PN-AAM-709

2630017/65

TECHNICAL REPORT NO. 21

DESIGN REPORT

EL-HAMMAMI PIPELINE

By

The Fort Collins Staff Team

AID/NE-C-1351

Janaury 1983

EGYPT WATER USE AND MANAGEMENT PROJECT

22 El Galaa St., Bulak, Cairo, Egypt

DESIGN REPORT
EL-HAMMAMI PIPELINE

TECHNICAL REPORT NO. 21

Prepared under support of

WATER DISTRIBUTION RESEARCH INSTITUTE, WATER RESEARCH CENTER

MINISTRY OF IRRIGATION, GOVERNMENT OF EGYPT

Contracting agencies

Colorado State University
Engineering Research Center
Ft. Collins, Colorado 80523
USA

Consortium for International
Development
5151 E. Broadway, Ste., 1500
Tucson, Arizona 85711 USA

All reported opinions, conclusions or
recommendations are those of the writers
and not those of the supporting or contracting agencies.

ABSTRACT

Two proposed low pressure buried asbestos cement pipelines are designed to replace the present open channel irrigation delivery system in the El-Hammami area, Egypt. Together, the two pipelines will serve a gross area of 800 Feddans. Each pipeline is served by an individual pumping station consisting of three pumps. Additionally, a gated water stand connects the two pumping plants to allow the pumping load to be transferred should a pump breakdown occur. Venturi meters are proposed in each of the pipelines to monitor discharge requirements.

At intervals along the lines, risers capped with alfalfa valves spill the flow into weirs or flumes and in turn into concrete lined ditches. These ditches will generally serve more than one farmer and inverted syphons will be used to cross canals. The maximum water surface elevation at each alfalfa valve is determined as a function of the elevation of the highest field to be served by that valve and of all form and friction losses.

The designed discharge capacity of each pipeline is based upon a peak consumptive rate of 10 mm per day. Water table contribtuions are assumed to offset the conveyance and irrigation losses during the peak consumptive periods. The maximum pumping period is assumed to be 14 hours per day.

System head curves (total dynamic head versus discharge) and pumping system capacity curves are developed for the designed system. The system head curves are then matched with the pumping system capacity curves to obtain the final design flow rate through the system.

143 Pages 22 Tables 4 Figures

ملخص

فى منطقة الحمامى بمصر تم تصميم ماسورتين من الاسمنت الاسبستوس ذات الضغط المنخفض لاحلالها محل مجارى الرى المائيه الحاليه . هذا وسيخدم الماسورتين معا منطقه مساحتها ٨٠٠ فدان ،ويخدم كل ماسوره محطه طلمبات مستقله تتكون من ثلاث مضخات . وبالاضافه الى ذلك هناك بوابه مياه تتصل بالماسورتين بغرض السماح للمياه بالمرور فى حاله حدوث عطل فى احدى المضخات ـ ومركب على كل ماسوره عدد من الفنتورى ميتر للتحكم فى كمية التصرف المطلوبه .

ومركب على ابعاد مختلفه على طول الماسوره فتحات ذات غطاء وعليها صمامات من نوع الفا الفا لتسمح بمرور التصرف فى الهدارات ثم بعد ذلك فى المجارى المائيه المبطنه بالاسمنت . وهذه المجارى المائيه تخدم عاده اكثر من فلاح كما ستستعمل مواسير مدفونه لنقل المياه تحت الترع وقد تم تحديد اعلى منسوب للماء فى كل صمام كداله فى اعلى منسوب للحقل الذى يخدمه هذا الصمام مع حساب الفواقد الخاصه بالماسوره كفواقد الاحتكاك وغيرها .

وقد تم تحديد التصرف المصمم عليه لكل ماسوره على حسب اعتبار ان اعلى معدل للاستهلاك هو ١٠ مم / اليوم . كما انه لم يحسب تاثير منسوب الماء الارضى على التصميم على اعتبار انه يعادل الفواقد الخاصه بالتوصيل والرى اثناء فترات الاستهلاك القصوى وقد تم اعتبار فترة الضخ القصوى ١٤ ساعه / اليوم .

هذا وقد تم استنتاج ورسم منحنيات التصميم لارتفاع الضغوط (بين التصرف والارتفاع الديناميكى) وكذلك تم رسم منحنيات تصميم الطلمبه وللحصول على التصرف النهائى للتصميم المطلوب يتم تطابق منحنيات تصميم الطلمبه مع منحنيات الارتفاع الديناميكى .

١٤٣ صفحه ٢٢ جدول ٤ شكل

TABLE OF CONTENTS

		Page
Abstract (English and Arabic)		i
List of Figures		iv
List of Tables		v
Acknowledgements		vi
I.	Introduction	1
	1.1 History	1
	1.2 Design Concepts	2
	1.3 Appraisal of Given Design Concepts	5
II.	Pipe Routes and Areas Served	12
	2.1 Pipe Route	12
	2.2 Areas Served	13
	2.3 Design Flow Rate Per Outlet	13
	2.4 System Capacity	15
III.	Design of Surface Distribution Component Systems	21
	3.1 Minimum Needed Elevation of the Water Surface at the Top of the Improved Meskas	21
	3.2 Distributor Concrete Lined Ditch Design with Alfalfa Valves, Flumes Versus Weirs and Gates Versus Inverted Siphons	22
IV.	System Head Curves for the Pipeline Systems	100
	4.1 Design Rotation	100
	4.2 Pipe Selection	102
	4.3 The System Head Curves	104

LIST OF FIGURES

		Page
Figure 3.1	Flume Head Loss and Step Size.....	35
Figure 3.2	A Typical Inverted Siphon.....	44-A
Figure 4.1	System Head Curve for Pipeline 1.....	132
Figure 4.2	System Head Curve for Pipeline 2..... .	133

LIST OF TABLES

		Page
Table 1.1	Comparison of Alternatives	11
Table 2.1	Areas Served and Design Flow Rates in Pipeline 1	17
Table 2.2	Areas Served and Design Flow Rates in Pipeline 2	18
Table 3.1	Head Losses in Alfalfa Valves	30
Table 3.2	HP 25 Program for the Computation Q vs h_1 in Trapezoidal Flumes	31
Table 3.3	Q vs h_1 for Trapezoidal Flumes	32
Table 3.4	Head Loss and Step Size for Flumes	34
Table 3.5	Flow vs. Head Values for Fayyum Weir	42
Table 3.6	Concrete Lined Ditch Design	43
Table 3.7a	Head Losses in Inverted Siphons	47
Table 3.7b	HP 25 Program for Inverted Siphons	47
Table 3.7c	Inverted Siphon Computations	50
Table 3.8	Head Loss in Gates	51
Table 3.9	Design Flow Rates and Maximum Flow Rates of Outlets	98
Table 3.10	Details of Outlets	99
Table 4	Peak Consumptive Use of Water by Crops in Arizona	20
Table 4.1	Design Rotation Schedules Day 1 Water Issues	101
Table 4.2	HP 25 Program for System Head Values	118
Table 4.3	System Head Variation for Pipe 1	119
Table 4.4	System Head Variation for Pipe 2	131
Table 5	Minimum Water Surface Elevation at Improved Meska Pipeline 1	23
Table 6	Minimum Water Surface Elevation at Improved Meska Pipeline 2	24

ACKNOWLEDGEMENTS

The authors wish to express their thanks and appreciation for the contributions made to this paper by the staff of the Egypt Water Use and Management Project, and to the field teams at Mansuriya, Kafr El-Sheikh and El-Minya.

Special thanks are extended to Mr. Michael Moodie, Mr. Subramaniaiyer Sritharan, Mr. Robert Vandenberg, and Engineers Salah Abo El Ela, Mostafa Saleh and Fahim.

The Project is funded jointly by the Arab Republic of Egypt, and by the United States Agency for International Development. The United States Agency for International Development In Egypt is under the directorship of Mr. Michael P. W. Stone. Mr. John Foster is the United State Agency for International Development Project Officer for the Egypt Water Use and Management Project.

The Egypt Water Use and Management Project is implemented under the auspices of the Ministry of Irrigation's Water Management and Irrigation Technologies Research Institute and in collaboration with both the Ministry of Irrigation and the Ministry of Agriculture through the Soil and Water Research Institute and the Agriculture Economics Institute, which provide the Project with personnel and services.

The Consortium for International Development, with executive offices in Tucson, Arizona, is the United States Agency for International Development Contractor for the Project. American Project personnel are drawn from the faculties of Colorado State University, the lead American university taking part in the Project, Oregon State University, New Mexico State University, and Montana State University. The Project Director is Dr. Hassan Wahby and the Project Technical Director is Dr. Eugene Quenemoen. Dr. E. V. Richardson is the Campus Project Coordinator at Colorado State University.

CHAPTER 1

INTRODUCTION

1.1 HISTORY

The El-Hammami Pipeline Design, as presented in this paper, was begun on March 17, 1980 with briefings from Project members involved in the initial design. The original estimated completion date for this project was April 17, 1980. However, work on the actual pipeline design did not begin until April 24, 1980 after a general agreement had been reached on the design concept. The final pipeline design (as reported here) was completed on May 27, 1980.

The previous history of the pipeline scheme began in 1979 with the design of a high pressure pipeline along the present branch canal route. The design split the area into two roughly equal areas, each served by its own pumping plant and pipe.

Later a second pipeline design was conceived. This design kept the previous pipe routes and division of area. The pumping plant design was unavailable, making it impossible to determine if the second design envisioned a high or low pressure pipeline. Each outlet consisted of a vertical riser pipe linked by a 90° elbow to a horizontal pipe with a value. A second 90° elbow connected the horizontal section to a short drip pipe. The drip pipe discharged into a stilling basin, which in turn discharged through an orifice (short pipe) into the approach section of a faiyum weir, over the weir, and finally into the meska. The sill elevation of the weir was to have equalled the highest field to be served plus 20 cm. If one designed the weir for no submergence, in order to maximize the accuracy of flow measurement, then only 20 cm of head would be allowed for; the slope of the meska, turnout from meska

to farm ditch, slope of the farm ditch, turnout from farm ditch to field, and depth of flow on the field. The TDH (total dynamic head) of the farthest downstream outlet in the supply pipe (#7 for pipe 1 and #7 for pipe 2) was assumed to be 30 cm above the weir sill elevation. The maximum Q over the weir required a 15 cm head over the weir sill. This left 15 cm for head loss through the orifice (the short pipe from stilling basin to weir approach section), riser pipes, 2 elbows, valve, entrance (sideflow T), and exit. The minimum breadth of the weir was 1 cm, compared with a recommended minimum b = 5 cm.

The second design was based upon conflicting pumping data of 12 hours/day and 16 hours/day. The concrete lined circular cross section was expensive and it may be noted that a trapezoidal slipform is now available to EWUP. A design evapotranspiration (ET) rate of 8 mm/day was sued without allowing for leaching or losses in defining the system capacity. The gross area to be served was estimated to be 780 feddans, rounded up to 400 feddans per pumping plant, without adjusting for houses, roads, trails, canals, ditches, drains or fallow ground.

1.2 DESIGN CONCEPTS

The design considerations and concepts consisted of the following "givens":

1. buried pipeline to replace branch canals
2. 14 hours/day pumping at time of peak ET
3. modified demand system such that, at most, half of the outlets would be open at one time (with the outlets opened according to demand and supply)

4. during non-peak ET time, the three pumps would not be required to operate simultaneously for 14 hours/day, each day
5. rotation between meskas
6. rotation between farmers on each meska if they wished a large Q, or simultaneous use if they wished to split the Q
7. 2.0 cfs minimum flow capacity per pipe outlet
8. gravity flow from pipe to fields
9. pumping plant Q designed assuming no losses in the conveyance system in the irrigation application. The water table is high enough that such losses remain in the plant root zone and consequently are available to the plants.
10. eventual farmer control of the pipeline system
11. no night irrigation, unless it is needed to make up for water needs after mechanical breakdown or power outage
12. electrical power
13. flush and wasteway flow into drains
14. simple and strong construction
15. one meter cover over the pipe
16. adequate (not optimal) and transferable design
17. back-up equipment is not to be installed, but kept in storage
18. asbestos-cement pipe
19. 3 pumps

    1 x 50% or 75% design Q

    2 x 25% design Q
20. flexible design to allow the farmers to improve meska whenever they wish after the installation of the pipeline
21. flow measuring devices at each pipe outlet

22. venturi meter on the pipe near the pumping plant
23. pipe velocity greater than 2.0 ft/sec to reduce sedimentation
24. overflow weir in stand at meska 2 (pipe 1, outlet 7, north, pipe)
25. the 110 feddans in the northeast corner of the area are to be ignored
26. pipes 1 and 2 are to be connected in a gate stand
27. pipe route to be on the north side of the El Hammami Canal, the east side of the El Shimi Branch Canal, and the south side of the Remal Kafr Hakim Drain in general
28. large pump stands
29. laundry water is to be provided for the village
30. to provide a good bedding for pipe in the trench, the contractor may install tile drains beneath the pipe (with filter and/or gravel pack) with EWUP paying for materials, and contractor including installation in his bid price
31. a minimal number of alfalfa valve sizes
32. an extra 20% alfalfa valves for spares
33. one spare pump (and motor) for each pump size
34. eventual remote control (and automation) of outlets on pipe of 7N
35. TDH design criteria such that there is less than 12 feet TDH at mid-point of pipeline and less than 16 feet TDH at pumping plant
36. use of turbine or submerged centrifugal pumps so that there will be no priming problems
37. the two pumping plants should be mirror images of each other in pump/motor/motor control sizes, so design Q and TDH should be close

38. excavation starting in August or September

39. no limit on investment

40. and, finally, located near the inlet to El Shimi Branch Canal.

It was recommended that a slope of 0.00015 be used for the concrete-lined ditch (CLD). A list of available asbestos-cement pipe diameters was provided (75 mm, 100, 125, 150, 200, 225, 300, 375, 450, 525, 600 mm; 3 inches, 4, 5, 6, 7, 8, 9, 10, 12, 15, 18, 20, 24 inches). It was also recommended that; meska 1 (Pipe 1, outlet 5) be elevated and lined soon after pipeline construction, that meska 2 (pipe 1, outlet 7NP) be replaced by a pipeline with outlets spaced to serve every 2 or 3 farmers by the same outlet, and be remote controlled, and that a Hazen-Williams (C=140) be used for asbestos-cement pipe and rubber-gasket joints.

In addition, EWUP Project Technical Report #1 recommended giving priority to uniform distribution along the pipeline between meskas. However, the design priority was modified to give farmers equal access to the water.

1.3 APPRAISAL OF GIVEN DESIGN CONCEPTS

1.3.1 System Flow Rate Capacity

The design equation used previously was:

$$Q = C \left(\frac{A \cdot ET}{F}\right)$$

where

A = gross project area (ha)

ET = design evapotranspiration rate (mm/d)

F = pumping hours per day (h/d)

C = conversion factor

Originally, the fact that this previous design had ignored application losses was considered unacceptable. It was predicted that, if an optimal design of a surface application system (field) was made (considering that the shallow water table means that good quality water is cheaply available) for well drained El Hammami farms, the optimal E_s (storage efficiency) would be at least 95%, and optimal (application efficiencies) at most would be 50%. This optimal E_a is low because the value of the crops, the cheap water and labor, and the fact that soil textures range from sands to loamy sands. As a result, it was estimated that the above equation would result in a system with inadequate capacity, and consequently in severe crop stress.

A second alternative would be at minimum to double the above Q to allow for losses. In fact, however, losses will "deep" percolate to the water table, which is already high. It took Pakistan 100 years to develop severe waterlogging and salinization problems with a system with inadequate capacity. In Egypt, where they have more water than developed (irrigated) land, it only took 20 years.

A third alternative would be to provide pumping capacity from the main canal to supply only the peak ET (the above Q), provide pumping capacity for losses from the water table aquifer (shallow wells or tile drains), and provide for salt transport by off-peak ET pumping from the canal in excess of actual ET. This alternative assumes a conservative design, meaning the water table would be kept deep by drainage or conjunctive use.

The fourth alternative, would be to provide pumping capacity from the canal to supply only the peak ET (the above Q). It was assumed

that enough losses (deep percolation to the water table) would (over several irrigation intervals) return to the root zone of the deficiently irrigated portion of the field to avoid severe stress of that crop. So, the design relies on a high water table. If the water table had only small amplitude and frequency fluctuations, then this alternative would probably work well with regard to moisture stress. (Salt stress in the deficitly irrigated area will be severe, almost independently of the water table fluctuation.) Without better control of the high fluctuation and gradual rise of the water table over the summer, the crop in the deficiently irrigated area would probably suffer severe soil moisture stress as well as severe salt stress. (This hypothesis is best checked by pressure bomb readings of plant potential, rather than soil moisture and salinity measurements.) Because of the expected severe reduction in yield in the deficiently irrigated area, this alternative seems undersirable. However, it is an improvement on all previous alternatives because it considers water table contribution to the root zone, which earlier alternatives ignored.

The fifth alternative is similar to the fourth, except that an extensive tile drain system (spacing 15 m to 40 m) would be installed and managed to maintain a nearly constant water table level at a nearly constant (optimal) depth below the ground surface. This tile drain system would remove most of the fluctuations in the water table, and "optimally" manage its depth. Drainage water would be wasted to the large surface drains. This improvement would greatly reduce the soil moisture stress to the crop in the deficiently irrigated area. Because this alternative is best designed after land leveling, and because land leveling is now in progress and will likely accelerate after installation of this irrigation water supply pipeline, this alternative is still

feasible. Therefore, the design of the pipeline under alternative four can be supported while still supporting better management of the water table.

The sixth alternative, and the preferred choice, would be to pump back the drainage water of alternative five (except as needed for salt transport) into the pipeline (or other parts of the conveyance system before the water reaches the field) so that the volume of water available to each farmer would permit him to adequately surface irrigate a large fraction (more than 0.95) of his fields with a small area of deficient irrigation. Compared to alternative five, this alternative would greatly reduce the salt stress on the crop in the deficiently irrigated field. Compared to alternative four, this alternative would greatly reduce the soil moisture and salt stress of the crop, increase total plant potential in the deficiently irrigated part of the fields, and greatly reduce the fraction of the field that is deficiently irrigated. The increased crop yield of alternative six compared to alternative four would thus be significant.

Unfortunately, it would be very difficult to properly predict the performances of alternatives four and six. The ground water hydraulics and surface irrigation hydraulics could model the flow of water to the crop, but the response of the crop, to various salinity levels due to water table contribution to the root zone in an arid climate, is uncertain. Most American experience in subsurface irrigation (which is the additional component of alternative five) has involved projects in Florida, where the climate is humid and the soil is organic. Research based on such systems may not be transferable to the El Hammami area.

1.3.2 Pipeline vs Lined Ditch

It is predictable that detailed economic evaluations of El Hammami and other Egyptian irrigation water supply schemes will usually find lined ditches to be better than pipelines in mildly undulating ("flat") land. The most comparable American experience is in southern Arizona and southern California, where the winters are relatively mild and a 2 1/2 inch thick concrete can be used for lined ditches. The land is "flat," the ET (and therefore flow rate) is large, and the depth (volume per unit ground surface area) of required water per year (and therefore pumping hours per year) is large. In those areas of the U.S., pipelines are rarely used. Lined ditches serve almost all fields.

The following computation provides more substantial evidence for this prediction. It is possible to check an alternative consisting of lined ditches along the same routes as the following pipelines design, so the length of pipeline of alternative "A" will be the same as the length of CLD of alternative "B". In this case, the CLD is designed for twice the design Q, which is roughly comparable to the pipeline capacity when inflow comes from both ends. (Please see Table 1.1) For alternative B1 the CLD must be made by the currently available EWUP slipform:

$b = 1$ feet

$z = 1:1$

maximum depth = 30" = 2.5 feet

The slope of the CLD is to be set so that twice design Q (20.8 cfs) will have a normal depth of flow in the above ditch of 2.5 feet. This means that freeboard and ponding depths will be provided by hand plastered CLD tops. For $b = 1$, $y_n = 2.5$ ft the slope is 0.00060. This hand plastering would not be needed with an adequate slipform of $b = 2$ ft,

z = 1:1, and maximum depth = 36" = 3.0 ft. For alternative B2, b = 2', the slope is 0.00059, and y_n = (3.0 ft - 200m/0.3048 m/ft * 0.00059 x 3/4) = 1.95 ft. For alternative B3 (b = 2, y_n = 3.00 ft and hand plastering of the CLD top is needed for ponding and freeboard), the slope is slightly less than 0.00010.

As can be seen from Table 1.1, the CLD alternatives reduce 24% to 62% of the TDH, greatly reduce the number of low measuring devices, almost eliminate pipe, and eliminate stands, valves and medium pressure gates. Also, the "PIPELINE" alternative uses only 60% less CLD than the "CLD" alternative. Both the "PIPELINE" and "CLD" alternatives use about as many 20" gates and 15" and 18" inverted siphons with slide gates. Both alternatives can be remote controlled.

More detailed analysis would substantiate the prediction that lined ditches will be cheaper than pipelines for the El Hammami area.

TABLE 1.1 COMPARISON OF ALTERNATIVES

	ALTERNATIVES			
	A	B - CLD		
	Pipe	1	2	3
		b = 1 ft y_n = 2½ ft S_{CLD} = 0.00060	b = 2 ft y_n = 1. ft S_{CLD} = 0.00059	b = 2 ft y_n = 3 ft S_{CLD} = 0.00010
Critical elevation of water surface at top of meska CLD (m)				
Pipe 1	17.89	17.89	17.89	17.89
Pipe 2	18.44	18.36	18.36	18.44
Length to critical outlet: (m)				
Pipe 1	2470	2470	2470	2470
Pipe 2	2240	2405	2405	2240
Total dynamic head (estimated) (m)				
Pipe 1	4.37	2.87	2.85	1.64
Pipe 2	4.19	3.20	3.28	2.16
Pipe length (excluding inverted siphons for outlets)				
Pipe 1	2942	40	40	40
Pipe 2	2405	30	30	30
CLD length (excluding starts and stops)				
Pipe 1	1184	2942	2942	2942
Pipe 2	960	2405	2405	2405
Stands	6	0	0	0
Flumes				
Pipe 1	3	4	4	4
Pipe 2	7	2	2	2
Weirs				
Pipe 1	7	2	2	2
Pipe 2	1	1	1	1
Venturi Meter	3	0	0	0
Alfalfa Valves				
Pipe 1				
10"	6	0	0	0
14"	3	0	0	0
Pipe 2				
10"	4	0	0	0
14"	1	0	0	0

CHAPTER 2

2.1 PIPE ROUTE

The pipe route is largely determined by the following "given": "that the design be flexible enough to allow the farmers to be willing to improve meskas whenever they wish after the pipeline is installed." This means that the pipe route must serve the present system of meskas, sakias and branch canals, as well as some hypothetical (future) improved system. Therefore, the pipeline should follow the current canal route within the government's right-of-way.

Because the pipeline should serve the area served by the canals during its construction, the pipe should be set on the side of the canals. The previous design located the pipe on the north side of the El Hammami Canal and west side of the El Shimi Branch Canal.

An EWUP well is located northeast of the inlet to the El Shimi Branch Canal. Water from this well could be pumped into the pipeline. If a gate stand is located at the branching of the pipe, and is to serve also as a pump stand (for the well), and if the pipe from well to stand is to be short and straight for safety, then the stand should be to the northeast of the inlet to the El Shimi Branch Canal. Therefore, the pipe route should be north of the El Hammami Branch Canal and east of the El Shimi Branch Canal.

Another consideration concerning the planning of the pipe route is the number of inverted siphons neededfor each alternative route. If the pipe is located on the north side of El Hammami Canal, an inverted siphon and two pressure pipes are needed to service the south side of the canal (which will become the road). If it is located on the south side, nine inverted siphons and one pressure pipe are needed to serve the north side. If the pipe is located on the east side of the El Shimi Branch

Canal, sixteen inverted siphons and one pressure pipe are needed to serve the west side. If it is located on the west side, seventeen inverted siphons and one pressure pipe are needed to serve the east side. Thus, the inverted siphon consideration (in agreement with the well consideration) indicates the best route to be along the east side of the El Shimi Branch Canal. EWUP decided to route the pipe on the north side of the El Hammami Canal and east side of the El Shimi Branch Canal.

2.2 AREAS SERVED

A map of the region, delineating the present boundaries of the regions served by each of the proposed pipe outlets, was provided. The boundaries did not appear to fit together accurately. Due to lack of better information, however, and because the following procedures reduced the importance of accurate boundaries, the boundaries were accepted for the following analysis. The map also lists the gross area of each region. Without means of actual measurement, the areas had to be accepted as accurate.

Because of the "given" that the pumping plants should be mirror images of each other (so the design Q and TDH should be similar), the boundaries of the region to be served by each pipe were changed. The next table lists regions served by each pipe.

2.3 DESIGN FLOW RATE PER OUTLET

According to the present design, each pipe outlet will discharge into one or more of the irrigation regions. Usually each region is served by one outlet. However, as in the previous design, several of the regions are also served from two pipe outlets.

The general design procedure was as follows:

1. If a region is served by two pipe outlets, the part of the region served by each pipe outlet was estimated.

2. The flow rate required to satisfy the crop's needs in each adjusted region served by each pipe outlet during the time of peak ET with water flowing 50% of the time that the pump is pumping (0.50* 14 hours pumping per day/24 hours per day = 0.292 fractional time) was estimated.

3. The design Q for each outlet was set equal to the largest Q from (2) (after rounding up to two significant digits) from the several regions served by the pipe outlet.

4. For each pipe outlet, if the Q from (3) is less than the design minimum Q per outlet (2 ft^3/s = 0.0566 m^3/s), then the Q was rounded up to the minimum Q per outlet (see below).

5. If the design Q for an outlet must run for more than 80% of the pumping period to satisfy the water volume needs of all the areas served by that outlet, and if the outlet is not likely to be critical for TDH = f(L), then the design Q was increased to a convenient number. The adjusted design Q was checked to be certain it yields an acceptable fractional time of flow.

This procedure resulted in a range of flows per outlet from 0.0566 m^3/s (2.0 ft^3/s) to 0.167 m^3/s (5.9 ft^3/s). The next table lists the area of each region, the flow rates for each region (step 2) and final design flow rate from each pipe outlet (steps 3, 4, 5).

The minimum flow rate per outlet is a parameter of the optimal design of the irrigation system with special consideration for the application and use subsystems. Until it is contradicted by an optimal design, it is estimated that 57 L/s (2.0 ft^3/s) is a large enough flow

rate at the farmer's field to result in a high uniformity irrigation on a well designed and leveled half-acre border. 57 L/s was also chosen because it is greater, and hopefully better, than the largest irrigation time average flow rate (45 L/s) reported (p. 17 of ASAE #79-2566) in the El Hammami area.

2.4 SYSTEM CAPACITY

The gross area served by each pipeline is listed in Tables 2.1 and 2.2. It is fair to assume that, at minimum, 5% of the gross area consists of permanently established roads and housing, while another 5% of the gross area consists of branch canals, drains, meskas, farm ditches of such wide cross sections that roots cannot reach completely under them, and crop land out of production.

The design ET would be based on:

1. cultivated area
2. average ET of the crop mix of the cultivated areas
3. allowance for greater than the mean $\overline{ET}_{max}^{crops}$ (over crop seasons) by an optimal amount that takes into account the marginal cost of equipment and the marginal cost of lost crop yield due to undersized equipment.
4. Computation of the $\overline{\overline{ET}_{max}^{crops}}^{years}$ over the days of the irrigation interval. A downward adjustment for short term cumulative depletion of soil moisture is allowable as the soil moisture reservoir will alleviate the peaking problem.

Then, by equation 1-31 on page 1-41 of Hart (1975):

$$A = \left(\frac{C \cdot A \cdot Da}{F \cdot G}\right)$$

where C = a constant (1/360) = $\left(\frac{10{,}000\ m^2}{ha} \times \frac{m}{1000\ mm} \times \frac{hours}{3600\ seconds}\right)$

A = irrigated area (hectares)

Da = depth of applied irrigation water (mm/irrigation)

F = number of irrigation hours per day

If G = number of irrigation days per irrigation interval, then in the design (equations refer to Hart)

$Da = Du + De$ (equation 1-11)

$Du = SWD + Dt + Dd$ (equation 1-5)

$SWD = (FC - W)\ \gamma 6\ Y$ (equation 1-6)

$Dt = ET\ (Ti + Td)$ (equation 1-7)

$$SWD + Dt = \sum_{i=1}^{I} ET$$

I = irrigation interval $\leq I\ max$ (equation 1-13)

$Dd = LR \cdot Di$ (equation 1-8)
for design use $Dd = LR \cdot Du$

so

$$Du = \frac{\sum_{i=1}^{I} ET}{1 - LR} = \frac{I \cdot \overline{ET}\ max}{1 - LR}$$

where

De = depth of losses.

Because

G = 1 (pumping everyday)

and De = o by Section 1.3.1, $Q = \frac{C \cdot A\ ET}{F\ (1 - LR)}$

The design assumes that leaching occurs during times other than the peak ET time, i.e., when the pumping plant has excess capacity.

Thus,

$$Q = \frac{C \cdot A\ \overline{\overline{\overline{ET\ max}}}^{crops}}{F}^{years}\ \text{irrigation interval}$$

as used in Section 1.3.1.

Da	Depth of applied irrigation water.
Dd	Depth of water required for leaching.
De	Depth of water losses.
Di	Depth of applied water that enters the soil.
Dt	Depth of water consumptively used during the irrigation and drainage period.
Du	Total depth of irrigation water required.
Et	Evapotranspiration.
Fc	Field capacity.
I	Elapsed time between irrigations.
LR	Leaching requirement.
SWD	Soil water deficit.
Td	Time period for drainage.
Ti	Time period for irrigation.
W	Water content of the soil.
Y	Depth of root zone.
γ^6	Bulk specific gravity of the soil.

TABLE 2.1 AREAS SERVED AND DESIGN FLOW RATES IN PIPELINE 1

Pipe Outlet Number	Direction to Region Served	Area of Region (Feddans)	Q to each Region for 0.50 Fractional Pumping Time (Step 2)		Design Q for each Pipe Outlet		Remarks
			m^3/s	ft^3/s	m^3/s	ft^3/s	
1	N	3.0	0.0050	0.177	0.0566	2.0	
2	N	10.0	0.0167	0.589	0.0680	2.4	see #9
	S	40.0	0.0667	2.356			see #3
3	S	10.0	0.0167	0.589	0.0566	2.0	see #2S
4	N	2.0	0.0033	0.118	0.0765	2.7	see #8E
	S	45.0	0.0751	2.650			
5	N	30.0	0.0500	1.767	0.0566	2.0	elevated meska
6	N	10.0	0.0167	0.589	0.0680	2.4	
	S	40.0	0.0667	2.356			
	SE						potential
7	N,P	90.0	0.1501	5.301	0.1501	5.3	pipe
	S	31.0	0.0517	1.826			
BRANCH	PIPE - NEXT TO		EL -		SHIMI	BRANCH	CANAL
8	E	14.0	0.0234	0.81246	0.0566	2.0	see #4N
	W	11.9	0.0198	0.701			
9	W	19.2	0.0320	1.131	0.0566	2.0	see #2N
TOTAL		356.1 feddans in 14 regions = 149.59 ha			0.646 m^3/s	22.8 ft^3/s from 9 outlets	

Note: The above use of 5 significant figures does not represent corresponding accuracy. The number is used for design covenience.

TABLE 2.2 AREAS SERVED AND DESIGN FLOW RATES IN PIPELINE 2

Pipe Outlet Number	Direction to Region Served	Area of Region (Feddans)	Q to each Region for 0.50 Fractional Pumping Time (Step 2)		Design Q for each Pipe Outlet		Remarks
			m^3/s	ft^3/s	m^3/s	ft^3/s	
1	E	24.950	0.0416	1.470	0.0566	2.0	
	W	2.000	0.0033	0.118			see #2 NW
2	NE	27.472	0.0458	1.618	0.085	3.0	
	NW	9.851	0.0164	0.580			see #1 W
	SE	10.387	0.0173	0.612			
	SW	18.644	0.0311	1.098			
3	E	100.000	0.1668	5.890	0.1671	5.9	
	NW	16.950	0.0283	0.998			
	SW	8.720	0.0145	0.514			see #4 NW
4	E	23.000	0.0384	1.355	0.0566	2.0	see #5 NE
	NW	2.000	0.0033	0.118			see #3 SW
	W	10.521	0.0175	0.620			
	SW	4.000	0.0067	0.236			see #5 NW
5	NE	2.000	0.0033	0.118	0.0566	2.0	see #4 E
	SE	8.000	0.0133	0.471			see #6 NE
	NW	15.552	0.0259	0.916			see #4 SW
	SW	6.000	0.0100	0.353			see #6 W
6	NE	13.809	0.0230	0.813	0.0566	2.0	see #5 SE
	SE	14.000	0.0234	0.825			see #7 E
	W	15.600	0.0260	0.919			see #5 SW
7	E	8.000	0.0133	0.471	0.0566	2.0	see #6 SE
TOTAL		341.46 feddans in 21 regions = 143.44 ha			0.0535 m^3/s	18.9 ft^3/s from 7 outlets	

It was difficult to obtain reliable maximum ET data for the site area. However, data for Arizona, which is similar in soil, climate, and crops to the site area, is available. Because the design ET will most probably be greater than the ET max averaged over all crops, seasons, and irrigation intervals one week, the ET max calculated from the Arizona data will be applied to the gross area of the region as a crude compensation for inadequate information.

The design ET max used in the previous design was 8.0 mm/day. Previous project experience in the Delta region of Egypt indicated 9.0 mm/day might be appropriate. A listing of peak ET's for Arizona, which has a similar climate and evaporative demand is included in Table 4. The peak ET for all 10 crops was 8.45 mm/day. The peak ET for the 7 crops with largest ET's was 9.86 mm/day. A design ET of 10.0 mm/day was selected:

Pipe 1

$$Q = \left(\frac{149.59 \text{ ha} * \ 10.0 \text{ mm/day}}{360 * \ 14 \text{ hours/day}} \right)$$

$$Q = 0.2968 \text{ m}^3/\text{s} \quad (10.48 \text{ ft}^3/\text{s}) \quad (4720 \text{ gpm})$$

Pipe 2

$$Q \ (143.44 \text{ ha}) = 0.2846 \text{ m}^3/\text{s} \quad (10.05 \text{ ft}^3/\text{s}) \quad (4520 \text{ gpm})$$

(Note: The above use of 4 significant digits does not imply corresponding accuracy. The above numbers are used for design convenience.)

TABLE 4

PEAK CONSUMPTIVE USE OF WATER BY CROPS IN ARIZONA *

Crop	inch/day	mm/day
w Peas	.32	8.1
w Potatoes	.42	10.7
w Sweet Corn	.43	10.9
- Dry Onions	.38	9.7
w Green Onions	.2	
Sp Cantaloupe	.4	10.2
- Navel Oranges	.18	
- Grapefruit	.23	
Sp Wheat	.38	9.7
- Cotton	.38	9.7

$$\frac{\text{All Crops}}{\overline{ET_m}} = 8.45 \text{ mm/day}$$

$$\frac{\text{7 Crops}}{\overline{ET_m}} = 9.86 \text{ mm/day}$$

* Consumptive Use of Water by Crops in Arizona, Technical Bulletin 119, Ag. Experiment Station, University of Arizona, by Erie, French, Harris.

CHAPTER 3

DESIGN OF SURFACE DISTRIBUTION COMPONENT SYSTEMS

3.1 MINIMUM NEEDED ELEVATION OF THE WATER SURFACE AT THE TOP OF THE IMPROVED (ELEVATED AND LINED) MESKAS (COMMUNAL DITCHES, FARMER OWNED)

At the time of this design none of the meskas have been improved, and the results of land leveling are not available. The purpose of this chapter is to establish constraints for the design of the pipeline that will allow the designer of the improved meskas reasonable flexibility. The following equation was used:

The minimum needed elevation of the water surface at the top of the improved meska = elevation of high area served by meska + depth of flow in basin + head loss in turnout from farm ditch to basin + head loss in farm ditch (earthen) + head loss in turnout from improved meska to farm ditch + head loss in improved meska (L x T x S of the meska) + head loss in checks and corners (if any) + safety factor.

This equation becomes:

minimum elevation = high elevation + 0.20 m + 0.05 + 0.05 m + 0.10 +(L x T x S of the meska) + 0.0 +0.10

minimum elevation = high elevation + 0.50 m +(L x T x S) of the meska

where

L = the straight line distance (as measured on the map) from the distributor ditch to the high elevation spot,

T = a tortuosity factor (estimated) used to predict the actual length of the improved meska to the high elevation spot, and

S = the design slope of the water surface in the meska (usually equal to the slopes of the bottom and top of the ditch). It was recommended that an S = 0.00015 be used for the meskas.

Tables 5 and 6 give the minimum needed elevation of the water surface at the improved meskas in pipelines 1 and 2.

3.2 DISTRIBUTOR CONCRETE-LINED DITCH DESIGN WITH ALFALFA VALVES, FLUMES VERSUS WEIRS, AND GATES VERSUS INVERTED SIPHONS

3.2.1 Procedure

The following is the procedure used for ditches in which the alfalfa valve was at one end with a constant slope to the other end, so that flow was in one direction.

1. The design Q for the valve (Chapter 2), the TDH in the pipe below the valve, the head loss in the alfalfa valve for the Q design (Chapter 3), and the fluctuating head factor (which was used as a "safety" factor to ensure that outlets that were not critical for the design schedule of open outlets would not become unnecessarily critical for other schedules of open outlets) were determined.

2. The maximum elevation of the water surface over the alfalfa valve = TDH in pipe - h_L alfalfa valve - fluctuating head factor.

3. Tracing from the base map, the CLD route was sketched with the outlets, the proposed flumes or weirs, the proposed gates, the proposed pipe outlet location, and the proposed bottom end of the CLD.

4. The pipe outlet (alfalfa valve) was located 35 m upstream from the most upstream CLD outlet, the flume 25 m downstream from the alfalfa valve. If there was limited space or excess head,a weir and stilling basin were used.

5. The outlets were stationed at the end of the CLD = 5 m downstream from the most downstream outlet.

6. The minimum elevation of the water surface at the top of the meska CLD (see section 3.1) was determined.

TABLE 5 MINIMUM WATER SURFACE ELEVATION AT IMPROVED MESKA PIPELINE 1

Pipe Outlet Number	Direction to Region Served	Col.	High Elevation (m) ±0.01	L (m) ±25m	T (m) ±0.5	Minimum Needed Elevation of water surface at top of improved meska (m)	Remarks
1	N		18.04	100	2.0	18.57	
2	N		17.70	325	1.6	18.28	#9 serves high area
	S		17.87	450	1.6	18.48	
3	S		17.70	75	1.3	18.21	#2s serves high area
4	N		17.77	200	1.3	18.31	
	S		17.80	175	2.0	18.35	
5	N		17.73	275	1.6	18.30	elevated, lined
6	N		17.28	200	2.0	17.84	
	S		17.42	70	1.3	17.93	
	SE						potential
7	N,P		17.37	75	1.6	17.89	pipe
	S		17.33	150	1.6	17.87	
BRANCH	PIPE	-	NEXT	TO	EL	- SHIMI	CANAL
8	E		17.93	50	1.6	18.44	
	W		17.78	125	2.0	18.32	
9	W		17.73	75	1.6	18.25	

$\bar{x}_{14}$ = 17.68 18.22

s = 0.23 0.24

TABLE 6 MINIMUM WATER SURFACE ELEVATION AT IMPROVED MESKA PIPELINE 2

Pipe Outlet Number	Direction to Region Served	Color of Region	High Elevation (m)	L (m)	T (m)	Minimum Needed Elevation of $\overline{V}$ at top of improved meska (m)	Remarks
1	E	Yellow	17.49	100	1.3	18.01	
	W	Yellow	17.51	150	1.6	18.05	#2 NW serves high area
2	NE	Brown	17.75	125	2.0	18.29	
	NW	Yellow	17.80	125	2.0	18.34	longer L w/smaller high elev.
	SE	Green	17.72	325	2.0	18.32	
	SW	Green	17.81	175	2.0	18.36	
	SSW	Orange	17.65	225	1.6	18.20	potential
3	E	Yellow	17.73	175	2.2	18.29	
	NW	Orange	17.64	125	2.0	18.18	
	SW	Blue	17.52	100	2.0	18.05	
4	E	Green	17.61	175	1.6	18.13	
	NW	Blue	17.52	125	1.3	18.04	
	W	Yellow	17.44	175	2.0	17.99	
	SW	Brown	17.46	175	1.3	17.99	
5	NE	Green	17.65	75	1.6	18.17	
	SE	Blue	17.67	250	1.6	18.23	
	NW	Brown	17.60	25	1.3	18.10	
	SW	Orange	17.82	250	1.3	18.37	
6	NE	Blue	17.67	250	1.6	18.23	
	SW	Orange	17.68	100	1.3	18.20	
	W	Orange	17.93	50	1.3	18.44	
7	E	Orange	17.85	50	1.3	18.36	
		$\overline{x}$ = 17.66				18.20	
						s = 0.14	

7. For the proposed n, b and z from the CLD design, the normal depth of flow for the Q design (Chapter 3) was found.

8. Elevation of the meska CLD bottom = (6)-(7) was determined.

9. The head loss for the Q design through each standard structure (gate, port, inverted siphon [Chapter 3]) was found.

10. The excess head loss available = (2) - (6) - (9) (for gate or inverted siphon) - 0.10 m (for flume, or 0.30 m for weir) $-S_{CLD}\ L_{CLD}$ was estimated.

11. From (10), the Q maximum was estimated so that the excess h_L was consumed in the alfalfa valve, the flume (Chapter 3) and the CLD outlet structures and gates, and increased Y_n in meska CLD.

12. Using the equation of (2) above, the design elevation of the water surface over the alfalfa valve (Chapter 3), flume, (Chapter 3) and CLD outlet structures (see Section on gates and inverted siphons) was calculated.

13. Using the h_L in the alfalfa valve for Q max (See #11) the head loss for the Q maximum through each standard structure (flume, gate, inverted siphon) was found.

14. The normal depth of flow for the Q max (see [11]) was found.

15. The elevation of the water surface in the meska CLD = (8) + (14) was designed.

16. The elevation of the CLD bottom near the alfalfa valve = (12) - h_L (Chapter 3) - .01 m (if flume), or (12) - .30 m - .050 m (if weir) was designed.

17. The elevation of the water surface h_L (Chapter 3) over the alfalfa valve for the Q design = (16) + .01 (if flume), or (16) + 0.50 + head of water over sill for Q design (if weir) was designed.

18. The design elevation of the water surface at the entrace of each / CLD turnout = (12) - S_{CLD} * (5) station for each outlet - (13) for flume (or, 0.30 m for weir) was found.

19. The Y_{min} was found such that the Y_{min} = (the diameter of the inverted siphon pipe) + 1.5 ($\frac{V \text{ inverted siphon}^2}{2\ g}$). The elevation of CLD bottom at each turnout, and above and below the flow measuring device (flume or weir) = (18) - (14). If Y_n of (14) < Y_{min}, and an inverted siphon was used, then the elevation of the CLD bottom = (18) - Y_{min} for all outlets.

20. If $Y_n < Y_{min}$ in (19), and inverted siphons were used, then it was determined that the adjusted design elevation of the water surface at gate outlets = (18) for gate outlets + Y_n.

21. At each outlet the available head loss = (18) or (20) - (6) was determined.

22. If (21) $\geq$ (13), if (21) < (9) by a small amount at Q max, and if Q max > > Q design, then the resulting infringement on freeboard was accepted assuming that usually the flow rate from the alfalfa valve would be less than the Q max.

If (21) > > (9) by a large amount at Q max, then (11) to (21) was repeated with a smaller Q max that was still larger than Q design.

23. The freeboard was calculated such that the freeboard = [Y_n from (14)] or [Y_{min} from (19)], whichever is larger} + (23) + S_{CLD} L_{CLD} (and rounded up to the next even inch). If the needed CLD depth was greater than available, adding check gates (if S_{CLD} * L_{CLD} is large) or increasing the S_{CLD} was considered in order to reduce the normal depth of flow (Y_n) (if Y_n > > Y_{min}). In addition, hand plastering the earthen berm with concrete to provide the needed freeboard and ponding depth was also

considered (this alternative was not acceptable if Y_n was greater than the maximum available CLD size).

25. The elevation of the CLD top = (16) and (19) + (24) was designed.

This procedure allocates all extra available head to increased flow rate capacity, especially in the gates and inverted siphons. This extra head may also be used for extra head loss in remote control type valves, or for increased head in the meska CLD (by moving the meska CLD's bottom higher), for increased head loss in turnouts from meska CLD to farmer's ditch, or for increased slope of the meska CLD for uced ditch size.

3.2.2 Alfalfa Valves

According to section 2.3, the risers and alfalfa valves should be sized to carry a range of flow rates for pipe 1 from 0.0566 m^3/s (2.0 ft^3/s) to 0.1501 m^3/s (5.3 ft^3/s). For pipe 2 the range of flow rates should be from 0.0566 m^3/s (2.0 ft^3/s) to 0.1671 m^3/s (5.9 ft^3/s). It was decided that a limited number of alfalfa valve sizes would be utilized to simplify repair and replacement problems.

According to Hart (1975), page A-82, the flow range can be reasonably handled by risers and alfalfa valves of size 10", 12", and 14" with head losses ranging from 24 mm (0.08 ft) to 1271 mm (4.17 ft). Horte's figures were checked by comparison with Verne Scott (1950) and with House, Kruse, Dimick (1965). Initially, 10" valves were considered for all risers with a $Q \leq 0.0765$ m^3/s (2.7 ft^3/s). Later 8" valves were considered. Initially, 14" valves were considered for all risers with a $Q >$ 0 0765 m^3/s (2.7 ft^3/s) (all risers on the branch pipe from outlet 7 NP on pipe 1, and outlet 4 on pipe 2). Later, 16" valves were considered.

According to Meyer and Aljibury (1977), page 11, $Q = 0.7\ A \sqrt{2\ gh}$ where A = port area of valve, h = head loss (assumed to be piezometric head loss) and g = acceleration due to gravity. The constant, 0.7, can be assumed to apply only to a fully open valve. So, the piezometric head loss is:

$$hp = \frac{1}{2g} \left(\frac{Q}{0.7\ A}\right)^2$$

Total head lost in valve = piezometric head loss + loss of velocity head

$$= \frac{1}{2g} \left(\frac{1}{0.7\ A}\right)^2 Q^2 + \frac{1}{2g\ A^2} Q^2$$

$$\text{Total } h_L = \frac{1}{2g\ A^2} \left(\frac{1}{0.7^2} + 1\right) Q^2$$

$$= \frac{3.0408}{2 * g * A^2} Q^2$$

$$= \frac{4.9296}{2 * g * D^4} Q^2$$

$$\text{Total } h_L = \frac{0.2515}{D^4} Q^2 = Kv\ Q^2 \text{ for metric units, } Kv = \frac{0.2515}{D^4}$$

The head losses for alfalfa valves used are given in Table 3.1.

3.2.3 Flume Design

Trapezoidal flumes were recommended where flumes were specified in the distributor concrete-lines ditches (CLD). The specifications for a trapezoidal flume design and installation are ASAE S 3591 and discharge specifications are given in Appendix A, page 149 of Hart (1975), Irrigation System Design. Because the slipform available to EWUP makes CLD's with b = 1 foot, z = 1, and maximum depth of 30 inches, use of the standard calibrated trapezoidal measuring flume number 1, with designation 0.4 (1:1)-1.0 (1:1) , according to Table 1 is recommended. According to Table 3, the discharge equation is:

$$Q = 3.23\ h_1^{2.5} + 0.63\ h_1^{1.5} + 0.050$$

for 0.20 feet $\leq h_1 \leq$ 1.20 feet, and

for 0.05 $ft^3/s \leq Q \leq$ 5.96 ft^3/s

For easy TDH calculations (Chapter 4), data generated from the above discharge equation was fitted into an equation of the form

$$h_1 = a\ Q^b$$

over a limited range. The range of the design and maximum Q^s from sections 3.2.8 and 3.2.89 were found. The design Q^s ranged from 2.0 ft^3/s to 5.9 ft^3/s and averaged 2.6 ft^3/s. The maximum Q^s ranged from 2.0 ft^3/s to 5.9 ft^3/s and averaged 3.9 ft^3/s.

The needed data was generated by utilizing an HP-25 program of the discharge equation which is given in Table 3.2.

TABLE 3.1 HEAD LOSSES IN ALFALFA VALVES

Flow Rate		Head Lost in Fully Open Alfalfa Valve (m)		
		0.2540 m 10" Kv = 60.423	0.3048 m 12" Kv = 29.139	0.3556 m 14" Kv = 15.729
m^3/s	ft^3/s			
0.0566	2.0	0.19	0.09	0.05
0.0680	2.4	0.28	0.13	0.07
0.0765	2.7	0.35	0.17	0.09
0.0850	3.0	0.44	0.21	0.11
0.1133	4.0	0.78	0.37	0.20 extra
0.1501	5.3	1.36	0.66	0.35
0.1671	5.9	1.69	0.81	0.44
0.1982	7.0	2.37	1.14	0.62 extra
0.2860	10.1	4.94	2.38	1.29

Table 3.2 HP25 PROGRAM FOR THE COMPUTATION OF Q vs h_1 IN TRAPEZOIDAL FLUMES

```
3.23      STR 0
2.5       STR 1
0.63      STR 2
1.5       STR 3
0.050     STR 4
h1
STR       7
ENT
RCL       1
y^x
RCL       0
X
STR       5
RCL       7
ENT
RCL       3
y^x
RCL       2
X
ENT
RCL       5
 +
RCL       4
 +
read Q
```

The resulting data was:

Table 3.3 Q vs h_1 FOR TRAPEZOIDAL FLUMES

i	Q ft^3/s	h_1 ft.
1	1.74	0.7
2	2.35	0.8
3	2.55	0.83
4	2.70	0.85
5	2.77	0.86
6	2.92	0.88
7	3.07	0.9
8	3.31	0.93
9	3.56	0.96
10	3.72	0.98
11	3.91	1.0
12	4.19	1.03
13	4.47	1.06
14	4.88	1.1
15	5.97	1.2

Then pages 98 to 100 of the HP-25 Applications Program were used for the power curve fit of the above data. This resulted in:

$a = 0.550862$ sec b/feet $^{3b - 1}$

$b = 0.436887$

$r^2 = 0.999943$

The data was entered twice, and the program run each time. Each run resulted in the same a, b and r^2 valves.

The equation is changed to metric system by changing the "a" parameter:

$$0.550862 \left(\frac{sec^b}{feet^{3b-1}}\right) * \left(\frac{feet}{0.3048_m}\right)^{0.310660} = 0.550862 * 1.446425 =$$

$$0.796780 \frac{sec^b}{m^{3b-1}}$$

So the resulting equation is $h_1 = 0.796780 \frac{sec^{0.437}}{m^{0.311}} Q^{0.436887}$

The potential submergence problem can be evaluated using Table 2 of ASAE Standard -S 3591. The Table shows that at 70% and 75% submergence ($\frac{h_2}{h_1}$ * 100), the error in measurement is 0.7% and 1.6%, respectively, for flume 1. Box (1976), page 85, defined the modular limit as the submergence ratio at which the error in measurement is 1%. So, the modular limit is roughly at a 72% submergence ratio.

A figure was drawn depicting the normal depth of flow for the CLD, h_1 for the flume, 0.72 * h_1 for the flume and several curves of (0.72 * h_1 + C) with C's of 0.19 m, 0.29 m and 0.33 m. These curves showed that the modular limit needed to be checked only at the maximum flow rate. So the head loss in the flume and drop in CLD below the flume needed to avoid submergence could be found by an algebraic procedure.

Each pipe outlet was designed in sections 3.2.8 and 3.2.9 for a maximum flow rate. Only these maximum Q's were checked.

Table 3.4 HEAD LOSS AND STEP SIZE FOR FLUMES

Q m^3/s	h_1 m	$h_L = 0.28 * h_1$ m	$0.72 * h_1$ m	Step size = $Y_n - 0.72 * h_1$ m
0.0566	0.227	0.06	0.16	0.18
0.0680	0.246	0.07	0.18	0.18
0.0765	0.259	0.07	0.19	0.19
0.0850	0.271	0.08	0.20	0.20
0.1133	0.308	0.09	0.22	0.24
0.1501	0.348	0.10	0.25	0.27
0.1671	0.365	0.10	0.26	0.29

Notes:

"h_L" is head loss in the flume.

"Step size" is the drop in CLD that is needed.

"Y_n" is the normal depth of flow for the CLD downstream from the flume from section 3.2.4.

The water surface upstream of the flume was calculated to rise in an M-2 curve to approach $Y_n = 0.55$ m. If the flume is 25 m downstream from the alfalfa valve, the depth of flow by gradually varied flow at the valve was found following the method of Hart (1975) page 73:

1. Select Y_1, such that $A_1 \leq 1.2\ A_2$

$A = by + Zy^2 = 0.3048\ y + y^2$

$Y_2 = 0.365$ m

$A_2 = 0.3048 * 0.365\ m + (0.365\ m)^2$

$A_2 = 0.2445\ m^2$

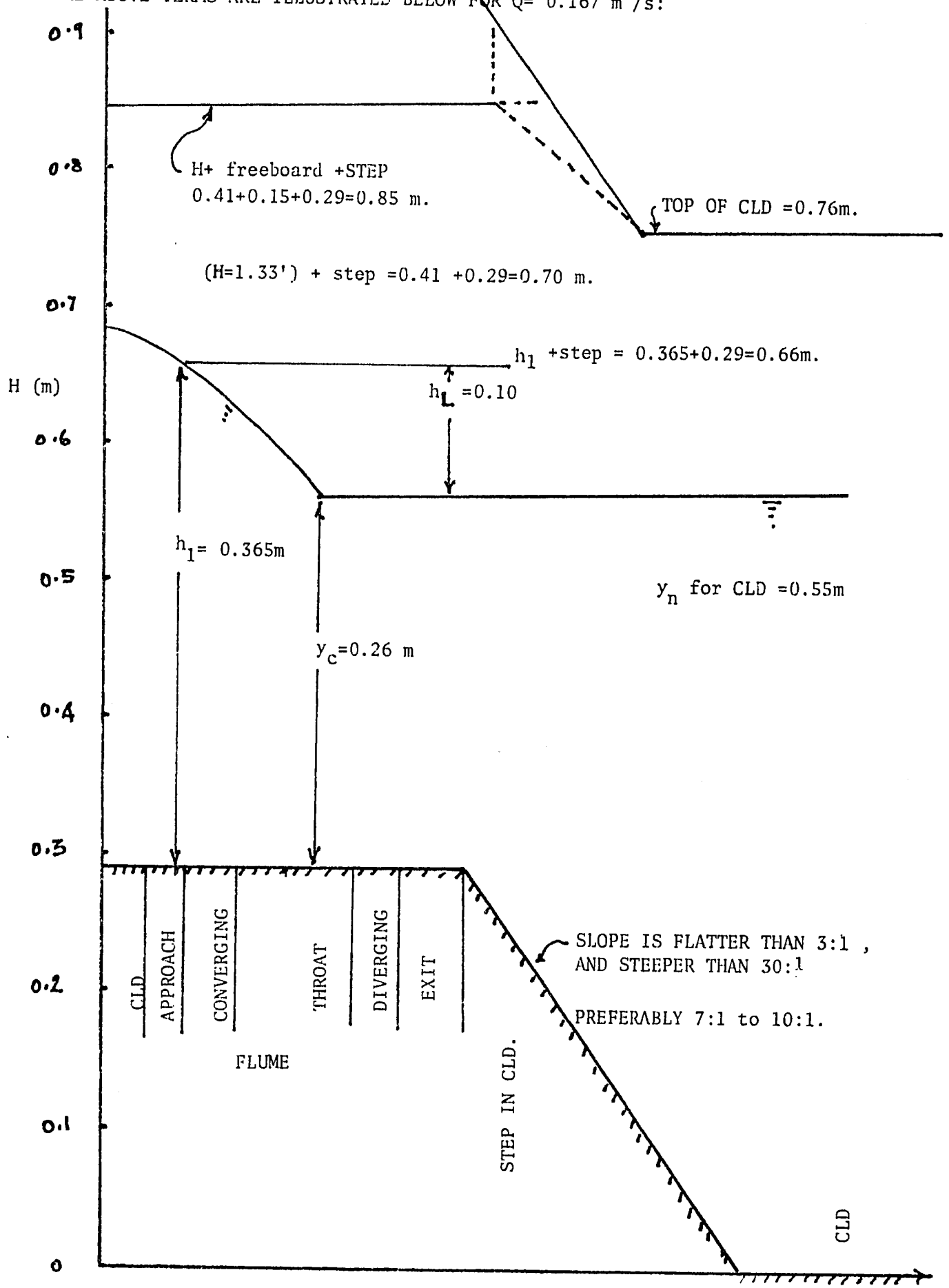
$A_1 \leq 0.2934\ m^2$

Try: $Y_1 = 0.5$ m

$A_1 = 0.3048 \times 0.5 + (0.5)^2$

$= 0.4024\ m^2$ - too much

THE ABOVE TERMS ARE ILLUSTRATED BELOW FOR Q= 0.167 m^3/s:



NOTE: DATUM FOR H IS CLD INVERT DOWNSTREAM FROM STEP.

Figure 3.1 FLUME HEAD LOSS AND STEP SIZE

If: $y_1 = 0.40$ m

$A_1 = 0.3048 \times 0.4 + (0.4)^2$

$A_1 = 0.2819\ m^2$

2. V and S can be calculated:

$$V_1 = \frac{Q}{A_1} = \frac{0.167\ m^3/s}{0.2819\ m^2} = 0.5924\ m/s$$

$$V_2 = \frac{Q}{A_2} = \frac{0.167\ m^3/s}{0.2445\ m^3}$$

$V_2 = 0.06830$ m/s

$P = b + 2\sqrt{y^2 + (Zy)^2} = 0.3048\ m + 2\ 2y^2$

$P = 0.3048 + 8\ y^2$

$P_1 = 1.4362$ m

$P_2 = 1.3372$ m

$\overline{P} = 1.3867$ m

$\overline{A} = 0.2632\ m^2$

$$S = \left[\frac{0.167\ m\ 3/5 * 0.014 * (1.3867\ m)^{2/3}}{1.486 \qquad (0.2632\ m^2)^{5/3}}\right] = 0.000,\ 327,\ 5898,\ 45$$

$$3.\quad x = \left[\frac{0.6830^2 - 0.5924^2}{2 \times 9.80665\ m\ 3/5} + 0.365 - 0.40\right] * \left[\frac{1}{0.00015 - 0.00032789845}\right]$$

$= 163.90$ m

So, $x = 164 >> 25$ m, so try again with a small Y_1.

1. Try $Y_1 = 0.37$ m, $A_1 = 0.3048 \times 0.37 + (0.37)^2 = 0.2497\ m^2$

$\overline{A} = 0.2471\ m^2$

2. $Y_1 = (\frac{0.167}{0.2497}) = 0.6688$ m/s

$P_1 = 1.3513$ m

$\overline{P} = 1.3443$ m

$$S = \left| \frac{0.167 \times 0.014 \times (1.3443)^{2/3}}{1.486 \qquad (0.2471)^{5/3}} \right|^2 = 0.000,\ 387,\ 899.78$$

$$3.\quad x = \left(\frac{(0.6830)^2 - (0.6688)^2}{2 \times 9.80665\ m/s^2} + 0.365 - 0.37\right) * \left(\frac{1}{0.00015 - 0.00038789978}\right)$$

$= 16.90\ m = 17\ m$

Y	X
0.400 m	154 m
	25 m
0.370 m	17 m
0.365 m	0 m

Probably, $y \approx 0.372$ m at X = 25 m.

So, there is an extra (0.372 - 0.365) + 0.00015 * 25 m = 0.007 m + 0.004 m = 0.011 m of head over the alfalfa valve relative to the h_1 of the flume for Q = 0.167 m 3/5. This extra head of 0.011 m plus head loss in flume of 0.100 m plus the head loss in the CLD from the flume to the first outlet means the total loss from the alfalfa valve to the first CLD outlet below the flume is 0.11 m, which in fact is the valve used for all flumes.

This head loss of 0.11 is a worst case analysis, as indicated by the following analysis for Q = 0.0566 m^3/s. As shown in the above table, the minimum needed head loss through the flume is 0.06 m. The step size is 0.18 m.

As was done above, the depth of flow over the alfalfa valve was estimated by using the gradually varied flow procedure.

$Y_2 = 0.227$ m

$A_2 = 0.1207\ m^2$

$V_2 = \left(\frac{0.0566\ m^3/s}{0.1207\ m^2}\right) = 0.4689$ m/s

$P_2 = 0.9469$ m

1. Y_1 was selected such that $A_1 \leq 1.2\ A_2 = 0.1448$ m

If: $Y_1 = 0.235$ m

$A_2 = 0.1269$ m

$\overline{A} = 0.1238$

2. $V_1 = (\frac{0.0566}{0.1269}) = 0.4460$ m/s

$P_1 = 0.9695$ m

$\overline{P} = 0.9582$ m

$$S = [\frac{0.0566\ m\ 3/5 \times 0.014 \times (0.9582)^{2/3}}{1.486 \qquad (0.1238)^{5/3}}]^2 = 0.000,\ 284,\ 04346$$

3. $$x = [\frac{(0.4689)^2 - (0.4460)^2}{2 \times 9.80665} + 0.227 - 0.235] * (\frac{1}{0.00015 - 0.00028404346}) = 51.7130\ m$$

Y	X
0.235	51.71
	25
0.227	0

Probably, Y (25 m) = 0.231 m.

So, there is an extra (0.231 m - 0.227 m) + 0.00015 * 25 m = 0.004 + 0.0004 = 0.008 m of head over the alfalfa valve relative to the h_1 of the flume for Q = 0.0566 m^3/s. This extra head of 0.008 m, plus head loss in the flume of at least 0.06 m, plus the head loss in the CLD from the flume to the first outlet (10 m * 0.00015) of 0.002 m means the total head loss from the alfalfa valve to the first CLD outlet below the flume is 0.07 m. This indicates that over the range of flow rates 0.11 m is the largest of the minimum required head losses that permit the flume to operate within the modular limit.

3.2.4 Faiyum Weir

In the early stages of this design the only location where a weir was thought necessary was on pipe 1 at outlet 7 NP which discharges into the pipe that replaces what is called meska 2. A weir was chosen over an orifice or venturi meter because a weir totally separates the downstream flow from the upstream flow, as long as the weir is not submerged. Thus, rotating the flow between alfalfa valves along pipe 7 NP will not affect the TDH upstream from the weir, and therefore, will not cause a varying flow rate in the alfalfa valves upstream of the weir. However, upstream conditions will determine the TDH at the weir, and therefore, the flow rate over the weir. Rotating flows between upstream valves will effect the flow rate over the weir into pipe 7 NP and out of those alfalfa valves. It was proposed that this be corrected by a moveable weir, as described by Bos (1976), page 125.

A Faiyum weir was selected (Bos, 1976 on page 149) over a roundnosed horizontal broad crested weir (Bos, 1976 on page 122), because apparently its construction and use was familiar to Egyptian engineers. The design procedure for the Faiyum weir was:

1. A minimum Q was selected

2. A minimum readable $h_1 \geq 0.06$ m (Bos, 1976 on page 152) such as 0.10 m was selected.

3. b was found by $Q = Cd\ Cv\ 2/3\ (2/3g)^{1/2}\ b\ h_1^{\ 1.50}$ assuming Cd = 0.89 because $\frac{h_1}{L} = 0.667$ (see Bos, page 147). That $b \geq 0.05$ m was checked (see Bos, 1976, page 153).

4. $L \leq 1.5 * h_1$ minimum was found $L \geq \left[\frac{h_1}{1.6}\right] = 0.625h_1$ (see Bos, 1976, page 153) was checked.

5. Maximum Q was selected.

6. h_1 maximum by (3) assuming Cd - 1.00 was found.

7. $\left[\frac{h_1}{L}\right]$ maximum was found.

8. Cd maximum (Box, 1976, page 147) was found.

9. The corrected h_1 maximum was found and it was decided whether it was necessary to repeat 7, 8, and 9.

10. That $L \geq 0.625\ h_1$ maximum was checked.

11. $\left[\frac{b}{L}\right]$ was found.

12. $\left[\frac{R}{L}\right]$ was found.

13. R = (12) * L was found.

14. $\left[\frac{h_{1\ maximum}}{L}\right]$ and $\left[\frac{h_{1\ minimum}}{L}\right]$ were found, and(Bos, 1976, page 151)the permissible submergence for 1% error was found.

15. The elevation of the downstream channel invert was designed so that the permissible submergence is never exceeded over the range of flows.

The specifications for a Faiyum Weir are:

1. $b\ h_1/A_1 \leq 0.35$, where b = breadth of weir crest, h_1 = head of water surface upstream from the weir relative to the weir crest, and A_1 = the flow area at the measurement station;

2. h_1 should be measured two or three times to determine the h_1 maximum upstream from the weir face;

3. the air pocket beneath the nappe should be fully aerated; and

4. the horizontal upstream edge of the weir crest should be constructed according to Bos, 1976, page 149, Figure 4.16

Other specifications that Bos listed are included in the above design procedure.

Following is a list of the steps for the design of the weir at the head of pipe 7 NP:

1. Minimum Q = 0.0566 m^3/s (2.0 ft^3/s)

2. Minimum readable h_1 = .10 m

3. $b = \left(\frac{Q}{Cd\ C_v\ 2/3\ (2/3\ g)^{1/2}\ h_1^{1.5}} \right) = \frac{Q}{1.5166\ h_1^{1.5}}$

   b = 1.180 m $\geq$ 0.05 m ✓

4. L = 1.5 * h_1 minimum

   = 1.5 * 0.10 m

   = 0.15 m $\geq$ 0.625*0.10m = 0.06 m ✓

5. Maximum Q = 0.1501 m^3/s (5.3 ft^3/s)

6. h_1 maximum = $\left(\frac{Q}{1.7040\ b} \right)^{2/3}$

   = 0.177 m

7. $\left(\frac{h_1}{L} \right)$maximum = $\left(\frac{0.177\ m}{0.15\ m} \right)$ = 1.182

8. Cd maximum = 0.99 OK

9. ✓ OK

10. L = 0.15 m $\geq$ 0.625 h_1 max

    $\geq$ 0.625 * 0.177

    $\geq$ 0.111 m

11. $\left(\frac{b}{L}\right) = \frac{1.180\ m}{0.15\ m} = 7.87$

12. from p 150, guess that $\left(\frac{R}{L} \right) = 0.5$

13. R = 0.5 * 0.15 = 0.075 m

14. $\left(\frac{h_1\ maximum}{L} \right)$= 1.182 from (7). This procedure results in determining a maximum permissable submergence of roughly 40% and maximum H_2/L=0.47 (Bos, 1976, page 151). So, maximum permissable h_2 (downstream head relative to weir crest) = $\left(\frac{H_2}{L} \right)$ · L = 0.47 * 0.15m = 0.071m at maximum Q, $\left(\frac{h_1\ minimum}{L} \right)$ = 0.6667 by the design procedure in step 4. This computation results in determining a maximum

permissible submergence of roughly 37%, and maximum $(H_2/L) = 0.25$ (Bos, 1976, page 151). So, maximum permissible $h_2 \approx (H_2/L) * L = 0.25 * 0.15$ m = 0.037 m at the minimum Q.

Because the hydraulics of pipelines are strongly affected by downstream flow conditions which are presently now well defined, the weir has been designed with a safety factor which allows no submergence.

$$\text{So, } h_1 = h_2 = \left[\frac{Q}{1.7040 \times 1.180 \ (m^{3/2}/S)} \right]^{2/3} = (Q/2.0107 \ m^{3/2}/S)^{2/3}$$

Table 3.5 FLOW VS HEAD VALUES FOR FAIYUM WEIR

Flow m^3/s	Rate ft^3/s	$h_1=h_2$ m
0		
0.0566	2.0	0.09
0.0680	2.4	0.10
0.0765	2.7	0.11
0.0850	3.0	0.12
0.1133	4.0	0.15
0.1501	5.3	0.18
0.1671	5.9	0.19
0.1982	7.0	0.21
0.2860	10.1	0.27

3.2.5 Concrete-Lined Ditch (CLD) Design

For this problem there is little flexibility available for CLD design. A ditch slope of 0.00015 was recommended. The slip form for making CLD that EWUP recently bought and sent to Egypt lines a ditch with a b = 1 foot,

z = 1 and has a maximum depth of 30 inches (2.50 feet, 0.76 m). If freeboard is (Y_n/3), then the maximum Y_n = 1.88 feet = 0.57 m. It can be assumed that the slip form is manufactured to produce ditches of even inch increments.

Table 3.6 lists normal depths of flow found from Hart, 1976, page A-44.

S_{CLD} = 0.00015

Z = 1

b = 1 foot

CLD $\leq$ 30" = 2.5' = 0.76 m

Table 3.6 CONCRETE-LINED DITCH DESIGN

Flow Rate		Normal Depth of Flow		Freeboard	Minimum CLD	Preliminary CLD design	
ft^3/s	$m^3/5$	ft	m	ft	inches	inches	m
2.0	0.0566	1.10	0.34	0.50	19.20	24"	0.61
2.4	0.0680	1.19	0.36	0.50	20.28	24"	0.61
2.7	0.0765	1.26	0.38	0.50	21.12	24"	0.61
3.0	0.0850	1.32	0.40	0.50	21.84	24"	0.61
4.0	0.1133	1.50	0.46	0.50	24.00	26"	0.66
5.3	0.1501	1.70	0.52	0.57	27.20	28"	0.71
5.9	0.1670	1.81	0.55	0.60	28.96	30"	0.76
7.0	0.1982	1.96	0.60	0.65	31.32	32" [a]	
10.1	0.2860	2.45	0.75	0.82	39.20	40" [b]	
10.4	0.2945	2.50 ±0.06	0.76	0.83	40.00	40" [b]	

a - use 30" CLD + 2" hand plastered concrete on earth berm

b - use 30" CLD + 10" hand plastered concrete on earth berm

3.2.6 Inverted Siphon Design

A road will be routed parallel to the pipeline. The pipelines with distributor concrete-lined ditches will occupy one side of the government's right-of-way. The road will use much of the remaining right-of-way. The meskas and fields next to the pipeline and ditches can be easily served by gates in the side of the ditches. But, the meskas and fields on the other side of the road cannot be so easily served. In such cases, inverted siphons are usually used because they are much cheaper than bridges for small flows.

According to Hisenbrey, Hayes, Warren, Winsett and Young (Bureau of Reclamation), 1974, "Design of Small Canal Structures," inverted siphons should be designed for downstream control, so that:

1. the pipe size is not too large for available head loss, and
2. there will be a seal (ponded water) over the entrance to avoid air entrainment.

However, downstream control depends on future needs in the design of the concrete-lined ditches to replace the meskas. This plan provides flexibility for the future of the meska CLD by distributing water at a reasonably high elevation. The plan also considers that the design Q provided by the canal pumping plant may not be all of the Q through the alfalfa valve, CLD, and inverted siphon, because conjunctive use of surface and ground water is possible by pumping the tile drainage water from sumps into the pipeline (at the downstream ends of the pipes). This pumped flow rate may be used to increase the flow rate to each meska, or to increase the time of flow into each meska (which would increase the number of outlets that could be opened simultaneously), or both.

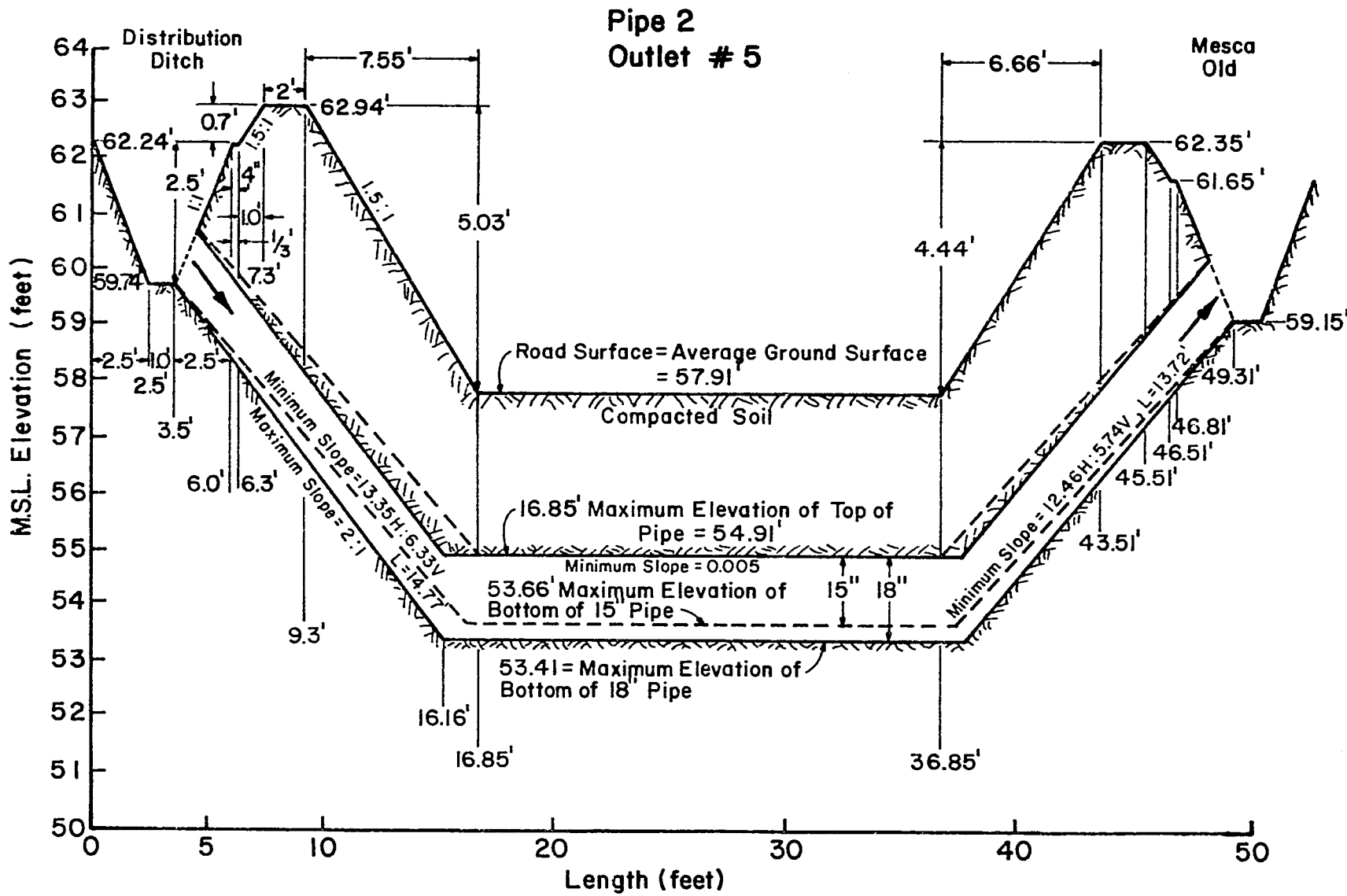


Figure 3.2 A TYPICAL INVERTED SIPHON DESIGN

For the following design only two standard design (pipe sizes) for inverted siphons are utilized. Just as in the alfalfa valve design, this will allow extra capacity for most outlets, and be close to the design for a few outlets.

According to "Design of Small Canal Structures," page 25, the pipe diameter of the inverted siphon is dependent on the design flow rate and maximum permissible velocity. According to page 28, the maximum velocity is a function of type (earth or concrete) of transitions, length of the siphon, available head and economics. According to page 25, considering the concrete transition, the short length of the inverted siphon, and the small head loss, the maximum velocity should be 5.0 feet per second. Without fully considering head loss,

$D = 12"$ for $2.0 \leq Q \leq 3.9$ ft ^{3}/s, and

$D = 15"$ for $3.9 < Q \leq 5.9$ ft ^{3}/s.

For more complete head loss consideration, a proposed inverted siphon design is needed. It is shown in the preceding figure. For the preceding figure, the important considerations are:

1. 0.2 m (0.7 foot) earth berm above lip of CLD ("Design of Small Canal Structures," page 15).

2. 0.1 m (4') lip on top of CLD (see Hart, 1975, page A-45),

3. 0.6 m (2.0 foot) wide top width berm (SCS standards),

4. all earth side slopes $1.5^{\#}$/H: $1^{\#}$/V or flatter (for sandy soils).

5. at least 1.0 m (3.00 ft) cover over pipe (see section 1.2),

6. 6 m (20 foot) wide road (estimated),

7. at most 2 H: 1 V slope in the inverted siphon ("Design of Small Canal Structures, page 31),

8. at least S = 0.005 in the section of inverted siphon that is below the road for easier pumping.

9. The total head loss through the inverted siphon is

$$\Sigma h_L = h_L \text{ entrance} + h_L \text{ exit} + 2h_L \text{ bends} + h_L \text{ pipe}$$

According to page 360 of "Design of Small Canal Structures," for single angle miter bends, the head loss coefficient for a 25.4° [$\tan^{-1}$ (6.33/13/35) upstream] bend is 0.090, and for a 24.7° [$\tan^{-1}$ (5.74/12.46), downstream] bend the head loss coefficient is 0.088.

The Hazen-William's equation was used for the head loss in a pipe. In metric units,

$$S = \left[\frac{3.5909\ m^{\frac{sec}{0.37}}\ Q}{C_1\ d^{2.63}}\right]^{1.3519}$$

The inverted siphon can be assumed to be made from pre-cast concrete pipe (mitered joints) so C_1 = 100. The syphon is roughly (14.77 + 20.00 + 13.72 = 48.49 feet (± 10 feet). Then,

$$\Sigma h_L = (0.50 + 1.00 + 0.090 + 0.088)\ \frac{V^2}{2g} +$$

$$48.44 \text{ feet} * 013048\ \frac{m}{ft} * 0.00211098\left[\frac{Q}{D^{2.63}}\right]^{1.8519} =$$

$$1.678 * 0.082711\ \frac{Q^2}{D^4} + 14.78\ m * 0.00211078\left[\frac{Q}{D^{2.63}}\right]^{1.8519}$$

$$\Sigma h_L = 0.13879\ \frac{Q^2}{D^4} + 0.031193\left[\frac{Q}{D^{2.63}}\right]^{1.8519}$$

Table 3.7a HEAD LOSSES IN INVERTED SIPHONS

		HEAD LOSS (M)			
FLOW RATE		DIAMETER OF INVERTED SIPHON			
$ft^{3/5}$	$m^{3/5}$	12" 0.2540 m	15" 0.3810 m	18" 0.4572 m	20" 0.5080 m
2.0	0.0566	0.10	0.04	0.02	0.01
2.4	0.0680	0.14	0.05	0.02	0.02
2.7	0.0765	0.18	0.07	0.03	0.02
3.0	0.0850	0.22	0.08	0.04	0.02
4.0	0.1133	0.39	0.15	0.07	0.04
5.3	0.1501	0.67	0.25	0.11	0.07
5.9	0.1671	0.82	0.31	0.14	0.09
7.0	0.1982	1.14	0.43	0.20	0.12
7.4	0.2095	1.27	0.48	0.22	0.14
10.1	0.2860	--	0.88	0.40	0.25
10.4	0.2945	--	0.93	0.42	0.27

Table 3.7b HP-25 PROGRAM FOR INVERTED SIPHONS

$$\Sigma h_L = a \left(\frac{Q}{D^{2.63}}\right)^{1.8519} + b \left(\frac{Q}{D^2}\right)^2$$

STR 0 = a = 0.031193
STR 1 = 2.63
STR 2 = 1.8519
STR 3 = 6 = 0.13879
STR 7 = 0.0254 (m/inch)
STR 4 = Q (m^3/s)

D (inches)

↓

ENT
RCL 7

X

optional start

w/ D (m) ↓

STR 5

ENT

RCL 1

f - y^x

RCL 4

x ⇄ y

·

·

ENT

RCL 2

f - y^x

ENT

RCL 0

X

STR 6

RCL 5

g - x^2

RCL 4

x ⇄ y

·

·

g - x^2

ENT

RCL 3

X

ENT

RCL 6

+

Σh_L (m)

The velocity should be 5.0 feet per second.

After considering the previous table of head losses, and the minimum inverted siphon pipe diameters, pipe sizes one increment larger than the minimum were chosen. So, for $Q \leq 0.0850\ m^3/s$ ($3\ ft^3/s$) 0.3810 m (15") pipe was used, and for $0.0850\ m^3/s$ ($3\ ft^3/s$) $< Q \leq 0.1671\ m^3/2$ ($5.9\ ft^3/2$) 0.4572 m (18") pipe was used. The Table 3.7c lists the $\frac{v^2}{2g}$ and Y min (see #18 of section 3.2.1) for the selected pipe sizes.

3.2.7 Gates

For a very conservative design, standard Soil Conservation Service analysis was used to estimate head loss through a large field turnout. It was assumed that all of the head loss in their analysis occurs in the gate. In their analysis they assume that the gate acts like a weir, so they apply the weir equation and design for 2/3 submergence, 1/3 head loss. For English units:

$$b = \left[\frac{Q}{3.1\ h_1^{3/2}}\right]$$

They design for $h_1 = 3\ h_L$

So,

$$b = \left[\frac{Q}{3.1\ (3\ h_L)^{3/2}}\right]$$

After finding b for several large Q's with $h_2 = 0.15$ m (0.49 ft), a 20" wide gate was selected. It is one of the smallest gates available. Then,

$$h_1 = \frac{h_1}{3} = 1/3 \left[\frac{Q}{3.1\ \frac{ft^{1/2}}{s}\ \frac{20\ in}{12\ in/ft}}\right]^{2/3}$$

$$h_1 = 0.11153\ Q^{2/3} \text{ (English)}$$

$$h_1 = 0.3676\ Q^{2/3} \text{ (S.I.)}$$

Table 3.8 (head loss in gates) lists head loss in the gate for various flow rates.

Table 3.7c INVERTED SIPHON COMPUTATIONS

Flow Rate		Inverted Siphon Diameter			
		15" 0.3810 m		18" 0.4572 m	
ft^3/s	m^3/s	$\frac{V^2}{2g}$ m	Y_{min} m	$\frac{V^2}{2g}$ m	Y_{min} m
2.0	0.0566	0.013	0.40	0.006	0.47
2.4	0.0680	0.018	0.41	0.009	0.47
2.7	0.0765	0.023	0.42	0.011	0.47
3.0	0.0850	0.028	0.42	0.014	0.48
4.0	0.1133	0.050	0.46	0.024	0.49
5.3	0.1501	0.088	0.51	0.043	0.52
5.9	0.1671	0.110	0.55	0.053	0.54
7.0	0.1982	0.154	0.61	0.074	0.57
7.4	0.2095	0.172	0.64	0.083	0.58
10.1	0.2860	0.321	0.86	0.155	0.69
10.4	0.2945	0.340	0.89	0.164	0.70

$$Y_{min} = D + 1.5 \frac{V^2}{2g}$$

$$Y_{min} = D + 1.5^* \left[0.082711 \left(\frac{Q}{D^2}\right)^2\right]$$

Table 3.8 HEAD LOSS IN GATES

Flow	Rate	Head Lost in 20" Gate
ft^3/s	m^3/s	(m)
2.0	0.0566	0.05
2.4	0.0680	0.06
2.7	0.0765	0.07
3.0	0.0850	0.07
4.0	0.1133	0.09
5.3	0.1501	0.10
5.9	0.1671	0.11
7.0	0.1982	0.12
7.4	0.2095	0.13
10.1	0.2860	0.16
10.4	0.2945	0.16

3.2.8 Distributer Concrete Lined Ditches for Pipe 1-Alfalfa Valves

(Section 3.2.1 "Procedures," delineates the calculation procedure).

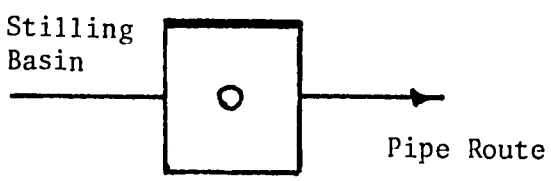
Outlet #1

1. Q design = 0.0566 m^3/s (2.0 ft^3/s) from section 2.3

   TDH in pipe below the valve = 21.00 from an earlier estimate of TDH = f(L) for the design rotation, which will be discussed in the next chapter. Head loss in the alfalfa valve for Q design = 0.19 m for a 10" valve, page 14. Fluctuating head factor = 0.40 m from analysis of drop in TDH when flow changes from 7N P to 7S. (See pipe diagram)

2. Maximum elevation of water surface over pipe outlet = 21.00 m - 0.19 m - 0.40 m = 20.41 m.

3. Tracing from base map of CLD route



10" valve

4. A weir should be used because of excess head.

5. Standard weir design should be used.

6. Minimum elevation of water surface at top of meska CLD = 18.57

7. Normal depth of flow for Q design = 0.34 m

8. Meska CLD bottom elevation = 18.57 - 0.34 = 18.23 m.

9. No special structures are needed.

10. Excess head loss available - 20.41 - 18.57 - 0.0 - 0.30 - 0.0 = 1.54 m.

11. If Q = 0.1133 m^3/s (4.0 ft^3/s), then extra head loss = (0.78 - 0.19) + (0.46 - 0.34) = 0.59 + 0.12 = 0.71 m. If Q = 0.1501 m^3/s (5.3 ft^3/s), then extra head loss = (1.36 - 0.19) + (0.52 - 0.34) = 1.50 m + 0.21 m = 1.71 m (too much). This procedure assumes that the elevation of the bottom of the improved meska at its upstream end will be the same as found in step (8) above. So, this outlet can handle as much as 0.1501 m^3/s (5.3 ft^3/s) with available head if the CLD is large enough (28" CLD).

12. Design elevation of water surface over alfalfa valve = 21.00 m - 1.36 m - 0.40 m = 19.24 m.

13. Not applicable.

14. Normal depth of flow for Q max = 0.52 m.

15. Design elevation of water surface in meska CLD = 18.23 + 0.52 = 18.75 m.

16. Design elevation of CLD bottom near alfalfa valve = 19.24 - 0.52 = 18.72 m. This is the elevation if there is a flume in the CLD. However, project engineers recommended placing the flow of the Faiyum weir 0.50 m below the sill elevation. Sill elevation = elevation of water surface over alfalfa valve (12) - 0.30 m head of water over sill. Sill elevation = 19.24 m - 0.30 m = 18.94. Stilling basin floor elevation = 18.94 m = 0.50 m = 18.44 m.

17. Design elevation of water surface over alfalfa valve for Q design = 18.56 + 0.50 + 0.09 = 19.15 m.

18. Not appropriate.

19. Not appropriate.

20. Not appropriate.

21. Not appropriate.

22. In this case the weir must not be submerged. From (15) the sill elevation is 19.06 m. From (14) the design elevation of the water surface in the meska CLD = 18.75 m. So, there is considerable extra head still available for use.

23. Freeboard = 0.17 m.

24. CLD depth = 0.52 + 0.17 + 0.0 = 0.69 m = 27.3 inches, so 28" CLD = 0.71 m.

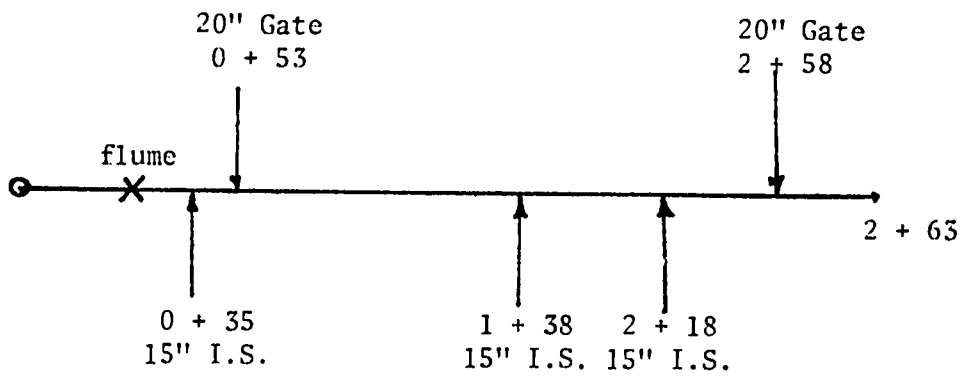
25. Design elevation of CLD top = 18.72 + 0171 m = 19.43 m above weir and 18.94 m below weir, CLD top = 18.23 (see #8) + 0.71 m.
= 18.94 m below weir.

Outlet #2

1. Q design = 0.0680 m^3/s (2.4 ft^3/s) TDH in pipe = 20.63 m, head loss in alfalfa valve for Q design (10") = 0.28 m, fluctuating head factor = 0.40 m

2. Maximum elevation of water surface over alfalfa valve = 20.63 - 0.28 - 0.40 = 19.95 m

3.



(I.S. = Inverted Siphon)

4. Flume

5. 2 X 20" gates, 3 X 15" I.S.

6. North - 18.28 m

   South - 18.48 m

7. Y_n = 0.36 m

8. Elevation of meska CLD bottom = 17.92 m (north), 18.18 m (south)

9. h_L gate = 0.06 m

   h_L 15" I.S. = 0.05 m

   h_L flume = 0.07 m

10. Excess head loss = 19.95 - 18.28 - 0.06 - 0.07 - 263 m *0.00015 = 1.5 m (north)

    17.95 - 18.48 - 0.05 - .07 - 263 *0.00015 = 1.31 m (south)

11. If Q max = 0.1133 m^3/s (4.0 ft^3/s), then extra head used = (0.78 - 0.28) + 0.15 - 0.05) + (0.09 - 0.07) + (0.46) - 0.36) = 0.72 m

    If Q max = 0.1501 m^3/s (5.3 ft^3/s) then extra head used = (1.36 - 0.28) + (0.25 - -.05) + (0.10 - 0.07) + (0.52 - 0.36) = 1.47 m (too much).

    So, Q max = 0.1133 m^3/s (4.0 ft^3/s)

12. Design elevation of water surface over alfalfa valve = 20.63 - 0.78 - 0.40 = 19.45 m

13. h_L 20" gate = 0.09 m

    h_L 15" inverted siphons = 0.15 m

    h_L for flume = 0.09 m

14. Y_n for Q max = 0.46 m

15. Design elevation of water surface in meska CLD = 17.92 m + 0.46 m = 18.38 m (north)
    = 18.18 m + 0.46 m = 18.64 m (south)

16. Design elevation of CLD bottom near alfalfa valve = 19.45 - 0.31 - 0.01 = 19.13 m

17. Design elevation of water surface over alfalfa valve for Q design = 19.13 m + 0.25 + 0.01 = 19.39 m

18. Design elevation of water surface at 0 + 35 = 19.45 - -.00015 * 35 m - 0.09 = 19.35

    at 0 + 53 = 19.35

    at 1 + 38 = 19.34

    at 2 + 18 = 19.33

    at 2 + 58 = 19.32

19. Elevation of CLD bottom above flume = 19.35 - 0.46 = 18.89,

below flume = 18.89 m

at 0 + 35 = 18.89

at 0 + 53 = 18.89

at 1 + 38 = 18.88

at 2 + 18 = 18.87

at 2 + 58 = 18.86

Y min = 0.46 m at Q max

Y n = Y min so o.k.

20. Not applicable.

21. Available head loss at 0.35 = 19.35 - 18.58 = 0.77 m

at 0 + 53 = 19.35 - 18.38 = .97 m

at 1 + 38 = 19.34 - 18.58 = 0.76 m

at 2 + 18 = 19.33 - 18.58 = .75 m

at 2 + 58 = 19.32 - 18.38 = .94 m

22. All of (21) $\geq$ all of (13)

23. Freeboard = 0.46/3 = 0.15 m

24. CLD depth = 0.46 m + 0.15 + 263 + 0.00015

= 0.63 m = 25.6"

use: 26" = 0.66 m

25. Design elevation of CLD top at alfalfa valve = 19.79 m

at 0 + 35 = 19.55

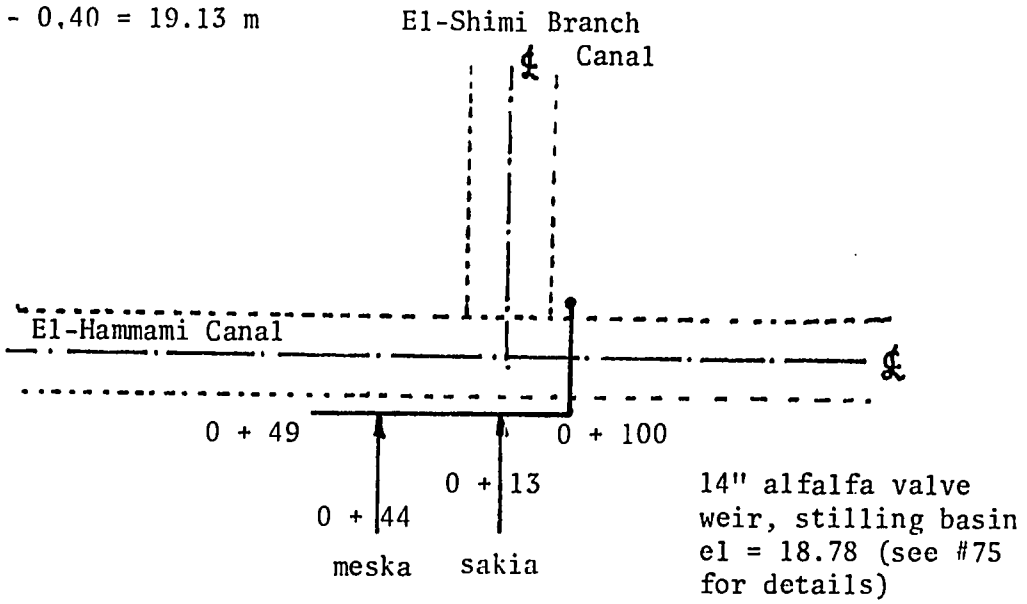
at 0 + 53 = 19.55

at 1 + 38 = 19.54

at 2 + 18 = 19.53

at 2 + 58 = 19.52

Outlet #3

1. Q design = 0.0566 m^3/s (2 ft^3/s), TDH in pipe = 20.13, loss in alfalfa valve and 14" pressure pipe for Q design = 0.60 m (see design for outlet #7S), fluctuating head factor = 0.40 m.
2. Maximum elevation of water surface over alfalfa valve = 20.13 - 0.60 - 0.40 = 19.13 m
3. 

4. Weir
5. 2 X 20" gates
6. Minimum elevation of water surface at top of meska CLD = 18.21 m
7. Normal depth of flow for Q design = 0.34 m
8. Elevation of meska CLD bottom = 18.21 - 0.34 = 17.87 m
9. h_L gate = 0.05 m
10. Excess head loss = 19.13 - 18.21 - 0.05 - 0.30 - 49 m *0.00015 = 0.56 m
11. IF Q = 0.1671 m^3/s (5.9 ft^3/s), extra head used = (0.76 - 0.60) + (0.11 - 0.05) + (0.55 - 0.34) = 0.43 m
12. Design elevation of water surface over alfalfa valve = 20.13 - 0.76 - 0.40 = 18.97 m
13. h_L 20" gate = 0.11 m
14. Y for Q max = 0.55 m

15. Design elevation of water surface in meska CLD = 17.87 + 0.55 = 18.42

16. Design elevation of CLD bottom near alfalfa valve = 18.97 - 0.30 - 0.50 = 18.17 m

17. Design elevation of water surface over alfalfa valve for Q design = 18.28 + 0.50 + 0.09 = 18.87 m

18. Design elevation of water surface at 0 + 13 = 18.67

    at 0 + 44 = 18.66 m

19. Elevation of CLD bottom at 0 + 13 = 18.12 m

    at 0 + 44 = 18.11 m

20. Not applicable

21. Available head loss at 0 + 13 = 18.67 - 18.21 = 0.46 m

    at 0 + 44 = 18.66 - 18.21 = 0.45 m

22. All of (21) > (13)

23. Freeboard = 0.55/3 = 0.18 m

24. CLD depth = 0.55 + 0.18 + 0.00015 *49 = 0.64 m = 29.2" use: 30" = 0.76 m

25. Elevation of CLD top at weir = 18.96 - 0.19 + 0.50 = 19.28 m

    at 0 + 13 = 18.88 m

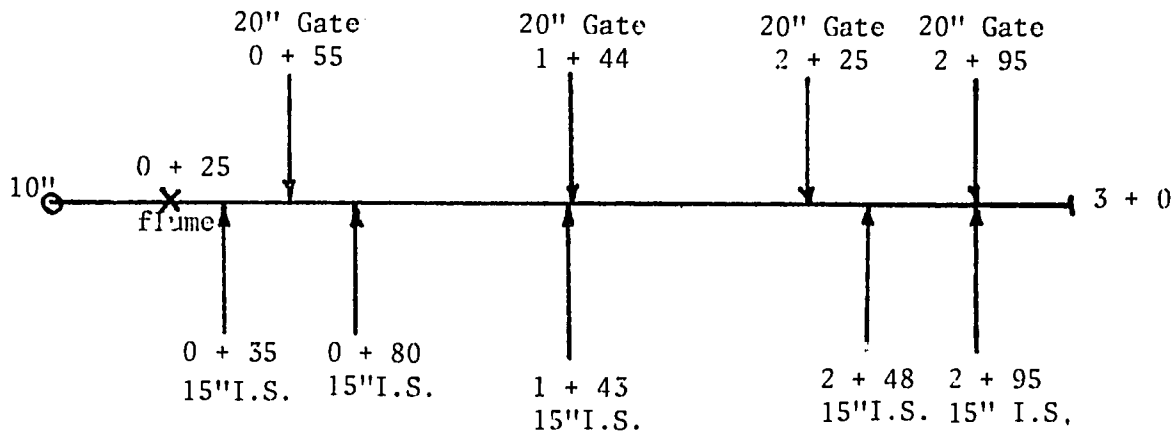
    at 0 + 44 = 18.87 m

Outlet #4

1. Q design = 0.0756 m^3/s (2.7 ft^3/s), TDH in pipe = 20.02 m, head loss in alfalfa valve = 0.35 m, fluctuating head factor = 0.40 m

2. Maximum elevation of water surface over pipe outlet = 20.02 - 0.35 - 0.40 = 19.27 m

3.



(I.S. = Inverted Siphon)

4. Flume

5. 5 X 15" I.S.

   4 X 20" gates

6. Minimum elevation of water surface at top of meska CLD = 18.31 (north) = 18.35

7. Yn for Q design = 0.38 m

8. Elevation of meska CLD bottom = 17.93 (north) 17.97 (south)

9. h_L 20" gate = 0.07 m

   h_L 15" I.S. = 0.07 m

   h_L flume = 0.07 m

10. Excess head loss available = 19.27 - 18.31 - 0.07 - 0.07 - 300 *0.0015 = .78 m. (north)
    = 0.74 m (south)

11. For Q maximum use flow through south as critical, if Q maximum = 0.1133 m 3/5 (4.0 ft 3/5), then extra head used = (0.78 - 0.35) + (0.15 - 0.07) + (.09 - .07) + (0.46 - 0.38) = 0.61 m

12. Design elevation of water surface over alfalfa valve = 20.02 - 0.78 - 0.40 = 18.84 m

13. h_L 20 " gate = 0.09 m

    h_L 15" inverted siphon = 0.15 m

    h_L flume = 0.09 m

14. Yn for Q max = 0.46 m

15. Design elevation of water surface in meska CLD =

    17.93 + 0.46 = 18.39 (north)

    17.97 + 0.46 = 18.43 (south)

16. Design elevation of CLD bottom near alfalfa valve = 18.84 - 0.31 - 0.01 = 18.52 m

17. Design elevation of water surface over alfalfa valve for Q design = 18.52 + 0.26 + 0 - 01 = 18.79 m

18. Design elevation of water surface at 0 + 35 = 18.84 - 0.00015 x 35 - .07 = 18.74 m

    at 0 + 55 = 18.74 m

    at 0 + 80 = 18.74 m

    at 1 + 43 = 18.73 m

    at 1 + 44 = 18.73 m

    at 2 + 25 = 18.72 m

    at 2 + 48 = 18.71 m

    at 2 + 95 = 18.71 m

    at 2 + 95 = 18.71 m

19. Y min = 0.42 < 0.46 of (13) elevation of CLD bottom at 0 + 35 = 18.74 - .46 = 18.28 m

    at 0 + 55 = 18.28 m

    0 + 80 = 18.28 m

    1 + 43 = 18.27 m

1 + 44 = 18.27 m

2 + 25 = 18.26 m

2 + 28 = 18.25 m

2 + 95 = 18.25 m

2 + 95 = 18.25 m

20. Not applicable

21. Available head loss at 0 + 35 = 18.74 - 18.43 = 0.31 m

at 0 + 55 = 18.73 = 18.31 = 0.43 m

0 + 80 = 18.73 - 18.35 = .39 m

1 + 43 = 18.72 - 18.35 = .38 m

1 + 44 = 18.72 - 18.31 = .42 m

2 + 25 = 18.61 - 18.31 = .42 m

2 + 95 = 18.70 - 18.31 = .40 (north) m

2 + 95 = 18.70 - 18.35 = .36 (south) m

22. All of (20) > all of (13)

23. Freeboard = 0.46/3 = 0.15 m

24. CLD depth = 0.46 + 0.15 + 0.00015 * 300 m = 0.66 m = 25.9"

use 26" = 0.66 m

25. Design elevation of CLD top at 0 + 00 = 18.52 + 0.66 = 19.18 m

0 + 35 = 18.94 m

0 + 55 = 18.94 m

0 + 80 = 18.94 m

1 + 43 = 18.93 m

1 + 44 = 18.93 m

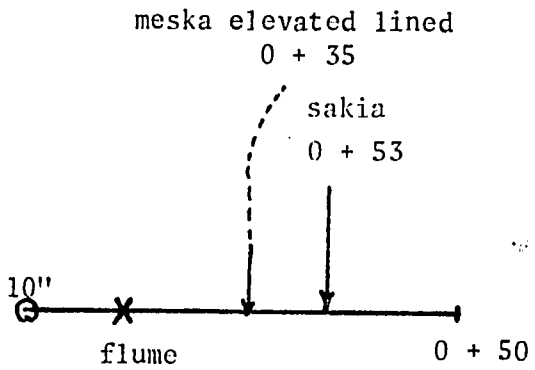
2 + 25 = 18.92 m

2 + 48 = 18.91 m

2 + 95 = 18.91 m

Outlet #5

1. Q design = 0.0566 m^3/s (2.0 ft^3/s), TDH = 19.68 m, head loss in alfalfa valve = 0.19 m, fluctuating head factor = 0.10 m
2. Maximum elevation of water surface over pipe outlet = 19.68 - 0.19 m - 0.40 m = 19.09 m
3.



4. Flume
5. 2 X 20" gates
6. Maximum elevation of water surface at top of meska CLD = 18.30 m
7. Normal depth of flow = 0.34 m
8. Elevation of meska CLD bottom = 18.30 - 0.34 = 17.96 m
9. h_L 20" gate = 0.05 m

   h_L flume = 0.06 m
10. Excess head available = 19.09 m - 18.30 - 0.05 - 0.05 - 0.00015 * 58 = 0.68 m
11. If Q max = 0.1133 m^3/s (4.0 ft^3/s), then extra head used = (0.78 - 0.19) + (0.09 - 0.05) + (0.09 - 0.06) + (0.46 - 0.34) = 0.78 m (too much). If Q max = 0.0850 m^3/s (3.0 ft^3/s), then the extra head used = (0.44 - 0.19) + (0.07 - 0.05) + (0.08 - 0.06) + (0.40 - 0.34) = 0.35 m

12. Design elevation of water surface over alfalfa valve = 19.68 - 0.44 - 0.40 = 18.84 m

13. h_L in 20" gate for Q max = 0.07 m

    h_L for flume = 0.08

14. Y n for Q max = 0.40 m

15. Design elevation of water surface in meska CLD = 17.96 + 0.40 = 18.36 m

16. Design elevation of CLD bottom near alfalfa valve = 18.84 - 0.27 - 0.01 = 18.56 m

17. Design elevation of water surface over alfalfa valve for Q design = 18.56 + .23 + .01 = 18.80 m

18. Design elevation of water surface at 0 + 35 = 18.84 - 0.01 = 18.75 m

    0 + 53 = 18.84 - 0.01 - .08 = 18.75 m

19. Elevation of CLD bottom at 0 + 35 = 18.75 - 0.40 = 18.35

    at 0 + 53 = 18.35 m

20. Not applicable

21. Available head loss at 0 + 35 = 18.75 - 18.36 = 0.39 m

    at 0 + 53 = 0.39 m

22. All of (21) > (13)

23. Freeboard = 0.40/3 = 0.13 m, use 0.15 m

24. CLD depth = 0.40 + 0.15 + 0.00015* 58 = 0.56 m = 22.0"

25. Elevation of CLD top at 0 + 00 = 18.56 + 0.56 = 19.12 m

    at 0 + 35 = 18.35 + 0.56 = 18.91 m

    at 0 + 53 = 18.91 m

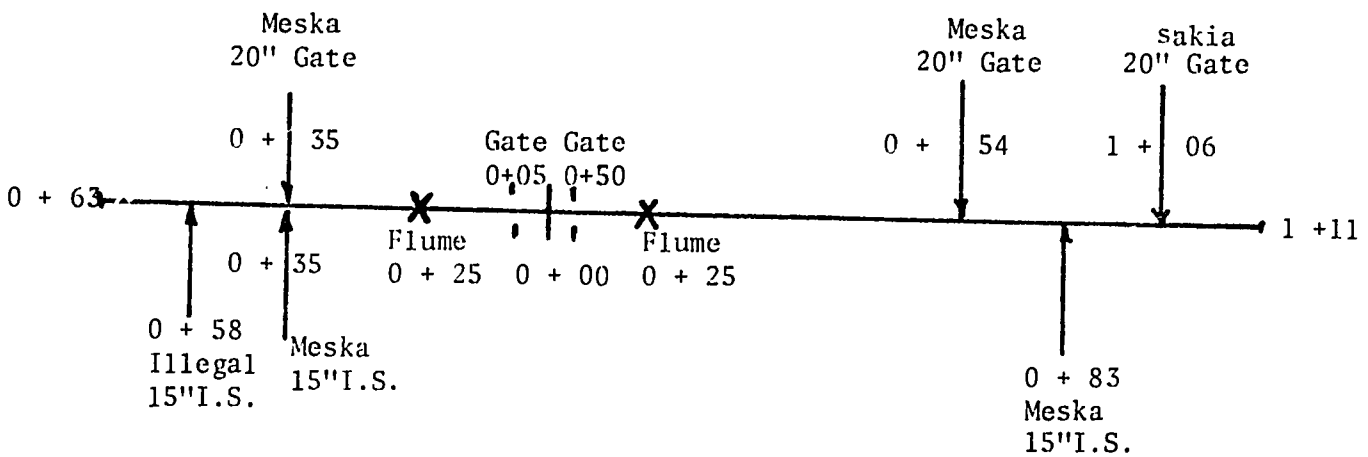
Outlet #6

1. Q design = 0.0680 m^3/s (2.4 ft^3/s), TDH in pipe = 19.60 m

   head loss in valve = 0.28 m, fluctuating head factor = 0.40 m

2. Maximum elevation of water surface over alfalfa valve = 19.60 - 0.28 - 0.40 = 18.92 m

3.



4. 2 X flumes

5. 3 X 20" gates

   3 X 15" inverted siphons

6. Minimum elevation of water surface at top of meska CLD = 17.84 m (north) & 17.93 m (south)

7. Yn for Q design = 0.36 m

8. Elevation of meska CLD bottom = 17.84 - 0.36 = 17.48 m (north)

   17.57 m (south)

9. h_L 20" gate = 0.06 m

   h_L 15" I.S. = 0.05 m

   h_L flume = 0.07 m

10. Excess head loss available = 18.92 - 17.84 - 0.06 - 0.07 - 111 * 0.00015 = 0.93 m (north)

    18.92 - 17.93 - 0.05 - 0.07 - 111 * 0.00015 = 0.85 m (south)

11. If Q max = 0.1133 m^3/s (4.0 ft^3/s), extra head used = (0.78 - 0.28) + (0.15 - 0.05)(.09 - .07) + (0.46 - 0.36) = 0.72 m

12. Design elevation of water surface over alfalfa valve = 19.60 - 0.78 - 0.40 = 18.42 m

13. h_L for 20" gate for Q max = 0.09 m

    h_L for 15" I.S. for Q max = 0.15 m

14. Yn for Q max = 0.46 m

15. Design elevation of water surface in meska CLD = 17.48 + 0.46 = 17.94 m (north)

    17.57 + 0.46 = 18.03 m (south)

16. Design elevation of CLD bottom near alfalfa valve = 18.42 - .31 - .01 = 18.10 m

17. Design elevation of water of surface over alfalfa valve for Q design = 18.10 + 0.23 + 0.01 = 18.32 m

18. Design elevation of water surface at 0 + 54 (east) = 18.42 - 54 * 0.00015 - 0.09 = 18.32 m

    at 0 + 83 (east) = 18.32 m

    at 1 + 06 (east) = 18.32 m

    at 0 + 35 (west) = 18.32 m

    at 0 + 58 (west) = 18.32 m

19. Y min = 0.46 m $\leq$ Y n of (14) elevation of CLD bottom at 0 + 54 (east) = 18.32 - .46 = 17.86 m

    0 + 83 (east) = 17.86 m

    1 + 06 (east) = 17.85 m

    at 0 + 35 (west) = 17.86 m

    at 0 + 58 (west) = 17.86 m

20. Not applicable

21. Available head loss at 0 + 54 (east) = 18.32 - 17.94 = 0.38 m

    at 0 + 83 (east) = 18.32 - 18.03 = .29 m

    at 1 + 06 (east) = 18.31 - 17.94 = 0.37 m

    at 0 + 35 (west) (north) = 18.32 - 17.94 = .38 m

    (south) 18.32 - 18.03 = 0.29 m

    at 0 + 58 (west) = 18.32 - 18.03 = 0.29 m

22. All of (21) > (13)

23. Freeboard = 0.46/3 = 0.15 m

24. CLD depth = 0.46 + 0.15 + 111 * 0.00015 = 0.63 m = 24.7", use 26" = 0.66 m

25. Elevation of CLD top at 0 + 00 = 18.10 + 0.66 = 18.76 m

    at 0 + 54 (east) = 18.52 m

    at 0 + 83 (east) = 18.52 m

    at 1 + 06 (east) = 18.51 m

    at 0 + 35 (west) = 18.52 m

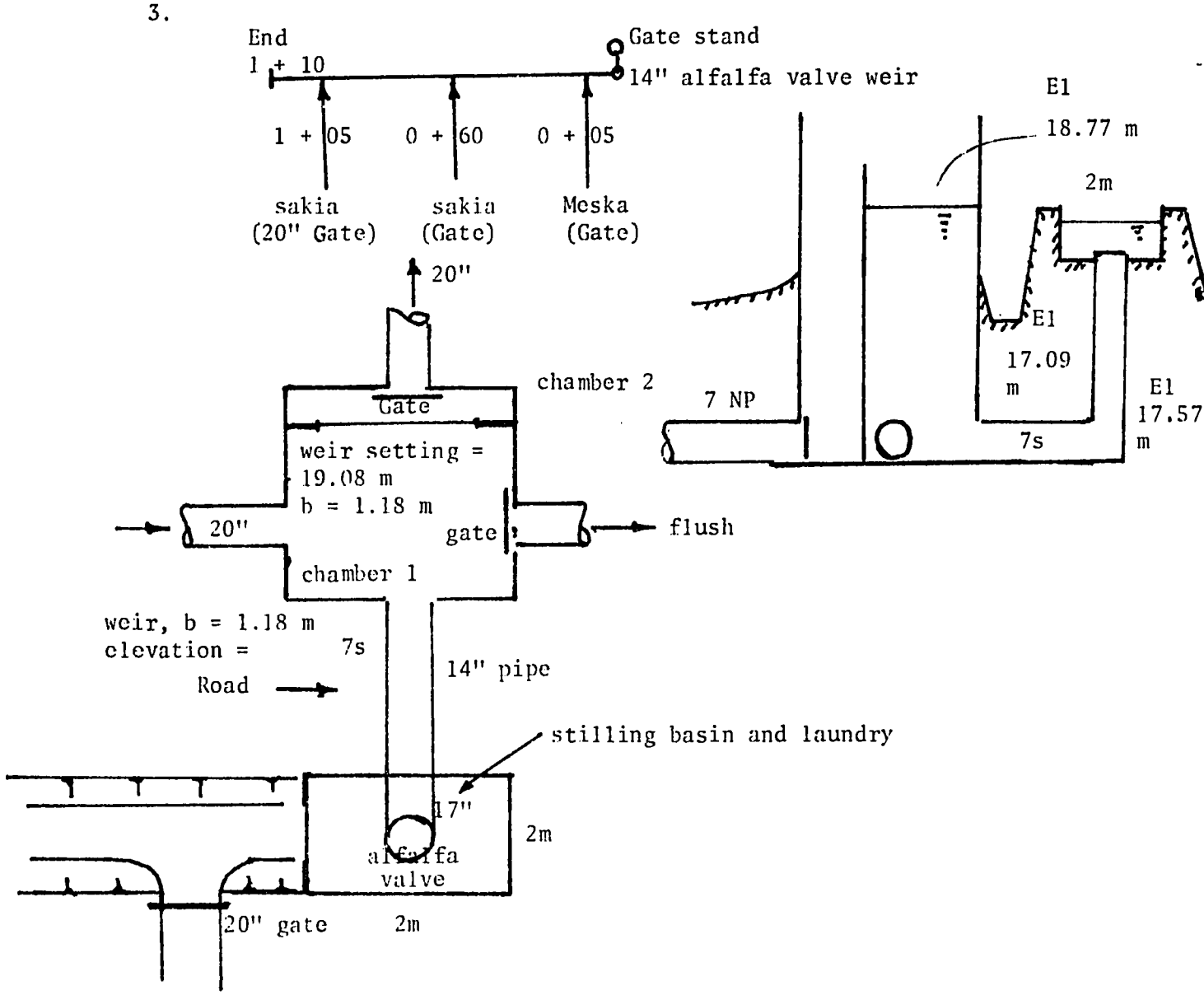
    at 0 + 58 (west) = 18.52 m

Outlet #7

1. Q design = 0.1501 m^3/s (5.3 ft^3/s), TDH in pipe stand = 18.97 m. This is less than the 19.08 m weir setting for 7 NP in the same stand, so that water will flow only to 7 S, not 7 NP (when desired). Head loss in alfalfa valve and pressure pipe for Q design = 0.60 m for 14.98 m in a 14" concrete pipe and riser, 14" alfalfa valve and exit (fully open), entrance (K = 0.50) and 90° bend alfalfa head losses. Fluctuating head factor is not appropriate because this is the critical outlet.

2. Maximum elevation of water surface over pipe outlet = 18.97 - 0.60 - 0.00 = 18.37 m.

3.



4. See sketches above

5. See above

6. Minimum elevation of water surface at top of meska CLD = 17.87 m

7. Yn for Q design = 0.52 m

8. Meska CLD bottom = 17.87 - 0.52 = 17.35 m

9. h_L in 20" gate for Q design = 0.10 m

10. Excess head loss = 18.37 - 17.87 - 0.30 - 0.10 - 110 m *
    0.00015 = 0.08 m

11 - 17. (skip)

18. Elevation of water surface at ST 0 + 05 = 18.37 - 0.0 - 0.30 = 18.07 m
    at ST 0 + 60 elevation = 18.37 - 0.01 - 0.30 = 18.06 m
    at ST 1 + 05 elevation = 18.37 - 0.02 - 0.30 = 18.05 m

19. Elevation of CLD bottom at bottom of weir = 17.87 - 0.52 = 17.35 m
    at 0 + 05 = 17.35 m
    at 0 + 60 = 17.34 m
    at 0 + 05 = 17.33 m

20. Not applicable

21. Available head loss at 0 + 05 = 18.07 - 17.87 = 0.30 m
    Available head loss at 0 + 60 = 18.06 - 17.87 = 0.29 m
    Available head loss at 1 + 05 = 18.05 - 17.87 = 0.28 m

22. All of (21) $\geq$ 0.10 m from (9)

23. Freeboard = 0.52 m/3 = 0.17 m

24. CLD depth = 0.52 + 0.17 + 105 m * 0.00015 = 0.71 m = 27.8"
    use 28" = 0.71 m

25. Elevation of CLD top at bottom of weir = 17.35 + 0.71 = 18.06 m
    at 0 + 05 = 17.35 + 0.71 = 18.06 m
    at 0 + 60 = 17.34 + 0.71 = 18.05 m
    at 1 + 05 = 17.33 + 0.71 = 18.04 m

Outlet #8 on branch pipe that parallels the El Shimi Branch Canal

1. Q design = $0.0566 m^3/s$ ($2.0 ft^3/s$)

   TDH in corner stand = 20.08 m

   head loss in alfalfa valve = 0.19 m

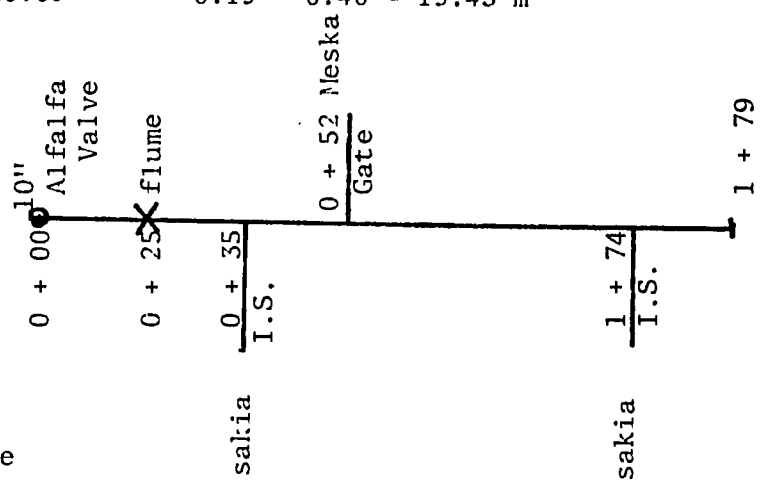
   head loss in pipe valve = 0.06 m

   fluctuating head factor = 0.40 m

2. Maximum elevation of water surface over pipe outlet =

   $= 20.08^{-0.06} - 0.19 - 0.40 = 19.43$ m

3.



4. Flume

5. 2 X 15" I.S.

   1 X 20" gate

6. Minimum elevation of water surface at top of meska CLD =

   = 18.44 m (east)

   = 18.32 m (west)

7. Y_n for Q design = 0.34 m

8. Elevation of meska CLD bottom = 18.10 m (east)

   = 17.98 m (west)

9. h_L 20" gate = 0.05 m (east)

   h_L 15" I.S. = 0.04 m (west)

10. Excess head loss available

    = 19.43 - 18.44 - 0.05 - 0.06 - 52 m * 0.00015 = 0.87 m (east)

    = 19.43 - 18.32 - 0.04 - 0.06 - 174 m * 0.00015 = 0.98 m (west)

11. For Q maximum, flow through east is critical if Q_{max} = $0.0850 m^3/s$ ($3.0 ft^3/s$), then the extra head used = (20.02 - 19.96) + (0.44 - 0.19) + (0.08 - .06) + (0.07 - 0.05) + (0.40 - 0.34) = 0.41 m

    If Q_{max} = $0.850 m^3/s$ ($3.0 ft^3/s$) to the west, then extra head used = (20.02 - 19.96) + (0.44 - 0.19) + (0.08 - .06) + (0.08 - 0.04) + (0.40 - 0.34) = 0.43 m

12. Design elevation of water surface over alfalfa valve = 19.96 - 0.44 - 0.40 = 19.12 m

13. h_L 20" gate = 0.07 m

    h_L 15" I.S. = 0.08 m

    h_L Flume = 0.08 m

14. Y_n for Q_{max} = 0.40 m

15. Design elevation of water surface in meska CLD

    = 18.10 + 0.40 = 18.50 (east)

    = 17.98 + 0.40 = 18.38 (west)

16. Design elevation of CLD botton near alfalfa valve = 19.12 - 0.27 - 0.01 = 18.84 m

17. Design elevation of water surface over alfalfa valve for Q design = 18.84 + 0.02 + 0.01 = 19.08 m

18. Design elevation of water surface at 0 + 35 = 19.12 - .00015*35 - .08 = 19.03 m

    at 0 + 52 = 19.03 m

    at 1 + 74 = 19.01 m

19. Y_{min} = 0.46 $\leq$ 0.46 of (14)

    Elevation of CLD bottom at 0 + 35 = 19.1 - .4 = 18.70 m

    at 0 + 52 = 18.63 m

    at 1 + 74 = 18.61 m

20. Not applicable

21. Available head loss at 0 + 35 = 19.03 - 18.38 = 0.65 m

at 0 + 52 = 19.03 - 18.50 = 0.53 m

at 1 + 74 = 19.03 - 18.38 = 0.63 m

22. All of (21) > all of (13)

23. Freeboard = 0.40/3 = 0.13 m, use 0.15

24. CLD depth = 0.40 m + 0.15 m + 0.0015 + 179

= 0.58 m = "22.7"

use "24" = 0.661 m

25. Design elevation of CLD top at 0 + 00 = 18.84 + .61 = 19.45 m

at 0 + 35 = 19.24 m

at 0 + 52 = 19.24 m

at 1 + 74 = 19.22 m

Outlet #9

1. Q design $0.566 m^3/s$ ($2.0 ft^3/s$)

   TDH in corner stand = 20.08 m

   Head loss in 500 m of 20" pipe, entrance, exit and pass through T = 0.08 m

   Head loss in 10" concrete pipe and riser, 14.98 m of 10" alfalfa valve and exit (fully open), entrance (k = 0.50), and 90° level (k = 0.50) (see #7S)

   $= 0.03615 \left(\frac{Q}{D^{2.63}}\right)^{1.8519} + 0.3342 \frac{Q^2}{D^4}$

   $= 25.04\ Q^{1.8519} + 80.29\ Q^2$

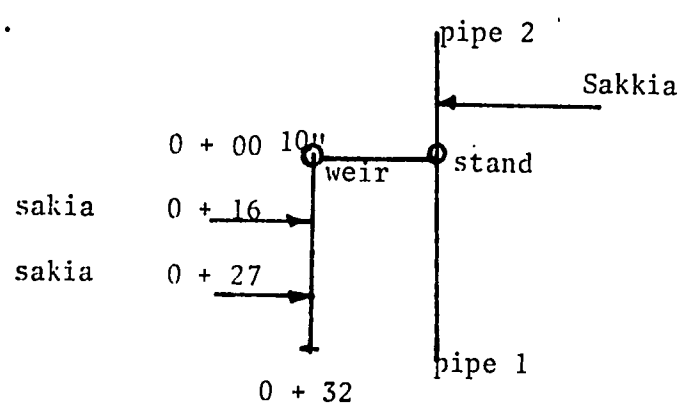
   = 0.38 m

   fluctuating head factor = 0.40 m

2. Maximum elevation of water surface over alfalfa valve =

   20.08 - 0.08 - 0.38 - 0.40 = 19.22 m

3.



4. Weir

5. 2 X 20" gates

6. Minimum elevation of water surface at top of meska CLD = 18.25 m

7. Y_n for Q design = 0.34 m

8. Elevation of meska CLD bottom = 17.91 m

9. h_L 20" gate = 0.05 m

10. Excess head available = 19.22 - 18.25 - 0.05 - 0.30 - 0.0015 + 32 = 0.62 m

11. If Q_{max} = 0.0765

    Extra head used = (0.15 - 0.08) + (0.68 - 0.38) + (0.07 - 0.05) + (0.38 - 0.34) = 0.43 m

12. Design elevation of water surface over alfalfa valve = 20.08 - 0.15 -0.68 - 0.40 = 18.85 m

13. h_L 20" gate = 0.07 m

14. Y_n = 0.38 m

15. Design elevation of water surface in meska CLD = 17.91 + 0.38 = 18.29 m

16. Design elevation of CLD bottom near alfalfa valve = 18.85 - 0.11 - 0.50 = 18.24 m

17. Design elevation of water surface over alfalfa valve for Q design = 18.24 + 0.50 + 0.09 = 18.22 m

18. Design elevation of water surface at 0 + 16 = 18.55 m

    at 0 + 27 = 18.55 m

19. Elevation of CLD bottom at 0 + 16 = 18.17 m

    at 0 + 27 = 18.17 m

20. Not applicable

21. Available head loss at 0 + 16 = 0.26 m

    at 0 = 27 = 0.26 m

22. All of (21) > (13)

23. Freeboard = 0.38/3 = 0.13, use 0.15 m

24. CLD depth = 0.38 m + 0.15 + 0.00 = 0.53 m = 20.9" use 22" = 0.56 m

25. Design elevation of CLD top at 0 + 00 = 19.35 m

    at 0 + 16 = 18.73 m

    at 0 + 27 = 18.73 m

3.2.9 Distributer Concrete Lined Ditches for Pipe 2 Alfalfa Valves.

Outlet #1

1. Q design = 0.0566 m^3/s (2.0 ft^3/s)

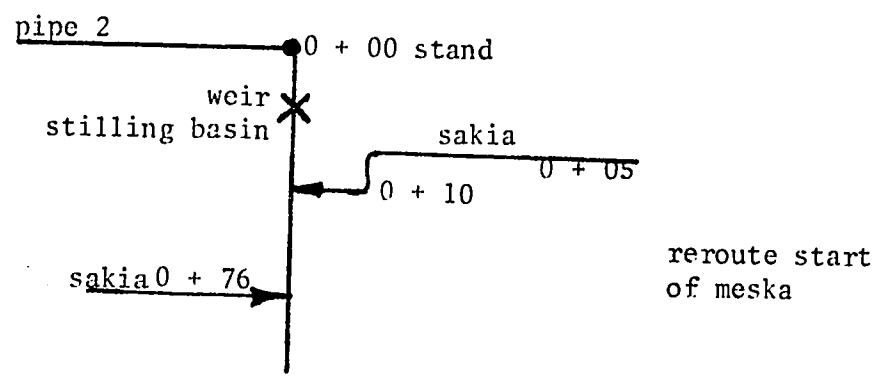
   TDH = 19.91 m

   Head loss (see #7, 26.64 Q^2, 12" orifice) = 0.09 m

   Fluctuating head factor = 0.20 m

2. Maximum elevation of water surface over alfalfa valve = 19.91 - 0.09 - 0.20 = 19.62 m

3.



4. Weir

5. 1 X 20" gate

   1 X 15" I.S.

6. Minimum elevation of water surface at top of meska CLD = 18.01 m (East)

   = 18.05 m (West)

7. Y_n for Q design = 0.34 m

8. Elevation of meska CLD bottom = 17.67 m (East)

   17.71 m (West)

9. h_L 20" gate = 0.05 m

   h_L 15" I.S. = 0.04 m

10. Excess head available = 19.62 - 18.01 - 0.05 - 0.30 - 10 *0.0015 = 1.26 m (East)

    or, 19.62 - 18.05 - 0.04 - 0.30 - 76 *0.0015 = 1.22 (West)

11. If Q_{max} = 0.1671 m^3/s (5.9 ft^3/s)

Extra head used = (0.74 - 0.09) + (0.31 - 0.04) + (0.55 - 0.34)

= 1.13 m

12. Design elevation of water surface at 0 + 00 = 19.91 - 0.74 - 0.20

= 18.97 m

13. h_L 20" gate = 0.11 m

h_L 15" I.S. = 0.31 m

14. Y_n for Q_{max} = 0.55 m

15. Design elevation of water surface in meska CLD

= 17.67 + 0.55 = 18.22 m (E)

= 17.71 + 0.55 = 18.26 m (W)

16. Elevation of CLD bottom of 0 + 00 = 18.97 - 0.30 - 0.50 = 18.17 m

17. Elevation of water surface at 0 + 00 for Q design = 18.28 +

0.50 + 0.09 = 18.87 m

18. Elevation of water surface at 0 + 10 = 18.97 - 0.0015* 10 = 0.30 =

18.67 m

at 0 + 76 = 18.97 - 0.0015* 76 - 0.30 = 18.66 m

19. Y_{min} = 0.55 m $\leq$ Y_n = 0.55 m

Elevation of CLD bottom at 0 + 10 = 18.67 - 0.55 = 18.12 m

at 0 + 76 = 18.66 - 0.55 = 18.11 m

20. Not applicable

21. Available head loss at 0 + 10 = 18.67 - 18.22 = 0.45 m

at 0.76 = 18.66 - 18.26 = 0.40 m

22. All of (21) > (13)

23. Freeboard = 0.55/3 = 0.18 m

24. CLD depth = 0.55 + 0.18 + 0.00015* 76 = 0.74 m = 29.2"

Use 30" = 0.76 m

25. Elevation of CLD top at 0 + 00 = 18.97 + 0.20 = 19.17 m

at 0 + 10 = 18.12 + 0.76 = 18.88 m

at 0 + 76 = 18.11 + 0.76 = 18.87 m

Outlet #2

1. Q design = $0.0850 m^3/s$ ($3.0 ft^3/s$)

   TDH = 19.69 m

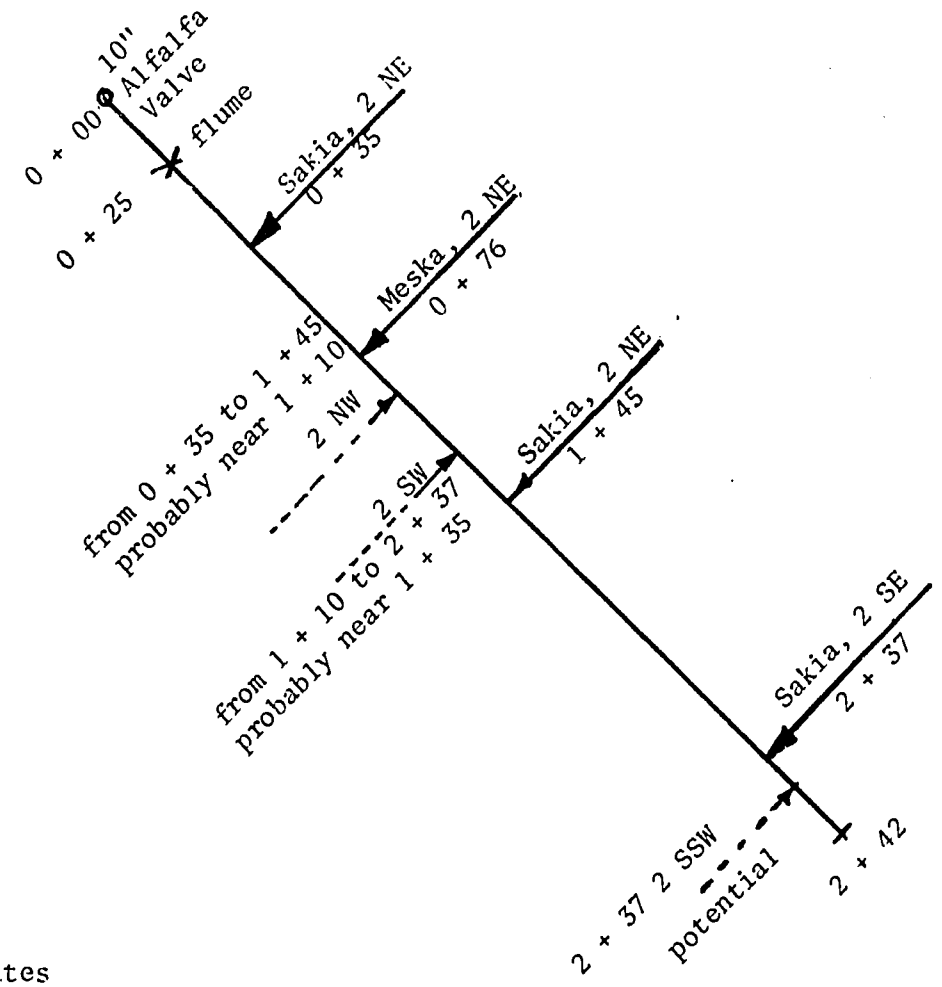
   Head loss in 10" valve = 0.44 m

   Fluctuating head factor = 0.20 m

2. Maximum elevation of water surface over alfalfa valve

   = 19.69 - 0.44 - 0.20 = 19.05 m

3.



4. Flume

5. 4 X 20" gates

   2 or 3 X 15" I.S.

6. Minimum elevation of water surface at top of meska CLD

   = 18.29 m (Northeast)

   = 18.34 m (Northwest)

   = 18.32 m (Southeast)

   = 18.36 m (Southwest)

   = 18.20 m (South-southwest) potential

7. Y_n for Q design = 0.40 m

8. Elevation of meska CLD bottom

   = 18.29 - 0.40 = 17.89 m (Northeast)

   = 17.94 m (Northwest)

   = 17.92 m (Southeast)

   = 17.96 m (Southwest)

   = 17.80 m (South-Southwest)

9. Head loss for Q design 20 gate = 0.09 m

   15" I.S. = 0.08 m

   Flume = 0.08 m

10. Excess head loss = 19.05 - 18.29 - 0.09 - 0.08 - 0.00015* 145 = 0.57 m

    = 19.05 - 18.34 - 0.08 - 0.08 - 0.0015*110 = 0.53 m (Northwest)

    = 19.05 - 18.32 - 0.09 - 0.08 - 0.00015*237 = 0.52 m (Southeast)

    = 19 05 - 18.36 - 0.08 - 0.08 - 0.00015*237 = 0.49 m (Southwest)

    = 19.05 - 18.20 - 0.08 - 0.08 - 0.00015*237 = 0.63 m (South-Southwest)

11. If Q_{max} = 0.1133 m^3/s (4.0 ft^3/s)

    Extra head used = (0.78 - 0.44) + (0.15 - 0.08) + (0.46 - 0.40) + 0.09 - 0.08 = 0.48 m

12. Elevation of water surface over alfalfa valve = 19.69 - 0.78 - 0.20 = 18.71 m

13. h_L 20" gate = 0.09 m

    h_L 15" I.S. = 0.15 m

    h_L Flume = 0.09 m

14. Y_n for Q_{max} = 0.46 m

15. Design elevation of water surface in meska CLD

    = 17.89 + 0.46 = 18.35 m (Northeast)

    = 18.40 m (Northwest)

    = 18.38 m (Southeast)

    = 18.42 m (Southwest)

    = 18.26 m (South-Southwest)

16. Elevation of CLD bottom at 0 + 00 = 18.71 - .31 - .01 = 18.39 m

17. Design elevation of water surface at 0 + 00 for Q design =

    18.39 + 0.27 + .01 = 18.67 m

18. Design elevation of water surface at 0 + 35

    = 18.71 - 0.00015*35 - 0.09 = 18.61 m

    at 0 + 76 = 18.60 + .01 = 18.61 m

    at 1 + 10 = 18.59 + .01 = 18.60 m

    at 1 + 35 = 18.59 + .01 = 18.60 m

    at 1 + 45 = 18.59 + .01 = 18.60 m

    at 2 + 37 = 18.57 + .01 = 18.58 m

19. $Y_{min} = 0.46 \text{ m} \geq Y_n = 0.46 \text{ m}$

    Elevation of CLD bottom at 0 + 35 = 18.61 - 0.46 = 18.15

    at 0 + 76 = 18.14 + .01 = 18.15 m

    at 1 + 10 = 18.13 + .01 = 18.14 m

    at 1 + 35 = 18.13 + .01 = 18.14 m

    at 1 + 45 = 18.13 + .01 = 18.14 m

    at 2 + 37 = 18.11 + .01 = 18.12 m

20. Not applicable

21. Available head loss at 0 + 35 = 18.61 - 18.35 = 0.16 m

    at 0 + 76 = 18.61 - 18.35 = 0.16 m

    at 1 + 10 = 18.60 - 18.40 = 0.20 m

    at 1 + 35 = 18.60 - 18.42 = 0.18 m

    at 1 + 45 = 18.60 - 18.35 = 0.25 m

    at 2 + 37 = 18.58 - 18.38 = 0.20 m

22. All of (21) > (13)

23. Freeboard = 0.46/3 = 0.15

24. CLD depth = 0.46 + 0.15 + 242 - 0.0015 = 0.65 m = 25.4, if

    26 " = 0.66 m

25. Design elevation of CLD top at 0 + 00 = 18.39 + 0.66 = 19.05 m

at 0 + 35 = 18.15 + 0.66 = 18.81 m

at 0 + 76 = 18.81

at 1 + 10 = 18.80

at 1 + 35 = 18.80

at 1 + 45 = 18.80

at 2 + 37 = 18.78

Outlet #3

1. Q design = $0.1671 m^3/s$ ($5.9 ft^3/s$)

   TDH = 19.14 m

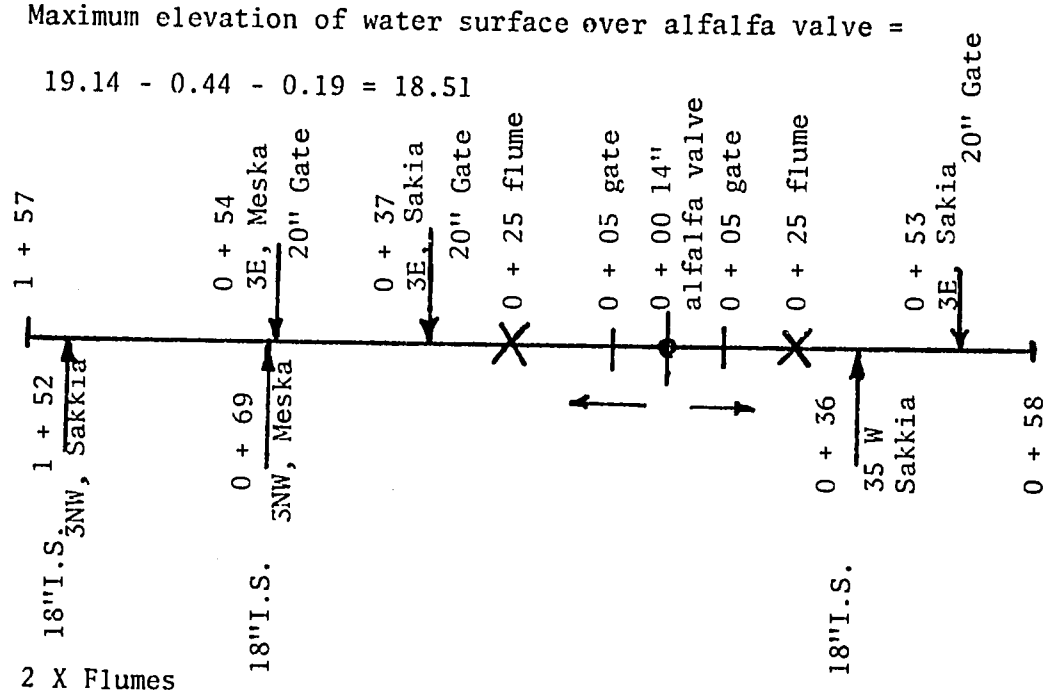
   Head loss in alfalfa valve (14") = 0.44 m

   Fluctuating head factor = 0.19 m

2. Maximum elevation of water surface over alfalfa valve =

   19.14 - 0.44 - 0.19 = 18.51

3.



4. 2 X Flumes

5. 3 X 18" inverted siphons

   3 X 20" gates

6. Minimum elevation of water surface at top of meska CLD = 18.29 (East)

   = 18.18 (Northwest)

   = 18.05 (Southwest)

7. Y_n for Q design = 0.55 m

8. Elevation of meska CLD bottom

   = 18.29 - 0.55 = 17.44 m (East)

   = 17.63 m (Northwest)

   = 17.50 m (Southwest)

9. h_L 20" gate = 0.11 m

   h_L 18" I.S. = 0.14 m

   h_L Flume = 0.10 m

10. Excess head loss available

= 18.50 - 18.29 - 0.11 - 0.10 - 45*0.00015 = 0.00 (East)

= 18.51 - 18.18 - 0.14 - 0.10 - 152*0.0015 = 0.07 (Northwest)

= 18.51 - 18.05 - 0.14 - 0.10 - 36*0.0015 = 0.21 (Southwest)

11. No excess hr

12. See #2

13. See #9

14. See #7

15. See #6

16. Elevation of CLD botton at 0 + 00 = 18.51 - 0.36 - 0.011 = 18.14 m

17. See #2

18. Elevation of water surface at 9 + 37 (north) = 18.51 - 37*0.0015 - 0.10 = 18.40 m

at 0 + 54 (north) = 18.51 = 54*0.0015 - 0.10 = 18.40 m

at 0 + 69 (north) = 18.40 m

at 1 + 52 (north) = 18.39 m

at 0 + 36 (south) = 18.40 m

at 0 + 53 (south) = 18.40 m

19. Y_{min} = 0.54 m < Y_n = 0.55

Elevation of CLD bottom at 0 + 37 (north) = 18.40 - 0.55 m = 17.85 m

at 0 + 54 (north) = 17.85 m

at 0 + 69 (north) = 17.85 m

at 1 + 52 (norht) = 17.84 m

at 0 + 36 (south) = 17.85 m

at 0 + 53 (south) = 17.85 m

20. Not applicable

21. Available head loss at 0 + 37 (north) = 18.40 - 18.29 = 0.11 m

at 0 + 54 (N) = 18.40 - 18.29 = 0.11 m

at 0 + 69 (N) = 18.40 - 18.18 = 0.22 m

at 1 + 52 (N) = 18.39 - 18.18 = 0.21 m

at 0 + 36 (S) = 18.40 - 18.05 = 0.35 m

at 0 + 53 (S) = 18.40 - 18.29 = 0.11 m

22. All of (21) $\geq$ (9)

23. Freeboard = 0.55/3 = 0.18 m

24. CLD depth = 0.55 + 0.18 + 157*0.0015 = 0.75 m = 29.7 use 30" = 0.76 m

25. Design elevation of CLD top at 0 + 00 = 17.96 + 0.76 = 18.72 m

at 0 + 37 (North) = 17.85 + 0.76 = 18.61 m

at 0 + 54 (North) = 18.61 m

at 0 + 69 (North) = 18.61 m

at 1 + 52 (North) = 18.60 m

at 0 + 36 (South) = 18.61 m

at 0 + 53 (South) = 18.61 m

Outlet #4

1. Q design = $0.0566 m^3/s$ ($2.0 ft^3/s$)

   TDH = 19.08 m

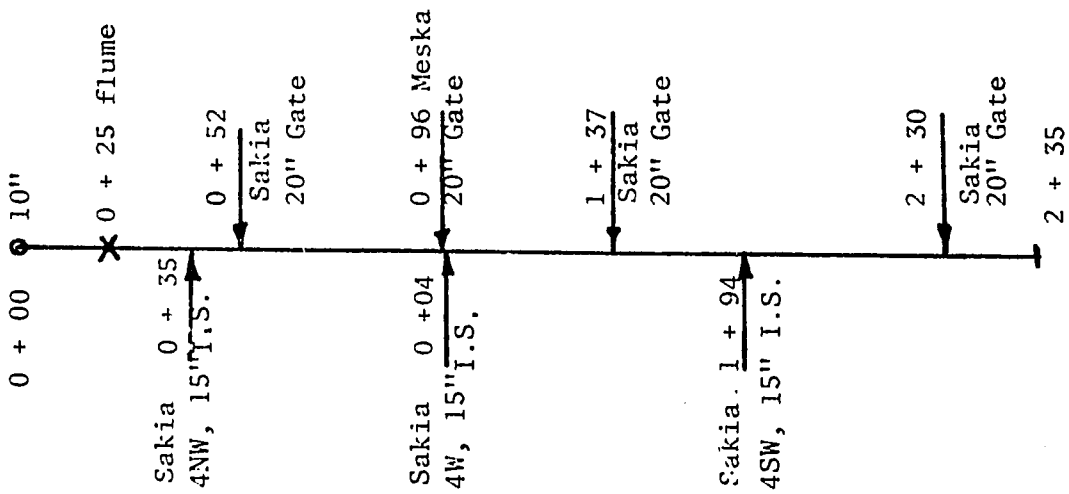
   Head loss in alfalfa valve = 0.19 m

   Fluctuating head factor = 0.20

2. Maximum elevation of water surface over alfalfa valve =

   19.08 - 0.19 - 0.20 = 18.69 m

3.



4. Flume

5. 3 X 15" I.S.

   4 X 20" gates

6. Minimum elevation of water surface at top of meska CLI

   = 18.13 m (East)

   = 18.04 m (Northwest)

   = 17.99 m (West)

   = 17.99 m (Southwest)

7. Y_n for Q design = 0.34 m

8. Evaluation of meska CLD bottoms

   = 18.13 - 0.34 = 17.79 m (East)

   = 17.70 m (Northwest)

   = 17.65 m (West)

   = 17.65 m (Southwest)

9. h_L 20" gate = 0.05 m

   h_L 15" I.S. = 0.04 m

   h_L Flume = 0.06 m

10. Excess head available

    = 18.69 - 18.13 - 0.05 - 0.06 - 230*.03*.00015 = 0.42 m (East)

    = 18.69 - 18.04 - 0.04 - 0.06 - 35*.01*.00015 = 0.54 m (Northwest)

    = 18.69 - 17.99 - 0.04 - 0.06 - 104*.02*.0015 = 0.58 m (West)

    = 18.69 - 17.99 - 0.04 - 0.06 - 195*.03*.0015 = 0.57 m (Southwest)

11. If Q_{max} = 0.0850 m^3/s (3.0 ft^3/s), extra head used = (0.44 - 0.19) + (.08 - .06) + (0.07 - 0.05) + (0.40 - 0.34) = 0.35 m (East)

12. Design elevation of water surface over alfalfa valve =

    19.08 - 0.44 - 0.20 = 18.44 m

13. h_L 20" gate = 0.07

    h_L 15" I.S. = 0.08 m

    h_L Flume = 0.08 m

14. Y_n for Q max = 0.40 m

15. Design elevation of water surface in meska CLD

    = 17.79 + 0.40 = 18.19 (East)

    = 17.70 + 0.40 = 18.10 (Northwest)

    = 18.05 (West)

    = 18.05 (Southwest)

16. Design elevation of CLD bottom near alfalfa valve = 18.44 - 0.27 - .01 = 18.16 m

17. Design elevation of water surface over alfalfa valve for Q design

    = 18.16 + 0.23 + 0.01 = 18.40 m

18. Design elevation of water surface at 0 + 35 = 18.44 - 0.00015*35 - 0.08 = 18.35 m

    at 0 + 52 = 18.35 m

    at 0 + 96 = 18.35 m

    at 1 + 04 = 18.34 m

at 1 + 37 = 18.34 m

at 1 + 95 = 18.33 m

at 2 + 30 = 18.33 m

19. $Y_{min} = 0.42 > Y_n = 0.40$

Elevation of CLD bottom at 0 + 35 = 17.93 m

at 0 + 52 = 17.93 m

at 0 + 96 = 17.93 m

at 1 + 04 = 17.92 m

at 1 + 37 = 17.92 m

at 1 + 95 = 17.91 m

at 2 + 30 = 17.91 m

20. Adjusted design elevation of the water surface at 0 + 52 = 18.33 m

at 0 + 96 = 18.33 m

at 1 + 37 = 18.32 m

at 2 + 30 = 18.31 m

21. Available head loss at 0 + 35 = 18.35 - 18.10 = 0.25 m

at 0 + 52 = 18.33 18.19 = 0.14 m

at 0 + 96 = 18.33 - 18.19 = 0.14 m

at 1 + 04 = 18.34 - 18.05 = 0.29 m

at 1 + 37 = 18.32 - 18.19 = 0.13 m

at 1 + 95 = 18.33 - 18.05 = 0.28 m

at 2 + 30 = 18.31 - 18.19 = 0.12 m

22. All of (21) > (13)

23. Freeboard = 0.40/3 = 0.13 use 0.15 m

24. CLD depth = 0.42 + 0.15 + 235*0.0015 = 0.61 m = 23.8" use 24" = 0.61 m

25. Elevation of CLD top at 0 + 00 = 18.16 + 0.61 = 18.77 m

at 0 + 35 = 18.54 m

at 0 + 52 = 18.54 m

at 0 + 96 = 18.54 m

at 1 + 04 = 18.53 m

at 1 + 37 = 18.53 m

at 1 + 95 = 18.52 m

at 2 + 30 = 18.51 m

Outlet #5

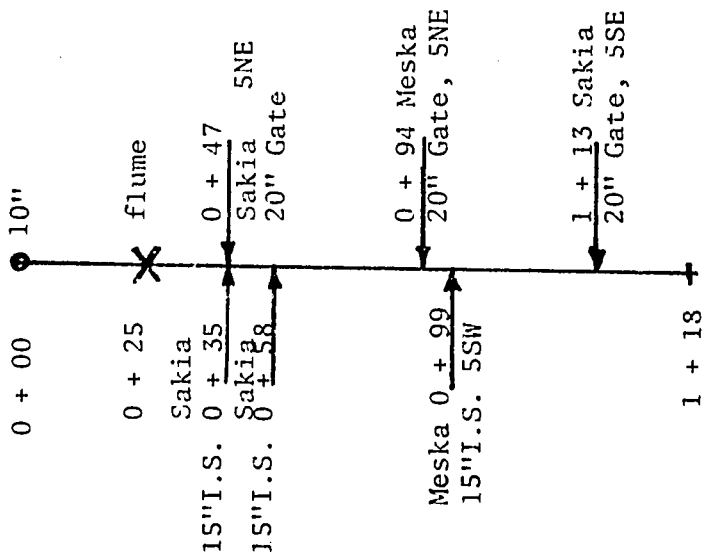
1. Q design = 0.0566 m^3/s (2.0 ft^3/s)

   TDH in pipe = 19.01 m

   Head losss in alfalfa valve (10") = 0.19 m

   Fluctuating head factor = 0.20 m (arbitrarily selected)

2. Maximum elevation of water surface over alfalfa valve = 19.01 - 0.19 - 0.20 = 18.62 m

3. 

4. Flume

5. 3 X 15" I.S.

   3 X 20" gates

6. Minimum water elevation of water surface at top of meska CLD =

   18.17 (Northeast)

   18.23 (Southeast)

   18.10 (Northwest)

   18.37 (Southwest)

7. Y_n for Q design = 0.34 m

8. Elevation of meska CLD bottom = 17.83 m (Northeast)

   = 17.89 m (Southeast)

   = 17.76 m (Northwest)

   = 18.03 m (Southwest)

9. h_L 20" gate = 0.05 m

   h_L 15" I.S. = 0.04

   h_L Flume = 0.06 m

10. Excess head available = 18.62 - 18.17 - 0.05 - 0.06 - 0.01 = 0.33 m (Northeast)

    = 18.62 - 18.23 - 0.05 - 0.06 - 0.02 = 0.26 m (Southeast)

    = 18.62 - 18.10 - 0.04 - 0.06 - 0.01 = 0.41 m (Northwest)

    = 18.62 - 18.37 - 0.04 - 0.06 - 0.01 = 0.14 m (Southwest)

11. There is not enough excess head (0.14 m) to try to design for higher Q.

12. See #2

13. See #9

14. See #7

15. See #6

16. Elevation of CLD bottom near alfalfa valve = 18.62 - .23 - .01 = 18.38 m

17. See #2

18. Elevation of water surface at 0 + 35 = 18.62 - .00015*35 - .06 = 18.51 m

    0 + 47 = 18.53 m

    0 + 58 = 18.55 m

    0 + 94 = 18.55 m

    0 + 99 = 18.55 m

    1 + 13 = 18.54 m

19. $Y_{min} = 0.40 > Y_n = 0.34$ m

    Elevation of CLD bottom at 0 + 35 = 18.55 - .4 = 18.15 m

    0 + 47 = 18.15 m

    0 + 58 = 18.15 m

    0 + 94 = 18.15 m

    0 + 99 = 18.15 m

    1 + 13 = 18.15 m

20. Adjusted water surface elevation at 0 + 47 = 18.15 + .34 = 18.49 m

0 + 94 = 18.49 m

1 + 13 = 18.48 m

21. Available head loss at 0 + 35 = 18.55 - 18.10 = 0.45 m

0 + 47 = 18.44 - 18.17 = 0.32 m

0 + 58 = 18.55 - 18.10 = 0.45 m

0 + 94 = 18.49 - 18.23 = 0.26 m

0 + 99 = 18.55 - 18.37 = 0.18 m

1 + 13 = 18.48 - 18.23 = 0.25 m

22. All of (21) > (9)

23. Freeboard = 0.34/3 = 0.11 m, use 0.15 m

24. CLD depth = 0.40 + 0.15 + 118*0.0015 = 0.57 m = 22.3 use 24" = 0.61 m

25. Elevation of CLD top at 0 + 00 = 18.38 + 0.61 = 18.99 m

at 0 + 35 = 18.15 + 0.61 = 18.76 m

at 0 + 47 = 18.76 m

at 0 + 58 = 18.76 m

at 0 + 94 = 18.76 m

at 0 + 99 = 18.76 m

at 1 + 13 = 18.75 m

Outlet #6

1. Q design = $0.0566 m^3/s$ (2.0 ft^3/s)

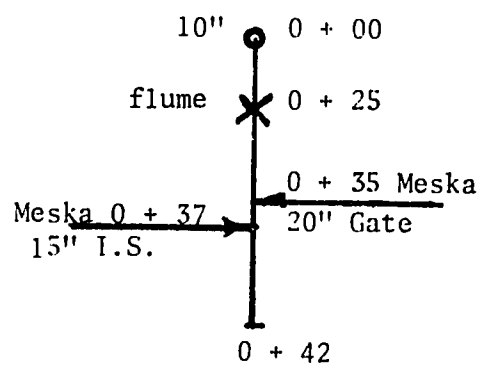
   TDH in pipe = 18.708 m

   Head loss (10") = 0.19 m

   Fluctuating head factor doesn't apply because this is a critical outlet

2. Maximum elevation of water surface over alfalfa valve = 18.78 - 0.19 - 0.00 = 18.59 m

3.



4. Flume

5. 1 X 15" I.S.

   1 X 20" Gate

6. Minimum elevation of water surface at top of meska CLD

   = 18.23 m (Northeast)

   = 18.20 m (Southwest)

   = 18.44 m (West)

7. Y_n for Q design = 0.34 m

8. Elevation of meska CLD bottom = 17.89 m (Northeast)

   17.86 m (Southwest)

   18.10 m (West)

9. h_L 15" I.S. = 0.04 m

   h_L 20" gate = 0.05 m

   h_L Flume = 0.06 m

10. Excess head available = 18.59 - 18.23 - 0.05 - 0.06 - 0.01 = 0.04 m (Northeast)

    = 18.59 - 18.20 - 0.04 - 0.06 - 0.01 = 0.28 m (Southwest)

    = 18.59 - 18.44 - 0.04 - 0.06 - 0.01 = 0.04 m (West)

11. Not applicable

12. See #2

13. See #9

14. See #7

15. See #6

16. Elevation of CLD bottom at 0 + 00 = 18.59 - 0.23 - 0.01 = 18.35 m

17. See #2

18. Design elevation of water surface at 0 + 35 = 18.59 - 0.0015*35 - 0.06 = 18.52 m

    at 0 + 37 = 18.52 m

19. Y_{min} = 0.40 m > Y_n = 0.34 m

    elevation of CLD bottom at 0 + 35 = 18.48 - 0.40 = 18.08 m at 0 + 37 = 18.08

20. Design elevation of the water surface at 0 + 35 = 18.08 + 0.34 = 18.46 m

21. Available head loss at 0 + 35 = 18.46 - 18.23 = 0.23 m

    At 0 + 44 = 18.48 - 18.44 = 0.08 m

22. All of (21) $\geq$ (9)

23. Freeboard = 0.34/3 = 0.11 m, use 0.15 m

24. CLD depth = 0.40 + 0.15 + 0.01 = 0.56 m = 22.05" use 24" = 0.61 m

25. Design elevation of CLD top at 0 + 00 = 18.35 + 0.61 = 18.96 m

    at 0 + 35 = 18.12 + 0.61 = 18.73 m

    at 0 + 37 = 18.12 + 0.61 = 18.73 m

Outlet #7

1. Q design = $0.0566 m^3/s$ ($2.0 ft^3/s$)

   TDH in stand = 18.78 - 196 m of 20" pipe - pass through T (K = 0.3) exit loss (K = 1.0)

   $= 18.78 - 6.0063\ Q^{1.8519} - 1.6142 Q^2$

   = 18.75 m

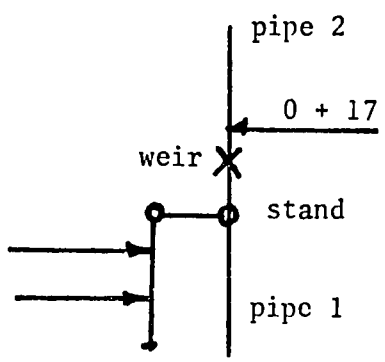
   head loss in 12" orifice = $0.2299\ sec^2/m\ (\frac{Q^2}{D^4})$

   = 0.09 m

   Fluctuating head factor does not apply because this is a critical outlet

2. Maximum elevation of water surface above weir = 18.75 - 0.09 - 0.00 = 18.66 m

3.



4. Weir

5. No I.S. or gate in CLD

   Only gate over orifice in stand

6. Minimum elevation of water surface at top of meska CLD = 18.36 m

7. Y_n for Q design = 0.34 m

8. Elevation of meska CLD bottom = 18.36 - 0.34 = 18.02 m

9. Not applicable

10. Excess head available = 18.66 - 18.36 - 0.30 - 0.00 = 0.00 m

11. Not applicable

12. See #2

13. See #9

14. See #7

15. See #6

16. Elevation of CLD bottom at 0 + 00 = 18.66 - 0.30 - 0.50 = 17.86 m

17. See #2

18. See #6

19. See #8

20. Not applicable

21. Not applicable

22. Not applicable

23. Freeboard = 0.34/3 = 0.11, use 0.15 m

24. CLD depth = 0.34 + 0.15 + 0.00 = 0.49 m = 19.3" use 20" = 0.51 m

25. Design elevation of CLD top above weir = 18.66 + 0.50 = 19.16 m

    below weir = 18.02 + 0.51 = 18.53 m

3.2.10 Summary

The previous two sections list the following for each pipe and outlet design parameters referenced by their listed item numbers:

Q design	(1)
Q maximum	(11)
Valve or orifice size	(3)
Number of flume(s) or weir(s)	(4)
If weir, weir sill elevation = first outlet of	(18) or (12) - 0.30 m
If flume, flume floor elevation	(16)
Elevation of distributer CLD bottom	(16) and (19)
Elevation of distributer CLD top	(25)
Distributer and meska CLD depth	(24)
Number and size of inverted siphons	(5)
Number of 20" gates	(5)
Elevation of upstream end of meska CLD bottom	(8)

The location of the outlets (alfalfa valves and orifices) should be (± 20 m):

Pipe 1 (Please see drawings #s 3EHPSPL1 and 4EHPSPL1BR of the spefications)

Outlet 1 90 m from El Mansouri Canal

Outlet 2 370 - 53 = 317 m from El Mansouri Canal

Outlet 3 Roughly 758 m from El Mansouri Canal

Outlet 4 861 - 35 = 826 m from El Mansouri Canal

Outlet 5 1284 - 53 = 1231 m from El Mansouri Canal

Outlet 6 1363 + 35 = 1398 m from El Mansouri Canal

Outlet 7 1668 m from El Mansouri Canal

Pipe 1 Branch 1

Outlet 8 334 + 35 = 369 m from El Hammami Canal

Outlet 9 500 m from El Hammami Canal

<u>Pipe 2</u> (Please refer to drawings #s, 5EHPS-PL2 and 6EHPS-PL2 of the specifications)

Outlet 1	1970 m from El Hammami Canal
Outlet 2	1828 + 35 = 1863 m from El Hammami Canal
Outlet 3	1290 + 26 = 1326 m from El Hammami Canal
Outlet 4	1167 + 35 = 1202 m from El Hammami Canal
Outlet 5	850 + 35 = 885 m from El Hammami Canal
Outlet 6	661 + 35 = 696 m from El Hammami Canal
Outlet 7	500 m from El Hammami Canal

Table 3.9 gives the design flow rates and the maximum flow rates at the outlets of pipe lines 1 and 2. Table 3.10 gives the details at the outlets.

Table 3.9 DESIGN FLOW RATES AND MAXIMUM FLOW RATES OF OUTLETS

Outlet	(1) Q design	(11) Q max
Pipe 1		
1	2.0 $ft^{3/5}$	5.3 $ft^{3/5}$
2	2.4	4.0
3	2.0	5.9
4	2.7	4.0
5	2.0	3.0
6	2.4	4.0
7	5.3	5.3
8	2.0	3.0
9	2.0	2.7
Pipe 2		
1	2.0	5.9
2	3.0	4.0
3	5.9	5.9
4	2.0	2.0
5	2.0	2.0
6	2.0	2.0
7	2.0	2.0
	$\bar{x}$ = 2.61 cfs	3.88 cfs
	s = 1.21	1.43

Table 3.10 DETAILS OF OUTLETS

	OUTLET		ELEVATION FLUME FLOOR OR WEIR SILL	REL ELEV	FLOW MEASURING DEVICES
PIPE 1	1		18.94	1.94	Weir
	2		19.13	2.13	Flume
	3		18.67	1.67	Weir
	4		18.52	1.52	Flume
	5		18.56	1.56	Flume
	6		18.10	1.10	Flumes x 2
	7	ND	19.08	2.08	Weir
			18.07	1.07	Weir
	8		18.84	1.84	Flume
	9		18.55	1.55	Weir
PIPE 2	1		18.67	1.67	Weir
	2		18.39	1.39	Flume
	3		18.14	1.14	Flumes x 2
	4		18.16	1.16	Flume
	5		18.28	1.28	Flume
	6		18.35	1.35	Flume
	7		18.36	1.36	Weir

CHAPTER 4

SYSTEM HEAD CURVES FOR THE PIPELINE SYSTEMS

4.1 DESIGN ROTATION

According to design considerations 3, 5 and 7 of Section 1.2, the pipeline will actually operate in a manner varying highly in time. Pipe outlets will be opened and closed according to need, not by a pre-set schedule. For pump selection, a design condition of flow rates from each valve was estimated which was thought to be slightly worse (larger TDH for design Q) than average. Thus, a safety factor exists in the design condition.

For Pipe 1 the irrigation rotation resulted in a TDH for design Q that was roughly "worst case." Furthermore, the early part of the day 2 rotation resulted in nearly the "best case." So, the day 2 cases were abandoned for pump selection.

The important conclusion from those early studies deals with pipe sizes. The TDH = f(L) curves of the previous design were compared with this design's TDH = f (L) curves for Day 1 and early Day 2 rotations. Though the previous design never considered a modified demand system, it was tested in a modified demand scheme for the purposes of this design. The results showed that the presently proposed pipe sizes save 1.61 m (4.48 m versus 6.09 m) on the Day 1 rotation, and the previous design saved 0.20 m (2.58 m versus 2.78 m) on the early Day 2 rotation. For this and other reasons (see section 4.2) the proposed pipe sizes of this design were selected.

For the derivation of design TDH curves for pump selection, the design conditions were:

Table 4.1 DESIGN ROTATION SCHEDULES - DAY 1 WATER ISSUES

Pipe 1

Outlet number	Q	
	m^3/s	ft^3/s
5	0.0566	2.0
6	0.0335	1.2
7 N.P	0.1501	5.3
8	0.0566	2.0
TOTAL	0.2968	10.5

Pipe 2

Outlet number	Q	
	m^3/s	ft^3/s
3	0.1671	5.9
4	0.0043	0.2
5	0.0566	2.0
6	0.0566	2.0
TOTAL	0.2846	10.1

Outlet 6 on Pipe 1 and outlet 4 on Pipe 2 were used as "floating" alfalfa valves. Their flow rates were set such that the total flow rates equalled the total design flow rates of Section 2.4 - System Capacity.

This may seem rather arbitrary, but is perfectly reasonable since the design condition itself is arbitrary. There are a host of possible design conditions. Working out all of the many possible design conditions would take much time, and would not prove very useful because the pump is to be selected on a particular design Q and TDH. This design Q and TDH can be found adequately by a well chosen design condition that gives a reasonable TDH at the design Q. The choice of the design condition is somewhat arbitrary, with the single criterion being that a reasonably "average"

TDH will result. This may appear to be sloppy, but because this pipeline will be highly dynamic and because pumps can rarely be selected very close to design conditions, more detailed analysis cannot be justified.

4.2 PIPE SELECTION

As stated above, the proposed pipe sizes of this design saved much TDH on Day 1 rotation compared to the pipe sizes proposed in the previous design. However, this design used a bit more TDH on the early Day 2 rotation. Another advantage of this design's pipe size is that the pipe sizes in the middle and lower reaches of the pipelines are much bigger than previously proposed because the TDH = f(L) changes less when a valve is closed and a neighboring valve of similar desired Q is opened. This means that the pipeline operator will find that the other open valves need less adjusting to continue to deliver the desired Q. If the pipeline operator fails to adjust the other open valves, then the actual Q from each valve will vary less over time in pipe sizes of this design than in the previous design.

The proposed pipe sizes of this design are:

1) For pipeline 1:24" (0.6096 m) from pump stand to outlet 6;20" (0.5080 m) from outlet 6 to the stand at outlet 7 and to the end of the pipe replacing meska 2 (7N.P), and from the stand at outlet 3 to the stand at outlet 9.
2) For pipeline 2:1.0 m pipe from the Mansouria Canal to the pump sump; 24" (0.6096 m) from the pump stand to outlet 4;20" (0.5080 m) from outlet 4 to the stand at outlet 7.

For both pipelines the diameters of the column pipe from the pump to the discharge head, and the pipe from the discharge head to the pump stand are 10" (0.2450 m) for the 25% Q pump, 12" (0.3048 m) for the 50% Q pumps. All of these pipes are made from cast iron, as selected by the

EWUP staff. For both pipelines the flush pipes may be 10" - 14" (alfalfa valve riser pipe size), 15", 18" (inverted siphon pipe sizes) or 20" (main line pipe sizes).

4.3 THE SYSTEM HEAD CURVES

4.3.1 Purpose

Pumps can rarely be selected to give the design flow rate (Q) at design total dynamic head (TDH). Drawing several pump curves on the same figure with the system head curve (TDH = f(Q) for design condition) is a useful technique to study the probable flow resulting from each pump alternative. This study is helpful in pump selection.

4.3.2 Basics

For the purposes of calculation, the following assumptions will be made:

1. Major (pipe) losses
   a. pipe sizes of the previous section.
   b. Hazen-Williams C = 140 (from Section 1.2) for the main line, new, asbestos-cement pipe,

$$h_L = L \left(\frac{3.5909 \quad Q}{C \quad D^{2.63}}\right)^{1.8519}$$

and

C = 130 (Morris and Wiggert, 1971, page 74) for the new, cast-iron pressure pipe from discharge head to pump stand, as selected by EWUP staff (the hydraulics of the cast-iron pressure pipe is being included in the pump's performance curves).

   c.

$$h_{L_{10''}} = 6.8143 \quad Q^{1.8357}$$

$$(r^2 = 1.0000, \text{ for } 0.0631 \text{ m } 3/s \leq Q \leq 0.1262 \text{ m/s}$$

for 10 feet of 10 inch column pipe with 1 3/16 inch shafting (sizing from EWUP staff), and

$$h_{L_{12''}} = 2.9655 \quad Q^{1.8588}$$

$(r^2 = 1.0000$, for $0.0631\ m^3/s \leq Q \leq 0.1893\ m^3/s$

for 10 feet of 12 inch column pipe with 1 3/16 inch shafting (sizing from EWUP staff; the hydraulics of the column pipe is being included in the pump's performance - see page 222 of Fairbanks Morse Pumps).

d. All new, asbestos-cement pipe head losses are increased by 10% and new, cast-iron pipe head losses are increased by 30% to estimate the average future head loss of roughened pipe (see Morris and Wiggert, 1971, page 74, and the previous design). This implies that only pipe surface friction loss changes with time, and the following minor losses are all from losses that do not change with time.

2. Minor (special structure) losses

$$h_L = K\left(\frac{V^2}{2g}\right) = K\left(\frac{8}{\pi^2 g}\right)\left(\frac{Q^2}{D^4}\right) = a\ Q^2$$

a. $K = 0.850$ and $a = \left(\frac{0.0413}{D^4}\right)$

for the pipe entrance with flush connections (Morris and Wiggert, 1971, page 112).

b. $K = 1.00$ and $a = \left(\frac{0.0827}{D^4}\right)$

for the projecting pipe entrance (Morris and Wiggert, 1971).

c. $K = 1.00$ and $a = \left(\frac{0.0827}{D^4}\right)$

for pipe exit into a 'reservoir' (Morris and Wiggert, 1971).

d. $K = 0.30$ and $a = (\frac{0.0248}{D^4})$

for flow passing through a "T" (Morris and Wiggert, 1971)

e. $K = 0.18$ and $a = (\frac{0.0149}{D^4})$

for the change in pipe diameters from 24" to 20" pipe (Morris and Wiggert, 1971)

f. $K \geq 3.0408$ and $a \geq (\frac{0.2515}{D^4})$

for alfalfa valves, see Section 3.2.2.

g. $h_{L_{10"}} = (6.7405\ s^{2.057}\ m^{-5.171})\ Q^{2.0571}$ $(r^2 = 0.9568)$ for a 10" cast iron discharge head for $0.05047\ m^{3/}s \leq Q \leq 0.1262\ m^{3/}s$, and $h_{L_{12"}} = (2.2006\ s^{1.8309}\ m^{-4.493})\ Q^{1.8309}$ $(r^2 = 0.9176)$ for a 10" cast iron discharge head for $0.07886\ m\ 3/s \leq Q \leq 0.1893\ m^{3/}s$, (see page 224 of Fairbanks Morse Pumps, as selected by EWUP staff). The hydraulics of the discharge head are being included in the pump's performance curves.

h. $h_{L_{24"}} = (0.152851\ s^{1.997}\ m^{4.9925})\ Q^{1.99749}$ $(r^2 = 0.999988$, for $0.5757\ m\ 3/s \leq Q \leq 1.2883\ m\ 2/s)$ for 24" (nominal line size) ratio and 17.550 inch throat diameter at 60°F water temperature, as selected by EWUP staff. The Q^s of the above range of Q^s, from which the equation was fitted, are much larger than the Q^s that are expected to run in the venturi meter in the application. Due to a lack of information, the above equation will be extrapolated without modification, into the range of Q^s that needs to be covered.

3. Elevations of

   a. water surface in pump sump of pipeline 1 equals the water surface of the El Mansouri Canal, equals 17.00 m (which is the low water surface elevation probably 8 days out of 12 days),

   b. outlets-elevations are listed in section 3.2.10, and

   c. water surface at inlet to 1 m pipe of pipeline 2 equals the water surface of the El Mansouri Canal, equals 17.00 m, which is the low water surface.

4. Operating head for

   a. flume is $h = (0.79678\ s^{0.437}\ m^{-0.311})\ Q^{0.436887}$ (see section 3.2.3), and

   b. weir is $h = (0.4973\ Q)^{2/3} m$ (see section 3.2.4)

There are three steps needed to determine the system head curves. The first step is to calculate the dynamic head for the design rotation, starting at the downstream end of the pipe. Progressing upstream the needed $K = h_L \times 2\ g/V^2$ ($\geq$ K min) for each alfalfa valve of the design rotation are found. For the second step the equation for each pipe branch, segment and section are written and the equation is plotted on graph paper. For the third step curves are graphically added (vertically for branches in series, horizontally for branches in parallel) to find the TDH = f(Q) that the pumps work against, as well as H = f(Q) at various critical points of the system.

All elevations and total heads (H) in the following analysis will be relative to the low water surface elevation of the El Mansouri Canal, which is 17.00 m above sea level.

4.3.3 Calculations for Pipe 1 System Total Dynamic Head at the Given Delivery

The outlet furthest downstream from pipe 1 is outlet 7. The design rotation calls for flow to the north into the pipe that replaces meska 2. The water flows over a weir set at 19.08 m, or 2.08 m relative elevation. At rotation Q = 0.1501 m 3/s, the operating head = 0.18 m (section 3.2.4). The exit loss from the 20" pipe into the gate stand,

$h_L = a\ Q^2\ (\frac{0.0827}{D^4})\ Q^2 = (\frac{0.0827}{0.5080^4})\ 0.1501^2 = 0.03$ m (see section 3.2.10)

the head loss of 20" (0.5080) pipe (outlet 6 to 7) is:

$$h_L = 1.1 * {}^{270\ m}(\frac{3.5909 * 0.1501}{140 * (0.5080)}\ \ 2.63)^{1.8519}$$

$$= 1.1 * 270\ m\ 0.0009143$$

$$= 0.267\ m$$

The head loss in the change of pipe diameters is:

$$h_L = (\frac{0.0149}{0.5080m^4})\ (0.1501) = 0.01\ m.$$

The head loss in the pass through "T" is approximately:

$$h_L = (\frac{0.0248}{D^4})\ Q^2$$

$$= (\frac{0.0248}{(0.6096)^4})\ (0.1501)^2$$

$$= 0.0024\ m$$

$$= 0.00\ m$$

The computation above results in a design rotation head slightly upstream of outlet 6 of:

$$H_6 = 2.08 + 0.18 + 0.03 + 0.27 + 0.01 + 0.00 = 2.57\ m$$

Part of this head is available for head loss in the 10" alfalfa valve of outlet 6. The water elevation over the valve is the elevation of the flume floor, plus the flume operating head, plus the backwater (M-2 curve), plus the drop of concrete-lined ditch bottom. This is:

$$H = 1.10\text{m (section 3.2.10)} + 0.79678 \quad (0.0335)^{0.43688} \text{ (section 4.3.2)}$$
$$+ 0.01\text{m (section 3.2.3)} + 0.00 \text{ m} = 1.29 \text{ m.}$$

Available head loss through the alfalfa valve is 2.57 - 1.29 = 1.28.

So the needed "a" is:

$$a = \frac{h_L}{Q^2} = \frac{1.28\text{m}}{(0.0335)^2} = 1140 \text{ s}^2/\text{m}^5, \text{ compared to a minimum} = 60.42 \text{ s}^2/\text{m}^5.$$

(see section 3.2.2). Under design flow the needed

$$a = \frac{2.57 - (1.10 + 0.25 + 0.01)}{(0.0680)^2} = 262.$$

Under maximum flow the needed

$$a = \frac{2.57 - (1.10 + 0.31 + 0.01)}{(0.0680)^2} = 89.59.$$

Returning to the main pipeline, the head loss in (1398m - 1231m =) 167 m of 24" (0.6096 m) pipe (outlet 5 to 6) is:

$$h_L = 1.1 * 167 \text{ m} \left[\frac{3.5909 * (0.1501 + 0.0335)}{170 * (0.6096)^{2.63}} \right]^{1.8519}$$

$$= 1.1 * 167 \text{ m} * 0.000576349$$

$$= 0.10 \text{ m.}$$

The head lost in the pass through "T" is approximately

$$h_L = \left(\frac{0.0248}{D^4}\right) Q^2$$

$$= \left(\frac{0.0248}{0.6096^4}\right) (0.1836)^2$$

$$= 0.0061 \text{ m}$$

$$= 0.01 \text{ m}$$

The computation above results in a design rotation head slightly upstream of outlet 5 of:

$$H_5 = 2.57 + 0.10 + 0.01$$

$$= 2.68 \text{ m}$$

Part of this head is available for head loss in the 10" alfalfa valve of outlet 5. The water surface over the valve is:

$$H = 1.56 + 0.23 + 0.01 + 0.00 = 1.80 \text{ m}$$

Available head loss through the valve is:

$$h_L = 2.68 - 1.80 = 0.88 \text{ m.}$$

So, the needed "a" is:

$$a = \frac{h_L}{Q^2}$$

$$= \frac{0.88 \text{ m}}{(0.0566)^2}$$

$$= 275.$$

Returning to the main pipeline, the head lost in (1231 - 758 m =) 473 m of 24" (0.6096m) pipe from the gate stand of outlet 3 to outlet 5 is:

$$h_L = 1.1 * 473 \text{ m} \left[\frac{3.5909 \ (0.1836 + 0.0566)}{140 \qquad (0.6096)^{2.63}} \right]^{1.8519}$$

$$= 1.1 * 473 * 0.0008986$$

$$= 0.467 \text{ m}$$

The head lost in the pass through "T" of outlet 4 is:

$$h_L = \left(\frac{0.0248}{D^4}\right) \quad Q^2$$

$$h_L = \left(\frac{0.0248}{0.6096^4}\right) \quad (0.2402)^2$$

$$= 0.01 \text{ m}$$

The head lost in the flush entrance to the pipe from the stand is:

$$h_L = \left(\frac{0.0413}{D^4}\right) \quad Q^2$$

$$= \left(\frac{0.0413}{0.6096^4}\right) \quad (0.2402)^2$$

$$= 0.02 \text{ m}$$

The computation above results in a design rotation head in the stand of outlet 3 of:

$$H_3 = 2.68 \text{ m} + 0.47 + 0.01 + 0.02$$

$$= 3.18 \text{ m}$$

Part of this head is available for head loss in the 10" valve of outlet 8. The head below the valve is the head in the stand, minus the flush entrance loss, minus the pipe head loss. The flush entrance head loss is:

$$h_L = \left(\frac{0.0413}{D^4}\right) \quad Q^2$$

$$= \frac{0.0413}{(0.5080)^4} \quad (0.0566)^2$$

$$= 0.002 \text{ m}$$

$$= 0.00$$

The head lost in 369 m of 20" (0.5080 m) from the gate stand to outlet 8 is:

$$h_L = 1.1 * 369 \text{ m} \left[\frac{3.5909 * 0.0566}{140 * (0.5080)^{2.63}}\right]^{1.8519}$$

$$= 1.1 * 369 \text{ m} + 0.0001502$$

$$= 0.06 \text{ m}$$

The computation above results in a head below the valve of:

$$H_8 = 3.18 - 0.00 - 0.06$$

$$= 3.102 \text{ m}$$

The water surface over the valve is at:

$$H_8 = 1.84 + 0.23 + 0.01 + 0.00$$

$$= 2.08 \text{ m}$$

The available head loss in the valve is:

$$h_{L,8} = 3.12 - 2.08 = 1.04 \text{ m}$$

So, the needed "a" is:

$$a = \frac{1.04 \text{ m}}{(0.0566)^2}$$

$$= 325.$$

Returning to the main pipeline, the head lost in (758 m - 12 m =) 746 m of 24" (0.6096 m) of pipe from the gate stand to the pump stand is:

$$h_L = 1.1 * 746\ m \left[\frac{3.5909\ (0.2968)}{140\ \ (0.6096)^{2.63}}\right]^{1.8519}$$

$$= 1.1 * 746 * 0.0013297$$

$$= 1.09\ m$$

The head lost in the exit into the gate stand of outlet 3 is:

$$h_L = \left(\frac{0.0847}{D^4}\right) \quad Q^2$$

$$h_L = \frac{0.0847}{(0.6096\ m)^4} \quad (0.2968)^2$$

$$= 0.05\ m.$$

The head lost in the pass through "T" of outlets 1 and 2 is:

$$h_L = 2 \times \frac{0.0248}{D^4} \quad Q^2$$

$$= 2 \times \frac{0.0248}{(0.6096)^4} \quad (0.2968)^2$$

$$= 0.03\ m$$

The head lost in the flush entrance to the pipe from the pump stand is:

$$h_L = \frac{0.0413}{D^4} \quad Q^2$$

$$= \frac{0.0413}{(0.6096)^4} \ (0.2968)^2$$

$$= 0.03\ m$$

The head lost in the venturi meter is:

$$h_L = 0.152851\ Q^{1.997497}$$

$$= 0.01\ m$$

So, the head in the pump stand is:

$$H_o = 3.18 + 1.09 + 0.05 + 0.03 + 0.03 + 0.01$$

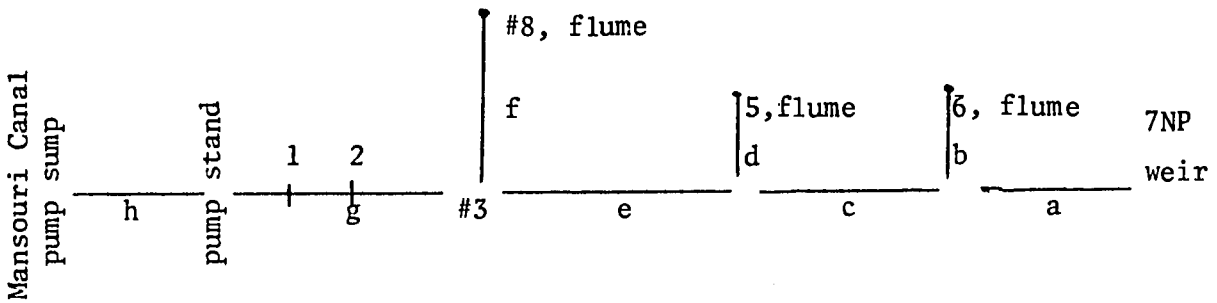
$$= 4.39\ m.$$

The last section of piping, from the pump sump to the pump stand is included by EWUP staff in the pumps' performance curves (H = f[Q]).

GENERAL TOTAL DYNAMIC HEAD VARIATION

PIPE 1

As stated in section 4.3.2, the next step is to write the equation for each pipe branch.



Branch a: (see figure)

$$H = 2.08 + 0.6277\ Q^{2/3} +$$

weir setting + weir operating head

$$+ 1.2418\ Q^2 + 9.1015\ Q^{1.8519}$$

+ exit loss + pipe friction

$$+ 0.2237\ Q^2 + 0.1796\ Q^2$$

+ change in pipe diameters + pass through "T"

$$H_a = 1.6451\ Q^2 + 9.1015\ Q^{1.8519}$$

$$+ 0.6277\ Q^{2/3} + 2.08$$

Branch b:

$$H = 1.10 + 0.79678\ Q^{0.436887}$$

+ flume setting + flume operating head

+ 0.01 + 0.00

+ M-2 backwater (roughly) + 25 m * 0.00015 drop in CLD

$$+ 1140\ Q^2$$

+ alfalfa valve head loss.

$$H_b = 1140\ Q^2 + 0.79678\ Q^{0.436887} + 1.11$$

Branch c:

$$H = 2.3164\ Q^{1.8519} + 0.1796\ Q^2$$

+pipe friction + pass through "T"

$$H_c = 0.1796\ Q^2 + 2.3164\ Q^{1.8519}$$

Branch d:

$$H = 1.56 + 0.79678\ Q^{0.436887}$$

+ flume setting + flume operating head

+ 0.01 + 0.00

+ M-2 backwater (roughly) + drop in CLD

+ $275\ Q^2$

+ alfalfa valve head loss

$$H_d = 275\ Q^2 + 0.79678\ Q^{0.436887} + 1.57$$

Branch e:

$$H = 6.5608\ Q^{1.8519} + 0.1796\ Q^2$$

pipe friction + pass through "T"

+ $0.2991\ Q^2$

+ flush entrance

$$H_e = 0.4787\ Q^2 + 6.5608\ Q^{1.8519}$$

Branch f:

$$H = 1.84 + 0.79678\ Q^{0.436887} + 0.01$$

+ flume setting + flume operating head + M-2 backwater

+ $0.00 + 325\ Q^2$

+ drop in CLD + alfalfa valve head loss

$$+ 12.4387\ Q^{1.8519} + 0.6201\ Q^2$$

$$+ \text{pipe friction} + \text{entrance losses}$$

$$H_f = 325.62\ Q^2 + 12.4387\ Q^{1.8519}$$

$$+ 0.79678\ Q^{0.436887} + 1.85$$

Branch g:

$$H = 10.3475\ Q^{1.8519} + 0.6133\ Q^2$$

$$+ \text{pipe friction} + \text{exit loss into stand of outlet 3}$$

$$+ 0.3592\ Q^2 + 0.2991\ Q^2$$

$$+ 2 \times \text{pass through "T"} + \text{entrance loss to pipe}$$

$$+ 0.152851\ Q^{1.997447}$$

$$+ \text{venturi meter head loss}$$

$$H_g = 1.2716\ Q^2 + 0.152851\ Q^{1.997497}$$

$$+ 10.3475\ Q^{1.8519}$$

Table 4.2 gives the HP 25 program for the calculation of Hg and Table 4.3 gives the system head variation with the pumped flow rate Q (m^3/s) for pipe line system 1. These curves for each branch are drawn on Figure 4.1, which also shows the intermediary and final system head curves. The tangent to the system head curve at the design point has a slope of 14.8 m of head per (m^3/s) of flow.

Table 4.2 HP 25 PROGRAM FOR SYSTEM HEAD VALUES

$$H = a\,Q^b + c\,Q^d + e\,Q^f + g$$

```
a
STR 0
b
STR 1
c
STR 2
d
STR 3
e
STR 4
f
STR 5
g
STR 6
Q
STR 7
ENT
RCL 1
f - y*
RCL 0
x
RCL 7
RCL 3
f   y*
RCL 2
x
+
RCL 7
RCL 5
f   y*
RCL 4
x
+
```

```
RCL 6
+
read H
```

Table 4.3 SYSTEM HEAD VARIATION FOR PIPE 1

H (m)

Q m^3/s	Branch a	b	c	d	e	f	g
0.0	2.08	1.11	0	1.57		1.85	
0.00001	2.08	1.12	$1x10^{-9}$	1.58		1.86	$6x10^{-9}$
0.005	2.10	1.22	0.0001	1.66		1.94	0.0006
0.01	2.11	1.33	0.00048	1.70	0.001	1.99	0.002
0.02	2.13	1.71	0.002	1.82	0.005	2.13	0.01
0.03	2.16	2.31	0.004	1.99	0.01	2.33	0.02
0.04	2.18	3.13	0.01	2.21	0.02	2.60	0.03
0.05	2.20	4.18	0.01	2.47	0.03	2.93	0.04
0.06	2.23		0.01	2.79	0.04	3.32	0.06
0.07	2.26		0.02	3.17	0.05	3.79	0.08
0.08	2.29		0.02	3.59	0.06	4.31	0.11
0.09	2.32		0.03	4.08	0.08	4.91	0.13
0.10	2.36		0.03	4.61	0.10	5.57	0.16
0.12	2.44		0.05		0.14	7.10	0.22
0.14	2.52		0.06		0.18		0.30
0.16	2.61		0.08		0.23		0.38
0.18	2.71		0.10		0.29		0.48
0.20	2.82		0.12		0.35		0.58
0.25	3.13		0.19		0.53		0.88
0.30	3.49		0.27		0.75		1.24
0.35	3.90		0.35		1.00		1.66
0.40	4.35				1.28		2.12
0.45	4.86				1.59		2.65
0.50	5.41				1.94		3.22
R0	1.6451	1140	0.1796	2.75	0.4787	325.62	1.2716
1	2.0	2.0	2.0	2.0	2.0	2.0	2.0
2	9.1015	0.79678	2.3164	0.79678	6.5608	12.4387	0.152851
3	1.8519	0.436887	1.8519	0.436887	1.8519	1.8519	1.997497
4	0.6277	0.0	0.0	0.0	0.0	0.79678	10.3475
5	2/3	1.0	1.0	1.0	1.0	0.436887	1.8519
6	2.08	1.11	0.0	1.57	0.0	1.85	0.0

4.3.4 Calculations for the Pipe 2 System Total Dynamic Head at the Given Delivery

The outlet furthest downstream from pipe 2 is outlet 7. The design rotation defines outlet 6 as the furthest downstream flowing outlet. Outlet 7 is ignored in the design rotation because it will flow for a very small fractional time (0.0133/0.0566 x 0.50 = 0.117, see section 2.3). It can be scheduled to flow when outlets 5 and 6 are closed so that there can be adequate head at outlet 7 without raising the TDH at the pump much above the design TDH.

The water flowing from outlet 6 flows over a flume set at 18.35 m above sea level, or 1.35 m above the datum of this analysis, which is the low water surface elevation of the El Mansouri Canal (17.00 m). At the rotation Q = 0.0566 m^3/s, the operating head is 0.23 m. The backwater (M-2 curve) is 0.01 m. The drop in the ditch bottom is 2.5 m * 0.00015 = 0.00 m. So the water elevation over the alfalfa valve is: 1.35 + 0.23 + 0.01 + 0.00 = 1.59 m. The head lost in the 10" alfalfa value is 0.19 m. So the head in the main pipeline below outlet 6 is 1.59 + 0.19 = 1.78 m. The head lost in (885 - 696 m =) 189 m (see section 3.2.10) of 20" (0.5080 m) pipe (outlet 5 to 6) is:

$$h_L = 1.1 * 189\ m \left(\frac{3.5909 * (0.0566\ m\ 3/s)}{140 * (0.05080)^{2.63}}\right)^{1.8519}$$

$$= 1.1 * 189\ m * 0.0001502$$

$$= 0.03\ m$$

The head lost in the pass through "T" is

$$h_L = \left(\frac{0.0248}{D^4}\right) Q^2$$

$$= \frac{0.0248}{(0.5080m)^4} (0.0566)^2$$

$$= 0.00$$

The above results in a design rotation head slightly upstream of outlet 5 of:

$$H = 1.78 + 0.03 + 0.00$$

$$H_5 = 1.81 \text{ m}$$

Part of this head is available for head loss in the 10" alfalfa valve of outlet 5. The water elevation over the valve is the elevation of the flume floor, plus the flume operating head, plus the backwater (M - 2 curve), plus the drop of the concrete-lined ditch bottom. This is:

$$H = 1.28 \text{ m} + 0.79678\ (0.0566)^{0.436887} + 0.01 + 0.00$$

$$H = 1.52 \text{ m}$$

Available head loss through the alfalfa valve is: 1.81 m - 1.52 m = 0.29 m

So the needed "a" is:

$$a = \frac{h_L}{Q^2} = \frac{0.29\text{m}}{(0.0566)^2} = 90.52$$

Returning to the main pipeline, the head lost in (1202 - 885 = m) 317 m of 20" pipe (0.5080 m) pipe (outlet 4 to 5) is:

$$h_L = 1.1 * 317\ \left(\frac{3.5909 * (0.0566 + 0.0566)}{140 \quad (0.5080)^{2.63}}\right)^{1.8519}$$

$$= 1.1 * 317 * 0.000\ 5422$$

$$= 0.19 \text{ m}$$

The head lost in change of pipe diameter is:

$$h_L = \frac{0.0149}{(0.5080\text{m})^4}\ (0.1132)^2 = 0.00$$

The head lost in the pass through "T" is:

$$h_L = \left(\frac{0.0248}{D^4}\right) \quad Q^2$$

$$h_L = \frac{0.0248}{(0.5080)^4} (0.1132)^2$$

$$= 0.00$$

The computation above results in a design rotation head slightly upstream of outlet 4 of:

$$H = 1.81 + 0.19 + 0.00 + 0.00$$

$$= 2.00 \text{ m}$$

Part of this head is available for head loss in the 10" alfalfa valve of outlet 4. The water surface over the valve is:

$$H = 1.16 + 0.79678\ Q^{0.436887} + 0.01 + 0.00$$

$$= 1.24 \text{ m}$$

Available head loss through the valve is:

$$h_L = 2.00 - 1.24 = 0.76 \text{ m}$$

So, the needed "a" is

$$a = \frac{h_L}{Q^2} = \frac{0.76}{(0.0043)^2} = 41{,}103$$

Under design flow the needed

$$a = \frac{[2.00 - (1.16 + 0.23 \quad 0.01 + 0.00)]}{(0.0566)^2}$$

$$= 187.$$

Under maximum flow the needed

$$a = \frac{2.00 - (1.10 + 0.27 + 0.01 + 0.00)}{(0.0850)^2}$$

$$= 77.51.$$

Returning to the main pipeline, the head lost in (1326 - 1202=) 124 m of 24" (0.6096 m) pipe (outlet 3 to 4) is:

$$h_L = 1.1 * 124 \left(\frac{3.5909 * (0.1132 + 0.0043)}{140 \quad (0.6096)^{2.63}}\right)^{1.8519}$$

$$= 1.1 + 124 * 0.000\ 23906$$

$$= 0.03 \text{ m}$$

The head lost in the pass through "T" of outlet 3 is:

$$h_L = \frac{0.0248}{D^4} \quad Q^2$$

$$= \frac{0.0248}{(0.6096)^4} \quad (0.1175)^2$$

$$= 600$$

The computation above results in a design rotation head slightly upstream of outlet 3 of:

$$H_3 = 2.00 + 0.03 + 0.00$$

$$= 2.03 \text{ m}$$

Part of this head is available for head loss in the 14" alfalfa valve of outlet 3. The water surface over the valve is:

$$H = 1.14 + 0.79678\ Q^{0.436887} + 0.01 + 0.00$$

$$= 1.14 + 0.36 + 0.01$$

$$= 1.51 \text{ m}$$

Available head loss through the valve is

$$h_L = 2.03 - 1.51 = 0.52 \text{ m}$$

So the needed "a" is

$$a = \frac{0.52}{(0.1671)^2} = 18.62.$$

Returning to the main pipeline, the head lost in (1970-1326 m =) 644 m of 24" (0.6096 m) pipe (outlet 1 to 3) is:

$$h_L = 1.1 * 644 \left(\frac{3.5909 \ (0.2846)}{140 \ \ (0.6096)^{2.63}}\right)^{1.8519}$$

$$..1 * 644 * 0.0012303$$

$$= 0.87 \text{ m}$$

The head lost in the pass through "T" of outlet 2 is:

$$h_L = \frac{0.0248}{(0.6096)^4} \quad (0.2846)^2$$

$$= 0.01 \text{ m}$$

The head lost in the flush entrance to the pipe from the corner stand is:

$$h_L = \frac{0.0413}{(0.6096)^4} \quad (0.2846)^2$$

$$= 0.02 \text{ m}$$

The computation above results in a design rotation head in the corner stand of:

$$H_1 = 2.03 + 0.87 + 0.01 + 0.02$$

$$= 2.93 \text{ m}$$

The head lost in 900 m of 24" (0.6096 m) pipe from the pump stand to the corner stand of outlet 1 is:

$$h_L = 1.1 * 900 \text{ m} * \left(\frac{3.5909 * (0.2846)}{140 * (0.6096)^{2.63}}\right)^{1.8519}$$

$$= 1.1 * 900 * 0.00\ 12303$$

$$= 1.22\text{m}$$

The head lost in the exit from the pipe to corner stand is:

$$h_L = \frac{0.0827}{(0.6096)^4} (0.2846)^2$$

$$= 0.05\,\text{m}$$

The head lost in the entrance to the pipe from the pump stand is:

$$h_L = \frac{0.0413}{(0.6096)^4} (0.2846)^2$$

$$= 0.02\,\text{m}$$

The head lost in the venturi meter is:

$$h_L = 0.152851\ Q^{1.997497}$$

$$h_L = 0.01.\ \text{m}$$

The computation above results in a design rotation head in the pump stand of:

$$H_o = 2.93 + 1.22 + 0.05 + 0.02 + 0.01$$

$$= 4.23 \text{ m } (+17.00 = 21.23)$$

The head lost in the section of piping between the pump sump and pump stand is included in each pump's performance curve.

The head lost in roughly 20 m of 1.0 m diameter pipe from the Mansouri Canal to the pump sump is:

$$h_L = 1.1 * 20 \text{ m} * \left(\frac{3.5909 * (0.2846)}{140 * (1.0)^{2.63}}\right)^{1.8519}$$

$$= 1.1 * 20 * 0.0001104$$

$$= 0.00 \text{ m}$$

The head lost in the exit from the pipe into the pump sump is:

$$h_L = \frac{0.0827}{(1.0)^4} (0.2846)^2$$

$$= 0.01 \text{ m}$$

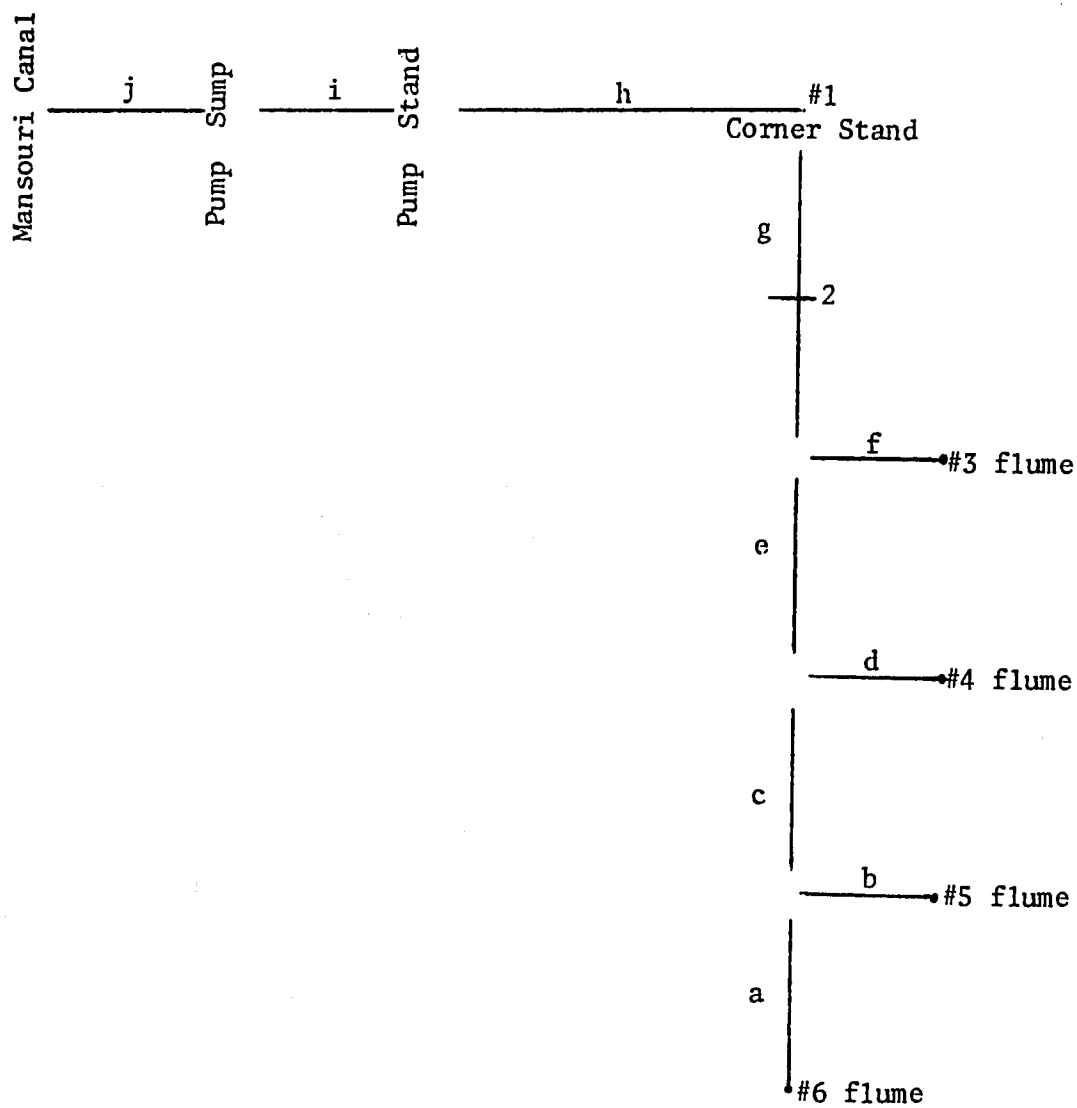
The head lost in the projecting entrance to the pipe from the Mansouri Canal is:

$$h_L = \frac{0.0827}{(1.0)^4} (0.2846)^2$$

$$= 0.01 \text{ m}$$

As stated in section 4.3.2, the next step is to write the equation for each pipe branch.

GENERAL TOTAL DYNAMIC HEAD VARIATION



PIPE 2

Branch a:

$$H = 1.35 + 0.79678\ Q^{0.436887}$$

$$+ \text{ flume setting + flume operating head}$$

$$+ 0.01 + 60.423\ Q^2$$

$$+ \text{ backwater + alfalfa valve head loss}$$

$$+ 6.3710\ Q^{1.8519} + 0.3724\ Q^2$$

$$+ \text{ pipe friction + pass through "T"}$$

$$H_a = 60.7954\ Q^2 + 6.3710\ Q^{1.8517}$$

$$+ 0.79678\ Q^{0.436887} + 1.36$$

Branch b:

$$H = 1.28 + 0.79678\ Q^{0.436887}$$

$$+ \text{ flume setting + flume operating head}$$

$$+ 0.01 + 90.52\ Q^2$$

$$+ \text{ M-2 backwater + alfalfa valve head loss}$$

$$H_b = 90.52\ Q^2 + 0.79678\ Q^{0.436887} + 1.29$$

Branch c:

$$H = 10.6858\ Q^{1.8519} + 0.2237\ Q^2$$

$$+ \text{ pipe friction + change in pipe diameters}$$

$$+ 0.3724\ Q^2$$

$$+ \text{ pass through "T"}$$

$$H_c = 0.5961\ Q^2 + 10.686\ Q^{1.8519}$$

Branch d

$$H = 1.16 + 0.79678\ Q^{0.436887}$$

+ flume setting + flume operating head

+ 0.01 + 41.103 Q^2

+ M-2 backwater + alfalfa value head loss

$$H_d = 41.103\ Q^2 + 0.79678\ Q^{0.436887} + 1.17$$

Branch e:

$$H = 1.7200\ Q^{1.8519} + 0.1796\ Q^2$$

+ pipe friction + pass through "T"

$$H_e = 0.1796\ Q^2 + 1.7200\ Q^{1.8519}$$

Branch f:

$$H = 1.14 + 0.79678\ Q^{0.436887} + 0.01$$

+ flume setting + flume operating head + M-2 backwater

+ 18.62 Q^2

+ alfalfa value head loss

$$H_f = 18.62\ Q^2 + 0.79678\ Q^{0.436887} + 1.15$$

Branch g:

$$H = 8.9327\ Q^{1.8519} + 0.1796\ Q^2$$

+ pipe friction + pass through "T"

+ 0.2991 Q^2

+ flush entrance

$$H_g = 0.4787\ Q^2 + 8.9327\ Q^{1.8519}$$

Branch f:

$$H = 12.6223\ Q^{1.8519} + 0.5989\ Q^2$$

+ pipe friction + exit losses

+ 0.2991 Q^2

$$+ \text{ entrance losses}$$

$$H_h = 0.8980\ Q^2 + 12.6223\ Q^{1.8519}$$

Branch i: included in pump's performance curves.

Branch j:

$$H = 0.02490\ Q^{1,8519} + 0.0827\ Q^2$$

$$+ \text{ pipe friction} + \text{inlet losses}$$

$$+ 0.0827\ Q^2$$

$$+ \text{ exit losses}$$

$$H_j = 0.1654\ Q^2 + 0.02490\ Q^{1.8519}$$

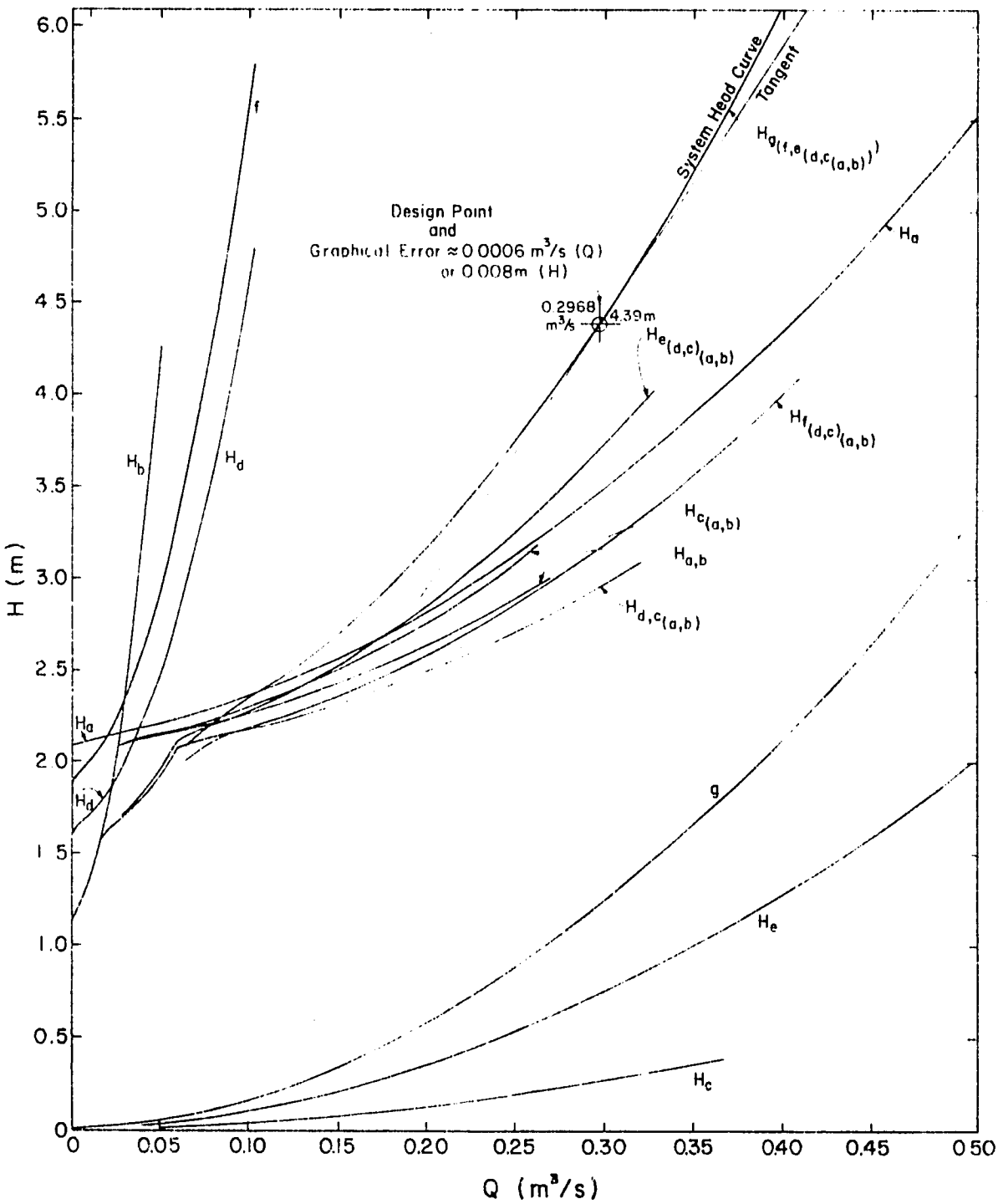
The curves for each branch are drawn on the accompanying large figure which also shows the intermediary and final system head curves. The tangent to the system head curve at the design point has a slope of 18.25 m of head per m^3/s of flow.

Table 4.4 gives the system head variation for pipe line system 2. Figure 4.2 gives the system head curves for system 2.

Table 4.4 SYSTEM HEAD VARIATION FOR PIPE 2

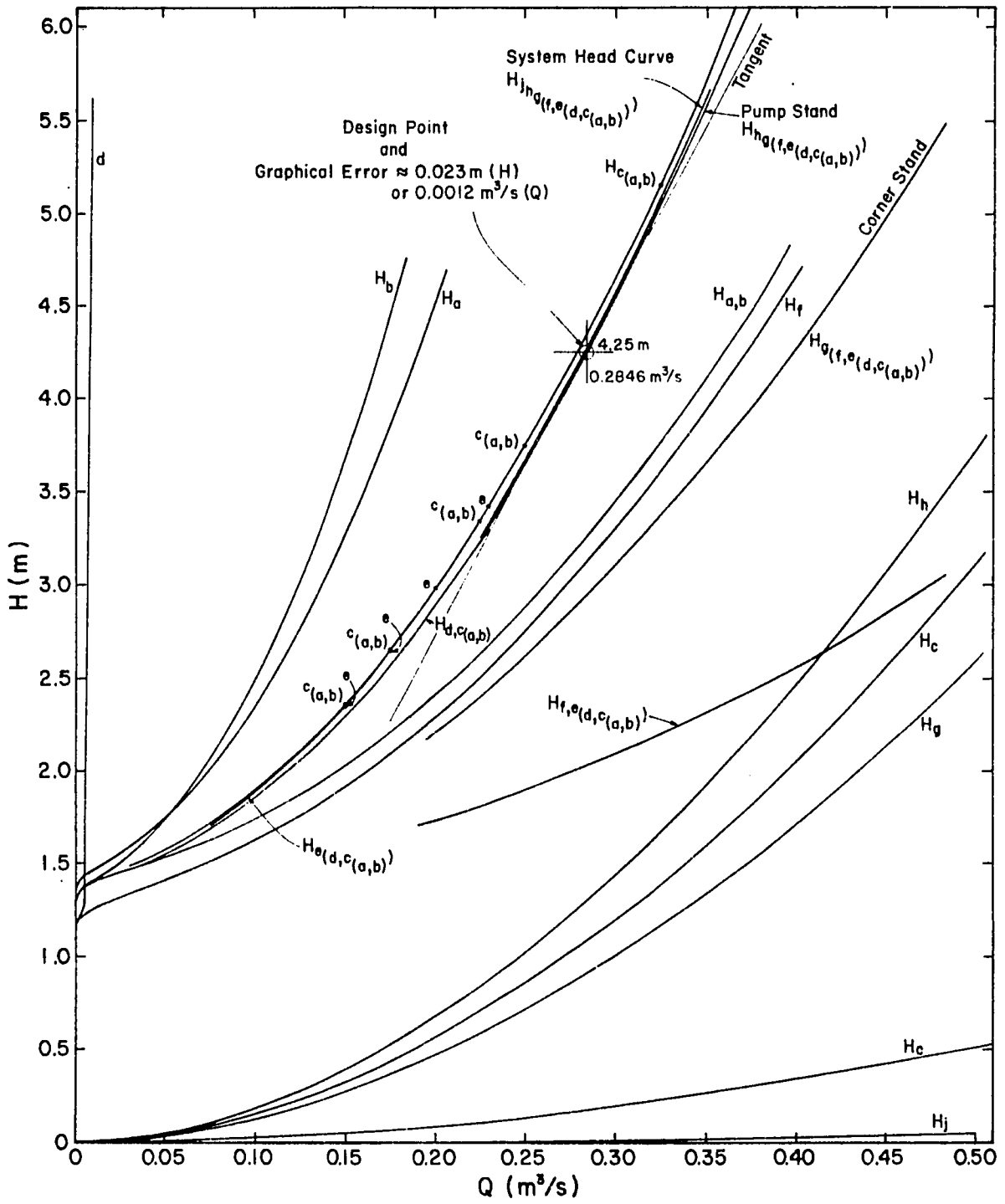
H (m)

Q m^3/s	Branch								
	a	b	c	d	e	f	g	h	j
	1.36	1.29		1.17		1.15			
0.00001	1.37	1.30	$6x10^{-9}$	1.18		1.16			
0.005	1.44	1.37	0.0006	2.28		1.23			
0.01	1.47	1.41	0.002	5.39	0.0004	1.26	0.002	0.003	
0.02	1.53	1.47	0.01	17.76	0.001	1.30	0.01	0.01	
0.03	1.60	1.54	0.02		0.003	1.34	0.01	0.02	
0.04	1.67	1.63	0.03		0.005	1.38	0.02	0.03	
0.05	1.75	1.73	0.04		0.01	1.41	0.04	0.05	
0.06	1.85	1.85	0.06		0.01	1.45	0.05	0.07	
0.07	1.95	1.98	0.08		0.01	1.49	0.07	0.10	
0.08	2.07	2.13	0.10		0.02	1.53	0.09	0.12	
0.09	2.20	2.30	0.13		0.02	1.58	0.11	0.15	0.002
0.10	2.35	2.49	0.16		0.03	1.63	0.13	0.19	0.002
0.12	2.68	2.91	0.22		0.04	1.73	0.18	0.26	0.003
0.14	3.06	3.40	0.29		0.05	1.85	0.24	0.35	0.004
0.16	3.49	3.97	0.37		0.06	1.98	0.31	0.45	0.005
0.18	3.97	4.60	0.47		0.08	2.13	0.39	0.56	0.006
0.20	4.51		0.57		0.09	2.29	0.47	0.68	0.008
0.25			0.86		0.14	2.75	0.72	1.02	0.012
0.30			1.20		0.20	3.30	1.00	1.44	0.018
0.35			1.60		0.27	3.93	1.34	1.92	0.024
0.40			2.05		0.34	4.66	1.71	2.46	0.03
0.45			2.56		0.43		2.13	3.06	0.04
0.50			3.11		0.52		2.59	3.72	0.05
RO	60.7954	90.52	0.5961	41103	0.1796	18.62	0.4787	0.898	0.1654
1	2.0								
2	6.3710	0.0	10.686	0.0	1.720	0.0	8.9327	12.6233	0.02490
3	1.8519								
4	0.79678		0.0	0.79678	0.0	0.79678	0.0		
5	0.436887								
6	1.36	1.29	0.0	1.17	0.0	1.15	0.0		



System Head Curve for Pipeline I

FIGURE 4.1



System Head Curve for Pipeline 2

FIGURE 4.2