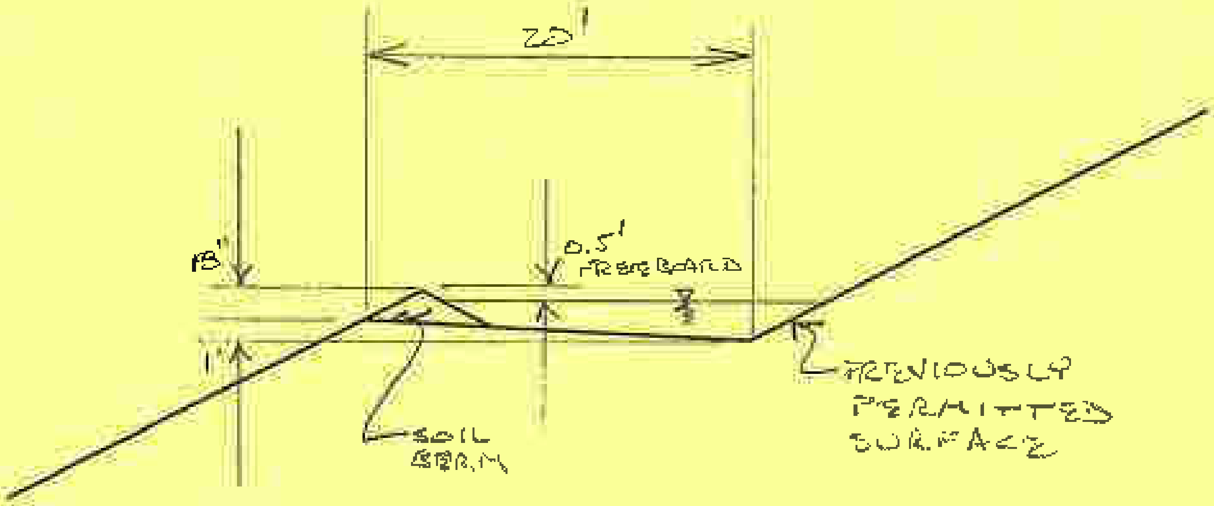
Reverse Bench Drainage Configuration:



BENCH AND BERM SECTION

SCALE 1" = 10'

SUBJECT KEYSTONE S[illegible] - FORM I-1-H

BY SER DATE 11/4/98 PROJ. NO. 92-220-73-07

CHKD. BY JMJ DATE 11/5/98 SHEET NO. 8 OF 10

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USE MANNING'S EQUATION TO ESTIMATE BENCH CAPACITY CONSIDERING 0.5 FEET FREEBOARD.

$A = 28\ FT^2$ FROM SHEET 7

P = WETTED PERIMETER = 21 FT FROM SHEET 7

$R = A/P = 1.33$

A & P MEASURED BY AUTOCAD, USED FOR SHEET 7

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

$n = 0.04$ AS BEFORE SEE SHEET 3

$S = 0.008$ FT/FT = 0.8% SEE SHEET 5

$$Q = \frac{1.49}{0.04} \cdot 28 \cdot 1.33^{2/3}\ 0.008^{1/2} = 111\ CFS > 60\ CFS$$

∴ OK

SUBJECT KEYSTONE STATION - FORM I-1-H

BY SRC DATE 11/5/98 PROJ. NO. 97-220-[illegible]

CHKD. BY JMJ DATE 11/5/98 SHEET NO. 9 OF 10

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THIS SHEET IS INTENTIONALLY LEFT BLANK

SUBJECT KEYSTONE STATION

BY KMB/SER DATE 11/22/95/3/17/97 PROJ. NO. 92-720-73-07

CHKD. BY JMJ DATE 3/17/98 SHEET NO. 1E OF 10

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EXISTING ROAD DITCH

THE EXISTING ROAD DITCH HAS BEEN DESIGNED FOR A FLOW OF 65 cfs, WITH A FULL-FLOW CAPACITY OF 240 cfs (SEE CALCS BY SER, DATED 3/27/97, 8 PAGES STAGE II - DRAINAGE)

THE CHANNEL DIMENSIONS AS DESIGNED WERE

BOTTOM WIDTH = 2 ft
SIDE SLOPES = 2:1
DEPTH = 2 ft
SLOPE = 10 %
MANNING'S n = 0.025

CHECK THE CAPACITY OF THE CHANNEL WITH A REDUCED TOTAL DEPTH OF 1.5 FT : (CONSIDERING 0.5' FREEBOARD)

AREA = $2\left[\frac{1}{2} \times 1.5 \times 3\right] + [2 \times 1.5] = 7.5\ ft^2$

PERIMETER = $2 + 2\left[\sqrt{3^2 + 1.5^2}\right] = 8.7\ ft$

$R = \frac{A}{P} = 0.86$

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2} = \left(\frac{1.49}{0.025}\right)(7.5)(0.86^{2/3})(0.1)^{1/2}$$

$$= 128\ cfs$$

THE PEAK FLOW CONSIDERING MAX. FLOW INTO EXISTING DITCH PLUS MAX FLOW ALONG REGRADED BENCH =

65 cfs (EXISTING) + 60 cfs (REGRADED) = 125 cfs

CAPACITY IS OK

SUBJECT KEYSTONE STATION
ENLARGED STAGE II C — (CULVERT DESIGN)

BY [illegible] DATE 1/20/98 PROJ. NO. 92-220-73-07

CHKD. BY JMJ DATE 2/17/98 SHEET NO. 1 OF 11

REVISED BY SER 5/19/98

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Culvert for Stage II C

SIZE AND LAY OUT THE CULVERT USED TO CARRY FLOW FROM THE REVERSE GRADE BENCH, UNDER THE HAUL ROAD, AND TO THE EXISTING DITCH ON THE FAR SIDE OF THE HAUL ROAD. MAINTAIN A MINIMUM 4' OF COVER UNDER THE ROAD.

PRELIMINARY HYDROLOGY CALCS PRODUCED AN ESTIMATE OF 60 cfs INTO THE BENCH.

CHECK INLET CONTROL + OUTLET CONTROL HEAD REQUIREMENTS FOR A PIPE OF AN ASSUMED LENGTH. USE 24", 30", AND 36" CORRUGATED METAL PIPES.

INLET / OUTLET CONDITIONS

THE INLET TO THE CULVERT WILL BE A DROP BOX. WATER WILL BE ALLOWED TO POOL UP INSIDE THE CONCRETE BOX — IT WILL FUNCTION AS A HEADWALL.

AT THE OUTLET OF THE CULVERT, A CONCRETE BOX WILL BE USED BOTH AS AN ENERGY DISSIPATOR AND AS A TRANSITION BACK TO THE ROAD DITCH.

SUBJECT Keystone Station

BY KMB DATE 1/20/98 PROJ. NO. 92-220-73-07

CHKD. BY JMW DATE 2/17/98 SHEET NO. 2 OF 11

REV'D BY SZR 5/11/98

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CULVERT ANALYSIS

INLET CONTROL

THE CHART ON THE NEXT PAGE SHOWS THE INLET CONTROL HEADWATER CHART FOR A CORRUGATED METAL PIPE.

24" 60 cfs → HW/D >> 6

30" 60 cfs → HW/D = 3.05 ⇒ HW = 7.6 ft

36" 60 cfs → HW/D = 1.6 ⇒ HW = 4.8 ft ↙

42" 60 cfs → HW/D = 1.12 ⇒ HW = 3.9 ft

THE 36" PIPE APPEARS TO BE THE BEST OPTION COMBINING HEADWATER AND PIPE DIAM.

PIPE INLET INVERT ELEV = 1266.0

INLET CONTROL HW ELEV = 1266 + 4.8 = 1270.8 FT

ALLOWABLE HW ELEV = 1275.2 ∴ OK

SUBJECT Keystone Station

BY SEIR DATE 5/11/98 PROJ. NO. 92-220-73-7

CHKD. BY JMJ DATE 5/20/98 SHEET NO. 3 OF 11

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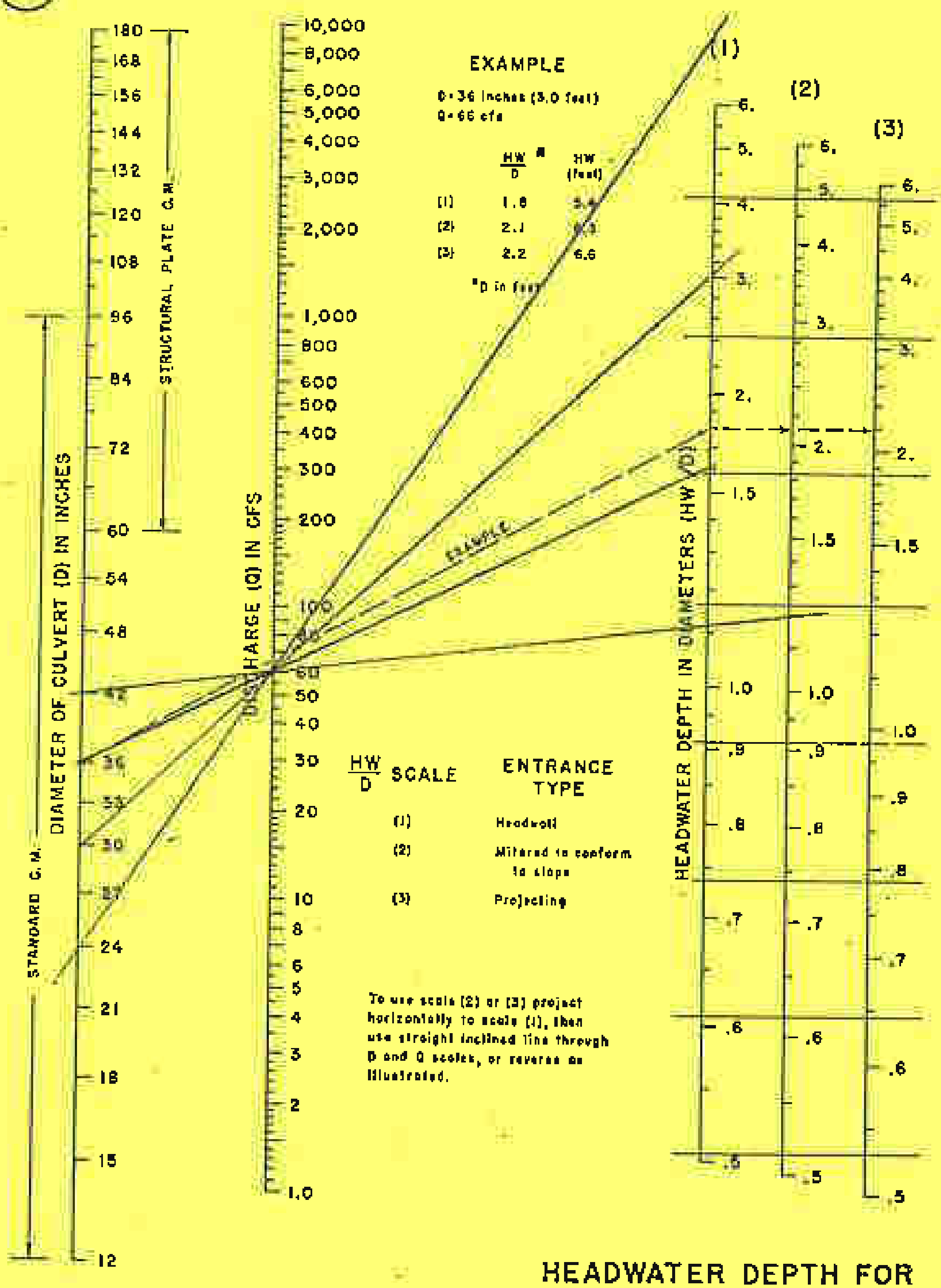
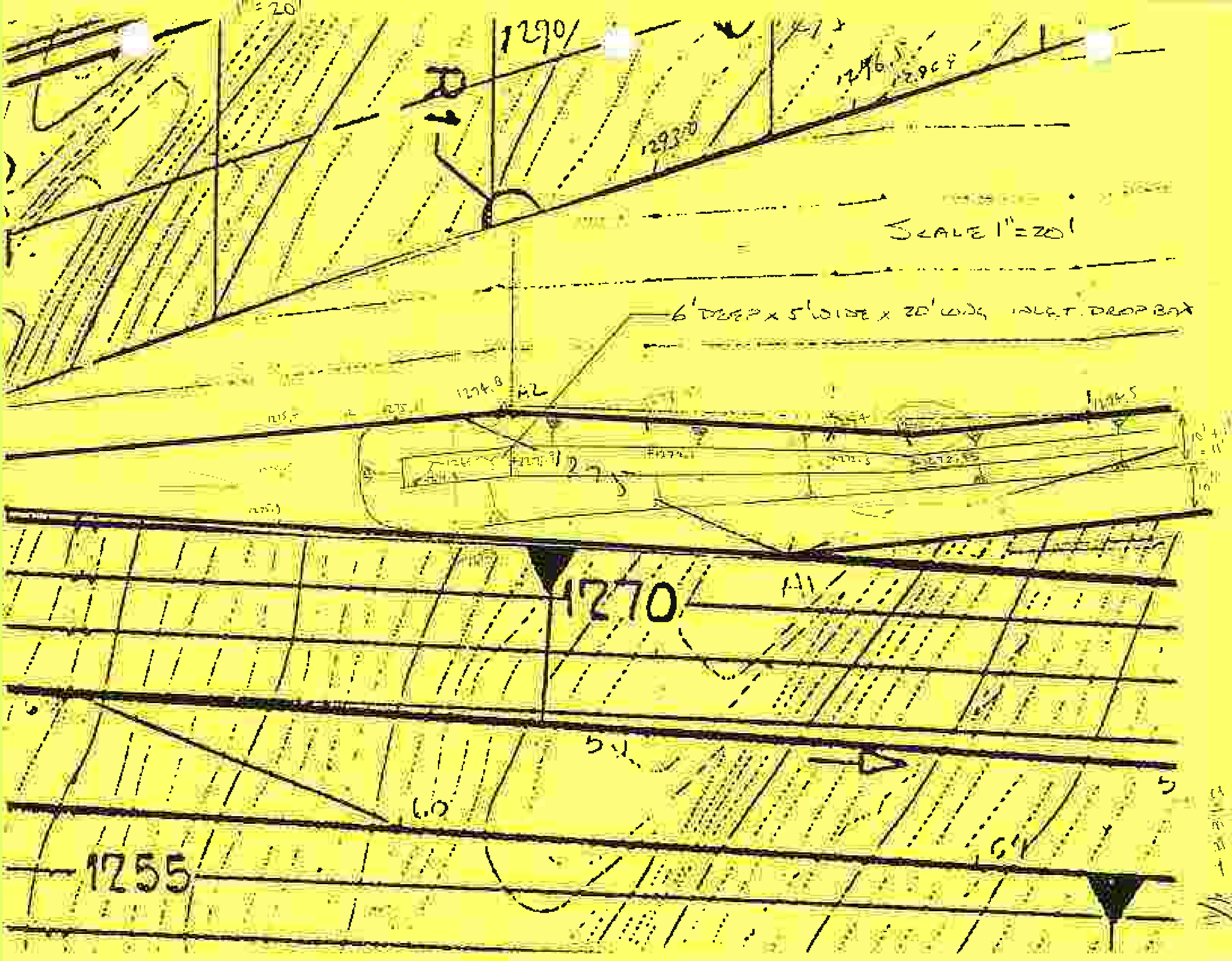
CHART 2



HEADWATER DEPTH FOR
C. M. PIPE CULVERTS
WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN. 1963



1290
R1
1293.0
SCALE 1"=20'
6' DEEP x 5' WIDE x 20' LONG INLET DROP BOX
1274.8
A2
1270
A1
1255

SUBJECT KEYSTONE STATION

BY KMB DATE 1/20/98 PROJ. NO. 92-220-73-07

CHKD. BY JMJ DATE 5/7/98 SHEET NO. 5 OF 11

REVISED BY SBR 5/17/98

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CULVERT ANALYSIS

OUTLET CONTROL

THE FOLLOWING EQUATION (FROM HDS-5) WILL BE USED TO DETERMINE NECESSARY HEAD TO PASS FLOW DURING OUTLET CONTROL.

$$H = \left[1 + K_e + (29 n^2 L) / R^{1.33}\right] \frac{V^2}{2g}$$

H = HEAD REQUIRED ABOVE OUTLET TAILWATER

K_e = ENTRANCE COEFFICIENT
USE $K_e = 0.5$ FOR SQUARE EDGED HEADWALL

n = MANNING'S ROUGHNESS COEFFICIENT
$n = 0.024$ FOR CMP (FROM HDS-5)
(THE PIPE MAY BE LINED; STILL, USE 0.024)

L = PIPE LENGTH USE 205' (SEE SKETCH NEXT SHEET)

A = 7.07 SQ FT FOR 36" ∅ PIPE

R = D/4 = 0.75' FOR 36" ∅ PIPE

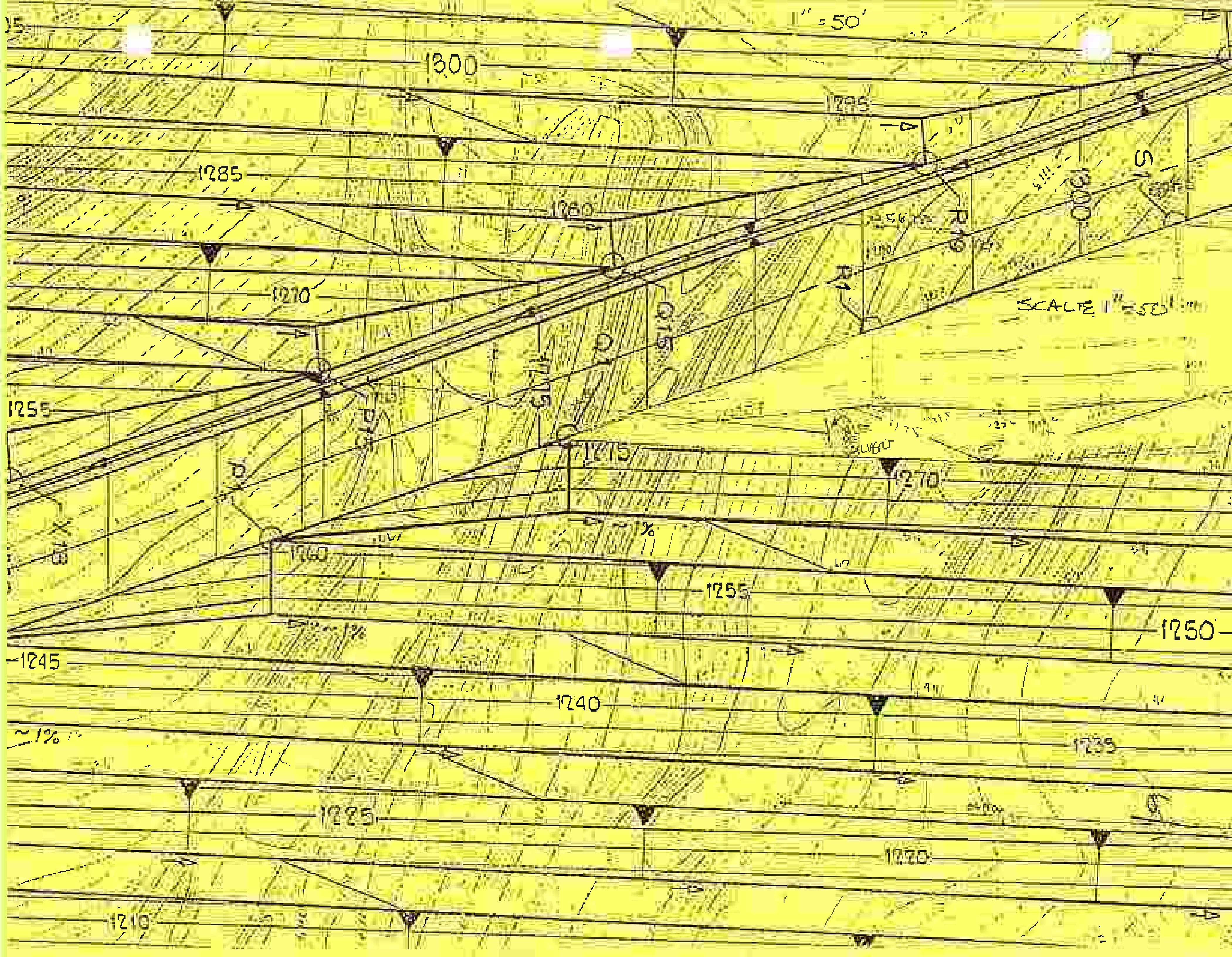
$$H = \left(1 + 0.5 + \frac{29(0.024)^2 \cdot 205'}{0.75^{1.33}}\right) \frac{60^2}{7.07^2} \frac{1}{2g}$$

H = 7.3 FT

ALLOWABLE HW ELEV = 1075.2

∴ ALLOWABLE TW ELEV = 1075.2 - 7.3 = 1067.9

THIS IS A MAXIMUM VALUE ALLOWABLE, ACTUAL TW IS ESTIMATED BELOW.


1" = 50'
1300
1295
1285
1280
1270
S1
R19
R1
Q15
Q1
SCALE 1" = 50'
1255
1275
1270
~1%
X13
1260
1255
1250
1245
1240
~1%
1235
1225
1220
1210

7/11

TABLE 12 - ENTRANCE LOSS COEFFICIENTS

Outlet Control, Full or Partly Full Entrance head loss

$$H_e = k_e \left(\frac{V^2}{2g} \right)$$

Type of Structure and Design of Entrance	Coefficient k_e
Pipe, Concrete	
Projecting from fill, socket end (groove-end)	0.2
Projecting from fill, sq. cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square-edge	0.5
Rounded (radius = 1/12D)	0.2
Mitered to conform to fill slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side-or slope-tapered inlet	0.2
Pipe, or Pipe-Arch, Corrugated Metal	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side-or slope-tapered inlet	0.2
Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of 1/12 barrel dimension, or beveled edges on 3 sides	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of 1/12 barrel dimension, or beveled top edge	0.2
Wingwall at 10° to 25° to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side-or slope-tapered inlet	0.2

*Note: "End Section conforming to fill slope," made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections, incorporating a closed taper in their design have a superior hydraulic performance. These latter sections can be

FROM "HYDRAULIC DESIGN OF HIGHWAY CULVERTS"

SUBJECT KEYSTONE STATION

BY SER DATE 5/11/98 PROJ. NO. 92-220-75-7

CHKD. BY JMJ DATE 5/20/98 SHEET NO. 8 OF 11

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ESTIMATE FLOW DEPTHS AT VARIOUS SECTIONS ALONG OUTLET CHANNEL SYSTEM WHICH IS SHOWN IN ELEVATION AND PLAN ON SHEET 4

SECTION A DOWNSTREAM OF BREAK IN SLOPE FROM 2% TO 10%

SLOPE = 10%

GROUTED RIPRAP LINING
n = 0.025

SECTION A | B
NTS

TOTAL FLOW IN CHANNEL SYSTEM DOWNSTREAM OF PIPE IS 125 CFS SEE SHEET 10 OF 10 CALC BY KMB/SER 12/14/97/5/11/98 "ENLARGED STAGE IC DRAINAGE"

FLOW DEPTH = 1.484 FT BY SOLUTION OF MANNING'S

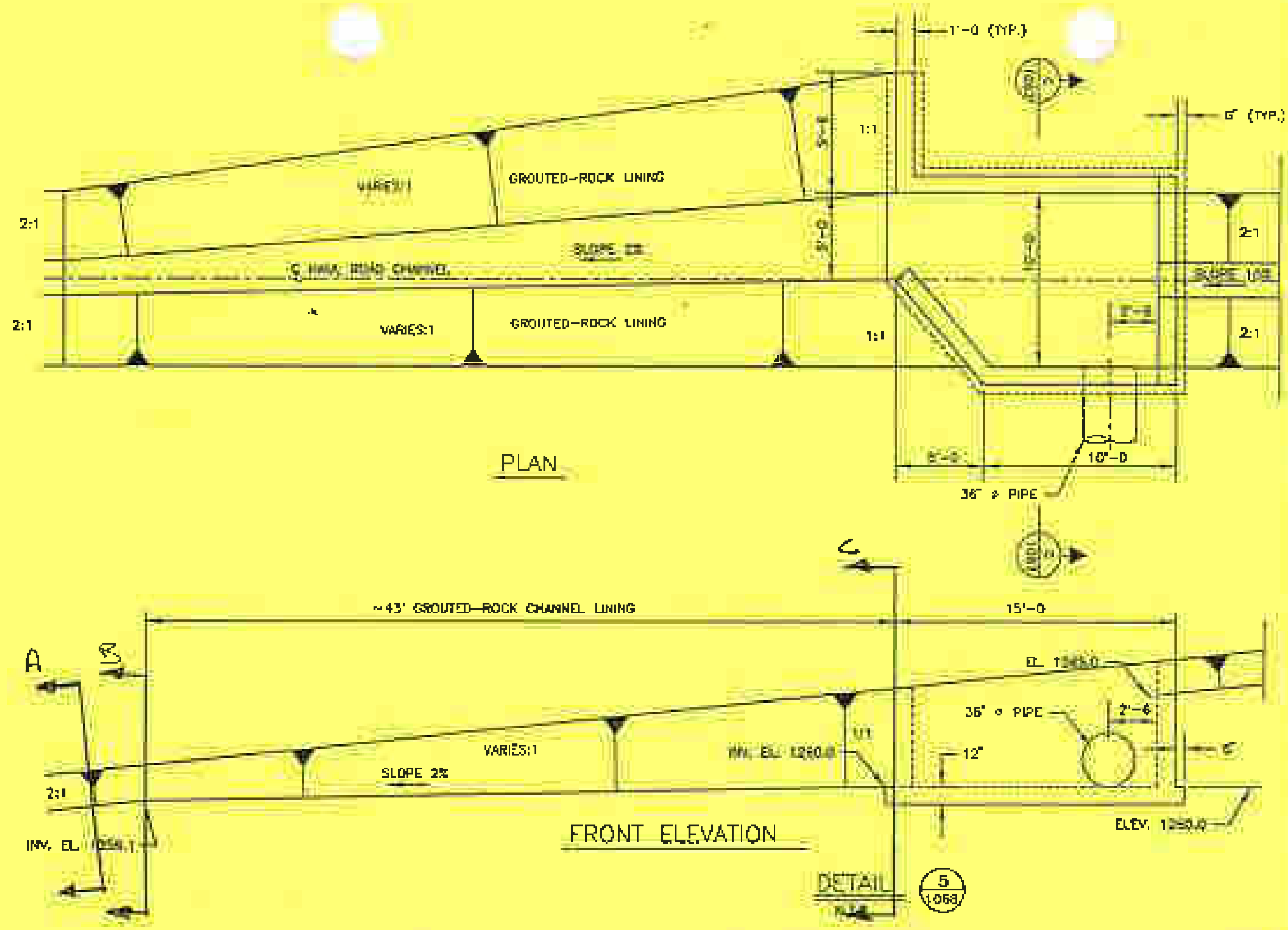
$A = 7.373\ FT^2$

$WP = 8.637\ FT$

$R = 0.854\ FT$

$V = 16.96\ FPS$

$F = V/\sqrt{gd} = 2.4$ ∴ FLOW IS SUPERCRITICAL



CULVERT NO. 11 CONCRETE OUTLET BOX AND GROUTED-ROCK CHANNEL

FROM CAD DRAWING 92-…-F4024

SHEET 9/11

SUBJECT Keystone Station

BY SER DATE 5/11/98 PROJ. NO. 92-220-75-7

CHKD. BY JMN DATE 5/20/98 SHEET NO. 10 OF 11

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SECTION B SAME AS SECTION A EXCEPT SLOPE = 2%

FLOW DEPTH = 2.134 FT BY SOLUTION OF MANNING'S
ACTUAL DEPTH WILL BE LESS

$A = 13.424\ FT^2$

$WP = 11.566\ FT$

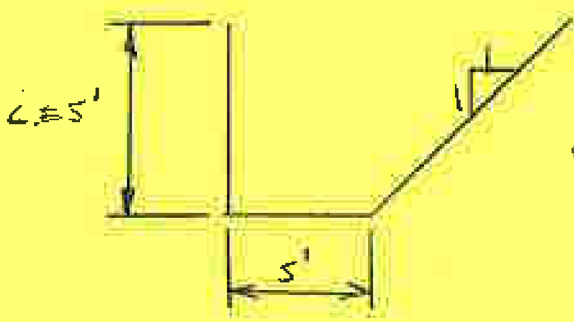
$R = 1.161\ FT$

$V = 9.311\ FPS$

$F = 1.122$ ∴ FLOW IS SUPERCRITICAL

SECTION C

SLOPE = 2%



GROUTED RIPRAP LINING

SECTION C
NTS

FLOW DEPTH = 2.067 FT BY SOLUTION OF MANNING'S

$A = 12.471\ FT^2$

$WP = 9.622\ FT$

$R = 1.296\ FT$

$V = 10.02\ FPS$

$F = 1.23$ ∴ FLOW IS SUPERCRITICAL

SINCE FLOWS DOWNSTREAM OF SECTION C WILL BE SUPERCRITICAL, FLOW MUST THROUGH CRITICAL SOMEWHERE NEAR SECTION C WHICH WILL ACT AS A WEIR

USE WEIR EQUATION

$$Q = CLH^{3/2}$$

$Q = 125$ cfs

$C = 3.1$ CONSERVATIVELY, SUBMERGENCE CONSIDERING, TRIANGULAR FLOW NOT CONSIDERED IN WEIR EQUATION

$L = 5$ FT

$$H^{3/2} = \frac{Q}{CL} = \frac{125}{3.1 \cdot 5} = 8.06$$

$H = 4.0$ FT

THIS FLOW DEPTH WILL BE CONTAINED WITHIN OUTLET SYSTEM AND IS LESS THAN THE ALLOWABLE TWO

APPENDIX B

Calculations from July 2013 Stage IV Leachate Improvements, Form I Supplemental Calculations

Purpose:

To design appropriate additional site drainage features as part of the 2013 Stage IV Minor Permit Modification Application for the Keystone Station Disposal Site. Specifically, this calculation package will serve as a supplement to Form I Appendix I-1-A (Reference 1 - identified below) and provides design of additional culverts and a channel which are needed for installation of a leachate pipe cleanout access road adjacent to previously designed and permitted diversion and collection channels (identified on the "Permit Drawings"). Calculations have been provided for each proposed drainage feature and follow this Narrative. All calculations have been completed in accordance with the PaDEP Solid Waste Regulations, as applicable.

References:

1. Form I in Volume 4 of PaDEP Residual Waste Major Permit Modification: Keystone Station Disposal Site, dated July 1996;
2. PA Erosion and Sediment Control BMP Manual; and
3. PaDEP Solid Waste Management Regulations.

List of Figures:

List of Tables:

List of Attachments:

Introduction:

The Keystone Station Disposal Site is located in Shelocta, Pennsylvania and is currently utilized under Solid Waste/NPDES Permit No. 300837, with mine refuse and coal combustion byproducts (CCB's) from the Keystone Generating Station currently being placed in the Stage III portion of the disposal area. Storm water runoff from all active disposal areas drains to an on-site sedimentation pond and is routed away from nearby waterways. Storm water runoff from vegetated and/or reclaimed areas is permitted to drain to nearby waterways provided that adequate erosion control measures are provided. This approach of site drainage development will continue throughout the life of the site. This calculation package provides the technical basis for modifying the current Form 3R "Permit Drawings" and Form I as further discussed below.

The current "Stage IV Permit Drawings" in Form 3R did not provide for a maintenance road to access the Stage IV leachate collection and detection pipe cleanouts and did not detail drainage features needed for an access road. This calculation set will be used as the design basis to revise the current "Stage IV Permit Drawings" to add access roads and associated drainage details.

Design Considerations:

1. The original Form I Calculations for Stage III and Stage IV drainage features are valid as presented in the PaDEP Residual Waste Major Permit Modification for the Keystone Station Disposal Site: Volume 4, Dated July 1996.
2. Proposed new culverts will be designed to convey the 25-year, 24-hour storm event as identified in the original Permit Form I Calculations.
3. Proposed culverts identified in Table 1 have been designed per the culvert design parameters defined on the following page.
4. Currently permitted channel design will remain unchanged as proposed site features will result in a slightly reduced drainage area. Specifically, the design calculations for the currently permitted southwest diversion channel along the Stage IV toe were not revised due to the relatively minor reduction of drainage area due to the newly proposed "outer diversion" channel adjacent to the proposed cleanout access road.
5. Actual culvert, channel and/or grading alignments (as shown on Figures 1 and 2) are subject to minor change based on future construction layout.

Analysis:

The culverts and channel identified in the attached Tables 1 and 2 have been designed per the parameters defined below. The following parameters are the governing hydraulic case for each proposed culvert and channel, respectively.

A: Culvert Design Parameters

Culvert ID	Storm Event	Peak Flow[1]
-	x-year, 24-hour	cfs
Proposed Culvert 12	25	91.0
Proposed Culvert 13	25	69.0
Proposed Culvert 14	25	90.0
Proposed Culvert 15	25	90.0
Proposed Culvert 16	25	51.0
Proposed Culvert 17	25	91.0
Proposed Culvert 18	25	6.3
Proposed Culvert 19	25	5.6

Notes:

1. The entire original Permit Form I calculations are not included within this calculation package as peak flows for Proposed Culverts 12 - 17 correspond to "permitted channels" identified in various locations in the Form I Calculations.

B: Channel Design Parameters

Channel ID	Storm Event	Peak Flow
-	x-year, 24-hour	cfs
Proposed Stage IV Southwest Access Road Diversion Ditch	25	0.4

Analysis and Design Summary:

The calculations required to adequately size the drainage features have been included as the Attachments. Erosion protection measures based on discharge velocities and shear stresses are included with each proposed drainage design. Proposed culvert and channel schedules have been provided in Tables 1 and 2, respectfully.

Conclusion:

The proposed drainage features for the access road have been designed in accordance with the applicable regulations and will be incorporated into the "Permit Drawing Revisions".

Design/Computing References:

1. Autodesk, Inc. (2010). AutoCAD Civil 3D 2010, Version 3, Version D.215.0.0.
2. Kibler, D. D., Hodges, C. C., Thomas F. Smith, I. P., & F. Brian Thye, E. (1986). Virginia Tech/Penn State Urban Hydrology Model, Version 6.0.
3. United States Department of Agriculture. (June 1986). *Urban Hydrology for Small Watersheds, Technical Release 55, Second Edition.* Natural Resources Conservation Service, Conservation Engineering Division.
4. United States Department of Transportation. (September 2001). *Hydraulic Design Series Number 5, Hydraulic Design of Highway culverts.* Federal Highway Administration, National Highway Institute.

FIGURES

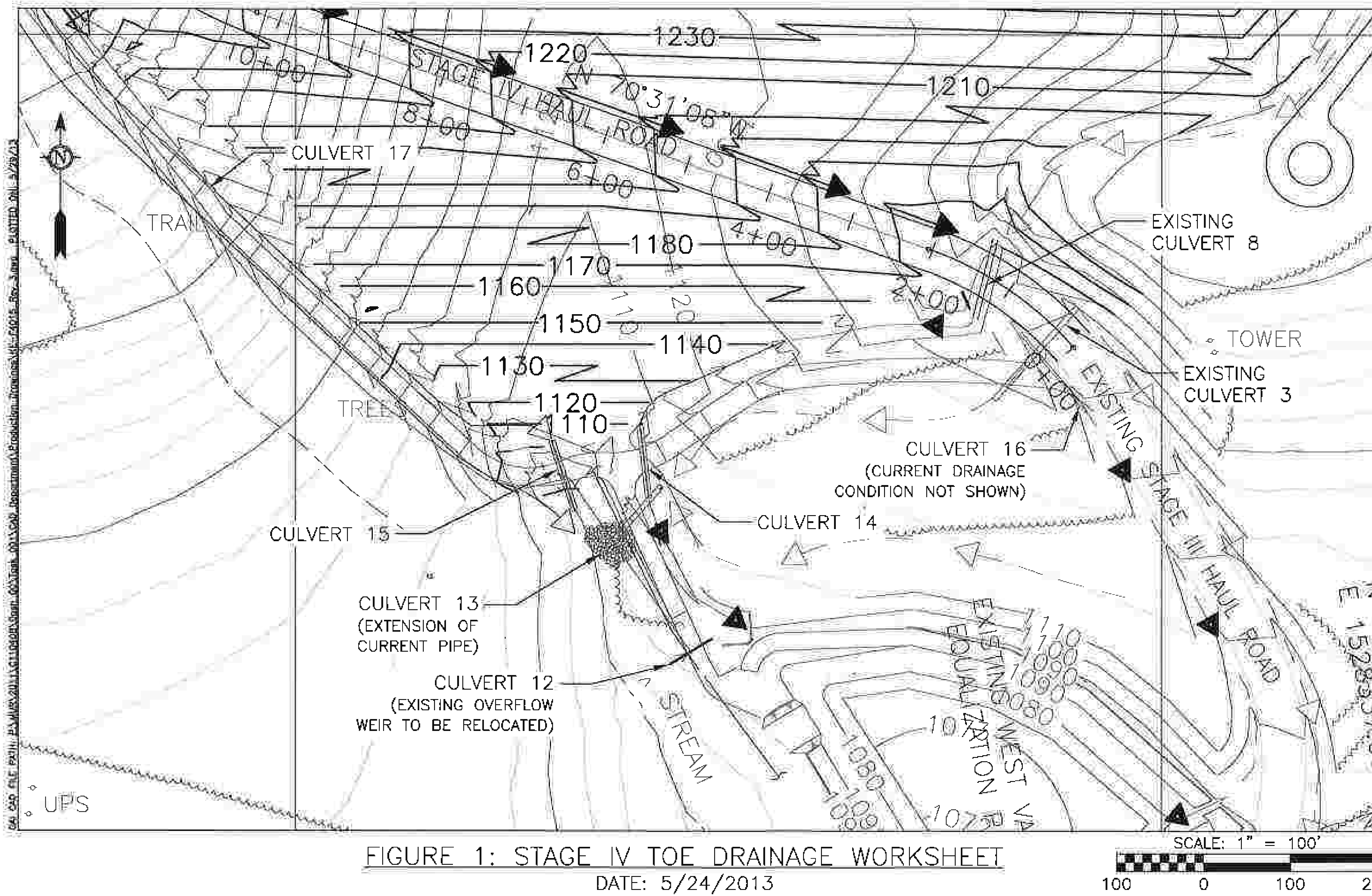
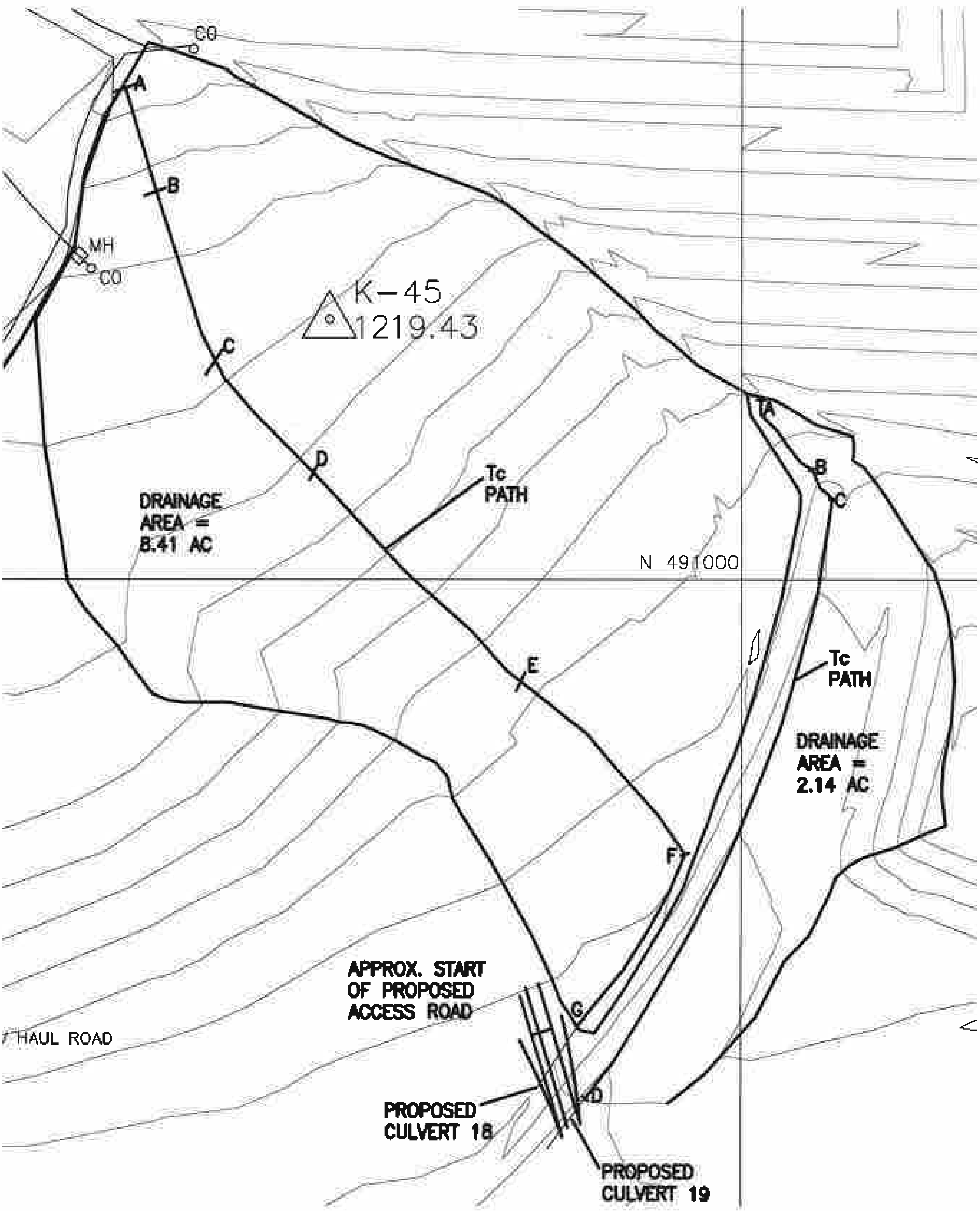


FIGURE 1: STAGE IV TOE DRAINAGE WORKSHEET

DATE: 5/24/2013

By: DMD
Date: 1/3/12

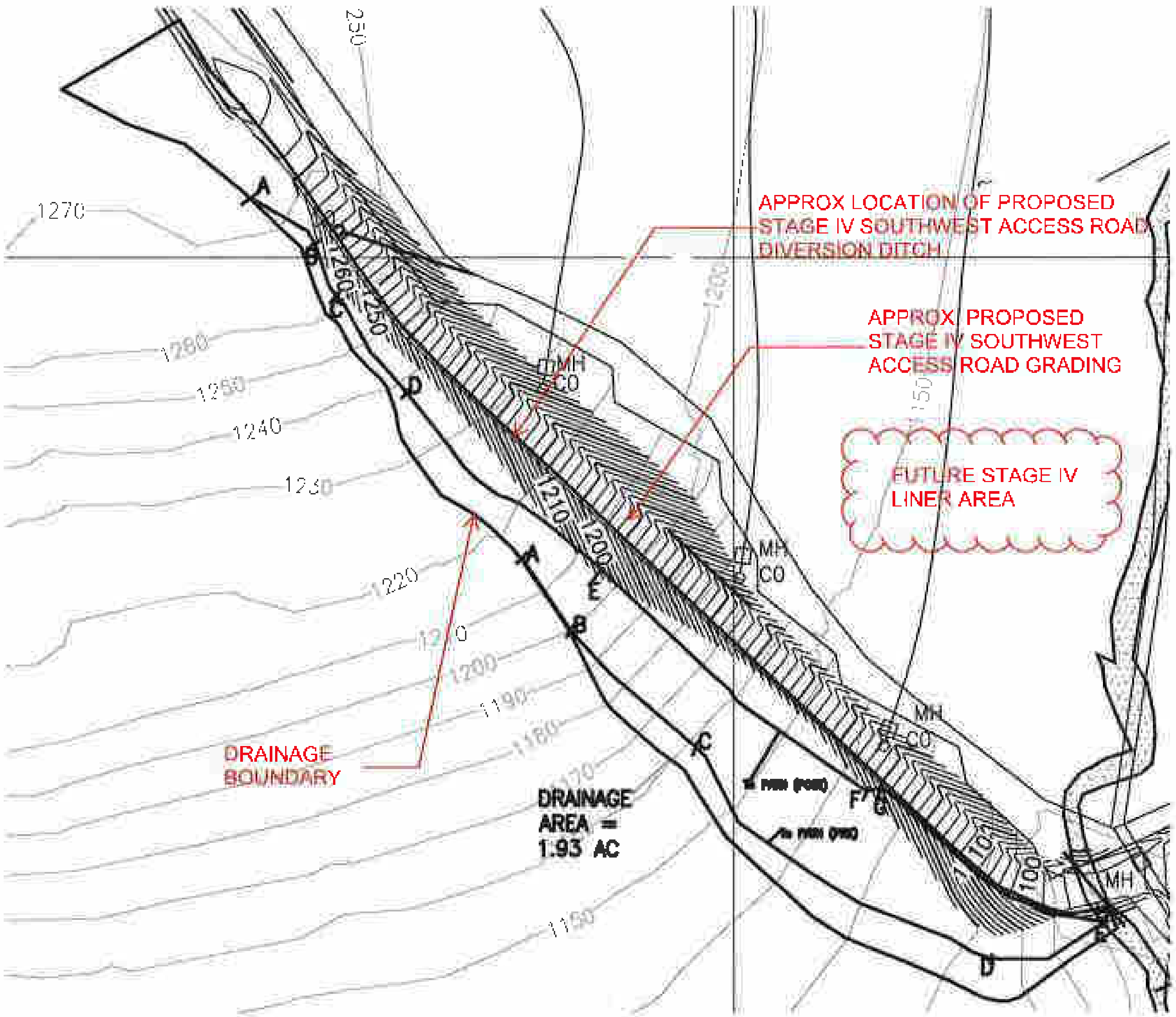
FIGURE 2: PROPOSED CULVERTS 18 AND 19 DRAINAGE WORKSHEET



SCALE: 1" = 100'

BY: DMD
DATE: 1/5/12

FIGURE 3: PROPOSED STAGE IV SOUTHWEST ACCESS ROAD DIVERSION DITCH



SCALE: 1" = 100'

TABLES

Table 1: Culvert Schedule

Culvert Number	Material	Size (in. Dia)	Approx. Length (ft)	Inlet Invert Elev.	Outlet Invert Elev.	Outlet Velocity (fps)	Remarks	Outlet Protection Type (min)
Proposed Culvert 12	Pre-cast Concrete Box	24 x 144	75.0	1096.00	1083.00	4.68	Shaped Entrance Required	Grouted R-4 Riprap
Proposed Culvert 13[1]	BCCMP	36	80.0	1095.50	1094.70	8.83	-	R-4 Riprap Required
Proposed Culvert 14	HDPE	Dual 36	80.0	1104.00	1101.00	6.92	Type D-W Endwall at Inlet	R-4 Riprap Required
Proposed Culvert 15	HDPE	Dual 36	80.0	1100.50	1099.70	6.92	Type D-W Endwall at Inlet	R-4 Riprap Required
Proposed Culvert 16	HDPE	36	40.0	1156.00	1151.00	7.56	-	R-4 Riprap Required
Proposed Culvert 17	HDPE	Dual 36	60.0	1179.00	1170.00	6.99	-	R-4 Riprap Required
Proposed Culvert 18	HDPE	18	70.0	1130.50	1129.50	3.88	-	R-3 Riprap Required
Proposed Culvert 19	HDPE	18	45.0	1116.50	1115.50	3.62	-	R-3 Riprap Required

1. Calculations verify that an extension of existing culvert to an approximate length of 80 feet is acceptable. The existing pipe shall be extended as needed.

Table 2: Channel Schedule

Channel ID	Base Width (ft)	Depth (ft)	Side Slopes (zH: 1V)	Outlet Velocity (fps)	Lining Type
Proposed Stage IV Southwest Access Road Diversion Ditch	0	1.5	2	5.54	Grouted Riprap

ATTACHMENTS

ATTACHMENT 1: ULTIMATE CONDITIONS DRAINAGE SKETCH AND HYDRAULIC SUMMARY (ORIGINAL FORM I PERMIT CALCULATIONS SHEETS 4 AND 26 OF 45)

SUBJECT KEYSTONE WEST VALLEY

PHASE II PERMITTING

BY SRR DATE 3/19/96 PROJ. NO. 92-220-73-7

CHKD. BY [illegible] DATE 6/10/96 SHEET NO. 4 OF 45

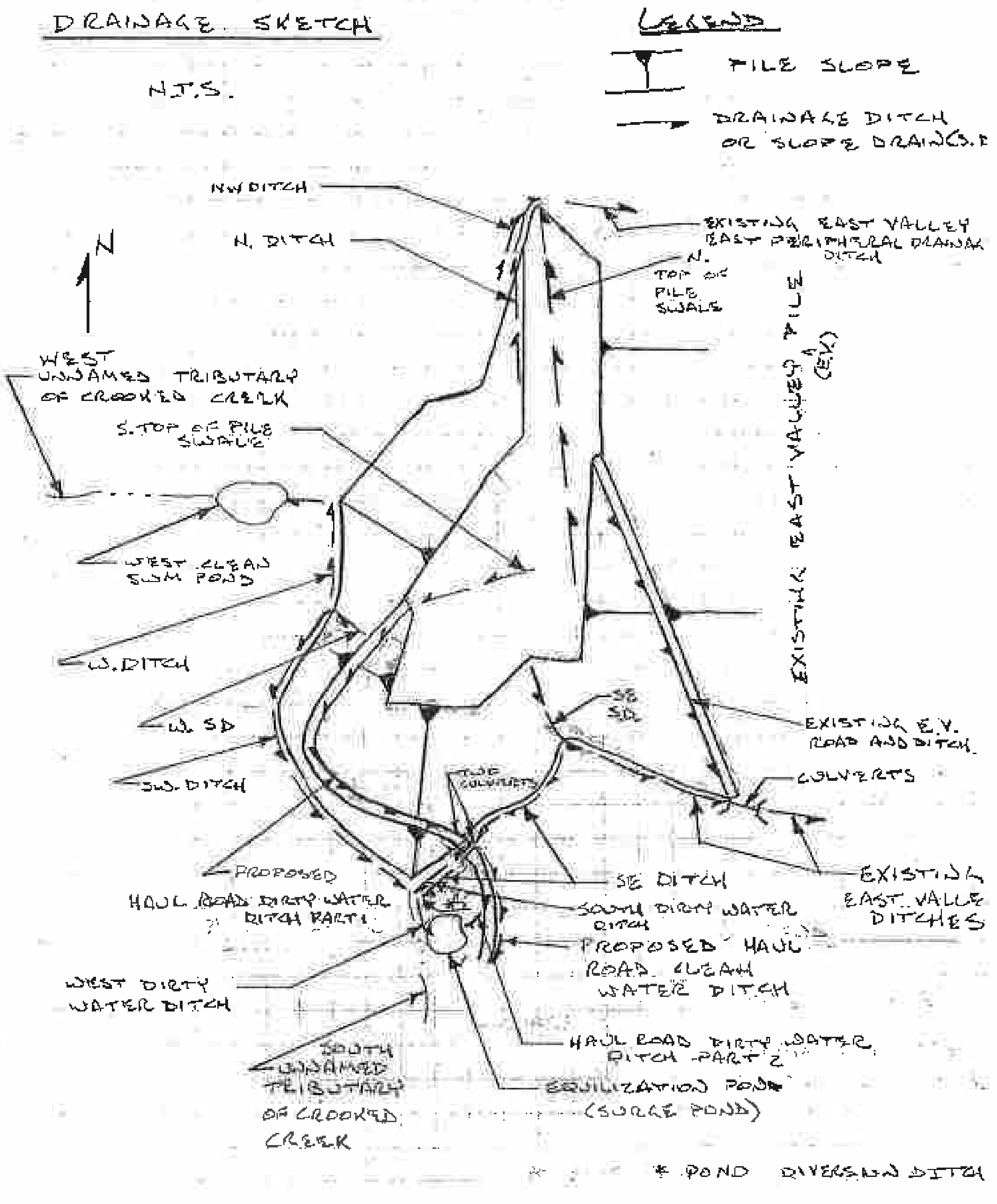
GAI CONSULTANTS, INC

Engineers • Geologists • Planners
Environmental Specialists

ULTIMATE CONDITIONS

DRAINAGE SKETCH

N.T.S.



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Environmental Specialists

Hydraulics

The design flow, lining, bottom width (b), side slope (z), and maximum and minimum slope for each drainage structure is summarized below.

Drainage Structure	Design Flow (cfs)	Maximum Slope	Minimum Slope	Lining	Bottom Width	Side Slopes, z
West Ditch	29	$\frac{5}{18}=0.278$	$\frac{5}{110}=0.045$	Grouted Rock	2	2
North Ditch -						
Part 1	7	$\frac{5}{70}=0.071$	$\frac{5}{85}=0.059$	Grass	2	2
Part 2	20	$\frac{5}{25}=0.2$	$\frac{5}{50}=0.1$	Grouted Rock	2	2
Part 3	69	$\frac{15}{250}=0.06$	$\frac{15}{250}=0.06$	Grouted Rock	2	2
Northwest Ditch	13	$\frac{5}{270}=0.019$	$\frac{5}{270}=0.019$	Grass	2	2
Southwest Ditch -						
Part 1	90	0.01	0.01	Grass	2	2
Part 2	90	$\frac{5}{15}=0.333$	$\frac{5}{35}=0.143$	Grouted Rock	2	2
Southeast Ditch - Part 1*	22	$\frac{5}{32}=0.156$	$\frac{5}{150}=0.033$	Grouted Rock	2	2
Haul Road Clean Water Ditch	5	0.1	0.1	Grouted Rock	2	2
North Top of Pile Swale	53	$\frac{25}{415}=0.06$	$\frac{5}{135}=0.037$	Grass	0	3
South Top of Pile Swale	85	$\frac{5}{110}=0.045$	$\frac{5}{330}=0.015$	Grass	0	3
Southeast Slope Drain	71	0.4	0.05	Concrete Revetment Uniform Section Mat	2	2
West Slope Drain	60	0.4	0.05	Concrete Revetment Uniform Section Mat	2	2
Existing East Valley West Side Collection Channel -						
Part 1	108	$\frac{45}{255}=0.176$	$\frac{5}{160}=0.031$	Grouted Rock	3	2
Existing East Valley Haul Road Ditch	51	$\frac{25}{250}=0.1$	$\frac{25}{250}=0.1$	Grouted Rock	2	2

* The Southeast Ditch - Part 1 is the Southeast Ditch above the proposed haul road and is designed within this calc. set.
The Southeast Ditch - Part 2 is the Southeast Ditch below the proposed haul road and is designed in another calc. set.

ATTACHMENT 2: PROPOSED CULVERT 12 THROUGH 19 DESIGN DETAILS

TABLE 5.1
Pennsylvania Rainfall by Counties
(For Use with Technical Release 55 – Urban Hydrology for Small Watersheds)
<u>NOT TO BE USED WITH THE RATIONAL EQUATION</u>

COUNTY	24 HR RAINFALL FOR VARIOUS FREQUENCIES							COUNTY	24 HR RAINFALL FOR VARIOUS FREQUENCIES						
	1 yr.	2 yr.	5 yr.	10 yr.	25 yr.	50 yr.	100 yr.		1 yr.	2 yr.	5 yr.	10 yr.	25 yr.	50 yr.	100 yr.
Adams	2.52	3.02	3.77	4.43	5.48	6.45	7.59	Lackawanna	2.12	2.55	3.15	3.69	4.55	5.35	6.30
Allegheny	1.97	2.35	2.88	3.30	3.90	4.40	4.92	Lancaster	2.51	3.02	3.85	4.56	5.63	6.56	7.59
Armstrong	2.03	2.42	2.95	3.40	4.01	4.53	5.06	Lawrence	1.99	2.37	2.90	3.33	3.94	4.44	4.96
Beaver	1.97	2.35	2.87	3.30	3.90	4.40	4.91	Lebanon	2.50	3.02	3.84	4.55	5.64	6.59	7.67
Bedford	2.19	2.62	3.27	3.81	4.60	5.27	5.99	Lehigh	2.69	3.24	4.05	4.73	5.75	6.63	7.60
Berks	2.65	3.19	4.00	4.68	5.67	6.50	7.41	Luzerne	2.37	2.84	3.53	4.13	5.08	5.96	6.99
Blair	2.23	2.68	3.33	3.87	4.63	5.28	5.96	Lycoming	2.38	2.85	3.53	4.12	5.04	5.88	6.87
Bradford	2.05	2.44	2.98	3.41	3.99	4.45	4.93	McKean	2.08	2.48	3.03	3.48	4.13	4.66	5.21
Bucks	2.72	3.26	4.10	4.80	5.81	6.67	7.59	Mercer	2.05	2.44	2.99	3.43	4.07	4.58	5.13
Butler	2.02	2.40	2.93	3.37	3.98	4.49	5.02	Mifflin	2.36	2.83	3.52	4.10	4.95	5.68	6.49
Cambria	2.17	2.59	3.18	3.68	4.39	4.97	5.59	Monroe	2.63	3.16	3.92	4.60	5.68	6.70	7.91
Cameron	2.11	2.53	3.10	3.60	4.35	5.02	5.80	Montgomery	2.67	3.21	4.03	4.70	5.68	6.50	7.38
Carbon	2.74	3.29	4.09	4.79	5.92	6.96	8.20	Montour	2.35	2.82	3.50	4.09	5.05	5.94	6.99
Centre	2.20	2.64	3.29	3.82	4.58	5.22	5.91	Northampton	2.64	3.16	3.95	4.61	5.60	6.45	7.41
Chester	2.70	3.25	4.07	4.75	5.73	6.55	7.44	Northumberland	2.32	2.78	3.45	4.04	4.96	5.82	6.83
Clarion	2.09	2.49	3.05	3.50	4.14	4.67	5.22	Perry	2.34	2.81	3.49	4.08	5.03	5.90	6.92
Clearfield	2.13	2.54	3.12	3.60	4.28	4.85	5.44	Philadelphia	2.72	3.28	4.12	4.83	5.85	6.72	7.68
Clinton	2.18	2.61	3.19	3.67	4.34	4.89	5.47	Pike	2.45	2.94	3.64	4.26	5.23	6.13	7.20
Columbia	2.38	2.85	3.54	4.14	5.10	5.99	7.04	Potter	2.01	2.40	2.96	3.44	4.21	4.91	5.74
Crawford	2.08	2.49	3.04	3.50	4.14	4.67	5.23	Schuylkill	2.77	3.33	4.14	4.85	5.96	6.97	8.17
Cumberland	2.35	2.82	3.50	4.11	5.08	5.97	7.02	Snyder	2.60	3.12	3.88	4.55	5.59	6.56	7.7[illegible]
Dauphin	2.50	3.01	3.78	4.45	5.50	6.44	7.52	Somerset	2.06	2.46	3.08	3.61	4.44	5.16	5.97
Delaware	2.69	3.25	4.10	4.82	5.87	6.75	7.72	Sullivan	2.54	3.04	3.73	4.30	5.12	5.82	6.58
Elk	2.08	2.48	3.02	3.48	4.12	4.65	5.21	Susquehanna	2.23	2.67	3.26	3.74	4.41	4.96	5.55
Erie	2.13	2.56	3.19	3.71	4.46	5.09	5.76	Tioga	1.96	2.34	2.88	3.35	4.07	4.73	5.49
Fayette	2.08	2.47	3.02	3.46	4.08	4.60	5.13	Union	2.41	2.89	3.58	4.19	5.13	6.01	7.04
Forest	2.06	2.46	3.00	3.45	4.08	4.59	5.14	Venango	2.05	2.45	2.99	3.44	4.07	4.58	5.12
Franklin	2.44	2.94	3.65	4.26	5.17	5.97	6.86	Warren	2.07	2.47	3.01	3.47	4.11	4.63	5.19
Fulton	2.27	2.73	3.39	3.93	4.75	5.40	6.13	Washington	1.99	2.38	2.91	3.35	3.96	4.46	4.99
Greene	2.01	2.40	2.92	3.36	3.96	4.45	4.96	Wayne	2.38	2.86	3.53	4.12	5.03	5.86	6.83
Huntingdon	2.21	2.65	3.29	3.83	4.60	5.25	5.94	Westmoreland	2.05	2.45	2.99	3.43	4.06	4.57	5.11
Indiana	2.15	2.57	3.14	3.62	4.29	4.85	5.44	Wyoming	2.16	2.58	3.18	3.69	4.46	5.14	5.91
Jefferson	2.09	2.50	3.05	3.50	4.14	4.67	5.23	York	2.45	2.96	3.80	4.53	5.65	6.64	7.76
Juniata	2.36	2.83	3.52	4.11	5.02	5.84	6.79								

NWS NOAA Atlas 14, Sept 25-29, 2008

NOTE: Data from this table should not be used for final <u>design</u> of Erosion Control or PCSM BMPs.

Culvert Report

Hydraflow Express Extension for AutoCAD® Civil 3D® 2010 by Autodesk, Inc. Tuesday, Jan 31 2012

Proposed Culvert No. 12

Invert Elev Dn (ft)	= 1083.00
Pipe Length (ft)	= 75.00
Slope (%)	= 17.33
Invert Elev Up (ft)	= 1096.00
Rise (in)	= 24.0
Shape	= Box
Span (in)	= 144.0
No. Barrels	= 1
n-Value	= 0.012
Inlet Edge	= 0
Coeff. K,M,c,Y,k	= 0.0145, 1.75, 0.0419, 0.64, 0.5

Embankment

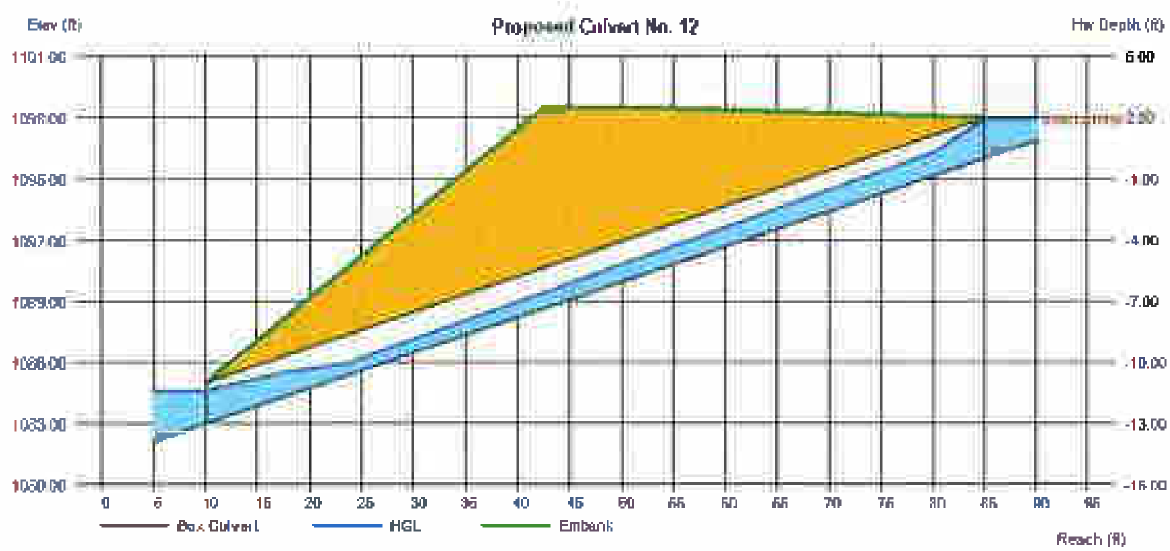
Top Elevation (ft)	= 1098.50
Top Width (ft)	= 10.00
Crest Width (ft)	= 10.00

Calculations

Qmin (cfs)	= 0.00
Qmax (cfs)	= 91.00
Tailwater Elev (ft)	= 0

Highlighted

Qtotal (cfs)	= 90.00
Qpipe (cfs)	= 90.00
Qovertop (cfs)	= 0.00
Veloc Dn (ft/s)	= 4.68
Veloc Up (ft/s)	= 6.23
HGL Dn (ft)	= 1084.60
HGL Up (ft)	= 1097.21
Hw Elev (ft)	= 1097.94
Hw/D (ft)	= 0.97
Flow Regime	= Inlet Control



Culvert Report

Proposed Culvert 13

Invert Elev Dn (ft)	= 1094.70
Pipe Length (ft)	= 80.00
Slope (%)	= 1.00
Invert Elev Up (ft)	= 1095.50
Rise (in)	= 36.0
Shape	= Cir
Span (in)	= 36.0
No. Barrels	= 1
n-Value	= 0.022
Inlet Edge	= 0
Coeff. K,M,c,Y,k	= 0.0045, 2, 0.0317, 0.69, 0.5

Embankment

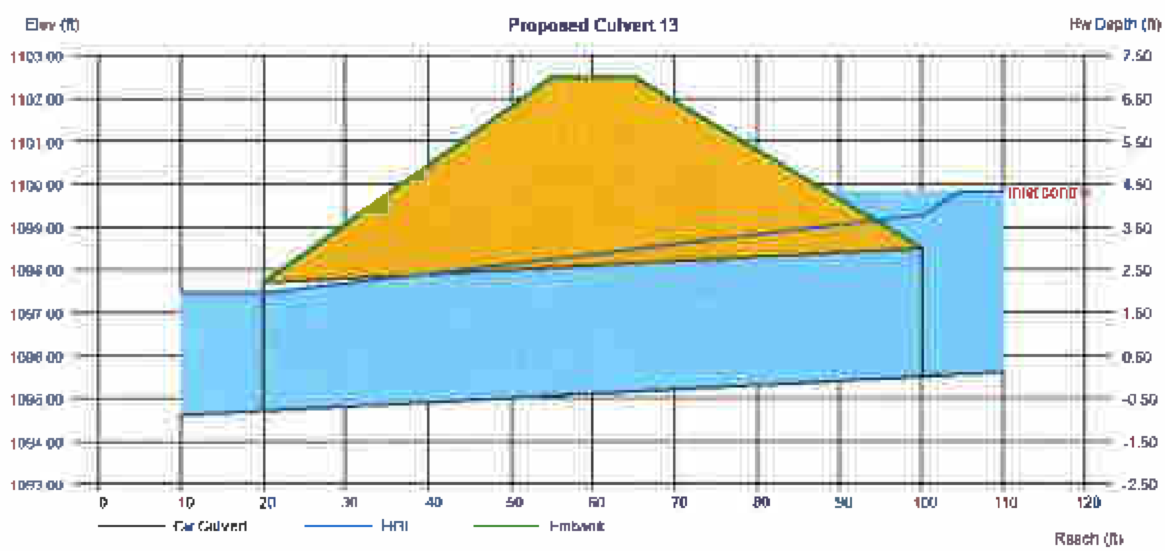
Top Elevation (ft)	= 1102.50
Top Width (ft)	= 10.00
Crest Width (ft)	= 10.00

Calculations

Qmin (cfs)	= 0.00
Qmax (cfs)	= 69.00
Tailwater Elev (ft)	= 0

Highlighted

Qtotal (cfs)	= 60.00
Qpipe (cfs)	= 60.00
Qovertop (cfs)	= 0.00
Veloc Dn (ft/s)	= 8.83
Veloc Up (ft/s)	= 8.49
HGL Dn (ft)	= 1097.45
HGL Up (ft)	= 1099.28
Hw Elev (ft)	= 1099.84
Hw/D (ft)	= 1.45
Flow Regime	= Inlet Control



Culvert Report

Proposed Culvert 14

Invert Elev Dn (ft)	= 1101.00
Pipe Length (ft)	= 80.00
Slope (%)	= 3.75
Invert Elev Up (ft)	= 1104.00
Rise (in)	= 36.0
Shape	= Cir
Span (in)	= 36.0
No. Barrels	= 2
n-Value	= 0.011
Inlet Edge	= 0
Coeff. K,M,c,Y,k	= 0.0098, 2, 0.0398, 0.67, 0.5

Embankment

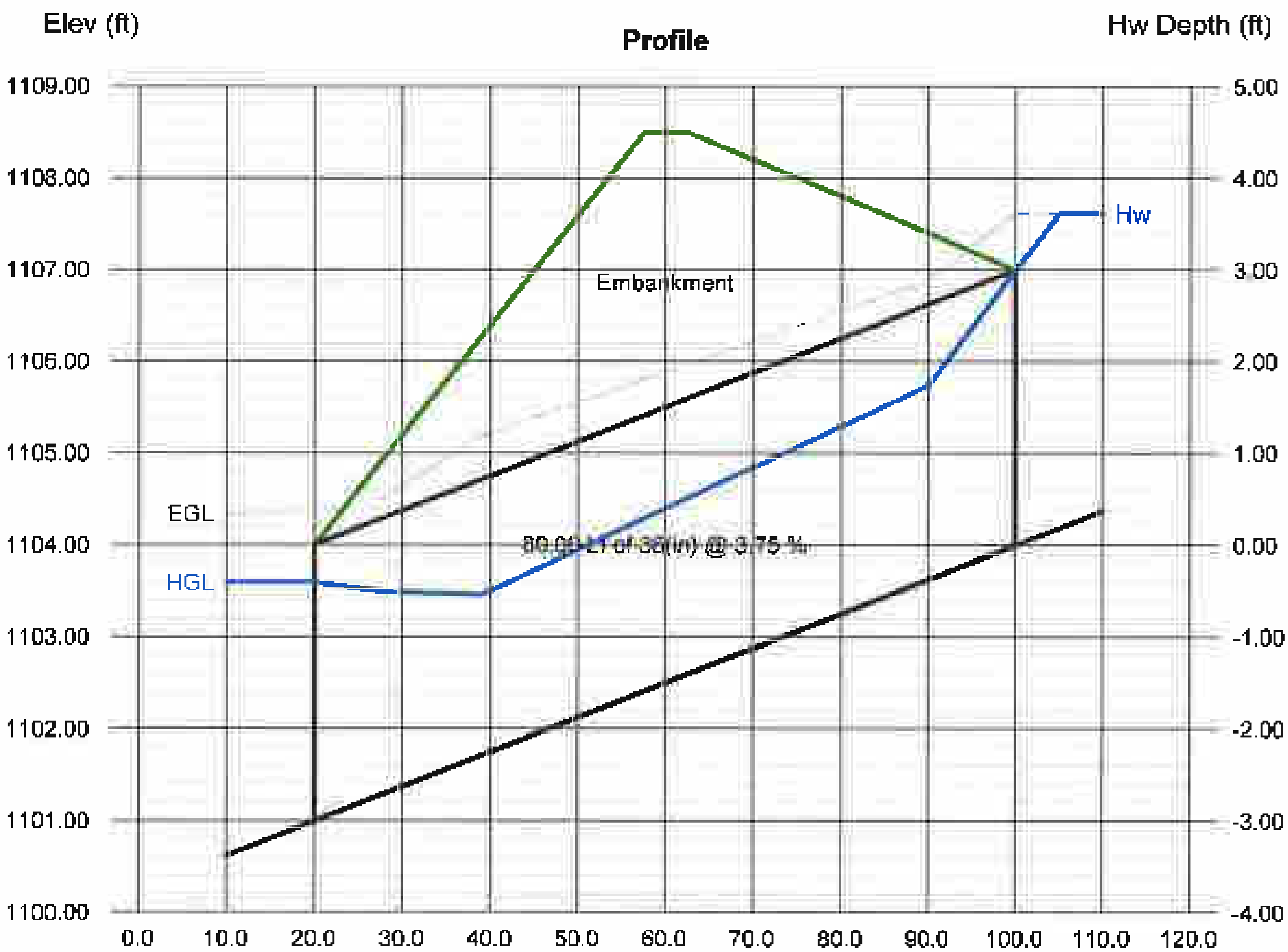
Top Elevation (ft)	= 1108.50
Top Width (ft)	= 5.00
Crest Width (ft)	= 5.00

Calculations

Qmin (cfs)	= 0.00
Qmax (cfs)	= 90.00
Tailwater Elev (ft)	= 0

Highlighted

Qtotal (cfs)	= 90.00
Qpipe (cfs)	= 90.00
Qovertop (cfs)	= 0.00
Veloc Dn (ft/s)	= 6.92
Veloc Up (ft/s)	= 8.13
HGL Dn (ft)	= 1103.60
HGL Up (ft)	= 1106.19
Hw Elev (ft)	= 1107.62
Hw/D (ft)	= 1.20
Flow Regime	= Inlet Control



Culvert Report

Hydraflow Express Extension for AutoCAD® Civil 3D® 2010 by Autodesk, Inc. | Tuesday, Jan 31 2012

Proposed Culvert 15

Invert Elev Dn (ft)	= 1099.70
Pipe Length (ft)	= 80.00
Slope (%)	= 1.00
Invert Elev Up (ft)	= 1100.50
Rise (in)	= 36.0
Shape	= Cir
Span (in)	= 36.0
No. Barrels	= 2
n-Value	= 0.012
Inlet Edge	= 0
Coeff. K,M,c,Y,k	= 0.0045, 2, 0.0317, 0.69, 0.5

Embankment

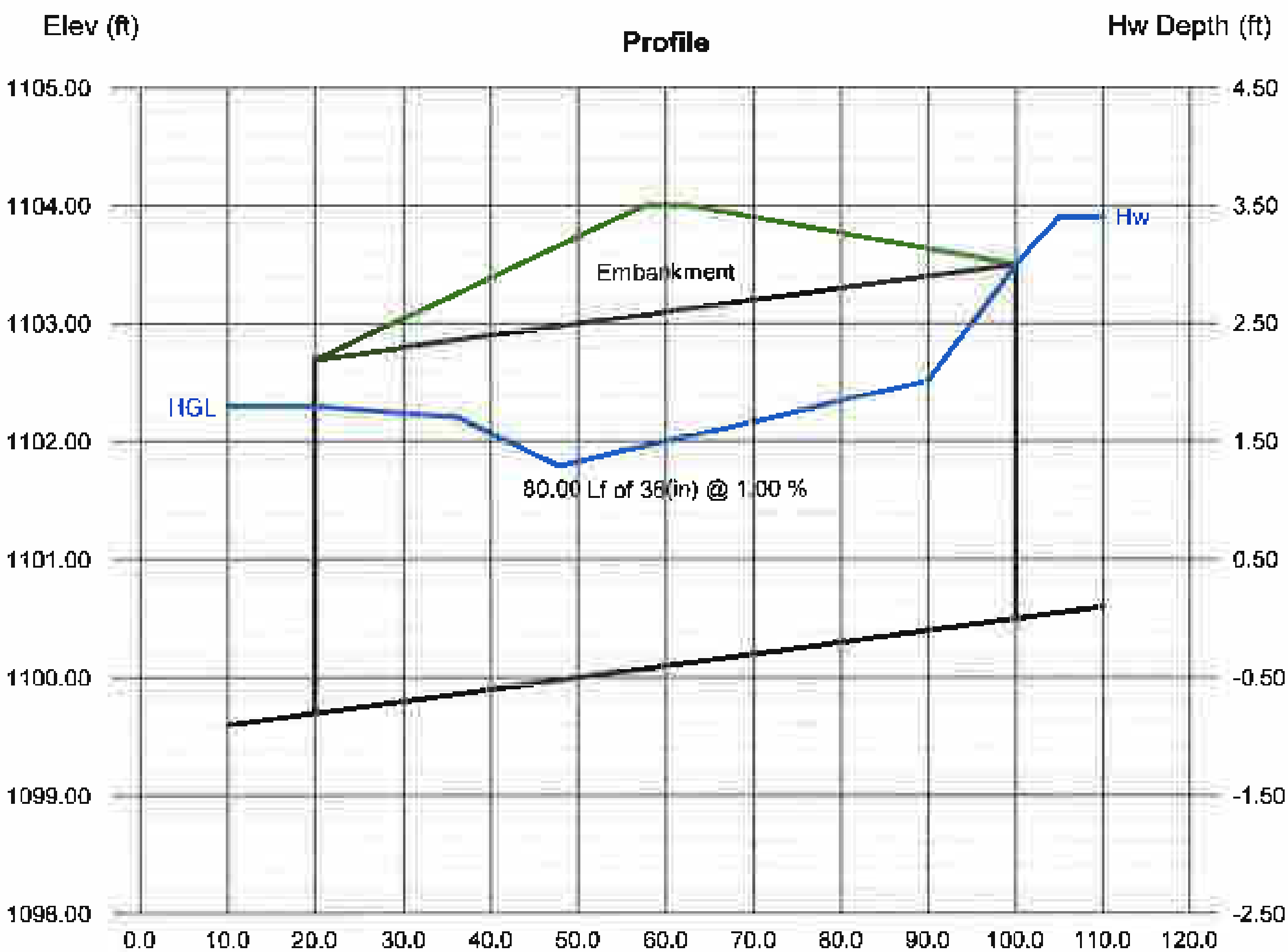
Top Elevation (ft)	= 1104.00
Top Width (ft)	= 5.00
Crest Width (ft)	= 5.00

Calculations

Qmin (cfs)	= 0.00
Qmax (cfs)	= 90.00
Tailwater Elev (ft)	= 0

Highlighted

Qtotal (cfs)	= 90.00
Qpipe (cfs)	= 90.00
Qovertop (cfs)	= 0.00
Veloc Dn (ft/s)	= 6.92
Veloc Up (ft/s)	= 8.13
HGL Dn (ft)	= 1102.30
HGL Up (ft)	= 1102.69
Hw Elev (ft)	= 1103.90
Hw/D (ft)	= 1.13
Flow Regime	= Inlet Control

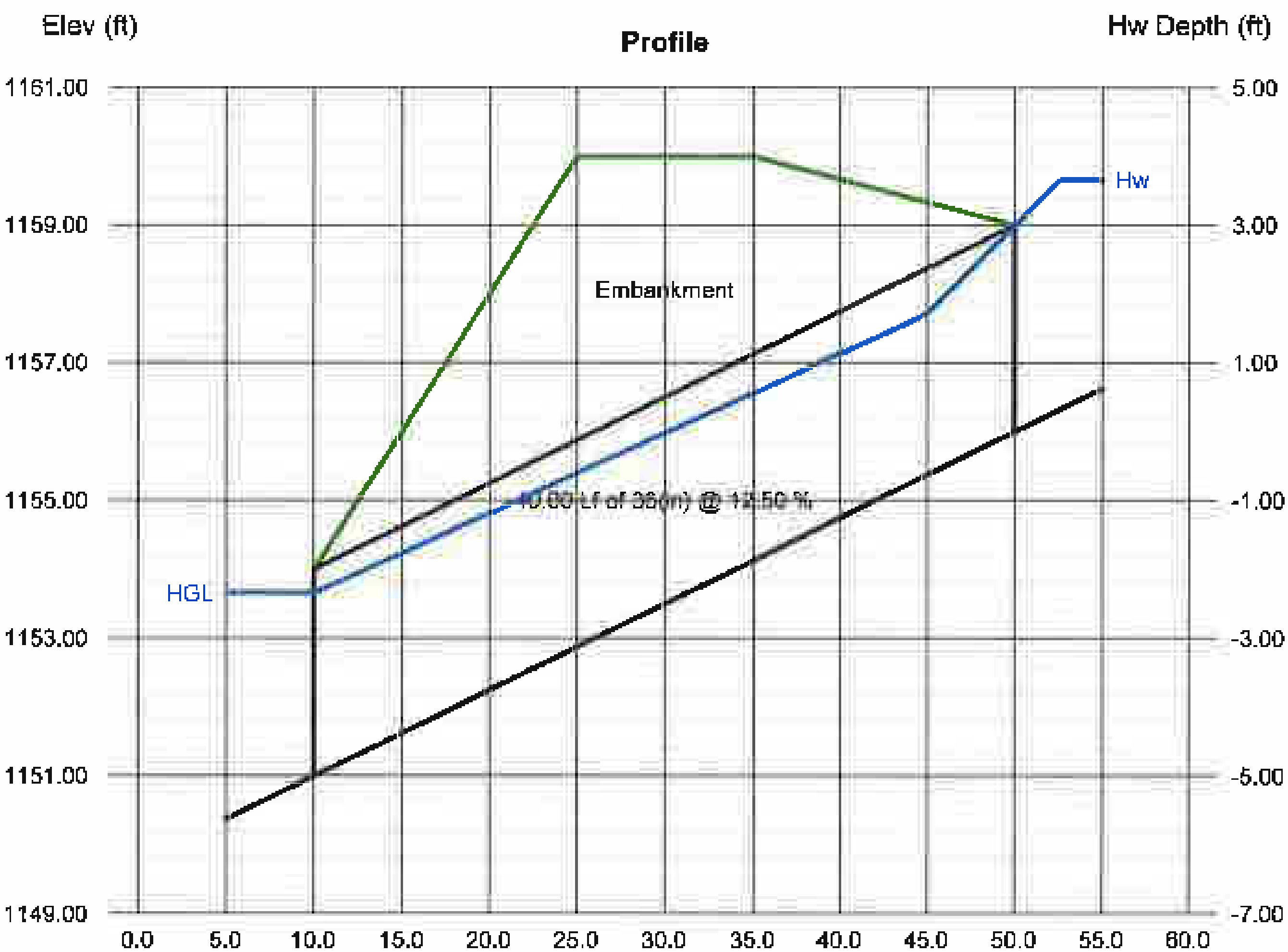


Culvert Report

Hydraflow Express Extension for AutoCAD® Civil 3D® 2010 by Autodesk, Inc. Tuesday, Jan 31 2012

Proposed Culvert 16

Invert Elev Dn (ft)	= 1151.00	**Calculations**	
Pipe Length (ft)	= 40.00	Qmin (cfs)	= 0.00
Slope (%)	= 12.50	Qmax (cfs)	= 51.00
Invert Elev Up (ft)	= 1156.00	Tailwater Elev (ft)	= 0
Rise (in)	= 36.0		
Shape	= Cir	**Highlighted**	
Span (in)	= 36.0	Qtotal (cfs)	= 50.00
No. Barrels	= 1	Qpipe (cfs)	= 50.00
n-Value	= 0.011	Qovertop (cfs)	= 0.00
Inlet Edge	= 0	Veloc Dn (ft/s)	= 7.56
Coeff. K,M,c,Y,k	= 0.0045, 2, 0.0317, 0.69, 0.5	Veloc Up (ft/s)	= 8.57
		HGL Dn (ft)	= 1153.65
Embankment		HGL Up (ft)	= 1158.31
Top Elevation (ft)	= 1160.00	Hw Elev (ft)	= 1159.65
Top Width (ft)	= 10.00	Hw/D (ft)	= 1.22
Crest Width (ft)	= 10.00	Flow Regime	= Inlet Control



Culvert Report

Hydraflow Express Extension for AutoCAD® Civil 3D® 2010 by Autodesk, Inc. — Tuesday, Jan 31 2012

Proposed Culvert 17

Invert Elev Dn (ft)	= 1170.00
Pipe Length (ft)	= 60.00
Slope (%)	= 15.00
Invert Elev Up (ft)	= 1179.00
Rise (in)	= 36.0
Shape	= Cir
Span (in)	= 36.0
No. Barrels	= 2
n-Value	= 0.011
Inlet Edge	= 0
Coeff. K,M,c,Y,k	= 0.0045, 2, 0.0317, 0.69, 0.5

Embankment

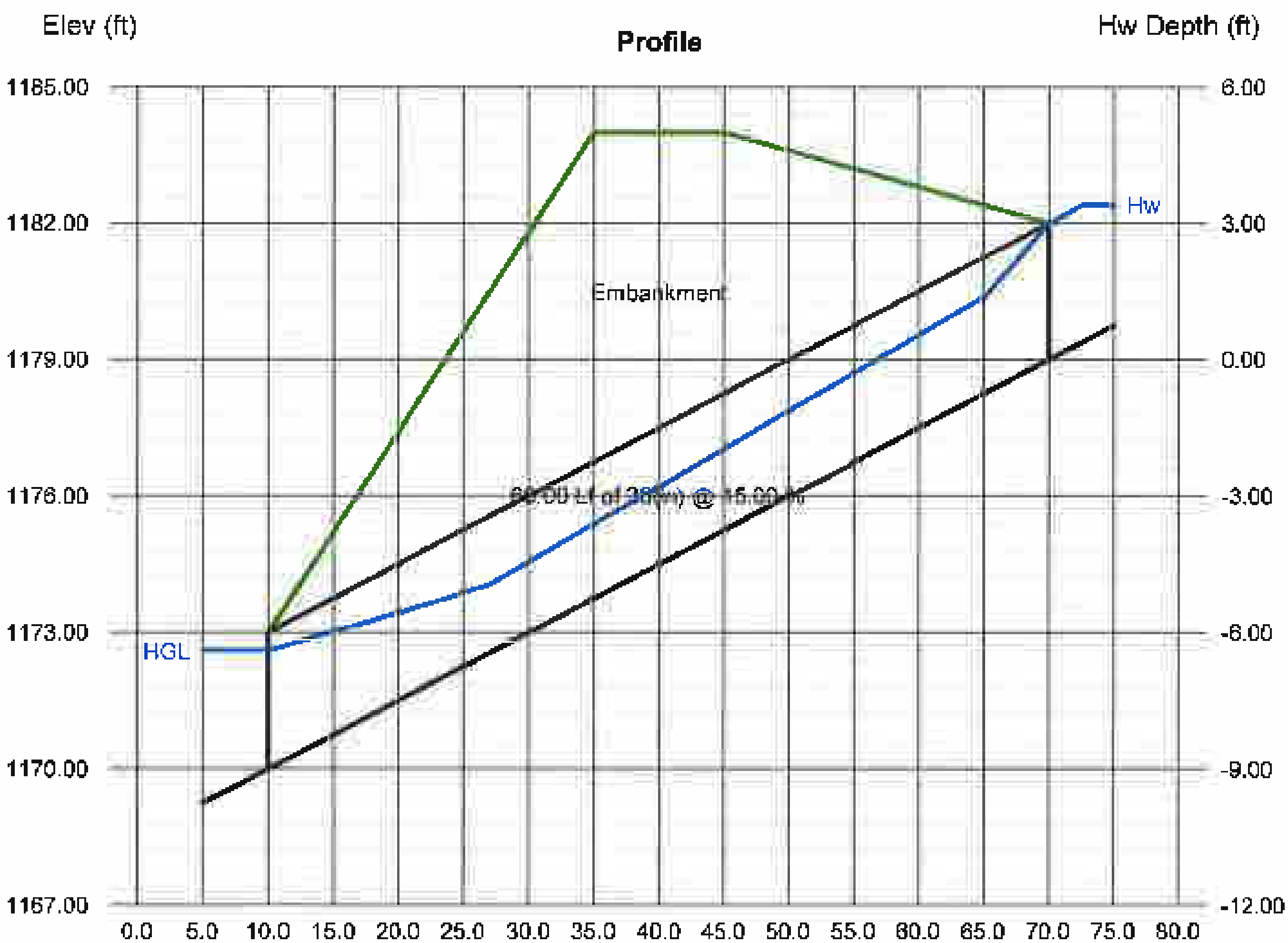
Top Elevation (ft)	= 1184.00
Top Width (ft)	= 10.00
Crest Width (ft)	= 10.00

Calculations

Qmin (cfs)	= 0.00
Qmax (cfs)	= 91.00
Tailwater Elev (ft)	= 0

Highlighted

Qtotal (cfs)	= 91.00
Qpipe (cfs)	= 91.00
Qovertop (cfs)	= 0.00
Veloc Dn (ft/s)	= 6.99
Veloc Up (ft/s)	= 8.17
HGL Dn (ft)	= 1172.60
HGL Up (ft)	= 1181.21
Hw Elev (ft)	= 1182.40
Hw/D (ft)	= 1.13
Flow Regime	= Inlet Control



Runoff Curve Number

Project: Keystone Generating Station: Stage IV Minor Permit Modification Application	By: DMD	Date: 1/4/2012
Location: Proposed Access Road Culvert 18 (East of West Valley Stage IV Expansion)	Checked:	Date:

Check one: ■ Present ❑ Developed

Runoff Curve Number and Hydrologic Group	Cover Description	CN			Area	Product of CN x Area
		Table 2-2	Figure 2-3	Figure 2-4	■ Acres ❑ $miles^2$ ❑ %	
B	Woods	60			8.14	488.40
			TOTALS		**8.14**	**488.40**

CN (weighted) = Total Product / Total Area

CN	**60**

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Runoff Curve Number

Project:	By:	Date:
Keystone Generating Station:	DMD	1/4/2012
Stage IV Minor Permit Modification Application	Checked:	Date:
Location:		
Proposed Access Road Culvert 18 (East of West Valley Stage IV Expansion)		

Check one:	☐ Present	■ Developed

Runoff Curve Number and Hydrologic Group	Cover Description	CN			Area	Product of CN x Area
		Table 2-2	Figure 2-3	Figure 2-4	■ Acres ☐ miles² ☐ %	
B	Gravel Access Road	85			0.47	39.95
B	Woods	60			7.67	460.20
			TOTALS		**8.14**	**500.15**

CN (weighted) = Total Product / Total Area

CN	**61**

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Time of Concentration

Project: Keystone Generating Station: Stage IV Minor Permit Modification Application	By: DMD	Date: 1/4/2012
Location: Proposed Access Road Culvert 18	Checked:	Date:

Check one: ☐ Present ■ Developed

Sheet Flow

	Segment ID	
	A to B	
Surface Description (Table 3-1)	Woods	
Manning's Roughness Coefficient, n (table 3-1)	0.4	
Flow Length, L	100	ft
Two-year 24-hour Rainfall, P_2	2.42	in
Land Slope, s	0.17	ft/ft
Travel Time, $T_t = (0.007*(n*L)^{0.8}) / (P_2^{0.5}*s^{0.4})$	**0.1748**	**hrs**

Shallow Concentrated Flow

Segment ID	B to C	C to D	D to E	E to F	
Surface Description (Paved / Unpaved)	Unpaved	Unpaved	Unpaved	Unpaved	
Surface Description Coefficient, C	16.1435	16.1435	16.1435	16.1435	
Flow Length, L	166	138	274	222	ft
Watercourse Slope, s	0.09	0.10	0.17	0.09	ft/ft
Average Velocity, $V = C*s^{0.5}$	4.85	5.14	6.61	4.85	ft/sec
Travel Time, $T_t = (L) / (3600*V)$	**0.0095**	**0.0075**	**0.0115**	**0.0127**	**hrs**

Channel Flow

Segment ID	F to G	
Section Base, b	0	
Section Depth, d	2	
Section Side Slope, z	2	
Cross Sectional Flow Area, $a = b*d + z*d^2$	8	
Wetted Perimeter, $p_w = b + (2*d)*(z^2 + 1)^{0.5}$	8.94	
Hydraulic Radius, $r = a / p_w$	0.89	
Channel Slope, s	0.04	
Manning's Roughness Coefficient, n	0.025	
Average Velocity, $V = (1.49*r^{2/3}*s^{1/2}) / (n)$	11.35	ft/sec
Flow Length, L	190	ft
Travel Time, $T_t = (L) / (3600*V)$	**0.0046**	**hrs**

Time of Concentration

Sheet Flow T_t	0.1748	hrs
Shallow Concentrated Flow T_t	0.0412	hrs
Channel Flow T_t	0.0046	hrs
Time of Concentration, T_c	**0.2207**	**hrs**
	13.24	**mins**

SCS TR55 Tabular Method

Watershed Title: Proposed Culvert 18 (Pre-Construction)

25 Year Type II Storm: Precipitation = 4.01 inches

Summary of Input Parameters

Subarea	Area (acres)	Curve Number	IA/P	Runoff (in)	Tc (min)	Adj. Tc (min)	Tt (min)	Adj. Tt (min)
1	8.140	60	0.333	0.77	13.240	12.000	0.000	1.260
Composite	**8.140**	**60**		**0.77**				

SCS TR55 Tabular Method

Watershed Title: Proposed Culvert 18 (Pre-Construction)

25 Year Type II Storm: Precipitation = 4.01 inches

Individual Subarea and Composite Hydrographs

Subarea	Time (hrs) 11.0	11.9	12.2	12.5	12.8	13.2	13.6	14.0	15.0	17.0	20.0	26.0
1	0.00	0.26	5.75	2.19	1.16	0.83	0.68	0.57	0.44	0.31	0.22	0.00
Composite	**0.00**	**0.26**	**5.75**	**2.19**	**1.16**	**0.83**	**0.68**	**0.57**	**0.44**	**0.31**	**0.22**	**0.00**

The peak flow is 5.8 cfs at 12.2 hrs.

SCS TR55 Tabular Method

Watershed Title: Proposed Culvert 18 (Post Construction)

25 Year Type II Storm: Precipitation = 4.01 inches

Summary of Input Parameters

Subarea	Area (acres)	Curve Number	IA/P	Runoff (in)	Tc (min)	Adj. Tc (min)	Tt (min)	Adj. Tt (min)
1	8.140	61	0.319	0.82	13.240	12.000	0.000	1.260
Composite	**8.140**	**61**		**0.82**				

SCS TR55 Tabular Method

Watershed Title: Proposed Culvert 18 (Post Construction)

25 Year Type II Storm: Precipitation = 4.01 Inches

Individual Subarea and Composite Hydrographs

Subarea	Time (hrs)											
	11.0	**11.9**	**12.2**	**12.5**	**12.8**	**13.2**	**13.6**	**14.0**	**15.0**	**17.0**	**20.0**	**26.0**
1	0.00	0.29	6.33	2.37	1.23	0.87	0.72	0.60	0.46	0.33	0.23	0.00
Composite	**0.00**	**0.29**	**6.33**	**2.37**	**1.23**	**0.87**	**0.72**	**0.60**	**0.46**	**0.33**	**0.23**	**0.00**

The peak flow is 6.3 cfs at 12.2 hrs.

Culvert Report

Proposed Culvert 18

Invert Elev Dn (ft)	= 1129.50
Pipe Length (ft)	= 70.00
Slope (%)	= 1.43
Invert Elev Up (ft)	= 1130.50
Rise (in)	= 18.0
Shape	= Cir
Span (in)	= 18.0
No. Barrels	= 1
n-Value	= 0.012
Inlet Edge	= 0
Coeff. K,M,c,Y,k	= 0.0045, 2, 0.0317, 0.69, 0.5

Embankment

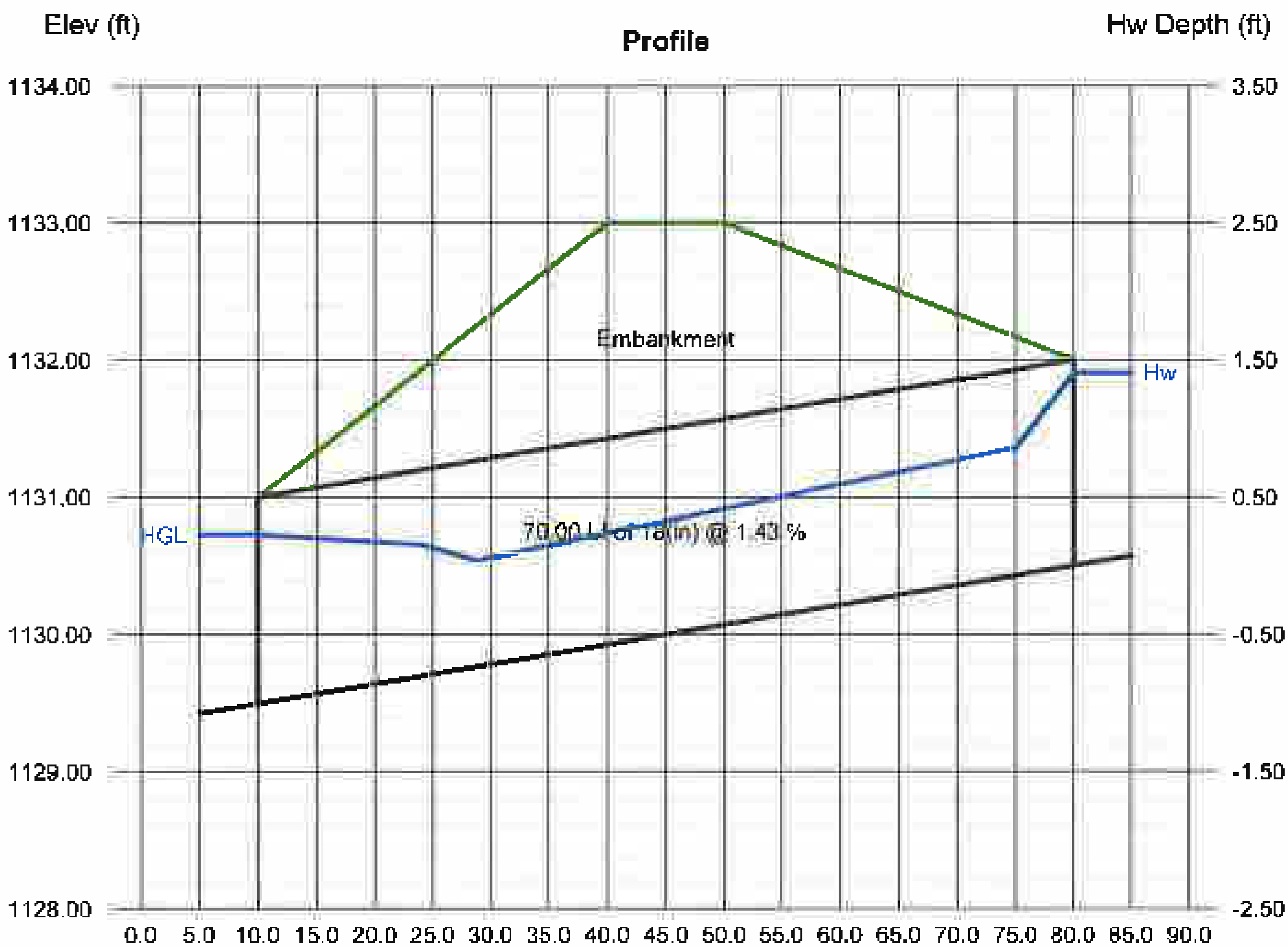
Top Elevation (ft)	= 1133.00
Top Width (ft)	= 10.00
Crest Width (ft)	= 10.00

Calculations

Qmin (cfs)	= 0.00
Qmax (cfs)	= 6.30
Tailwater Elev (ft)	= 0

Highlighted

Qtotal (cfs)	= 6.00
Qpipe (cfs)	= 6.00
Qovertop (cfs)	= 0.00
Veloc Dn (ft/s)	= 3.88
Veloc Up (ft/s)	= 5.09
HGL Dn (ft)	= 1130.73
HGL Up (ft)	= 1131.45
Hw Elev (ft)	= 1131.90
Hw/D (ft)	= 0.94
Flow Regime	= Inlet Control



Runoff Curve Number

Project: Keystone Generating Station: Stage IV Minor Permit Modification Application	By: DMD	Date: 1/4/2012
Location: Proposed Access Road Culvert 19 (East of West Valley Stage IV Expansion)	Checked:	Date:

Check one: ■ Present ❑ Developed

Runoff Curve Number and Hydrologic Group	Cover Description	CN			Area	Product of CN x Area
		Table 2-2	Figure 2-3	Figure 2-4	■ Acres ❑ miles^2 ❑ %	
B	Gravel Access Road	85			1.93	164.05
B	Brush (good)	48			0.48	23.04
			TOTALS		2.41	187.09

CN (weighted) = **Total Product / Total Area**

CN	**78**

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Runoff Curve Number

Project: Keystone Generating Station: Stage IV Minor Permit Modification Application	By: DMD	Date: 1/4/2012
Location: Proposed Access Road Culvert 19 (East of West Valley Stage IV Expansion)	Checked:	Date:

Check one: ☐ Present ■ Developed

Runoff Curve Number and Hydrologic Group	Cover Description	CN			Area	Product of CN x Area
		Table 2-2	Figure 2-3	Figure 2-4	■ Acres ☐ $miles^2$ ☐ %	
B	Gravel Access Road	85			1.93	164.05
B	Brush (good)	48			0.48	23.04
			TOTALS		**2.41**	**187.09**

CN (weighted) = Total Product / Total Area

CN	**78**

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Time of Concentration

Project: Keystone Generating Station: Stage IV Minor Permit Modification Application	By: DMD	Date: 1/4/2012
Location: Proposed Access Road Culvert 19	Checked:	Date:

Check one: ☐ Present ■ Developed

Sheet Flow

Segment ID	A to B	
Surface Description (Table 3-1)	Woods	
Manning's Roughness Coefficient, n (table 3-1)	0.4	
Flow Length, L	84	ft
Two-year 24-hour Rainfall, P_2	2.42	in
Land Slope, s	0.18	ft/ft
Travel Time, $T_t = (0.007*(n*L)^{0.8}) / (P_2^{0.5}*s^{0.4})$	**0.1491**	**hrs**

Shallow Concentrated Flow

Segment ID	B to C	
Surface Description (Paved / Unpaved)	Unpaved	
Surface Description Coefficient, C	16.1435	
Flow Length, L	34	ft
Watercourse Slope, s	0.35	ft/ft
Average Velocity, $V = C*s^{0.5}$	9.55	ft/sec
Travel Time, $T_t = (L) / (3600*V)$	**0.0010**	**hrs**

Channel Flow

Segment ID	C to D	
Section Base, b	0	
Section Depth, d	2	
Section Side Slope, z	2	
Cross Sectional Flow Area, $a = b*d + z*d^2$	8	
Wetted Perimeter, $p_w = b + (2*d)*(z^2 + 1)^{0.5}$	8.94	
Hydraulic Radius, $r = a / p_w$	0.89	
Channel Slope, s	0.04	
Manning's Roughness Coefficient, n	0.025	
Average Velocity, $V = (1.49*r^{2/3}*s^{1/2}) / (n)$	10.76	ft/sec
Flow Length, L	608	ft
Travel Time, $T_t = (L) / (3600*V)$	**0.0157**	**hrs**

Time of Concentration

Sheet Flow T_t	0.1491	hrs
Shallow Concentrated Flow T_t	0.0010	hrs
Channel Flow T_t	0.0157	hrs
Time of Concentration, T_c	**0.1658**	**hrs**
	9.95	**mins**

SCS TR55 Tabular Method

Watershed Title: Proposed Culvert 19

25 Year Type II Storm: Precipitation = 4.01 inches

Summary of Input Parameters

Subarea	Area (acres)	Curve Number	IA/P	Runoff (in)	Tc (min)	Adj. Tc (min)	Tt (min)	Adj. Tt (min)
1	2.410	78	0.141	1.89	9.950	12.000	0.000	0.000
Composite	**2.410**	**78**		**1.89**				

SCS TR55 Tabular Method

Watershed Title: Proposed Culvert 19

25 Year Type II Storm: Precipitation = 4.01 inches

Individual Subarea and Composite Hydrographs

Subarea	Time (hrs)											
	11.0	**11.9**	**12.2**	**12.5**	**12.8**	**13.2**	**13.6**	**14.0**	**15.0**	**17.0**	**20.0**	**26.0**
1	0.13	1.24	5.56	1.23	0.65	0.46	0.38	0.31	0.23	0.16	0.10	0.00
Composite	**0.13**	**1.24**	**5.56**	**1.23**	**0.65**	**0.46**	**0.38**	**0.31**	**0.23**	**0.16**	**0.10**	**0.00**

The peak flow is 5.6 cfs at 12.2 hrs.

Culvert Report

Hydraflow Express Extension for AutoCAD® Civil 3D® 2010 by Autodesk, Inc. — Tuesday, Jan 31 2012

Proposed Culvert 19

Invert Elev Dn (ft)	= 1115.50
Pipe Length (ft)	= 45.00
Slope (%)	= 2.22
Invert Elev Up (ft)	= 1116.50
Rise (in)	= 18.0
Shape	= Cir
Span (in)	= 18.0
No. Barrels	= 1
n-Value	= 0.012
Inlet Edge	= 0
Coeff. K,M,c,Y,k	= 0.0045, 2, 0.0317, 0.69, 0.5

Embankment

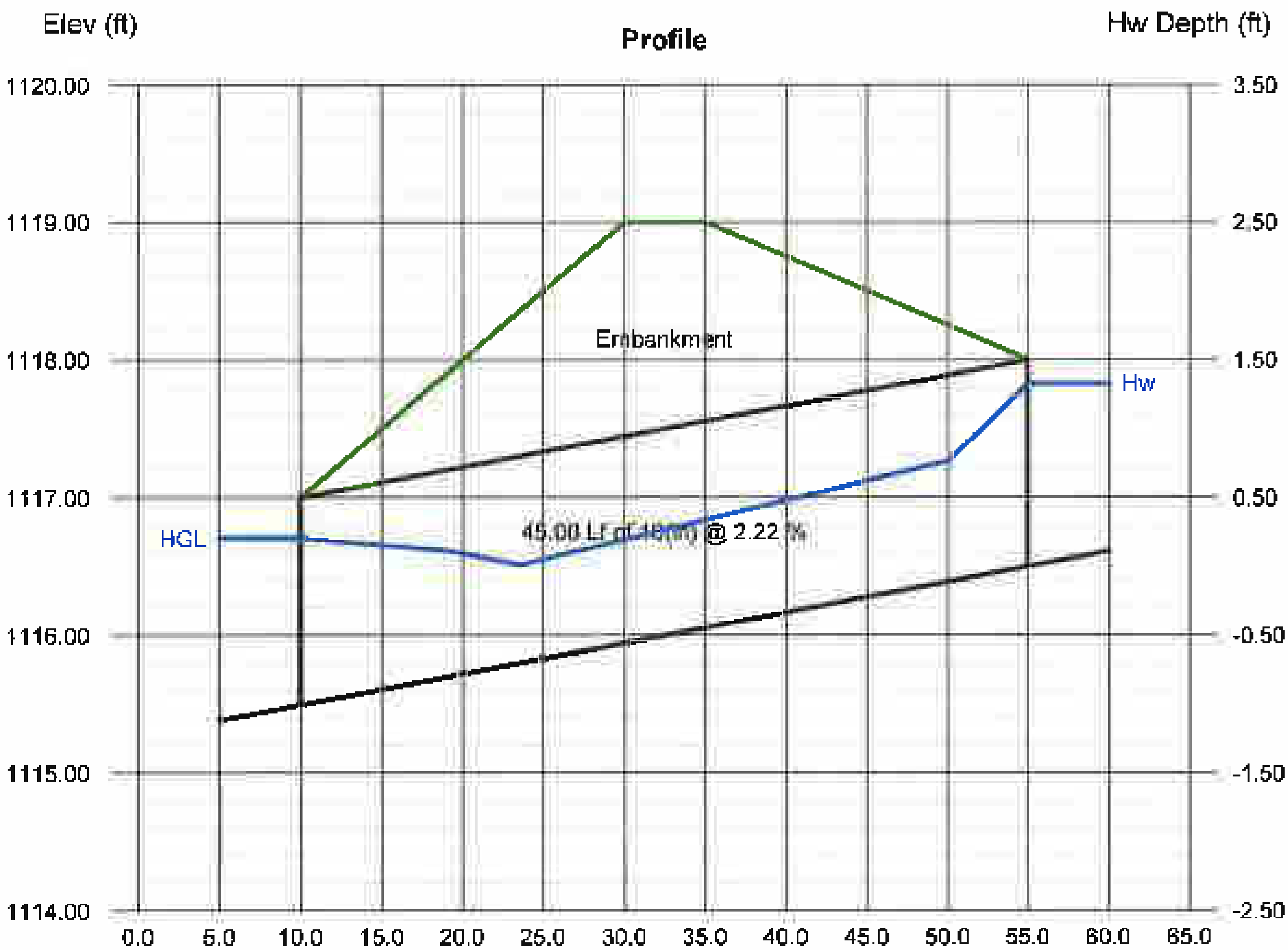
Top Elevation (ft)	= 1119.00
Top Width (ft)	= 5.00
Crest Width (ft)	= 5.00

Calculations

Qmin (cfs)	= 0.00
Qmax (cfs)	= 5.60
Tailwater Elev (ft)	= 0

Highlighted

Qtotal (cfs)	= 5.50
Qpipe (cfs)	= 5.50
Qovertop (cfs)	= 0.00
Veloc Dn (ft/s)	= 3.62
Veloc Up (ft/s)	= 4.91
HGL Dn (ft)	= 1116.71
HGL Up (ft)	= 1117.41
Hw Elev (ft)	= 1117.83
Hw/D (ft)	= 0.88
Flow Regime	= Inlet Control



ATTACHMENT 3: PROPOSED STAGE IV SOUTHWEST ACCESS ROAD DIVERSION DITCH DESIGN

Runoff Curve Number

Project:	By:	Date:
Keystone Generating Station:	DMD	1/4/2012
Stage IV Minor Permit Modification Application	Checked:	Date:
Location:		
Proposed Stage IV Access Road Southwest Diversion Ditch		

Check one:	■ Present	☐ Developed

Runoff Curve Number and Hydrologic Group	Cover Description	CN			Area	Product of CN x Area
		Table 2-2	Figure 2-3	Figure 2-4	■ Acres ☐ $miles^2$ ☐ %	
B	Brush (good)	48			1.93	92.64
			TOTALS		**1.93**	**92.64**

CN (weighted) = Total Product / Total Area

CN	**48**

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Time of Concentration

Project: Keystone Generating Station: Stage IV Minor Permit Modification Application	By: DMD	Date: 1/4/2012
Location: Proposed Stage IV Access Road SW Diversion	Checked:	Date:

Check one: ■ Present ☐ Developed

Sheet Flow

Segment ID	A to B	
Surface Description (Table 3-1)	Woods	
Manning's Roughness Coefficient, n (table 3-1)	0.4	
Flow Length, L	70	ft
Two-year 24-hour Rainfall, P_2	2.42	in
Land Slope, s	0.23	ft/ft
Travel Time, $T_t = (0.007*(n*L)^{0.8}) / (P_2^{0.5}*s^{0.4})$	**0.1168**	**hrs**

Shallow Concentrated Flow

Segment ID	B to C	C to D	D to E	
Surface Description (Paved / Unpaved)	Unpaved	Unpaved	Unpaved	
Surface Description Coefficient, C	16.1435	16.1435	16.1435	
Flow Length, L	266	302	100	ft
Watercourse Slope, s	0.16	0.13	0.30	ft/ft
Average Velocity, $V = C*s^{0.5}$	6.41	5.88	8.84	ft/sec
Travel Time, $T_t = (L) / (3600*V)$	**0.0115**	**0.0143**	**0.0031**	**hrs**

Channel Flow

Segment ID	C to D	
Section Base, b	0	
Section Depth, d	2	
Section Side Slope, z	2	
Cross Sectional Flow Area, $a = b*d + z*d^2$	8	
Wetted Perimeter, $p_w = b + (2*d)*(z^2 + 1)^{0.5}$	8.94	
Hydraulic Radius, $r = a / p_w$	0.89	
Channel Slope, s	0.00	
Manning's Roughness Coefficient, n	0.025	
Average Velocity, $V = (1.49*r^{2/3}*s^{1/2}) / (n)$	0.00	ft/sec
Flow Length, L	0	ft
Travel Time, $T_t = (L) / (3600*V)$	**#DIV/0!**	**hrs**

Time of Concentration

Sheet Flow T_t	0.1168	hrs
Shallow Concentrated Flow T_t	0.0289	hrs
Channel Flow T_t		hrs
Time of Concentration, T_c	**0.1457**	**hrs**
	8.74	**mins**

Runoff Curve Number

Project:	By:	Date:
Keystone Generating Station: Stage IV Minor Permit Modification Application	DMD	1/4/2012
Location: Proposed Stage IV Access Road Southwest Diversion Ditch	Checked:	Date:

Check one: ☐ Present ■ Developed

Runoff Curve Number and Hydrologic Group	Cover Description	CN			Area	Product of CN x Area
		Table 2-2	Figure 2-3	Figure 2-4	■ Acres ☐ $miles^2$ ☐ %	
B	Gravel	85			0.14	12.06
B	Brush (good)	48			1.79	85.83
			TOTALS		**1.93**	**97.89**

CN (weighted) = Total Product / Total Area

CN	**51**

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Time of Concentration

Project: Keystone Generating Station: Stage IV Minor Permit Modification Application	By: DMD	Date: 1/4/2012
Location: Proposed Stage IV Access Road SW Diversion	Checked:	Date:

Check one: ☐ Present ■ Developed

Sheet Flow

	Segment ID	A to B	
Surface Description (Table 3-1)		Woods	
Manning's Roughness Coefficient, n (table 3-1)		0.4	
Flow Length, L		64	ft
Two-year 24-hour Rainfall, P_2		2.42	in
Land Slope, s		0.06	ft/ft
Travel Time, $T_t = (0.007*(n*L)^{0.8}) / (P_2^{0.5}*s^{0.4})$		**0.1826**	**hrs**

Shallow Concentrated Flow

	Segment ID	B to C	C to D	D to E	E to F	F to G	
Surface Description (Paved / Unpaved)		Unpaved	Unpaved	Unpaved	Unpaved	Unpaved	
Surface Description Coefficient, C		16.1435	16.1435	16.1435	16.1435	16.1435	
Flow Length, L		46	97	214	280	12	ft
Watercourse Slope, s		0.13	0.25	0.15	0.25	0.50	ft/ft
Average Velocity, $V = C*s^{0.5}$		5.83	8.03	6.24	8.07	11.42	ft/sec
Travel Time, $T_t = (L) / (3600*V)$		**0.0022**	**0.0034**	**0.0095**	**0.0096**	**0.0003**	**hrs**

Channel Flow

	Segment ID	G to H	
Section Base, b		0	
Section Depth, d		2	
Section Side Slope, z		2	
Cross Sectional Flow Area, $a = b*d + z*d^2$		8	
Wetted Perimeter, $p_w = b + (2*d)*(z^2 + 1)^{0.5}$		8.94	
Hydraulic Radius, $r = a / p_w$		0.89	
Channel Slope, s		0.19	
Manning's Roughness Coefficient, n		0.025	
Average Velocity, $V = (1.49*r^{2/3}*s^{1/2}) / (n)$		24.40	ft/sec
Flow Length, L		216	ft
Travel Time, $T_t = (L) / (3600*V)$		**0.0025**	**hrs**

Time of Concentration

Sheet Flow T_t	0.1826	hrs
Shallow Concentrated Flow T_t	0.0250	hrs
Channel Flow T_t	0.0025	hrs
Time of Concentration, T_c	**0.2100**	**hrs**
	12.60	**mins**

SCS TR55 Tabular Method

Watershed Title: Proposed Stage IV Southwest Access Road Diversion Ditch (Pre)

25 Year Type II Storm: Precipitation = 4.01 inches

Summary of Input Parameters

Subarea	Area (acres)	Curve Number	IA/P	Runoff (in)	Tc (min)	Adj. Tc (min)	Tt (min)	Adj. Tt (min)
1	1.930	48	0.500	0.27	8.740	6.000	0.000	2.760
Composite	**1.930**	**48**		**0.27**				

SCS TR55 Tabular Method

Watershed Title: Proposed Stage IV Southwest Access Road Diversion Ditch (Pre)

25 Year Type II Storm: Precipitation = 4.01 inches

Individual Subarea and Composite Hydrographs

Subarea	Time (hrs)											
	11.0	11.9	12.2	12.5	12.8	13.2	13.6	14.0	15.0	17.0	20.0	26.0
1	0.00	0.00	0.30	0.13	0.09	0.07	0.06	0.05	0.05	0.03	0.02	0.00
Composite	**0.00**	**0.00**	**0.30**	**0.13**	**0.09**	**0.07**	**0.06**	**0.05**	**0.05**	**0.03**	**0.02**	**0.00**

The peak flow is 0.4 cfs at 12.1 hrs.

SCS TR55 Tabular Method

Watershed Title: Proposed Stage IV Southwest Access Road Diversion Ditch (Post)

25 Year Type II Storm: Precipitation = 4.01 inches

Summary of Input Parameters

Subarea	Area (acres)	Curve Number	IA/P	Runoff (in)	Tc (min)	Adj. Tc (min)	Tt (min)	Adj. Tt (min)
1	1.930	51	0.479	0.37	12.600	12.000	0.000	0.600
Composite	**1.930**	51		**0.37**				

SCS TR55 Tabular Method

Watershed Title: Proposed Stage IV Southwest Access Road Diversion Ditch (Post)

25 Year Type II Storm: Precipitation = 4.01 inches

Individual Subarea and Composite Hydrographs

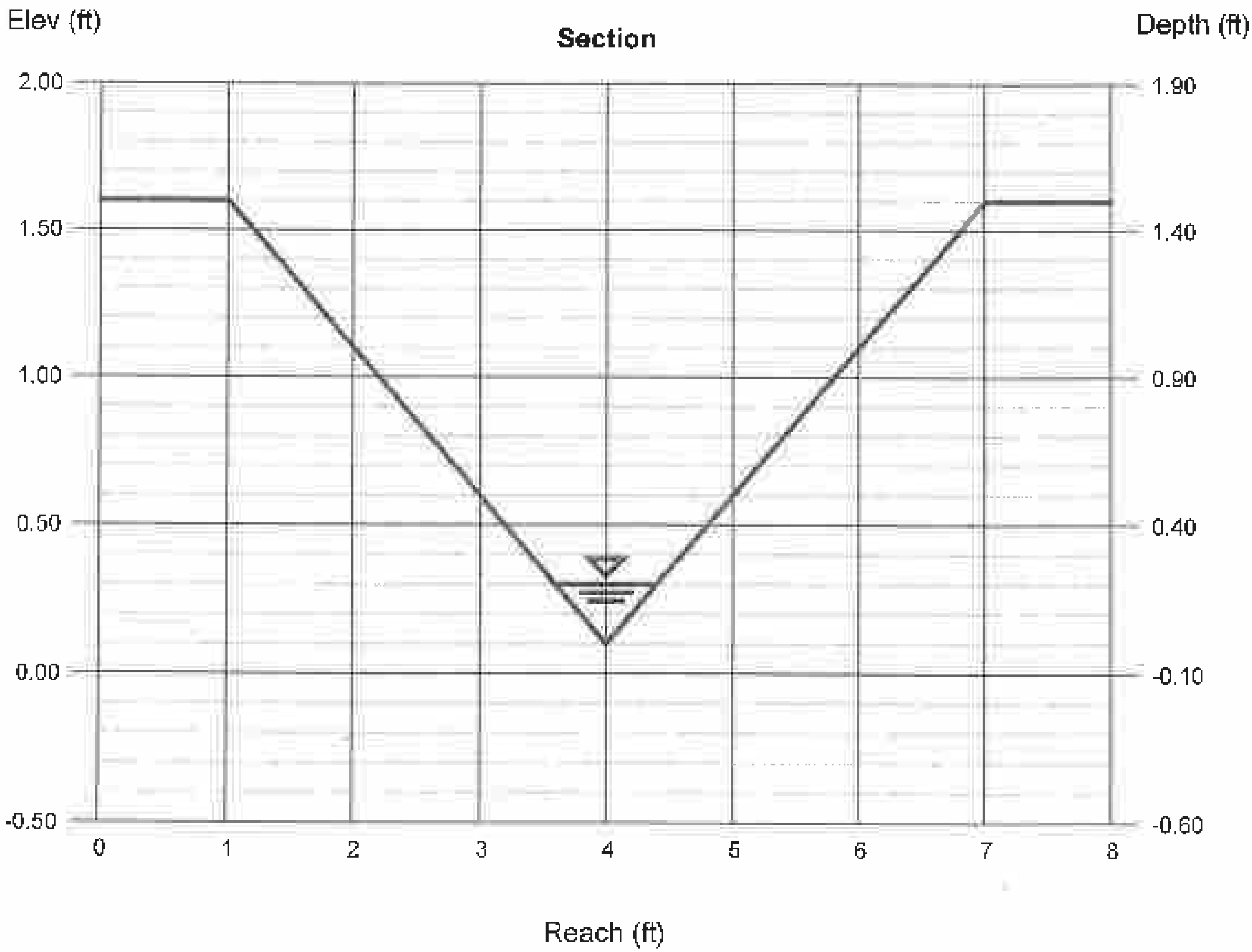
Subarea	Time (hrs)											
	11.0	**11.9**	**12.2**	**12.5**	**12.8**	**13.2**	**13.6**	**14.0**	**15.0**	**17.0**	**20.0**	**26.0**
1	0.00	0.00	0.44	0.21	0.14	0.10	0.09	0.08	0.06	0.05	0.03	0.00
Composite	**0.00**	**0.00**	**0.44**	**0.21**	**0.14**	**0.10**	**0.09**	**0.08**	**0.06**	**0.05**	**0.03**	**0.00**

The peak flow is 0.4 cfs at 12.2 hrs.

Channel Report

Proposed Stage IV Southwest Access Road Diversion Ditch (min slope)

Triangular		**Highlighted**	
Side Slopes (z:1)	= 2.00, 2.00	Depth (ft)	= 0.20
Total Depth (ft)	= 1.50	Q (cfs)	= 0.400
		Area (sqft)	= 0.08
Invert Elev (ft)	= 0.10	Velocity (ft/s)	= 5.00
Slope (%)	= 18.00	Wetted Perim (ft)	= 0.89
N-Value	= 0.025	Crit Depth, Yc (ft)	= 0.31
		Top Width (ft)	= 0.80
Calculations		EGL (ft)	= 0.59
Compute by:	Known Q		
Known Q (cfs)	= 0.40		



Channel Report

Hydraflow Express Extension for AutoCAD® Civil 3D® 2010 by Autodesk, Inc. Thursday, Jan 5 2012

Proposed Stage IV Southwest Access Road Diversion Ditch (max slope)

Triangular

Side Slopes (z:1)	= 2.00, 2.00
Total Depth (ft)	= 1.50
Invert Elev (ft)	= 0.10
Slope (%)	= 25.00
N-Value	= 0.025

Calculations

Compute by:	Known Q
Known Q (cfs)	= 0.40

Highlighted

Depth (ft)	= 0.19
Q (cfs)	= 0.400
Area (sqft)	= 0.07
Velocity (ft/s)	= 5.54
Wetted Perim (ft)	= 0.85
Crit Depth, Yc (ft)	= 0.31
Top Width (ft)	= 0.76
EGL (ft)	= 0.67