IRRIGATION DEVELOPMENT

A PRELIMINARY DESIGN AND ECONOMIC ANALYSIS

A Thesis

Presented to

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Master of Science in Civil Engineering

By

Roderick C. Bower

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INTRODUCTION

The purpose of this thesis is to present a potential irrigation development project in such a way that the preliminary design and economic analysis of the development embodies sufficient interesting alternatives to be of value as a tool for academic instruction in the field of Water Resources.

A model irrigation project with suitable topographic and hydrologic parameters has been established to satisfy the foregoing requirements. Ample data has been presented to enable the student to analyze the Hydraulic and Civil Engineering aspects of the problem and to carry out an analysis of the benefits and costs of the project.

It is expected that the work to be carried out by the student will require sufficient in-depth calculations and study to introduce the student to the methods and scope of work that must be undertaken by the practising engineer for a major irrigation project.

The material presented has been summarized and is only intended for use in a preliminary design study. The data presented in the chapter entitled "Available Information" are typical of the sort of information which the engineer would be required to assemble from earlier reports and Government Agencies and present in the body of his report. A brief discussion of computational formulae for consumptive use calculations has also been included.

The problem statement has been presented as a Letter of Intent from the Governor of the Province of Assiniboia to a consulting firm. The model has been assembled from Canadian and overseas resource studies and reports.

The main dam site on the Rio Saska is similar to the Tobin Rapids hydro development site on the South Saskatchewan River. River flows are recorded flows from the South Saskatchewan at Saskatoon. River flows on the Rio Seco are directly proportional to those for the Rio Saska but greatly reduced. Rainfall records are also proportional to river flows with a total annual rainfall equivalent to that experienced in the southern United States. Rainstorm records were obtained from mass curves of rainfall for Bar Harbour as reproduced by Bruce and Clarke (1966).

The rain gauging stations have been distributed so that a Thiessen polygon network can be constructed and rainfall values for the stations have been set so that when a frequency curve is constructed the 1-in-10 dry year and average year rainfall occur within the period of record.

In order to ensure that irrigation water demands would be large enough to be of significance in the multi-purpose dam analysis, which forms an integral part of this group thesis project, climatological data were obtained from a standard atlas for latitude 26° in North America, high consumptive use crops were chosen and the size of the development area was set at 300,000 acres.

The topography of the canal route and irrigation district is based on the Morden-Winkler area of Manitoba. Subsoil conditions along the canal are such that an unlined canal suffers high seepage losses. The rather tortuous stream crossing along the canal route was introduced so that alternatives to a gravity canal such as aqueducts, syphons, tunnels and open cut excavation would have to be investigated.

The Rio Seco dam site topography is based on the Treherne dam site on the Boyne River in Manitoba. The river valley width and potential storage volume has been increased considerably. Foundation conditions were selected so that a zoned earth fill dam would be satisfactory at this location.

LETTER OF INTENT

Saska Consult,
University of Manitoba,
Winnipeg, Manitoba.

Dear Sirs:

In accordance with the powers vested in me, as Governor of the Province of Assiniboia, I hereby inform you of the intention of the Province of Assiniboia, to engage the services of Saska-Consult for the purpose of an Engineering Feasibility Study of the irrigation of the Rio Seco Basin. As discussed with your representatives, it will be your responsibility to conduct an Engineering feasibility study and economic analysis for the irrigation development of 300,000 acres of agricultural land in the Rio Seco Basin. These studies shall include the following general topics as detailed in the Scope of Work of the Engineering Agreement of July 31, 1970.

1. Determine total diversion requirements and most economical source and mode of conveyance of water for the total development of the area. This analysis should include, but not be limited to, a storage dam on the Rio Saska, a lift station from the Rio Saska to a gravity canal, a storage dam on the Rio Seco and ground water wells in the project area.

2. Determine the most economical mode of water conveyance within the project area and the necessity for and total cost of surface and subsurface drainage.

3. Submit a feasibility report with preliminary design drawings, quantity and cost calculations, economic analysis of project alternatives and summary and recommendations for implementation of works, within one year from the date of this Letter of Intent. This preliminary report shall be in such form as to enable the Governor of the Province to apply to International Agencies for funds to finance further studies of the project.

Yours very truly,

Governor,
Province of Assiniboia.

PREAMBLE

In response to a request from the Planning and Development Agency of the Province of Assiniboia in June of 1965, a team of United Nations experts conducted a coordinated resources evaluation study of the Province. On the basis of their investigations and subsequent report, it was recommended that the Rio Seco Basin Agricultural Intensification scheme be further studied as a potential irrigation development project.

This particular scheme was singled out for further investigations as the Basin was entirely within Provincial boundaries, possessed a viable agricultural economy including some irrigated farms, enjoyed reasonable proximity to a large dependable source of water and had existing transportation, crop processing and storage facilities. The existence of these pre-requisites for irrigation development would greatly assist in the possible establishment of the area as a pilot project for future schemes and therefore stimulate further agricultural intensification in the Province.

In accordance with recommendations contained in the United Nations' Report on Water Resources Development of the Rio Seco Basin, the Provincial Authorities established an automatic stream gauge recording station upstream of Blaine on the Rio Seco and supplemented the meteorological data from the station at Blaine with the establishment of seven additional rain gauging stations in the basin. Provincial Departments, operating under the direction of United Nations advisors also carried out reconnaissance surveys in the area to determine soil classifications and capabilities, existing and potential land use patterns, crop market values and typical farm budgets.

A detailed survey and preliminary cost analysis of a single-purpose hydro-electric development, with a dam and reservoir situated on the Rio Saska, has been completed under the direction of the Provincial Power Authority. This dam and reservoir site was also recommended for potential multi-purpose development, for single-purpose development as a flood control storage dam or for municipal or irrigation water supply pending further studies. Available data on this storage site includes topographic maps and foundation exploration results, mean monthly discharges, stage-discharge, elevation-storage capacity and dependable flow-storage capacity relationships. Estimates of capital cost versus a range of full supply levels (FSL), based on current prices are also available.

Engineering surveys were conducted to establish topographical maps of potential dam sites and canal routes. Subsurface investigations were conducted at potential dam sites and mineral exploration drill holes were relogged for geological mapping and a few existing on-farm wells were tested for potential ground water yields and ground water quality.

In order to facilitate cost computations and to ensure that project comparisons are based on consistent parameters, the Provincial Planning and Development Corporation has established cost relationships based on recent construction projects in the Province. The Corporation has similarly established data relating to economic analysis, specifically, the period of analysis or depreciation for various items of civil works, annual operation and maintenance costs and the prevailing interest rates at which funds can be obtained for capital projects.

The relevant information is summarized in Chapter I.

CHAPTER I

AVAILABLE INFORMATION

1-1 General

The Rio Seco Basin Irrigation Development shown in the key plan, Figure 1 and in Figure 7, consists of 300,000 acres of land presently under cultivation. The land is capable of moderate yields under existing cropping patterns as the soils are fertile and wet season precipitation provides sufficient dependable moisture for crops such as cotton, corn, dryland grains and market and forage crops.

1-2 Climate

The project area is located at approximately 26° north latitude and the climate has been described as tropical savannah. Heavy rainfall occurs in four to five months of the year as shown in Tables V and VI and potential evapotranspiration exceeds rainfall in six to eight months. Irrigation would be required for most crops during the dry season and with irrigation the growing season could be extended over the full year.

1-3 Streamflows

Continuous flow records and a stage discharge relationship are available for the Rio Saska. Values for the period 1941 to 1970 are shown in Table III. Stage records have been kept on the Rio Seco since 1959 and average monthly discharges are shown in Table IV. The stage discharge relationship was checked during the last four years with an automatic stage recorder and continuous flow metering equipment. There was no significant change in the curve and therefore flows determined from this stage discharge relationship are satisfactory.

1-4 Precipitation

Monthly rainfall records have been maintained at the meteorological stations at Blaine for the period 1959 to 1970 inclusive. Records for the seven additional rain gauging stations shown in Figure 7 are available for the period 1967 to 1970. Precipitation data is shown in Tables V and VI. Storm rainfall records for the maximum and annual 24-hour rain storms are given in Table VII.

1-5 Meteorology Records

The average annual cloud cover is seven-eighths, with 11 hours of daylight per day on an average annual basis.

Average monthly values for relative humidity, air temperature, and pan evaporation from the observation station at Blaine are given in Table I. The figures are based on 12 years of records.

TABLE I

SUMMARY OF METEOROLOGY RECORDS

	Relative Humidity	Air Temperature	Evaporation	Wind Speed
	Percent	Degrees F	In. Water	M.P.H.
January	68	65	4.60	7
February	70	72	5.29	7
March	72	75	5.70	7
April	74	82	6.62	7
May	73	87	8.07	7
June	70	84	8.21	7
July	67	79	7.65	7
August	68	75	6.86	8
September	67	70	5.93	8
October	66	65	4.93	7
November	63	60	4.34	7
December	64	63	4.46	7
Total	-	-	72.66	-
Average	69	73	-	7.1

The pan evaporation data is from a Class A pan and the pan coefficient for converting pan evaporation to equivalent lake evaporation has been experimentally established as 0.7 for Lake Manitou. This lake is located in the same region as the development zone, approximately 100 miles due north.

1-6 Subsurface Explorations

The drill hole information summarized below is the data obtained from the series of drill holes shown in Figure 6 for a potential storage dam site on the Rio Seco.

TABLE II

SUMMARY OF DRILL HOLE INFORMATION
FOR THE RIO SECO DAM SITE

Properties	Units	SC	CL	Rock
Moisture Content	percent	9	22	n/a
Void Ratio		0.35	0.5	n/a
Unit Weight	lb/cu. ft.	115	110	170
Liquid Limit	percent	20	40	n/a
Plasticity Index	percent	10	20	n/a
Unconfined Compression Strength	kips/ sq. ft.	2.0	3.0	2,500
Permeability	cm/sec.	$5x10^{-4}$	$1x10^{-5}$	$1x10^{-10}$
Relative Density	percent	60	80	100
Carbonate Content	percent	0.12	0.10	n/a

General Description

SC - Firm, well graded, clayey, gravelly, sand, low to medium permeability, homogeneous.

CL - Stiff, sandy clay, dense, low compressibility, low permeability, layered.

Rock - Hard, massive, few cracks, no cavities, horizontal layers.

Information from the test pits along the proposed canal route indicate that the material would be stable at 2-to-1 side slopes. The material is similar to the surface material encountered at the Rio Seco dam site with an average in situ permeability of 1.0 feet per day.

1-7 Land Capability

Reconnaissance studies indicate that the land is capable of supporting, with added fertilization, perennial production of selected crop varieties and the soils are very good to fair irrigation soils. With irrigation, 60 percent of the development zone would be utilized for double cropping of rice and 35 percent would be cultivated with cotton from October to April and corn from May to September. The remaining 5 percent of the area would be taken up by roads, canals and farmsteads.

1-8 Land Classifications

The study area has been divided into three subzones as shown in Figure 8. The subzone boundaries coincide roughly with the soil classifications and farm sizes. The subzone numbering indicates the soil classes, Class I having the highest existing dryland yields and potential yields under irrigation. Average farm sizes decrease from 640 acres in subzone I to 160 acres in subzone III as indicated in the Farm Budget in Table VIII.

1-9 Water Availability

Good quality irrigation water is available from the dam and reservoir site on the Rio Saska. Minimum downstream releases for municipal dilution requirements have been fixed at 1,000 cubic feet per second (cfs). Potential use of water for hydro-electric schemes or downstream irrigation have been excluded from the analysis. Maximum groundwater yields from existing wells have been established at 0.5 cfs of medium salinity-medium sodium quality. The quantity and quality would not be satisfactory for extensive irrigation.

1-10 Drainage

In the upper part of the basin, surface drainage is adequate for all but the most severe conditions of saturated ground conditions and intensive rainfall. In the lower reaches of the basin, the flat terrain and high water table, combined with heavy rainfall and high water levels in the river during the wet months of June and July cause periodic flooding. Drainage problems, primarily in subzone III are a major limiting factor in present agricultural production.

1-11 Water Distribution Studies

Irrigation efficiency, or the percentage of water that remains in the root zone and is available for crop growth, has been established at 60 percent of farm-delivered water from studies of existing pump-irrigated farms along the Rio Saska valley.

Distribution canal losses, plus the waste of water due to poor operation, breaks and overflows have been estimated at 20 percent from similar studies.

Main canal losses, for various canal linings, should be based on the following soil permeabilities:

Canal Type	Permeability (ft/day)
Unlined	1.0
Clay-lined	0.2
Concrete-lined	0.01

1-12 Economic Data

Cost estimates shall be based on the cost relationships in Figures 9 to 12 inclusive. Where these curves are shown on a graph, the upper curve gives costs for difficult conditions, the lower curve for easy conditions and the central curve gives costs for average construction conditions. Electric power may be purchased from the Provincial Power Authority at rates of $18 per annum per kilowatt for capacity and 4 mills per kilowatt hour for power consumption.

Allowances for Engineering, Contingencies and Interest during construction shall be calculated as 30% of the estimated direct cost of construction.

The period of analysis or depreciation period shall encompass that period of time over which the project will usefully serve its intended purpose. In any case, this period shall not exceed a useful life span of 50 years.

Annual operation and maintenance costs are based on a percentage of capital cost of construction. As cost of fuel is a variable, it has not been included in the percentage factor.

The following may be used for irrigation projects.

	Useful Life (Years)	Operation & Maintenance (Percent)
Dams, earth and concrete	50	0.1
Intake and outlet works	50	1.0
Gates and hoists	25	1.5
Unlined canals	50	2.0
Lined canals	50	1.0
Concrete conduits	50	1.0
Distribution control structures	50	3.0
Bridges, concrete and steel	50	3.0
Pumps, large	25	2.0
Wells and well pumps	15	2.0

The interest for amortizing costs and for discounting benefits was established as the average rate on outstanding Provincial Government interest-bearing marketable securities running for 15 years or more. The most recent determination by the Treasury Department has fixed this average at 6 percent.

Existing dryland farming conditions and farm budget for irrigation farming have been estimated for the three irrigation subzones. These figures were established from on-farm interviews and from an analysis of soil potential and crop market values and are given in Table VIII.

1-13 Transportation

There are several good all-weather roads in the project area connecting with the Railhead in Blaine. These roads are two-lane asphalt paved highways in good repair, although continued maintenance and reconstruction of certain sections will be necessary in the near future. The inter-connecting farm roads, not shown on Figure 7, are only pass-able with four-wheel-drive or farm vehicles during the greater part of the wet season. Connections from Blaine to the City of Portage La Prairie are good and rail car avail-ability during the harvest season is adequate.

1-14 Marketing

The marketing of rice is under Government control and farmers are given guaranteed floor prices for their pro-duce. At present production is primarily for domestic consumption. Rice milling operations and storage facilities in Blaine are adequate for average production although storage facilities are strained in wet years when rice production is high. This situation is also aggravated when grain cars are in short supply and the harvest season is too wet for outside storage of milled grain.

1-15 Rural Population

The population of the Development Area is about 10,000 people, with some 2,000 concentrated in the service centre of Blaine. There are several small settlements located throughout the area concentrated primarily along the rivers and streams. Subzones II and III, where small farms are worked by intensive hand cultivation, are the most densely populated, figures range from 20 to 25 people per square mile. The larger farms in Subzone I are more mechanized and the population density decreases to 5 per square mile. These farms, however, are the primary employers of rural farm labour and during harvest season, migrant workers add to the population density of this subzone.

1-16 Credit Availability

Credit availability and use varies markedly with farm sizes and types of operation. The farmers in subzone III pay interest rates on capital of up to 12 percent due to the year to year variability in yields and quality of crops. Low land values limit the use of land as col-ateral. In subzone II, higher land values and higher, more dependable crop yields and quality enable landowners to obtain credit at lower rates ranging from 10 to 12 percent depending on the individual farmer and crop variety. Subzone I landowners are generally more prosperous and require financing primarily for capital purchases such as machinery. Again there is a range of interest rates from 8 to 10 percent depending on the use of credit and the individual farmer.

Generally, large farmers can obtain credit most easily, and at reasonable interest rates, by using their land and equipment as collateral at the bank. Smaller farmers have more difficulty and pay, on the average, higher interest rates.

RIO SASKA AT PORTAGE

Summary of Mean Monthly Discharge, in Second-feet, of Rio Saska at Portage

Year*	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	April	May	June	July	Aug.	Sept.	Total Acre-feet
1940	9,293	7,414	3,300e	1,247	1,981	2,432	15,852	11,937	32,436	24,232	14,854	9,143	8,106,373
1941	7,909	5,874	3,752	2,702	2,130	3,038	6,319	13,876	26,375	22,694	9,762	7,945	6,797,396
1942	10,315	8,151	3,204	3,379	2,345	3,318	13,472	19,813	36,144	60,542	33,704	16,357	12,798,628
1943	12,714	6,118	3,855	2,545	2,452	15,342	19,894	13,474	51,612	56,800	26,707	29,721	14,610,259
1944	14,071	8,067	4,952	3,557	3,112	2,746	16,694	25,303	57,662	30,232	11,958	8,914	11,314,976
1945	5,590	4,562	2,006	2,562	3,133	4,335	13,252	12,920	23,090	14,847	8,739	6,143	6,109,209
1946	5,140	4,239	2,007	1,882	2,484	1,925	7,619	9,424	19,365	10,312	9,638	6,146	4,841,237
1947	3,206	1,088	1,612	2,062	1,924	2,503	15,615	35,532	25,394	31,464	13,526	5,804	8,524,721
1948	4,609	2,854	2,081	1,430	1,625	2,934	11,835	12,902	26,022	15,061	8,126	4,273	5,666,236
1949	3,018	2,793	1,800	1,604	1,458	1,981	11,687	15,370	26,224	13,757	7,871	5,786	5,638,919
1950	3,590	2,460	1,470	1,620	1,220	1,450	6,620	8,490	56,300	32,000	13,900	9,310	8,356,324
1951	5,610	3,310	2,680	1,580	2,090	2,450	8,040	11,600	20,700	18,600	13,400	7,670	5,918,539
1952	5,020	2,960	2,470	2,160	2,290	3,450	24,600	16,000	26,400	17,600	9,180	8,820	7,298,838
1953	8,610	5,830	3,810	2,550	2,700	8,830	10,500	9,240	6,430	15,000	6,100	19,900	6,010,000
1954	17,200	9,090	4,110	3,130	2,650	5,010	25,100	22,500	51,300	35,800	19,700	20,800	13,100,000
1955	17,000	7,280	5,140	9,130	4,360	10,000	17,900	19,400	40,500	46,000	14,300	8,390	12,100,000
1956	6,730	5,060	2,590	2,210	1,770	4,960	7,430	12,200	42,800	12,000	5,340	3,820	6,450,000
1957	2,270	2,120	1,270	1,600	2,260	6,170	10,500	13,300	22,200	16,000	7,660	5,020	5,460,000
1958	3,570	3,560	2,890	1,870	2,460	2,720	4,660	4,450	10,300	11,400	5,300	4,500	3,490,000
1959	3,380	1,870	1,380	1,220	1,350	3,190	8,330	15,900	42,800	17,500	6,530	7,270	6,690,000
1960	4,430	3,250	2,590	2,060	1,660	2,240	12,300	17,400	27,000	18,500	6,440	6,030	6,290,000
1961	3,730	5,780	2,840	2,700	4,140	4,890	10,000	19,900	24,500	10,300	4,080	3,320	5,800,000
1962	3,820	3,580	2,130	1,140	2,520	3,440	10,400	9,370	20,000	17,100	10,200	3,950	5,294,000
1963	2,630	1,970	2,000	1,150	1,130	3,600	14,600	15,000	18,100	5,370	3,150	2,880	4,224,000
1964	1,840	1,180	997	1,010	1,100	1,540	6,690	4,760	17,800	11,450	5,910	3,300	3,477,000
1965	3,340	3,300	2,290	1,630	1,530	1,940	12,400	12,300	33,500	22,400	7,580	4,940	6,469,000
1966	3,950	2,490	1,850	1,480	1,570	2,190	11,000	7,470	18,800	22,500	5,170	3,540	4,963,000
1967	3,150	2,380	1,920	1,540	1,520	1,850	15,970	16,730	14,380	7,090	5,540	4,920	4,652,000
1968	6,190	2,710	2,740	2,160	1,980	2,390	11,000	4,760	8,430	5,840	3,780	5,030	3,440,000
1969	5,140	3,650	2,380	1,630	1,520	2,000	6,160	13,490	35,510	29,600	14,710	10,070	7,620,000
1970	8,460	3,390	2,610	2,340	2,490	2,910	26,100	12,800	21,600	21,500	9,200	4,330	7,111,000
Average	6,310	4,160	2,650	2,210	2,160	3,750	12,710	14,140	28,460	22,270	11,190	8,630	7,052,000
Average Acre-ft.	388,000	247,500	162,900	135,900	121,200	230,600	756,300	869,400	1,694,000	1,369,000	688,100	513,500	7,176,000

e Estimated

* The year refers to the months of January to September inclusive.

TABLE IV

SUMMARY OF MEAN MONTHLY DISCHARGES IN SECOND-FEET FOR RIO SECO AT BLAINE

*	OCT.	NOV.	DEC.	JAN.	FEB.	MARCH	APRIL	MAY	JUNE	JULY	AUG.	SEPT.	TOTAL SECOND-FEET	TOTAL ACRE-FEET
1959	90	90	70	45	60	68	117	111	257	285	132	112	1,437	86,220
1960	84	46	34	31	33	79	208	390	1070	437	163	181	2,756	165,360
1961	110	81	64	52	42	56	307	435	675	462	161	150	2,595	155,700
1962	91	142	71	65	103	122	250	498	625	251	102	83	2,403	144,180
1963	96	89	53	28	63	86	260	234	500	425	251	96	2,181	130,860
1964	66	49	50	29	28	90	365	375	452	134	78	72	1,788	107,280
1965	46	28	24	25	25	37	157	119	445	286	147	83	1,422	85,320
1966	83	82	57	41	38	48	310	308	835	560	189	123	2,674	160,440
1967	98	62	46	37	39	54	275	186	470	562	128	87	2,044	122,640
1968	77	59	48	38	38	46	397	418	359	153	138	123	1,894	113,640
1969	154	67	68	54	49	58	275	119	211	146	94	126	1,421	85,260
1970	128	91	59	40	38	50	152	237	893	740	363	252	3,143	188,580
TOTAL	1123	886	644	485	556	794	3073	3530	6792	4441	1946	1488		
AVERAGE	93	73	53	40	46	66	256	294	566	370	162	124		

* The year refers to the months January to September inclusive.

TABLE V

RECORDED PRECIPITATION (INCHES) IN THE RIO SECO BASIN

	O	N	D	J	F	M	A	M	J	J	A	S	TOTAL ANNUAL
METEOROLOGICAL STATION NO. 1													
1959	-	-	-	-	-	0.47	2.37	5.12	10.23	9.16	4.06	1.89	33.30
1960	T	-	-	-	T	1.30	3.57	7.31	12.31	9.12	4.13	2.13	39.87
1961	-	-	-	-	T	1.63	3.21	6.87	11.51	9.62	3.92	1.95	38.71
1962	-	-	-	-	-	0.73	3.17	6.18	11.21	10.05	4.21	2.56	38.11
1963	-	-	-	-	-	0.85	2.76	6.25	10.38	8.53	4.80	2.32	35.89
1964	T	-	-	-	-	0.53	2.73	6.18	10.03	8.86	3.72	1.71	33.76
1965	T	T	-	-	-	0.73	2.65	4.47	10.31	8.23	3.71	2.43	32.53
1966	T	-	-	T	T	1.42	3.27	6.32	10.93	10.78	5.37	1.73	39.82
1967	-	-	-	-	-	1.05	2.82	6.37	11.04	9.18	3.36	1.42	35.24
1968	T	-	-	-	-	0.97	3.23	5.21	10.52	8.21	3.61	2.36	34.11
1969	-	-	-	-	-	0.47	2.49	5.08	10.17	8.73	3.47	1.38	31.79
1970	0.50	T	T	T	T	1.57	2.96	6.80	12.07	10.23	4.85	2.49	41.79
Total	0.50					11.72	35.23	72.16	130.70	110.70	49.21	26.28	434.92
Average	0.04	T	T	T	T	0.97	2.94	6.01	10.89	9.22	4.10	2.10	36.22
RAIN GAUGE STATION NO. 2													
1967	-	-	-	-	-	0.40	2.25	4.87	9.73	8.70	3.02	1.06	30.03
1968	-	-	-	-	-	0.41	1.97	4.80	10.05	8.75	3.89	1.08	30.95
1969	-	T	-	-	-	0.72	2.10	4.35	9.62	(8.03)	(3.15)	1.93	29.90
1970	T	-	-	-	T	0.98	2.80	6.35	10.37	10.61	5.35	1.71	38.17
Total	T	T	-	-	T	2.61	7.12	20.37	39.77	36.09	15.41	5.78	129.05
Average	-	-	-	-	-	0.65	1.78	5.09	9.94	9.02	3.85	1.44	32.26
RAIN GAUGE STATION NO. 3													
1967	-	-	-	-	-	0.45	2.65	5.12	10.20	8.73	3.51	1.05	31.71
1968	-	-	-	-	-	0.50	2.51	5.16	10.27	9.11	3.91	1.97	33.43
1969	-	-	-	-	-	0.73	2.70	4.75	10.41	8.31	3.96	2.41	33.27
1970	-	-	-	-	-	1.46	3.44	6.51	11.00	10.71	4.87	1.95	39.94
Total	-	-	-	-	-	3.04	11.20	21.54	42.08	36.86	16.25	7.38	138.35
Average	-	-	-	-	-	0.76	2.80	5.38	10.52	9.21	4.06	1.84	34.58
RAIN GAUGE STATION NO. 4													
1967	-	-	-	-	-	1.12	3.17	6.53	11.21	9.35	3.21	1.25	34.59
1968	T	-	-	-	-	1.43	3.31	6.36	11.00	10.63	4.27	2.36	39.36
1969	-	-	-	-	-	0.71	2.32	5.97	10.28	9.01	3.32	1.38	32.99
1970	1.05	0.23	T	T	0.37	1.68	3.17	6.98	12.25	10.81	5.31	2.59	41.80
Total	1.05	0.23	T	T	0.37	4.94	11.97	25.79	44.74	39.80	16.11	7.58	148.74
Average	0.26	0.06	-	-	0.09	1.23	2.99	6.45	11.18	9.95	4.03	1.89	37.19
RAIN GAUGE STATION NO. 5													
1967	-	-	-	-	-	0.45	2.82	5.72	10.37	8.79	3.27	1.15	32.57
1968	T	-	-	-	-	0.61	2.69	5.56	10.27	9.53	4.21	2.04	34.91
1969	-	-	-	-	-	0.83	2.70	4.93	10.59	8.52	4.21	2.39	34.17
1970	0.61	0.17	T	T	0.22	1.82	3.51	6.39	11.27	10.63	5.14	2.14	41.90
Total	0.61	0.17	T	T	0.22	3.71	11.72	22.60	42.50	37.47	16.83	7.72	143.55
Average	0.15	0.04	-	-	0.05	0.93	2.93	5.65	10.62	: 37	4.21	1.93	35.89

NOTE: "T" signifies trace of precipitation.

TABLE VI

RECORDED PRECIPITATION (INCHES) IN THE RIO SECO BASIN

RAIN GAUGE STATION NO. 6

	O	N	D	J	F	M	A	M	J	J	A	S	TOTAL ANNUAL
1967	-	-	-	-	-	0.50	2.71	5.35	10.29	8.76	3.43	1.10	32.14
1968	-	-	-	-	-	0.54	2.60	5.36	10.35	9.37	4.13	2.01	34.36
1969	-	T	-	-	-	0.80	2.73	4.81	10.48	8.44	4.17	2.40	33.83
1970	T	T	-	-	T	1.63	3.57	6.42	11.13	10.72	5.00	2.10	40.57
Total	T	T	-	-	T	3.47	11.61	21.94	42.35	37.29	16.73	7.61	140.90
Average	-	-	-	-	-	0.87	2.90	5.48	10.59	9.32	4.18	1.90	35.23

RAIN GAUGE STATION NO. 7

	O	N	D	J	F	M	A	M	J	J	A	S	TOTAL ANNUAL
1967	-	-	-	-	-	1.23	2.94	6.39	11.22	9.17	3.57	1.56	36.08
1968	T	-	-	-	-	1.04	2.84	6.47	11.61	8.27	3.07	1.35	34.65
1969	-	-	-	-	-	0.73	2.61	5.22	10.10	8.92	3.61	1.54	32.73
1970	0.71	0.23	T	-	T	1.82	3.15	5.97	11.24	11.04	4.98	2.71	42.85
Total	0.71	0.23	T	-	T	4.82	11.54	24.05	44.17	37.40	15.73	7.16	146.31
Average	0.18	0.06	-	-	-	1.20	2.88	6.01	11.04	9.35	3.94	1.79	36.58

RAIN GAUGE STATION NO. 8

	O	N	D	J	F	M	A	M	J	J	A	S	TOTAL ANNUAL
1967	-	-	-	-	-	1.18	3.14	6.46	11.17	9.21	3.39	1.41	35.96
1968	T	-	-	-	-	1.23	3.32	6.92	11.88	9.33	3.82	2.27	38.77
1969	-	-	-	-	-	0.72	2.47	5.61	10.19	8.98	3.47	1.46	32.90
1970	0.85	0.21	T	T	0.22	1.75	3.18	6.05	11.31	10.84	5.14	2.67	42.22
Total	0.85	0.21	T	T	0.22	4.88	12.11	25.04	44.55	38.36	15.82	7.81	149.85
Average	0.21	0.05	-	-	0.05	1.22	3.03	6.26	11.14	9.59	3.95	1.95	37.46

TABLE VII

MAXIMUM 24 HOUR RAINSTORMS

STORM NO.: 7 BEGAN: June 15 @ 5:00 a.m. ENDED: June 16 @ 12:00 noon

STATION	ABSOLUTE MAX. PRECIP. (duration in hours)				CONTEMPORANEOUS ACCUMULATED PRECIP. (in inches) Time in Hours at End of Period								
	6	12	18	24	6	9	12	15	18	21	24	27	30
1	3.0	3.4	5.6	6.4	0.20	0.71	1.84	2.94	3.29	3.33	3.40	4.92	6.40
2	-	-	-	-	0.15		1.63		3.10		3.10		5.89
3	-	-	-	-	0.20		1.75		3.26		3.32		6.07
4	-	-	-	-	0.23		1.83		3.31		3.40		6.13
5	-	-	-	-	0.18		1.87		3.41		3.52		6.20
6	-	-	-	-	0.21		1.78		3.53		3.61		6.25
7	-	-	-	-	0.22		1.93		3.44		3.50		6.56
8	-	-	-	-	0.18		2.08		3.47		3.52		6.38

Maximum Annual 24 Hour Rainfall at Station 1

Year	1959	1960	1961	1962	1963	1964	1965	1966	1967	1968	1969	1970
Rainfall (inches)	3.20	5.08	4.40	4.21	3.92	3.48	3.02	4.57	3.85	3.67	2.73	6.40
Duration (hours)	20	21	25	26	22	24	23	29	24	22	20	24

NOTE: 1) Mass curve constructed from hourly values obtai ed for Station 1.

2) Storm durations ± 25% of 24 hours.

TABLE VIII

FARM BUDGET SUMMARIES

	FOR IRRIGATION SUB ZONE I TYPICAL FARM SIZE 640 ACRES		FOR IRRIGATION SUB ZONE II TYPICAL FARM SIZE 320 ACRES		FOR IRRIGATION SUB ZONE III TYPICAL FARM SIZE 160 ACRES	
	WITHOUT IRRIGATION	WITH IRRIGATION	WITHOUT IRRIGATION	WITH IRRIGATION	WITHOUT IRRIGATION	WITH IRRIGATION
CAPITAL INVESTMENT						
Land	$ 55,000	$ 55,000	$ 30,000	$ 30,000	$ 11,000	$ 11,000
Buildings	10,000	10,000	5,000	5,000	3,000	3,000
Machinery	12,000	18,000	7,500	12,000	4,000	6,000
Livestock	13,000	3,000	7,500	2,500	2,000	2,000
Land Shaping and Levelling		70,000	-	40,000	-	15,000
Total Investment	$ 90,000	$156,000	$ 49,000	$ 89,500	$ 20,000	$ 37,000
ANNUAL EXPENSES						
Interest and Depreciation	$ 7,000	$ 12,000	$ 3,400	$ 6,000	$ 2,100	$ 4,200
Seed, Fertilizer, Weed Control	1,700	3,500	900	2,000	400	900
Operation and Maintenance	2,300	4,000	1,300	2,300	950	1,300
Property Tax, Insurance	1,700	2,500	800	800	250	600
Hired Labour	1,000	5,000	300	800	-	-
Family Labour	5,000	5,000	2,000	3,000	2,000	3,000
Family Allowance	2,000	2,000	2,000	2,000	2,000	2,000
Total Annual Expenses	$ 20,700	$ 34,000	$ 10,700	$ 18,800	$ 7,600	$ 11,000
GROSS FARM INCOME	$ 28,000	$ 56,400	$ 14,000	$ 28,400	$ 9,000	$ 14,520
NET INCOME	$ 7,300	$ 22,400	$ 3,300	$ 9,600	$ 1,400	$ 3,520
NET INCOME PER ACRE	$ 11.40	$ 35.00	$ 10.31	$ 30.00	$ 8.75	$ 22.00

CHAPTER II

WATER REQUIREMENTS

2-1 General

The following discussion pertains to basic data, assumptions and procedures used to investigate the availability of water and to estimate the irrigation water requirements for the Rio Seco Basin development scheme. Data from previous studies of the Rio Saska reservoir have been utilized.

2-2 Irrigation Water Requirements

Irrigation water requirements consist of all water used for irrigation of crop lands plus seepage losses in the main canal and distributaries and operational wastes.

Hydrologic criteria for estimating irrigation water requirements are discussed in the following paragraphs.

2-3 Consumptive Use

Consumptive use of water by crops is the total water requirement for growth. It includes water lost in evaporation, transpiration and water utilized in building plant tissue. Consumptive use of water by plants is highly dependent on environmental factors such as weather, soil moisture and ground water. In most instances where consumptive use must be determined, meteorological information is available and the majority of computing formulae endeavour to estimate consumptive use based on this information. Selection of a relationship for calculating crop water requirements is a most important consideration, a poor choice of equation could result in excess expenditure on capital works or in insufficient water supply for the complete development of the project. The example calculations which follow aptly demonstrate this point.

In order to select a satisfactory computing formula which would give realistic results, full use was made of the knowledge that potential evapotranspiration and lake evaporation are very nearly equal in humid regions, Bruce and Clark (1966). This comparison would be valid as actual evapotranspiration will approach potential evapotranspiration where application of irrigation water is contemplated.

The following are example calculations utilizing available meteorological data and selected computing formulae from Table X.

Lowry-Johnson

$U = 0.000156H + 0.8$ .... Equation 2

This formula requires the accumulated degree-days of maximum daily temperatures above $32^{o}F$ during the growing season. The available figure on temperature is the average annual temperature of $73^{o}F$.

$H = (73-32) \times 365$ days or 14,200 degree days

$U = (1.56 \times 10^{-4} \times 1.42 \times 10^{4}) + 0.8 = 3.0'$ or 36"

The average equivalent lake evaporation is 72.66 x 0.7 or 50.86 inches. The Lowry-Johnson value is low.

Hargreaves

$$U = \sum_{1}^{m} kd\ (0.38 - 0.0038h)\ x(t-32^{o}) \text{ ..... Equation 8}$$

This relationship requires two coefficients, k a seasonal coefficient and d a monthly daytime coefficient. These values are available in Chow (1964). The values for h, relative humidity, and t, average monthly temperature, are from Table I. The following computation is for the month of October:

$$U = 1.07 \times 0.97 \quad 0.38 - (0.0038 \times 66) \quad \times (65-32) = 4.45"$$

The comparable figure for equivalent lake evaporation for the same month is 4.93 x 0.7 or 3.45 inches. The Hargreaves value is high.

Blaney-Criddle

U = k (pt/100) .... Equation 6

The parameters have been fully explained in the notes to Table X. The following values for consumptive use by corn and cotton have been computed with the Blaney-Criddle formula and are tabulated alongside observed Class A pan evaporation and equivalent lake evaporation for comparison.

TABLE IX

CONSUMPTIVE USE BY THE BLANEY-CRIDDLE FORMULA VERSUS EQUIVALENT LAKE EVAPORATION

Month	Consumptive Use Corn and Cotton	Class A Pan Evaporation	Equivalent Lake Evaporation
October	3.25	4.93	3.45
November	2.73	4.34	3.04
December	2.87	4.46	3.12
January	3.02	4.60	3.22
February	3.17	5.29	3.70
March	3.91	5.70	3.99
April	4.40	6.62	4.64
May	6.12	8.07	5.65
June	5.79	8.21	5.82
July	5.40	7.65	5.35
August	5.12	6.86	4.70
September	4.36	5.93	4.15

The comparison indicates good correlation between Blaney-Criddle results for corn and cotton and equivalent lake evaporation. The comparison is less satisfactory for rice, but rice has a uniquely high requirement for water and the strength of the correlation for cotton and corn, crops that are more similar to native vegetation, was felt to be adequate reason for selection of the Blaney-Criddle formula for all consumptive use calculations.

2-4 Effective Precipitation

The rainfall frequency curve for the index station at Blaine, from 12 years of records, was used to determine the average annual and 1-in-10 dry year rainfalls. When compared with more recent rainfall data from the additional 7 gauging stations in the basin, it was determined that the rainfall recorded at these stations in 1967 corresponds to the mean annual rainfall and in 1969 to the 1-in-10 dry year. The 1-in-10 dry year rainfall was selected for computation of irrigation requirements. Based on existing records this would mean that 1 year out of 10 there would be a shortage of irrigation water. The difference in rainfall between the wettest and driest years of record is some 9", a shortage which would limit production but would not result in a crop failure.

The method of Thiessen polygons was used for computing the average rainfall over the development area. This method assumes that the precipitation at each station affects the average rainfall in proportion to the ratio of the area of influence of the station to the total area under consideration. Weighting factors for each station are given in Table XI for the polygons shown in Figure 14.

2-5 Crop Irrigation Requirements

The crop irrigation requirement is the quantity of water, in addition to rainfall, necessary to ensure optimum crop production. It was computed as the difference between the monthly consumptive use and the monthly precipitation.

2-6 Irrigation Demand

In supplying the crop irrigation requirements, various losses occur on the farm unit as a result of seepage and waste. These losses plus the crop irrigation requirements make up the irrigation demand or farm delivery requirements. Farm irrigation efficiency is an indication of these losses, since it represents the useful water portion of total water delivered to the farm.

Farm efficiency was estimated at 60 percent. The irrigation demand was calculated by dividing crop irrigation requirement by the farm efficiency.

2-7 Canal Seepage Losses

When considering the relative merits and comparative costs of unlined canals versus different types of canal lining, consideration must be given to the relative quantities of water lost from the various types of canals through seepage. Seepage losses may be determined from the Moritz (1952) formula, which is as follows:

$$\text{Seepage losses (cfs)} = 0.2\ C\ (Q/V)^{1/2}$$

where C = soil permeability, feet per day

Q = canal capacity in cfs

V = velocity of flow fps

L = length of canal in miles

Soil permeability factors for canal lining for the main canal are as follows:

(1) Unlined. The material is a light sandy soil with C = 1 foot per day.

(2) Clay-lined. The material is a medium to heavy textured soil with C = 0.2 feet per day.

(3) Concrete-lined. Theoretically a concrete lined canal would be water tight, but seepage through construction joints and expansion cracks would account for some losses. A soil permeability rate of 0.01 feet per day has been selected as a working value.

In order to confirm the relative magnitude of the foregoing figures, the following comparison between soil test results at the Rio Seco and working values has been made.

Material at Rio Seco	Permeability		Working Values	Lining Material
	cm/sec.	ft/day	ft/day	
SC	$5x10^{-4}$	1.41	1.0	clayey sand
CL	$1x10^{-5}$	.05	0.2	average soil
Rock	$1x10^{-10}$	negligible	0.01	concrete

NOTE: 1 cm/sec is equivalent to 28.3 x 10^2 ft/day.

Soil permeability factors for canal linings for the distribution canals are as follows:

(1) Unlined. The material in which the canals would be excavated has a typical disturbed hydraulic conductivity of 2 inches per hour. In situ measurements of soil infiltration rates have established an infiltration rate of 0.25 inches per hour or C = 0.5 feet per day.

(2) Clay and concrete-lined. As for the main canal.

2-8 Diversion Requirements

Monthly irrigation diversion requirements in Table XII were estimated by increasing the irrigation demand requirements by 20 percent to account for conveyance losses in the distributaries. Main canal losses were calculated and added to irrigation water requirements to determine the diversion and storage requirements at the storage reservoir where different types of main canal sections were analyzed.

Table XII gives the net monthly irrigation and diversion requirements for 10,000-acre areas of specific crops.

It was assumed that 60 percent of the development would be used for rice and pasture, 35 percent alternating between cotton and corn and the remaining 5 percent allowed for farmsteads, roads, canals and drains. This distribution is in accordance with earlier recommendations.

In the calculation of total diversion requirements as shown in Table XIII it was assumed that farm requirements would remain undiminished during the harvest seasons. This assumption appeared realistic considering that harvesting seasons for the various crops would not be coincident, for example, rice would be under irrigation while corn was being harvested. Pre-saturation of rice fields for seeding and weed control would also necessitate a continued supply of water prior to the actual growing season.

2-9 Irrigation Return Flows

Return flow is water which is not consumed in evaporation and transpiration and returns to a surface stream or drain. Annual return flow can be calculated from annual precipitation plus annual diversion less consumptive use and corrected for seepage and waste. As re-use of return flow has not been considered in this analysis, and would be a speculative quantity, it has been assumed that return flows would amount to 20 percent of the monthly diversion requirements.

TABLE X

EVAPOTRANSPIRATION EQUATIONS

NAME	PERIOD FOR U	UNIT FOR U	EQUATION	
Hedke	Annual	Feet	$U = kH$	1
Lowry-Johnson	Annual	Feet	$U = 0.000156H + 0.8$	2
Blaney-Morin	m months	Inches	$U = k\sum_{1}^{m} pt(114-h)$	3
Thornthwaite	Monthly	Centimeters	$U = 1.6 \left(\frac{10t}{TE}\right)^{a}$ where $a = 0.000000675(TE)^3 - 0.0000771(TE)^2 + 0.01792TE + 0.49239$	4
Penman	Daily	Millimeters	$U = \frac{AH - 0.27E}{A - 0.27}$ where $E = 0.35(e_a - e_d)(1 + 0.0098w_2)$; $H = R(1 - r)(0.18 + 0.55S) - B(0.56 - 0.092e_d^{0.5})(0.10 + 0.90S)$	5
Blaney-Criddle	m months	Inches	$U = k\sum_{1}^{m} pt = kF$ where $F = \sum_{1}^{m} pt$	6
Halkias-Veihmeyer-Hendrickson	Monthly	Inches	$U = SD$	7
Hargreaves	m months	Inches	$U = \sum_{1}^{m} kd(0.38 - 0.0038h)(t - 32)$	8

A = slope of saturated-vapor-pressure curve of air at absolute temperature in ^{o}F, or de_a/dt in mm Hg/^{o}F

B = a coefficient depending on temperature

D = difference in evaporation between white and black atmometers in cm^3

d = monthly daytime coefficient dependent upon latitude

e_a = saturation vapor pressure at mean air temperature in mm Hg

e_d = saturation vapor pressure at mean dew point (i.e. actual vapor pressure in the air) in mm Hg, being equal to e_a multiplied by relative humidity in per cent

E = daily evaporation in mm

h = mean monthly relative humidity at noon, in Eq. (8), or annual mean relative humidity in per cent in Eq. (3)

H = accumulated degree-days above minimum growing temperature for growing season, in Eq. (1), or accumulated degree-days of maximum daily temperatureabove 32° F for growing season in Eq. (2), or daily heat budget at surface in mm. of water, in Eq. (5).

k = annual, seasonal, or monthly consumptive-use coefficient.

p = percent of daytime hours of the year, occurring during the period, divided by 100

r = estimated percentage of reflecting surface

R = mean monthly extraterrestrial radiation in mm. of water evaporated per day

S = estimated ratio of actual duration of bright sunshine to maximum possible duration of bright sunshine, or slope of regression line between D and U in Eq. (7)

TE = Thornwaite's temperature-efficiency index, being equal to the sum of the 12 monthly values of heat index $i = (t/5)^{1.514}$, where t is mean monthly temperature in °C in Eq. (4)

t = mean monthly temperature in °F, in Eqs. (3), (6), and (8), or in °C in Eq. (4)

U = evapotranspiration or consumptive use for given period

w_2 = mean wind velocity at 2 m. above the ground in miles/day, or equal to $w_1(\log 6.6/\log h)$, where w_1 is measured wind velocity in miles/day at height h in ft.

NOTES

Computing Formulae:

Evapotranspiration (U) = $k\left(\frac{pt}{100}\right)$ (Blaney-Criddle)

U = Evapotranspiration in inches per month.

k = Consumptive use coefficient.

p = Monthly percentage of daytime hours of the year.

t = Average monthly temperature °F.

Crop Requirements = U - rainfall

Irrigation Demand = $\frac{\text{Crop Requirements (ft.) x 10,000 Acres}}{\text{0.6 Irrigation Efficiency}}$

Diversion Requirements = $\frac{\text{Irrigation Demand (Acre Feet) x 1.2 Crop Losses}}{\text{60 Acre Feet/Second Foot Month}}$

Rainfall inches calculated from Thiessen Polygon method for rainfall equalled or exceeded 90% of the time.

TABLE XI

THIESSEN POLYGON
STATION INFLUENCE AREAS
WEIGHTING FACTORS

STATION	1	2	3	4	5	6	7	8	TOTAL
AREA (Thousands of Acres)	72.1	45.6	63.5	29.6	12.5	30.4	26.2	20.1	300
WEIGHTING FACTOR (Percent)	24	15	21	10	4	10	9	7	100

TABLE XII

MONTHLY IRRIGATION DEMAND AND
DIVERSION REQUIREMENTS FOR 10,000 ACRE UNIT
OF COTTON & CORN, ALTERNATING CROPS

Month	t	p	k	U	Rain-Fall	Crop Requirements		Irrigation Requirements	Diversion Requirements
					inches	inches	feet	acre feet	S.F.M.
			COTTON						
October	65	8.06	0.62	3.25	0	3.25	0.27	4,500	90
November	60	7.36	0.62	2.73	0	2.73	0.23	3,850	77
December	63	7.35	0.62	2.87	0	2.87	0.24	4,040	80
January	65	7.49	0.62	3.02	0	3.02	0.25	4,180	84
February	72	7.12	0.62	3.17	0	3.17	0.26	4,350	87
March	75	8.40	0.62	3.91	0.66	3.25	0.27	4,510	90
April	82	8.64	0.62	4.40	2.52	1.88	0.16	2,680	54
			CORN						
May	87	9.38	0.75	6.12	5.02	1.10	0.09	750	15
June	84	9.30	0.75	5.79	10.36	(4.57)	-	-	-
July	76	9.49	0.75	5.40	8.42	(3.02)	-	-	-
August	75	9.10	0.75	5.12	3.61	1.51	0.13	1,090	22
September	70	8.31	0.75	4.36	1.83	2.53	0.21	1,750	35
							2.11	TOTAL	634
								AVERAGE	53

MONTHLY IRRIGATION DEMAND AND
DIVERSION REQUIREMENTS FOR 10,000 ACRE UNIT
OF PERENNIAL RICE & PASTURE

			RICE & PASTURE						
October	65	8.06	1.00	5.25	0	5.25	0.44	7,350	147
November	60	7.36	1.00	4.41	0	4.41	0.37	6,170	123
December	63	7.35	1.00	4.63	0	4.63	0.39	6,500	130
January	65	7.49	1.00	4.86	0	4.86	0.41	6,850	137
February	72	7.12	1.00	5.13	0	5.13	0.48	7,180	144
March	75	8.40	1.00	6.30	0.66	5.64	0.47	7,850	157
April	82	8.64	1.00	7.09	2.52	5.57	0.46	7,670	153
May	87	9.38	1.00	8.15	5.02	3.13	0.26	4,350	87
June	84	9.30	1.00	7.71	10.36	(2.65)	-	-	-
July	76	9.49	1.00	7.20	8.42	(1.22)	-	-	-
August	75	9.10	1.00	6.82	3.61	3.21	0.26	4,350	87
September	70	8.31	1.00	5.81	1.83	3.98	0.33	5,500	110
							3.67	TOTAL	1,275
								AVERAGE	107

TABLE XIII

DIVERSION REQUIREMENTS WITHOUT
ADJUSTMENT FOR MAIN CANAL CONVEYANCE LOSSES

	10,000 ACRE UNIT		NUMBER OF UNITS		ALL UNITS		TOTAL
	Rice Pasture	Cotton Corn	Rice Pasture	Cotton Corn	Rice Pasture	Cotton Corn	second-foot-months
Oct.	147	90	19.5	9	2,870	810	3,680
Nov.	123	77	19.5	9	2,400	690	3,090
Dec.	130	80	19.5	9	2,540	720	3,260
Jan.	137	84	19.5	9	2,670	760	3,430
Feb.	144	87	19.5	9	2,810	780	3,590
March	157	90	19.5	9	3,060	810	3,870
April	153	54	19.5	9	2,980	495	3,475
May	87	15	19.5	9	1,690	135	1,825
June	-	-	19.5	9	-	-	-
July	-	-	19.5	9	-	-	-
Aug.	87	22	19.5	9	1,690	190	1,880
Sept.	110	15	19.5	9	2,140	990	3,130

GRAND TOTAL SFM 31,150

AVERAGE FLOW REQUIRED cfs 2,600

CHAPTER III

SOURCES OF WATER

3-1 Groundwater

Groundwater usage in the Irrigation District to date has been limited to excavation of dugouts for livestock watering purposes and the construction of 10 wells for municipal supply. The maximum well depth is 100 feet and the maximum yield 0.5 cfs.

An indication of total annual yield of the clastic formation into which the existing wells extend can be obtained from inspection of flow records in the Rio Seco. The lowest flow of record at Blaine is 31 cfs. Additional inflow downstream of Blaine may increase this flow to 100 cfs but it is apparent that the total groundwater yield, if mining groundwater is to be excluded, is insufficient for development of the entire area and is of low quality for irrigation.

Water samples from wells indicate an SAR of 10 and a conductivity of 300 micromhos per centimeter, which classifies this water as medium salinity-medium sodium irrigation water. The majority of these wells are shallow, less than 100 feet, and it is probable that the deeper layers of the clastic formation contain water of higher salinity.

Further studies may indicate the feasibility of groundwater mining in subzone III to lower the water table and improve surface drainage. For the purposes of this analysis, groundwater wells have been excluded.

3-2 Surface Water

Mass curve analysis of the existing stream flow records on the Rio Seco indicate that the maximum annual dependable flow from a reservoir on the Seco would be 170 cfs or 10,200 acre-feet (AF). This quantity of water would be sufficient to irrigate 2,000 acres with 5 feet of water.

Storage-dependable flow and mass curve relationships on the Rio Saska indicate that diversion flows would be more than adequate for a 300,000-acre development. The average annual flow from the existing stream flow records is 7,176,000 AF. Dry year flows of 3,300,000 AF alone would be sufficient to irrigate twice as much acreage as the development area and only the lack of suitable irrigation land has limited the scope of the proposed development.

Chemical analyses of water samples from the Rio Saska and Rio Seco have been carried out. In general, if stored and mixed, the composite chemical quality of the water from the two rivers would be suitable in every respect for irrigation. Estimated total dissolved solids would average 200 parts per million (ppm) which would increase due to reservoir evaporation, but annual drawdown conditions in the proposed Rio Seco reservoir would maintain solid buildup well below acceptable levels. The average sodium absorption ratio (SAR) of about 1.2 is low and toxic elements are only found in trace amounts. For municipal use the river water would be acceptable in all respects, except for hardness.

3-3 Water Rights

Water rights are based on the Riparian Doctrine of reasonable usage of the stream's flow, provided the water is used on that land which is contiguous or adjacent to a stream or other body of surface water. With the exception of a few pump irrigators along the Rio Saska, who use limited amounts of water during the normal growing season to supplement precipitation, the only existing formalized water rights are for downstream municipal requirements. This riparian flow is 1,000 cfs on the Rio Saska and 25 cfs on the Rio Seco.

CHAPTER IV

DAMS AND RESERVOIRS

4-1 Summary

The largest dependable source of water for irrigation diversion would be the Rio Saska. As dry season flows are less than diversion requirements and as the Irrigation District would be approximately 50 feet above river stage during low flow periods, storage and some means of elevating the water level would have to be provided.

The following discussion considers the feasibility of a dam and reservoir on the Rio Saska for irrigation diversion requirements, of pumping from the Rio Saska and of a dam and reservoir on the Rio Seco for flow regulations.

4-2 Dam on the Rio Saska

The detailed analysis of the dam and reservoir on the Rio Saska has been completed in an earlier study and does not form a part of the scope of work of this present study. The following discussion has been inserted however to ensure that a complete picture of the Irrigation Development is available in this report, and is made up of details from the original report supplemented by observations from on site inspections.

The dam site at Section A in Figure 2 was selected as the most satisfactory for single purpose irrigation water supply. The river bed is narrow at this section and foundation conditions are adequate for gravity or earth fill dam construction. The configuration of the right bank also facilitates investigation of a large range of canal intake elevations. Foundation conditions immediately downstream are excellent, eliminating the need for an expensive stilling basin. Possible left bank erosion may necessitate model studies of the spillway alignment.

Concrete gravity and earthfill with concrete spillway dams were both investigated as foundation conditions were suitable for both types and adequate construction material was available in the reservoir area. It was found that the concrete gravity section was most economical up to a reservoir level of 950 feet mean sea level (MSL) and that, thereafter, the concrete spillway and earthfill section, gated after 1,150 feet MSL, was the most economical. Diversion and care of water during construction was found to be the governing factor in cost comparison. Possible diversion procedures for concrete gravity sections would allow flood flows to pass through blockouts in the dam proper and overtopping of the section during extreme flows could be tolerated. Earth dams, on the other hand, would require extensive coffer-damming, continuous pumping out of seepage so that materials could be placed in the dry and early installation of the riparian conduit for diversion capacity.

The entire reservoir area is overlain by sand and gravel. This material would require some processing for concreting purposes but processing would be limited to washing and screening.

The natural sand and gravel is also suitable for pervious fill and has a sharp angle or repose. Slopes of 2 to 1 would be stable for dam heights up to 75 feet. The clay material is a suitable impervious core material and is available in layers along the upstream flood plain of the river.

A clean, unweathered outcropping of rock is exposed on the left bank of the river approximately 1 1/2 miles upstream of the dam site. It is estimated that this potential quarry could yield up to 20,000 cubic yards of material and would be suitable for concrete and rip rap.

Water quality is discussed in Paragraph 3-2. The quality is satisfactory for wetting earth fill but may require treatment for concreting operations.

4-3 Pumping

The Irrigation District is situated at elevation 950 feet MSL and the unregulated water level in the Rio Saska is approximately 900 feet MSL during low flow periods. One possibility for supply of water requirements to the district would be to pump the water into a gravity canal from a low level storage reservoir. Lifting water above that in the reservoir could also prove economical in that the slope of the canal could be increased, thus decreasing the cross-sectional area and cost of the canal.

Foundation conditions on the right bank of the Rio Saska at the proposed dam site are satisfactory for the construction of a large pumping installation.

A bank of centrifugal pumps, with electric motor prime movers would be housed in a reinforced concrete structure on the right bank of the river above maximum flood level. Pump efficiencies of 0.85 have been assumed as a working value. Water could be conveyed in reinforced concrete pipes or steel penstocks from the reservoir to a stilling basin at the entrance to the gravity canal.

The availability of construction materials for this site was discussed earlier.

4-3-1 Cost Estimates

Costs for pumping have been based on the relationship in Figure 11 for plant construction and on charges of $18 per annum per kilowatt for capacity costs and 4 mills per kilowatt hour for power consumption. The latter two figures have been supplied by the Provincial Power Authority. Conversion of the cost of power to an annual cost per kilowatt of 86,700 hours/year times

$.004/kilowatt hour yields $34.56 per kilowatt per year for continuous energy. This gives a total of some $53 per kilowatt per year. Cost curves have been established for a range of discharges and total dynamic head (TDH) and are shown in Figure 17. Table XV shows example calculations for a TDH of 100 feet.

4-4 Dam on the Rio Seco

The site on the Rio Seco, shown in Figure 6, was selected as a desirable site for a regulation reservoir for irrigation flows. There were no alternative dam sites available. Foundation conditions and availability of construction materials favor an earthfill structure. Extensive foundation treatment would be required for a concrete gravity dam. The drill logs indicate that the left bank would be the most satisfactory for a spillway structure as rock is fairly close to the surface. River bed conditions downstream of the dam site would require a stilling basin to prevent erosion.

A concrete gravity type dam, although it would result in a considerable saving in conduit costs, was eliminated due to extensive foundation treatment which would be required both for consolidation and prevention of seepage, and due to a lack of readily available rock for aggregate. The earthfill dam and concrete spillway structure was found to be the most economical. Diversion and care of water during construction would not be a special problem for this dam site as river flows are low in the dry season and the conduit capacity for large irrigation releases would be capable of handling the maximum flood of record, even at low reservoir levels.

The overburden material is clayey sand and gravel and would require extensive treatment for use as concrete aggregate. As the quantity of sand and gravel required would not be extensive, this material could be imported without greatly affecting costs. The material is very satisfactory as pervious fill, slopes of 2 1/2 to 1 being common for dam heights up to 70 feet.

The clayey material is similar to that encountered at the Rio Saska dam site and is suitable impervious core material for a zoned fill dam.

Surface rock outcrops are not available within a 10 mile radius of the site. The average depth to rock is 25 feet, as indicated by the exploratory drill holes.

Rip rap may be obtained in the reservoir area by processing surface materials.

A zoned fill dam was selected due to the availability of suitable materials for the pervious and impervious zones. Due to the perviousness of the foundation, the impervious core would be extended into clay material as a seepage cutoff.

Introduction of this cutoff wall and construction of the downstream portion of the dam of the same pervious material as the foundation would avoid construction of a drainage blanket. This is desirable as rock is not readily available for blanket construction. A toe drain has been provided to ensure that any seepage that would come through the foundation or embankment would be collected and so that groundwater would be kept below the surface sufficiently to avoid the creation of unsightly boggy areas below the dam.

The maximum water requirement for the full development of the irrigation district has been established as 3,870 second-foot-months (SFM). The outlet works capacity must range from a maximum of 3,870 cubic feet per second (cfs) to a minimum monthly requirement of 1,825 cfs. Flows through the outlet during June and July would be diverted into the Rio Seco and would range from run of river to downstream bankfull capacity of 750 cfs. The outlet must also function as a spillway up to a discharge of 4,000 cfs.

Pipe outlets with baffle stilling basins were considered for the outlet structures. Experimental data are only available for a maximum size of 6 feet in diameter, with 30 feet per second (fps) maximum velocity and maximum discharge capacity of 400 cfs. A bank of 7 pipes with gates would be required to handle this discharge. Cost comparison between a bank of pipes and a rectangular reinforced concrete section was carried out and a gated rectangular section was found to be the least costly alternative.

The maximum flood of record is 1,500 cfs. This flow could be discharged through the irrigation outlet and diverted into the stream bed at the canal regulator. Based on the frequency curve of maximum annual flood flows from 12 years of records the 1,500-cfs flood has a 20-year return period. The consequence of failure due to floods of greater magnitude would be loss of the dam, considerable flood damage downstream to irrigation structures and the municipality of Blaine and loss of terminal storage. The loss of terminal storage would mean that canal irrigation water supplies of 2,600 cfs continuous would not be adequate for irrigation of the total acreage.

As a result of these considerations, a design flood of 1-in-10,000 years was selected. This flow of 4,000 cfs can be handled by the irrigation outlet, but as gate openings during peak flow months would be pre-set for bankfull releases only and in the event of delays in opening the gates for conduit spillway operation, a 1000-cfs emergency spillway should be provided in the left bank.

A standard ungated ogee crest, concrete chute spillway and concrete wing walls was selected. This type of spillway is simple to construct, is automatic in its operation and as it does not have gates or gate piers is much less costly to construct and maintain than a gated structure.

The spillway would discharge into a concrete stilling basin from where flows could be conveyed via an excavated canal into a convenient stream bed which empties into the Rio Seco. The canal section would be excavated to dimensions similar to that of the Rio Seco at Blaine. The tailwater rating curve for Blaine was used for stilling basin design.

The layout of the dam, outlet works and spillway is shown in Figure 18. Details of the dam cross section, outlet works, spillway, stilling basin and special structures are shown in Figures 20 to 21 inclusive.

4-4-1 Cost Estimates

Quantities for a range of dam heights have been calculated and used to establish the relationship between full supply level (FSL) and capital cost. The cost estimate for an FSL of 1015 feet is shown in Table XVI.

TABLE XIV

ACCUMULATED FLOWS IN SECOND-FOOT-MONTHS FOR RIO SECO AT BLAINE

	OCT	NOV	DEC	JAN	FEB	MARCH	APRIL	MAY	JUNE	JULY	AUG	SEPT
1959	90	180	250	295	355	423	540	651	908	1193	1325	1437
1960	1521	1567	1601	1632	1665	1744	1952	2342	3412	3849	4012	4193
1961	4303	4384	4448	4500	4542	4598	4905	5340	6015	6477	6638	6788
1962	6879	7021	7092	7157	7260	7382	7632	8130	8755	9006	9108	9191
1963	9287	9376	9429	9457	9520	9606	9866	10100	10600	11025	11276	11372
1964	11438	11487	11537	11566	11594	11684	12049	12424	12876	13010	13088	13160
1965	13206	13234	13258	13283	13308	13345	13502	13621	14066	14352	14499	14582
1966	14665	14747	14804	14845	14883	14931	15241	15549	16384	16944	17133	17256
1967	17354	17416	17462	17499	17538	17592	17867	18053	18523	19085	19213	19300
1968	19377	19436	19484	19522	19560	19606	20003	20421	20780	20933	21071	21194
1969	21348	21415	21483	21537	21586	21644	21919	22038	22249	22395	22489	22615
1970	22743	22834	22893	22933	22971	23021	23173	23510	24403	25143	25506	25758

TABLE XV

ANNUAL PUMPING COSTS

TOTAL DYNAMIC HEAD OF 100 FEET

Continuous Discharge	Pump Capacity	$\frac{Q \times TDH}{1,000}$	Capital Cost	ANNUAL COSTS (Millions of Dollars)			
				Interest & Depreciation	Operation & Maintenance	Capacity & Fuel	Total Cost
cfs	kilowatts		$ Million				
500	5,000	50	2	0.16	.04	0.27	0.47
1,000	10,000	100	5	0.39	.10	0.53	1.02
2,000	20,000	200	10	0.78	.20	1.06	2.04
3,000	30,000	300	14.5	1.13	.29	1.59	3.01

NOTES:

Curve 2 in Figure 11 was utilized for determination of capital costs.

$$\text{Power (kilowatts)} = \frac{\text{Discharge x head}}{11.8 \times \text{efficiency}} = \frac{Qh}{11.8 \times 0.85} = \frac{Qh}{10}$$

Annual Costs - Interest and Depreciation 0.078 for 25 year life at 6% interest.
Operation & Maintenance 0.02 for large pumps.

TABLE XVI

COST ESTIMATE

RIO SECO DAM AND IRRIGATION OUTLET

ITEM	QUANTITY	UNIT	UNIT PRICE	TOTAL COST DOLLARS
DAM				
Acquisition	LS			1,000
Clearing	LS			1,000
Excavation	65,000	yd^3	0.45	29,300
Pervious Fill	145,000	yd^3	0.08	11,600
Impervious Fill	350,000	yd^3	0.10	35,000
Borrow	400,000	yd^3	0.80	320,000
Rip-rap	10,000	yd^3	10.00	100,000
Toe Drain	LS			1,000
OUTLET, STILLING BASIN AND GATE WELL				
Reinforced Concrete	1,500	yd^3	100.00	150,000
Gates	120	ft^2	325.00	39,000
Gravel and Drains	LS			1,000
Rip-rap	150	yd^3	10.00	15,000
SPILLWAY				
Reinforced Concrete	1,500	yd^3	100.00	150,000
Gravel and Drains	LS			1,000
Rip-rap	150	yd^3	10.00	15,000
TOTAL COST				$869,000
Engineering, contingencies and interest during construction				260,970
GRAND TOTAL				$1,130,870

CHAPTER V

CANALS AND LATERALS

5-1 Main Canal

The natural topographic features of the proposed development area, particularly the range of hills running from the Rio Saska to the irrigation district allows for the investigation of a wide range of canal slopes with only a few miles variation in total length of canal.

Canal alignments above the 1,010 MSL contour line must wind rather tortuously to traverse two tributaries of the Rio LaSalle and circumvent a prominent nose created by these streams. Alternatively, the canal alignment could cross the streams in elevated flumes or via an inverted syphon and could cut through the intervening nose in a tunnel or deep cut section.

Unlined, clay-lined and concrete-lined canal sections were investigated. The unlined canal would be the least costly alternative for a given discharge and slope but would be subject to high seepage losses. The selection of canal type, size and slope is an economic problem and is considered in Chapter VII in combination with the two storage dams and pumping.

The material along the canal alignment is a clayey sand and gravel similar to that encountered at the dam sites. Open cuts of 2-to-1 would be stable in this material and with rigid concrete lining slopes as steep as 1-to-1 could be adopted. The material would be suitable for concrete aggregate after screening and washing.

Surface rock is available along the canal route and several suitable quarry sites have been located with sufficient quantities of clean rock for concrete aggregate.

Typical canal cross sections are shown in Figure 23. It has been assumed that the final alignment of the canal would be designed so that cut and fill quantities would balance. The downstream fill bank has been designed to accommodate single lane traffic for operation and maintenance. Canal capacities analyzed for full development, with an allowance for seepage, were, (1) demand supply requiring a maximum discharge of 3,870 cfs, (2) 10-month steady discharge of 3,115 cfs and (3) 12-month steady discharge of 2,600 cfs, the latter two would require a regulation reservoir.

5-1-1 Cost Estimates

The canal capacity calculations were based on the Manning equation for discharge. Roughness coefficients of 0.015 for concrete-lined canals and 0.030 for clay-lined and unlined canals were adopted as working values. Relationships for size versus cost per mile of canal construction and lining and cost per mile for a range of discharges and canal slopes have been established. Unit cost figures for common excavation, fill and concrete were obtained from Figures 9 to 11 inclusive. The relationships are shown in Figures 24 to 28 inclusive. An example calculation for an unlined canal, capacity 2,950 cfs and slope 0.0001 is shown in Table XVIII.

5-2 Canal Structures

A rectangular reinforced concrete conduit in the right abutment of the dam on the Rio Saska was chosen for the canal intake structure. The configuration of the structure is identical to that of the conduit on the proposed Rio Seco dam as shown in Figure 20. Intake trash racks, roller gates and a stilling basin with concrete guide walls would be provided for exclusion of floating debris, flow regulation and energy dissipation respectively.

Canal flows discharging into the Rio Seco reservoir would pass over a series of simple energy dissipators or drop structures as shown in Figure 32. These structures would be provided to restrict the velocity of inflows and minimize bank erosion.

Cost comparisons between elevated flumes,inverted syphons and contour canals were carried out. In spite of the considerable decrease in canal length by utilizing flumes and syphons, these structures were not competitive primarily due to the high capital cost of the necessary concrete works. Cost comparisons are shown in Chapter VII.

Cost comparisons between tunnels, open cut and contour canals were also carried out. Again, in spite of the decrease in canal length by straightening the alignment, the tunnel and open cut alternatives were not competitive. The high capital cost of tunnel lining and the excavation quantities and stability problems to be expected with deep open cuts were the governing cost factors. Cost comparisons are shown in Chapter VII.

5-3 Distribution Canals and Laterals

The general flatness of the proposed irrigation district limits the available slope for supply canals. This necessitates rather wide cross sections to convey irrigation water at non-scouring velocities.

Unlined, clay-lined and concrete-lined canals were considered. Unlined canals would be the least costly alternative and seepage losses would be well within the 20 percent allowance for losses as the soil is generally of medium to low permeability compared with the material in which the main canal would be excavated.

The material in which the canals must be excavated is stable at 1 1/2-to-1 side slopes and has a disturbed hydraulic conductivity averaging 1×10^{-5} cm per second at depths of six feet.

The distribution canals and laterals have been laid out primarily to take advantage of the natural slope of the ground within the Irrigation District. Property boundaries follow the system in which the area is sub-divided into one square mile sections and further sub-division of these sections are quarter or half sections on a quarter mile grid basis. Road allowances are provided every mile, coincident with property boundaries, and where possible these road allowances have been utilized for main distributaries and branch laterals. Egyptian practise, Leliavski (1965), is to locate canals no further than 2.5 kilometers or 1.35 miles apart. This practise is followed in order to limit the number of private interests centering on one canal. Inasmuch as the average farm size in the Irrigation District is large, averaging 320 acres, both of the above requirements can be adhered to with a canal spacing of 2 miles. As stated, the canals have been laid out primarily according to the available topographic features and property lines. Consideration was also given to minimizing the total length of canal and to avoiding main road and river crossings where possible. Crossing of farm service roads at frequent intervals was unavoidable and provision has been made in the estimates for bridges, syphons and standard roadway culvert underpasses.

A horizontal radius of curvature of canals of from 10-to-15 times the bottom width of the canal was adopted to ensure that head losses in bends could be kept to a minimum. The curvature of the canals is not visibly apparent in Figure 34 but the minimum radii of curvature recommended would be as follows:

Discharge in cfs	Radius in feet
3000-1000	3,000
1000- 500	2,000
500- 100	1,000
100- 10	500
Less than 10	300

The water level in the feeder canals, or the canal from which the farmer would obtain irrigation flows directly, has been set at 10 inches above the existing ground level. This difference in elevation would enable the farmer to obtain water by gravity, utilizing one of the methods illustrated in Figure 32, thus avoiding forced lifts by pumping and the inherent annual costs which pumping entails.

Each feeder canal would be equipped with a head sluice gate and flow measuring device at the take off from the branch canal. The recommended flow measuring device is the Parshall flume. These measuring devices would also be spaced every 2 miles along the canals. The Parshall type flume, shown schematically in Figure 32 is recommended as head losses are less than those over weirs, an important consideration where natural slopes are flat, and as the flume is sturdy and simple to operate and maintain. Each canal would also be equipped with a tail escape or a weir check structure at the extreme end of the canal. This structure would be operated to raise water levels in the canal and to release excess flows into the tail drain. As drains would be excavated to much greater depths below the ground surface, the tail escape must be provided for all branch canals.

Canal cross sections have been designed for the water releases required by the cultivators plus an additional 60 percent of this discharge so that flows that have not been utilized by upstream cultivators can be conveyed in the canal without endangering adjacent lands from overbank flows. This additional discharge capacity is called escaping power, Leliavski (1965).

5-3-1 Cost Estimates

Canal capacities have been based on the Manning formula with an average slope of 0.0004 and a roughness

TABLE XVII

TOTAL MILEAGE & COST OF DISTRIBUTARIES
FOR FULL DEVELOPMENT

Discharge Range cfs	Subzone I	Subzone II	Subzone III	Total miles	Capital Cost Dollars Per Mile	Capital Cost Dollars Total millions
0- 60	28	29	50	107	2,150	0.23
60- 125	11	-	33	44	3,850	0.17
125- 225	-	25	-	25	5,400	0.14
225- 350	19	10	11	40	7,700	0.31
350- 800	5	5	2	12	10,550	0.13
800-1500	21	18	-	39	19,000	0.74
	TOTAL COST					1.82
	Main Distributaries, Full Development					
3,000				13	90,000	1.18
2,500				4	73,000	0.29
1,600				6	45,000	0.27
	TOTAL COST					1.64
	GRAND TOTAL					$3.46

Analysis of various partial developments indicate an average cost of $10 per acre for distribution canals.

TABLE XVIII

COST ESTIMATE FOR UNLINED CANAL

Capacity 2,950 cfs   Slope 0.0001   Dimension H = 20 feet

ITEM	QUANTITY	UNIT	UNIT PRICE	TOTAL COST DOLLARS
Acquisition	14	acres	$ 50.	700
Clear and Grub	14	acres	200.	2,800
Excavation	210,000	yd^3	0.25	52,500
Compacted Fill	85,000	yd^3	0.10	8,500
Common Fill	125,000	yd^3	0.07	8,750
TOTAL COST				73,250
Engineering and contingencies and interest during construction				22,000
TOTAL COST PER MILE				$95,250

NOTE: Calculations of canal dimension H is shown in Table XX.

CHAPTER VI

DRAINAGE

6-1 Main Surface Drains

Existing drainage is insufficient to cope with the runoff from heavy rain storms. The groundwater level in the basin is at an average depth of 5 feet below the surface during the dry season. It rises to within 3-4 feet of the surface in the upper subzones and to within 1 foot of the surface in the lower reaches of subzone III in the wet season. With additional irrigation water supplies, drainage problems would be increased and further rising of the water table into the root zone could be expected. High water tables contribute to soil salinity due to increased surface evaporation and prohibit proper plant development and maturity.

The drainage system was laid out to remove irrigation operational water via tail escapes, to remove and control storm runoff and control groundwater levels by main drainage canals. The system pattern was laid out, as for the distribution canals, to take full advantage of the natural topography, property lines and existing streams, thereby providing the most economical drainage. The layout of drains was influenced by the layout of supply canals since gravity ditch irrigation flows should be toward a drain so that excess flows will not collect and waterlog the arable land.

Drain spacing is generally 2 miles center to center, providing a maximum distance of flow to a drain of 1 mile from the corresponding distributary. Crossing of farm service roads would be accomplished by the installation of standard corrugated metal pipe crossings as shown in Figure 31.

Head loss is not a governing factor for drainage canals. Radii in bends should not be less than 100 feet, however, in order to minimize erosion of canal banks.

A standard drain canal cross section as utilized by the United States Bureau of Reclamation (1968) has been adopted and is shown in Figure 31. The water level in the canal, when flowing full, should be a minimum of two feet below the surrounding fields. Normal flows would be approximately five feet below the surface to assist in groundwater control.

Drainage canals have not been provided with control structures unless discharges into streams would be considerably above natural stream beds, in which case energy dissipators would be provided.

Drain capacities were designed according to an empirical formula obtained from Schwab (1966) for areas south of 37° north latitude with improved pasture crops. The required rate of water removal has been determined as follows.

$$Q = kM^x$$

where Q = rate of removal in cfs

M = drainage area in square miles

k = constant

x = exponent

The value of the constant k is 25 and for the exponent x is 0.83, therefore, for the irrigation district, where the average length is five miles and the width of the area to be drained is two miles, the canal drainage capacity would be:

$$Q = 25 \times (10 \text{ miles}^2)^{0.83} = 168 \text{ cfs}$$

The cross-sectional area of the standard drain section is 125 square feet and for slopes in excess of 0.0004, the canal capacity is adequate for canal roughness up to 0.050 in the Manning equation.

6-1-1 Cost Estimates

The cost of drainage canals is $6,250 per mile, based on costs of common excavation of $0.25 per cubic yard. The total length of drain canals would be 180 miles for a total capital cost of $1,110,000 for full development. The unit costs for canal drains would be $5.00 per acre for subzone III which must have extensive drainage and $3.00 per acre for subzone I and subzone II.

6-2 Distribution System Drainage

As discussed in the subsection on distribution canals, tail escape drains must be provided at the terminus of all distributaries to convey unused irrigation water to main drainage canals or existing stream courses.

The drain capacity should be at least equal to the design capacity of the distribution canal immediately upstream of the weir check structure. Canal depths would be a minimum of six feet below the ground surface as for main drains and the bottom width equal to the bottom width of the upstream distributary. Steep slopes can be tolerated as weed growth would limit discharge velocities. In general the slope of tail escape drains would be limited by the elevations of the upstream distributary and the downstream collector. Manning's roughness coefficients of the order of .060 to 0.10 due to weed growth could be expected.

An average tailrace drain cross-section is shown in Figure 31. Canal slopes of 0.001 have been assumed and a Manning's roughness of 0.060 has been utilized for moderately well maintained canals.

6-2-1 Cost Estimates

The cost of tailrace drains, for the average section, would be $3,000 per mile, based on common excavation costs of $0.25 per cubic yard. The total length of these drains would be 50 miles for a total capital cost of $150,000 for full development. The unit cost for tailrace drains would be $0.50 per acre.

CHAPTER VII

COST ANALYSIS

7-1 Summary

Several alternatives were studied for delivering irrigation water to the Irrigation District. These alternatives were compared to find the least costly combination of civil works for complete and partial development of the District.

The alternatives studied for Scheme I, 285,000 developed acres, were as follows:

(1) Storage dam on the Rio Saska and gravity canal to deliver water requirements on demand. The canal would deliver a maximum discharge of 3,870 cfs.

(2) Storage dam on the Rio Saska, gravity canal and regulation reservoir on the Rio Seco. The canal would deliver continuous flows of 3,115 cfs for a ten-month period.

(3) Storage dam on the Rio Saska, gravity canal and regulation reservoir on the Rio Seco. The canal would deliver continuous flows of 2,600 cfs for a twelve-month period.

(4) Storage dam and lift station on the Rio Saska in lieu of the storage dam only for the least costly of the above alternatives.

All alternatives were investigated for unlined, clay-lined and concrete-lined canal sections.

Partial development was studied for that alternative that proved least costly for the full development. Canal capacities used in this analysis were for the development of subzone I only, Scheme III; subzones I and II combined, Scheme II; and left and right bank sections of these combinations of subzones.

7-2 Method of Analysis

The approach to the comparative cost analysis for supply from the Rio Saska was to determine the maximum terminal elevation to which gravity canal flows would be delivered. This elevation would be the full supply level of the terminal reservoir for the storage volume required in alternatives (2), (3) and (4) and the highest point of the Irrigation District for alternatives (1) and (4) without the Rio Seco reservoir. This elevation corresponds to the theoretical minimum elevation to which the head water at the Rio Saska must be raised to supply water to the District. The head of water required to convey water by gravity along the canal and the range in supply levels in the reservoir for storage capacity would then be added to this elevation and the least cost of canal storage above this elevation can be determined.

Due to seepage losses in the canal, the canal capacity must be increased so that the delivery requirements to the Irrigation District or regulation reservoir can be met. The additional discharge required also necessitates an increase in storage capacity at the supply reservoir.

7-2-1 Canal Seepage Losses

The seepage losses for the range of discharges have been calculated from the Moritz (1952) formula. As a first approximation a maximum allowable velocity of 5 fps has been assumed. An example calculation of seepage losses for an earth-lined canal and a summary of diversion requirements, seepage losses and average canal capacity for a delivery requirement of 3,870 cfs are shown below. The canal size would vary from a maximum capacity of delivery requirements plus seepage losses at the canal intake, to a minimum capacity of delivery requirements only at the terminal end of the canal.

$$\text{Losses (cfs)} = 0.2 \times 1 \times 87 \times (3870/5)^{1/2} = 485 \text{ cfs.}$$

		Unlined	Clay-Lined	Concrete-Lined
Delivery Requirement	cfs	3,870	3,870	3,870
Seepage Losses	cfs	485	87	5
Diversion Requirement	cfs	4,355	3,957	3,875
Average Canal Capacity	cfs	4,112	3,912	3,872
Storage Required	sfm	18,000	15,000	15,000

The values for alternatives 2 and 3 may be obtained by inspecting Figures 41 and 42 respectively.

7-3 Comparison of Alternatives

Alternative (1). The elevation of the Irrigation District at which canal discharges would be delivered is 950 feet MSL. This would be the minimum elevation of the lower supply level in the main reservoir. A contour canal at this elevation would be 87 miles long and canal head losses would be approximately 85 feet for an assumed maximum canal slope of 0.0002. The least costly combination of dam height and canal slope on an annual cost basis can now be determined by summing the respective annual costs for all combinations in this range.

Capital cost figures were obtained from Figure 3 and Figures 24 and 25 for a range of dam heights and canal slopes. These figures were then converted to annual costs according to the appropriate recommended analysis parameters.

The storage requirements at the Rio Saska for the required irregular flow demands have been established from a mass curve analysis of river flows and discharge requirements as shown in Figures 34 to 39 inclusive. The critical period of low river flows in the existing records was from October 1962 to June 1965. Allowance has been made for riparian releases of 1,000 cfs. The range of reservoir operation to obtain storage was obtained from the stage storage curve in Figure 3.

Alternative (2). The full supply level of the distribution reservoir to supply regulation storage for irrigation water supply from a ten-month steady canal discharge of 3,115 cfs is 995 feet MSL. A contour canal at this elevation would be 83 miles long and canal head losses would be approximately 80 feet for an assumed maximum canal slope of 0.0002.

The analysis of this alternative from this point is equivalent to that for Alternative (1) except that the storage-dependable flow curve for the Rio Saska was used to determine required storage rather than a mass curve analysis. This approximation was considered valid for cost comparisons. Further refinements in the analysis could be carried out if cost comparisons did not prove to be conclusive for project selection.

The cost of the Rio Seco distribution reservoir is constant for all combinations of storage reservoir and canal slope for this alternative.

Alternative (3). The full supply level of the distribution reservoir to supply regulation storage for irrigation water supply from a 12-month steady canal discharge of 2,600 cfs is 1,015 feet MSL. A contour canal at this elevation would be 82 miles long and canal head losses could be approximately 80 feet for an assumed maximum slope of 0.0002.

The analysis of this alternative was similar to Alternative (2).

Based on Figures 40, 41 and 42 the least costly alternative would be an unlined canal discharging demand flows directly to the district. The canal slope would be 0.00015, the canal intake elevation 1,010 feet MSL and the reservoir FSL would be 1,055 feet MSL. The unlined canal slope of 0.0001, intake elevation 1,055 feet MSL and reservoir FSL of 1,080 feet MSL is competitive and these two alternatives have been compared in detail with the conclusion that the inclusion of the distribution reservoir is uneconomical for full development of a single-purpose irrigation project.

Alternative (4). The most economical canal slope, in combination with a storage range of 1,010 to 1,055 feet MSL was 0.00015. The least costly combination of storage dam and pumping can now be determined by combining these costs over a range of head from 950 feet MSL, the elevation of the district and 1,055 feet MSL, the FSL of the dam. Pumping costs have been obtained from Figure 11. The combinations of costs are shown in Figure 43. From this figure it is found that pumping is uneconomical over the full range of total dynamic head for an average annual discharge of 2,600 cfs. Pump supply was not considered for the remainder of the analysis due to the high annual costs involved.

7-4 <u>Main Canal Structures</u>

In the vicinity of the La Salle River, it would be possible to shorten the main canal by (1) constructing elevated flumes over the two wide valleys of the La Salle tributaries (2) constructing inverted syphons in the two valleys, and (3) tunnelling or (4) open cut excavation in the prominent nose separating the two valleys or (5) by a combination of two or more of the above.

The total length of contour canal would be 13 miles whereas the total length of flume and tunnelling or open-cut excavation would be 6.5 miles. The lengths of the individual structures are tabulated below. The two tributaries have been labelled North and South.

	LENGTH OF AQUEDUCT IN MILES				Length of Contour Canal
	Flume	Syphon	Tunnel	Open-cut	
North Tributary	2.5	2.55			3.5
Nose			2.0	2.0	4.0
South Tributary	2.0	2.05			5.5

Figure 44 gives combinations of flume slope and dimension H for flume velocities below critical for the flume shape shown schematically in the same figure. A dimension of 12 feet and slope of 0.001 have been selected for an assumed canal discharge of 3,000 cfs. The thickness of the reinforced concrete walls and invert would average one foot.

A fully-lined tunnel 10 feet in diameter with inlet control would be adequate for the design discharge of 3,000 cfs. Tunnel excavation costs have been obtained from Figure 12.

This alternative has been considered to be at least as costly as the concrete flume and could be analyzed in detail if the flume proved to be economical. Concrete pipe, 6 feet in diameter and 8 inches thick, would be adequate.

A rather deep excavation would require extensive slope protection measures to ensure slope stability and would include costs of canal plus additional excavation and was not considered further in this analysis.

7-4-1 Cost Estimates

The ruling factor in first comparisons of the alternatives was assumed to be the cost of concrete for the structures. It was decided to estimate the cost of concrete per mile for each of the flumes and the tunnel and to compare the total cost of concrete works versus the cost of the longer main canal. If the shortened route did not prove economical on this basis, then further refinement of the calculations would not be necessary. The estimates for the first comparison have been given in Table XXI.

7-5 Partial Development

The water requirements during the peak demand month would be 3,870 SFM divided by the total developed acreage of 285,000 acres, or 0.013 SFM per acre. The peak demand and size of the various partial development possibilities would be as follows:

	Area in Acres		
Description	Left Bank	Right Bank	Total
Subzone I	22,800	73,000	95,000
Subzone I plus II	45,000	145,000	190,000
	Peak Demand - SFM		
Subzone I	285	950	1,235
Subzone I plus II	585	1,890	2,475

7-5-1 Cost Estimates

The cost estimates for water supply for the partial developments were calculated in the same manner as estimates for full development but only for the demand supply and unlined canal alternative. The costs are summarized in Tables XXII and XXIII.

7-6 Land Shaping and Levelling Costs

In order to distribute irrigation water by gravity, it would be necessary for individual cultivators to excavate private canals and level their acreage. A cost for land shaping and levelling of \$20 per acre for subzone I, \$24 per acre for subzone II and \$21 per acre for subzone III have been used for estimating purposes. The additional capital cost of irrigation to farmers, assuming the Provincial Government would pay for the major hydraulic works, drains and distributaries would be as follows:

	Acreage	Without Irrigation	With Irrigation	Increased Costs
Subzone I	640	$ 20,700	$ 34,000	$ 13,300
Subzone II	320	10,700	18,800	8,100
Subzone III	160	7,600	12,000	3,400

7-7 Annual Costs

Annual costs would be the sum of operation and maintenance of the entire civil works, excluding private canals and drains, plus interest and depreciation on the total capital investment. Annual costs have been based on an assumed life of project of 50 years and an interest rate of six percent.

7-8 Interest and Depreciation

Annual interest and depreciation costs have been based on the useful life and interest rate figures as supplied for irrigation projects. It has been assumed that development of the irrigation district would take place over a period of ten years. Initially it would be expected that 28,500 acres would be available for irrigation when the main hydraulic works have been completed. Additional acreage would be added annually so that at the end of ten years all 285,000 acres would be under irrigation.

Interest and depreciation on the dam and main canal would be paid by the sinking fund method of debt retirement over the 50-year life of the project. On the irrigation works the annual cost would vary from a minimum of $1,961,565 at the inception of the project to a maximum of $2,475,300 after ten years and thereafter would remain constant for the remaining useful life of the project of 40 years.

Constant annual cost calculations are shown in Table XXV for Scheme I.

7-9 Operation and Maintenance

Annual operation and maintenance costs for the project have been based on the percentage of capital cost figures supplied for irrigation projects. The projected development period for the project would be ten years, acreage being brought under irrigation at a rate of approximately 28,500 acres per year. During the development period therefore, the cost of operation and maintenance would be expected to increase in proportion to the developed acreage for each year. It has however been assumed that operation and maintenance would be a fixed annual cost from the date of inception of the project. This assumption is justified considering that the major portion of the annual costs would be required to maintain the major civil works, a prerequisite to irrigation. Possible operation problems in the early stages of the new development could also increase costs out of proportion to the acreage under development.

The annual cost has been calculated for the full development of 285,000 acres as follows; annual cost per acre would be \$1.33 or approximately 1 percent of the capital cost of construction. This percent figure has been used for calculating operation and maintenance costs for all partial developments. Cost calculations are shown in Table XXVI for Scheme I.

TABLE XIX

COST ESTIMATE FOR

STORAGE DAM, UNLINED CANAL

Storage Dam, Full Supply Level 1,055 feet MSL

Minimum Supply Level 1,010 feet MSL

Annual Cost	$1,400,000

Unlined Canal, Slope 0.00015, Capacity 4,100 cfs

Canal Dimensions

$$Q = \frac{1.5R^{2/3} S^{1/2}}{0.030} A$$

$$A = 3H^2$$

$$R = 0.55H$$

$$4100 = \frac{1.5}{0.030} \times (0.55H)^{2/3} \times (0.00015)^{1/2} \times 3H^2$$

$$H = 21 \text{ feet}$$

Capital Cost

$$\frac{\$105,000}{\text{Mile}} \times 78 \text{ Miles} = \$8,200,000$$

Annual Cost

Interest & Depreciation @ 0.0634 =	$ 520,000
Operation & Maintenance @ 0.02 =	$ 164,000
TOTAL ANNUAL COST	$2,084,000

TABLE XX

COST ESTIMATE FOR

STORAGE DAM, UNLINED CANAL & DISTRIBUTION RESERVOIR

Storage Dam, Full Supply Level 1,080 feet MSL

Minimum Supply Level 1,055 feet MSL

Annual Cost $1,550,000

Unlined Canal, Slope 0.0001, Capacity 2,950 cfs

Canan Dimensions

$$Q = \frac{1.5R^{2/3}}{0.030} S^{1/2} A$$

$$2950 = \frac{1.5}{0.030} \times (0.55H)^{2/3} \times (0.0001)^{1/2} \times 3H^2$$

$$H = 20 \text{ feet}$$

Capital Cost

$\frac{\$95,250}{\text{Mile}} \times 75 \text{ Miles} = \$7,150,000$

Annual Cost

Interest & Depreciation @ 0.0634	$ 453,000
Operation & Maintenance @ 0.02	$ 143,000

Distribution Reservoir

Capital Cost $1,130,370

Annual Cost

Interest & Depreciation @ 0.0634	$ 72,000
Operation & Maintenance @ 0.001	$ 1,130

TOTAL ANNUAL COST $2,219,130

TABLE XXI

COST ESTIMATE FOR

CONTOUR CANAL, FLUMES & TUNNEL

Contour Canal			
	Unlined	13 miles x \$ 78,000	\$ 1,000,400
	Lined	13 miles x \$130,000	1,690,000
	Concrete-lined	13 miles x \$180,000	2,340,000
Flume			
	Cost per mile	15,000 yds^3 x \$100.00	\$ 1,500,000
	Total cost	4.5 miles x \$1,500,000	\$ 6,750,000
Tunnel			
	Cost per mile	20,000 yds^3 x \$100.00	\$ 2,000,000
	Total cost	2.0 miles x \$2,000,000	\$ 4,000,000
TOTAL COST STRUCTURES			\$11,750,000
TOTAL COST CANAL			\$ 2,340,000

Canal - Unit cost for the canals were obtained from Figure 24 for a discharge of 3,000 cfs and a canal slope of 0.0002.

TABLE XXII

COST ESTIMATE
FOR
SCHEME I - 285,000 PRODUCTIVE ACRES

Storage Dam At Rio Saska		
(Includes engineering, contingencies and interest during construction)		$ 21,800,000
Unlined Canal		
(Includes engineering, contingencies and interest during construction)		$ 8,620,000
Canal Structures		
(Includes engineering, contingencies and interest during construction)		
Intake	$ 205,000	
Main Regulator	81,900	
Regulators and Waste Ways	213,100	$ 500,000
Irrigation Works		
Main Supply Canals	$1,640,000	
Distributaries	1,820,000	
Canal Drains	1,110,000	
Distribution System Drains	150,000	
Structures; Weirs, Flumes, Road Crossings	1,225,000	
	$5,945,000	
Engineering, contingencies and Interest During Construction 30%	$1,785,000	$ 7,730,000
TOTAL COST		$ 38,650,000

TABLE XXIII

COST ESTIMATE
FOR
SCHEME II - 190,000 PRODUCTIVE ACRES

Storage Dam At Rio Saska		
(Includes engineering, contingencies and interest during construction)		$ 20,800,000
Unlined Canal		
(Includes engineering, contingencies and interest during construction)		$ 5,510,000
Canal Structures		
(Includes engineering, contingencies and interest during construction)		
Intake	$ 150,000	
Main Regulator	60,500	
Regulators and Waste Ways	139,500	$ 350,000
Irrigation Works		
Main Supply Canals	$1,050,000	
Distributaries	850,000	
Canal Drains	570,000	
Distribution System Drains	95,000	
Structures; Weirs, Flumes, Road Crossings	950,000	
	$3,515,000	
Engineering, contingencies and Interest During Construction 30%	$1,054,500	$ 4,569,500
TOTAL COST		$ 31,229,500

TABLE XXIV

COST ESTIMATE
FOR
SCHEME III - 95,000 PRODUCTIVE ACRES

Storage Dam At Rio Saska		
(Includes engineering, contingencies and interest during construction)		$20,200,000
Unlined Canal		
(Includes engineering, contingencies and interest during construction)		$ 4,500,000
Canal Structures		
(Includes engineering, contingencies and interest during construction)		
Intake	$ 100,000	
Main Regulator	40,000	
Regulators and Waste Ways	110,000	$ 250,000
Irrigation Works		
Main Supply Canals	$ 600,000	
Distributaries	350,000	
Canal Drains	275,000	
Distribution System Drains	50,000	
Structures; Weirs, Flumes, Road Crossings	475,000	
	$1,750,000	
Engineering, contingencies and Interest During Construction 30%	$ 525,000	$ 2,275,000
TOTAL COST		$27,225,000

TABLE XXV

ANNUAL COST OF INTEREST AND DEPRECIATION

SCHEME I - 285,000 PRODUCTIVE ACRES

	Capital Cost	Interest & Depreciation Annual Cost Percent	Interest & Depreciation Annual Cost Dollars
Storage Dam at Rio Saska	$21,800,000	0.06344	1,382,992
Main Canal	8,620,000	0.06344	546,853
Canal Structures	500,000	0.06344	31,720
Irrigation Works			
Canals and Drains	6,136,000	0.06646	407,798
Structures	1,594,000	0.06646	105,947
TOTAL ANNUAL COST			2,475,300

ANNUAL COST PER ACRE $\frac{2,475,300}{285,000}$ = \$ 8.61

ANNUAL COST AS A PERCENTAGE OF CAPITAL COST $\frac{2,475,300}{38,150,000} \times 100$ = 0.06487

TABLE XXVI

ANNUAL COST OF OPERATION & MAINTENANCE

SCHEME I - 285,000 PRODUCTIVE ACRES

		Operation & Maintenance Annual Cost	
	Capital Cost	Percent	Dollars
Storage Dam at Rio Saska	$ 21,800,000	0.1	21,800
Main Canal	8,620,000	2.0	172,400
Canal Structures	500,000	3.0	15,000
Irrigation Works			
Canals and Drains	6,136,000	2.0	122,720
Structures	1,594,000	3.0	47,820
TOTAL ANNUAL COST			379,740

ANNUAL COST PER ACRE $\frac{379,740}{285,000} =$ \$ 1.33

ANNUAL COST AS A PERCENTAGE OF CAPITAL COST $\frac{379,740}{38,150,000} \times 100 =$ 1 %

CHAPTER VIII

BENEFITS

8-1 Annual Revenues

The annual value of the benefits of irrigation can be determined directly from the farm budget by calculating the difference between net income per acre without irrigation and net income per acre with irrigation. These figures are shown below for the three subzones.

	NET INCOME PER ACRE		Increased Income or Net Benefits
	Without Irrigation	With Irrigation	
Subzone I	$ 11.40	$ 35.00	$ 23.60
Subzone II	10.31	30.00	19.69
Subzone III	8.75	22.00	13.25
TOTAL FOR ALL THREE SUBZONES			$ 56.54
AVERAGE NET BENEFIT PER ACRE			$ 18.85

The exact net benefit figures for the individual subzones have been used for calculation of benefits for partial development. It has been assumed that phasing of full development would be such that portions of all three subzones would be developed simultaneously. Actual development would depend upon applications from farmers for irrigation water, but the recommended method of construction staging would be to complete one main distributary, service all farms from this distributary and then commence work on the next distributary etc. It has been assumed that this procedure of development would be adhered to and therefore the average net benefit per acre figure as above has been used for stages of full development.

Estimates of irrigable acres, initial revenues, development period and final revenues from irrigation for full and partial development would be as follows:

	Irrigable Acreage	Develop. Period years	Annual Benefits Initial	Annual Benefits Final
SCHEME I	285,000	10	$537,130	$5,371,000
Subzones I & II				
Left Bank	45,000	2	$486,500	$ 973,000
Right Bank	145,000	5	628,000	3,140,000
SCHEME II TOTAL	190,000	7	$589,000	$4,113,000
Subzone I				
Left Bank	22,000	1	$520,000	$ 520,000
Right Bank	73,000	3	575,000	1,720,000
SCHEME III TOTAL	95,000	4	$560,000	$2,240,000

It has been assumed that for all developments, a total acreage of approximately 28,500 acres, would be brought under irrigation each year.

8-2 Annual Income of Individual Farmers

The annual income of individual farmers, with and without irrigation could be as follows:

		Without Irrigation		With Irrigation	
	Acreage	Per Acre	Total	Per Acre	Total
Subzone I	640	$11.40	$7,300	$35.00	$22,400
Subzone II	320	$10.31	$3,100	$30.00	$ 9,600
Subzone III	160	$ 8.75	$1,400	$22.00	$ 3,520

8-3 Water Charges

Three alternatives for payment of water charges have been analyzed, (1) payment for operation and maintenance of all works on a per acre basis, (2) payment on the basis of a fixed sum per acre-foot of water used, considering the value of water used, and (3) sliding scale payment for water based on quantity of water used.

(1) The annual operation and maintenance costs would be $1.33 per acre. The revised benefit-cost ratios with irrigation on a per-farm basis would be:

	Annual Revenue	Annual Cost			B/C
		Expenses	O & M	Total	
Subzone I	$56,400	$34,000	$850	$34,850	1.61
Subzone II	28,400	18,800	425	19,225	1.48
Subzone III	14,520	11,000	212	11,212	1.30

(2) The value of water, per acre-foot, based on net benefits of irrigation of the project would be $1.25. The revised benefit-cost ratio would be substantially the same as the figures in the above table.

(3) The suggested sliding scale for water charges per acre-foot would assess the farmers in such a way as to equalize the benefit-cost ratio for the farmers in each of the three subzones. Suggested rates would be $0.50 for the first 500 acre-feet, $1.00 for the next 500 acre-feet and $2.00 thereafter. The revised benefit-cost ratios would be as follows:

	Annual Revenue	Annual Cost			B/C
		Expenses	Water	Total	
Subzone I	$56,400	$34,000	$1,380	$35,380	1.57
Subzone II	28,400	12,800	710	19,510	1.45
Subzone III	14,520	11,000	240	11,240	1.29

Alternative (3) would be the most acceptable of the above as it would tend to equalize the individual B/C ratios. However, the high cost of water for withdrawals over 1,000 acre-feet per year might act as a deterrent to development and payment for water by volume would require installation of extensive measuring devices and therefore greater operational costs.

A fourth alternative would be to collect water charges on a per-acre basis according to the farmers ability to pay. The charges calculated in Table XXVII using land classes and value of water as the criteria, are recommended. The analysis is based on Standard Practice adopted by the International Pembina River Engineering Board (1964).

8-4 Value of Water to Farmers

The average irrigation demand is three feet per acre over the entire basin, the value of irrigation water to the farmer would be the increase in income per annum divided by the total number of acre-feet of water utilized. The value of water would be as follows:

	Annual Income			Total Water Rqmnts. Acre-Feet	Value of Water $/Acre-Foot
	Without Irrigation	With Irrigation	Increase		
Subzone I	$ 7,300	$ 22,400	$15,100	1,920	9.86
Subzone II	3,300	9,600	6,300	960	6.65
Subzone III	1,400	3,520	2,120	450	4.43

TABLE XXVII

ANALYSIS OF PAYMENT CAPACITY BY LAND CLASS

Land Class	1	2	3
Subzone	I	II	III
Net Farm Income	$22,400	$9,600	$3,520
Less Dryland Income	7,300	3,300	1,400
	15,100	6,300	2,120
Less 10 percent of Increase in Income	1,510	630	212
	13,590	5,670	1,908
Less Increase in Family Labor	-	1,000	1,000
	13,590	4,670	908
Reduce by 20 percent Contingencies	2,718	934	182
	10,872	3,736	716
Per Acre Payment Capacity	$ 17.00	$ 10.40	$ 4.50

CHAPTER IX

ECONOMIC ANALYSIS

9-1 General

The preceding engineering analysis has concentrated on the feasibility of constructing civil engineering works for the provision of irrigation water to the proposed Irrigation District. The economic analysis which follows concentrates on the economic feasibility of the irrigation project. For the purposes of this report it has been assumed that market demands will continue to increase for all produce from increased agricultural production and at current prices. Farm budget figures, as supplied by the Provincial Government, have been accepted as valid for these assumptions and present credit availability and interest rates have been assumed to be indicative of future conditions.

9-2 Benefit-Cost Analysis

Benefit and cost figures from the preceding Chapters have been used to calculate benefit-cost ratios. As the length of the period of development is a significant factor in determining these ratios, calculations have been carried out to demonstrate the value and disadvantage of more and less rapid periods of development respectively. The period of development used in the example calculations is 10 years.

Comparisons have been made on the basis of present values of benefits and costs, as the annual values of these parameters vary over the useful life of the project.

9-3 Benefits of Scheme I

Benefits would be realized from scheme I one year after completion of the main hydraulic works. The initial annual benefits would amount to $537,130, increasing in equal increments over a ten year period to an annual value of $5,371,300. This value would then remain more or less constant for the remaining 40 years of useful life of the project.

The present value of these benefits have been determined by converting the 10-year gradient series of benefits to equal annual amounts and determining the present value of this annuity. The 40 years of constant benefits have been converted to a lump sum in year nine and then into present value as follows; all costs are in dollars.

A. Gradient series conversion

Value of benefits Year 1		$537,130
Annual increment		537,130
Value of benefits Year 10		5,371,000
Uniform value	537,130 x 4.02 =	2,160,000
Present value at January 1, Year 0	2,160,000 x 7.36=	$15,900,000

B. Equal annual benefits to present value

Value of benefits Year 11		$ 5,371,300
Value of benefits Year 50		5,371,300
Present value at January 1, Year 10	5,371,000 x 15.046	80,700,000
Present value at project beginning	80,700,000 x 0.5584	$45,000,000

C. Total present value of all benefits at beginning of project $60,900,000

9-4 Costs of Scheme I

Costs would be the capital cost of the hydraulic works at the beginning of the project of $30,920,000, the initial cost of $51,375 for distributaries and an additional $51,375 annually for 10 years to an annual value of $513,375. This value remains constant for the remaining 40 years of useful life of the project. In addition the regular annual costs of operation and maintenance must be included.

The present value of these costs have been calculated by converting the 10-year gradient of costs to equal annual amounts and determining the present value of this annuity. The 40 years of constant costs for distributaries have been converted to a lump sum in year nine and then into present value. To these figures the present value of 50 years of operation and maintenance and the initial cost of the hydraulic works have been added. Interest during construction has been included with engineering and contingencies in the percentage of capital cost figure added to all capital cost estimates.

A. Gradient series conversion

Value of costs, Year 0		$ 51,375
Annual increment		51,375
Value of costs, Year 9		513,750
Note: Expenditures must occur prior to the realization of benefits from construction of distributaries.		
Uniform value		206,000
Present value at January 1, Year 1	206,000 x 7.36	$ 1,520,000
Present value at project beginning	1,520,000 x 1.124	$ 1,710,000

B. Equal annual costs, distributaries, to present value

Value of costs, Year 10		$ 513,750
Value of costs, Year 50		513,750
Present value at January 1, Year 9	513,750 x 15.3801	$ 7,900,000
Present value at project beginning	7,900,000 x 0.5919	$ 4,680,000

C. Operation and maintenance to present value

Value of costs, Year 0-50		$ 379,740
Present value at project beginning	379,740 x 15.762	$ 5,960,000

D. Capital cost of hydraulic works to present value

Present value at project beginning $ 30,920,000

E. Total present value of all costs at project beginning $ 43,270,000

F. Benefit-cost ratio

$$\frac{60,900,000}{43,270,000} = 1.41 \text{ to } 1$$

The computations for partial developments have been conducted in a similar manner.

9-5 Benefit-Cost Ratios

The benefits and costs, benefit-cost ratios and net benefits for all three schemes are summarized below.

	Scheme I	Scheme II	Scheme III
Direct Benefits	$61,900,000	$46,150,000	$30,340,000
Present Value of Costs	43,270,000	35,665,000	31,900,000
B/C	1.41	1.29	0.9
Present Value Net Benefits	18,630,000	10,485,000	negative
Net Benefits per acre	65.36	55.18	negative

The benefit and cost, benefit-cost ratio and net benefits for Scheme I assuming 285,000 acres under irrigation one year after completion of the main hydraulic works would be $84,600,000 direct benefits, $44,120,000 present value of costs, 1.92 B/C and $40,480,000 present value net benefits.

With more rapid development, the B/C ratio increases from 1.41 to 1.92 and net benefits would more than double in dollar value. This calculation demonstrates the importance of a rapid development period and the significance of the period of development on the feasibility of the project.

9-5-1 Farmer's Benefit-Cost Ratio

The benefit-cost ratio, to the individual farmer, with and without irrigation would be as follows:

	Subzone I	Subzone II	Subzone III
Without Irrigation			
Annual Revenue	$28,000	$14,000	$ 9,000
Annual Cost	20,700	10,700	7,600
B/C	1.35	1.31	1.18
With Irrigation			
Annual Revenue	$56,400	$23,400	$14,520
Annual Cost	34,000	18,800	11,000
B/C	1.66	1.51	1.32

CHAPTER X

SUMMARY AND RECOMMENDATIONS

10-1 Summary

The investigation which forms the subject of this report was undertaken at the request of the Governor of the Province of Assiniboia. Its objective has been to conduct a preliminary study and economic analyses of the feasibility of Irrigation in the Rio Seco Basin and to determine the extent of the works which should be constructed for the most economic and beneficial development of the basin.

The computational procedures followed were intended to result in preliminary estimates, on the basis of which, comparison of development alternatives could be carried out and capital requirements established.

It was found that the most feasible plan for irrigation development would be a combination of the Rio Saska storage dam at a cost of $21,800,000, an unlined gravity canal at a cost of $9,120,000 and a network of unlined distribution canals, laterals and associated structures and open drainage ditches in the irrigation district at a cost of $7,730,000. This combination of works would provide sufficient irrigation water for year-round crop production for the 285,000 irrigable acres within the Rio Seco Basin. The benefit-cost ratio of the project, based on present depreciated values of benefits and costs was found to be 1.4 for a 10-year development period commencing from the date of completion of the storage dam and main canal.

10-2 Recommendations

Irrigation development of the Rio Seco basin is feasible and it is recommended that further studies be conducted to finalize designs and bring the project to the tendering stage as soon as possible.

As the storage dam, gravity canal and distribution reservoir alternate was found to be competitive on the basis of annual costs, it is further recommended that this combination of structures be investigated for possible multi-purpose use of storage on the Rio Saska.

A multi-purpose development with storage allocated to hydro power, flood control and irrigation at a full supply level of approximately 1,170 feet MSL could prove beneficial to all three consumers thus decreasing the capital cost of storage for the irrigation project. Irrigation flows, if available above elevation 1,010 feet MSL would also result in savings in canal costs as a steeper canal slope would allow for a decrease in the canal cross section. The distribution reservoir would become a significant component in a multi-purpose scheme as irrigation water could be drawn off during peak flow periods where withdrawals of 4,000 SFM would have little effect on available head for hydro power generation. This diversion would also have the effect of reducing downstream flows and making storage available for flood control. The distribution reservoir would provide storage for irrigation flows until required.

With further regard to multi-purpose development, it should be noted that the cost analyses and comparison of various canal types did not assign a value to the loss of water due to seepage from the main canal. Costs were compared on the basis of the additional storage and increased cost of the storage reservoir to make up the seepage losses. Where multi-purpose use is to be considered, the value of lost benefits to other users could become substantial, in which case canal lining to restrict seepage losses should be considered. The difference in costs between unlined and clay-lined canals for a maximum discharge of 3,870 cfs was established as $350,000 or $72.50 per cfs. If the value of water to hydro power were to exceed this figure, canal lining could prove to be economical.

Assuming single-purpose development for irrigation water supply is considered to have priority over other uses, again it is recommended that further investigations of the Rio Seco reservoir be carried out. It is estimated that the construction period, from award of contract to beginning of project, would encompass a period of six years. During this time, with additional possible delays in reservoir filling for example, considerable capital costs would have been incurred and benefits would still be several years in developing. If the Rio Seco distribution reservoir were to be constructed as Phase 1 of the project, with an estimated construction period of two years, a small portion of the Irrigation District could be supplied from Rio Seco storage, possibly on a supplementary basis. There would be two advantages to such a plan, early realization of benefits from irrigation and additional time in which farming techniques could be studied and improved and a model farm could be established for demonstrating irrigation farming and fertilization methods.

In the event that further studies of the project will analyze the main storage dam for multi-purpose use, the following figures would prove useful in determining separable costs and remaining benefits of irrigation for cost allocation purposes.

Total estimated benefits of irrigation	$60,900,000
Total estimated costs	$38,650,000
Maximum possible contribution	$22,250,000
Total cost of storage dam	$21,800,000

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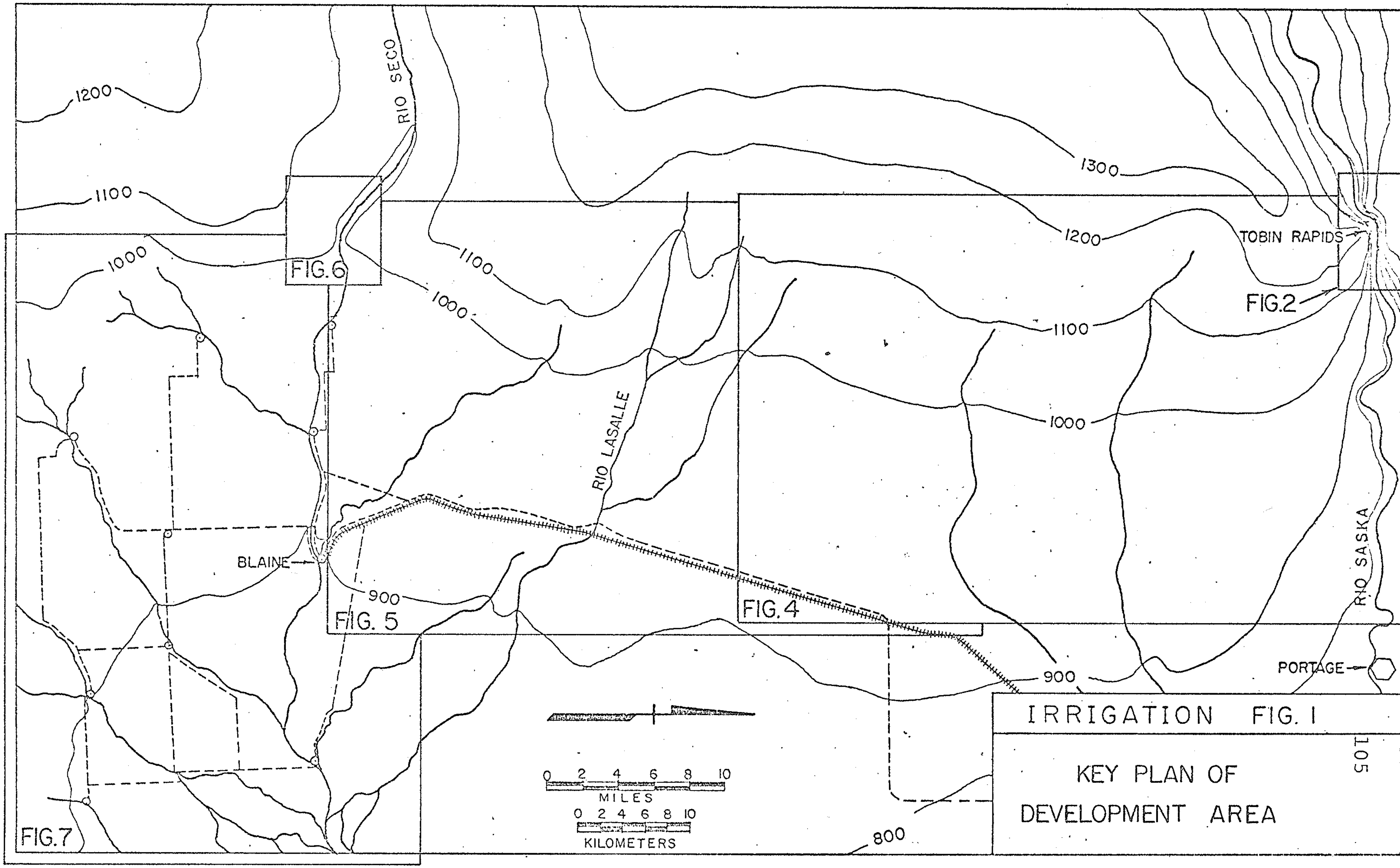
FIGURES

Unless otherwise noted, Figures have not been drawn to scale and dimensions should not be scaled from the drawings. Major structures have been dimensioned as required. Drawings of typical structures are intended to convey a concept of the proportions of the structure only.

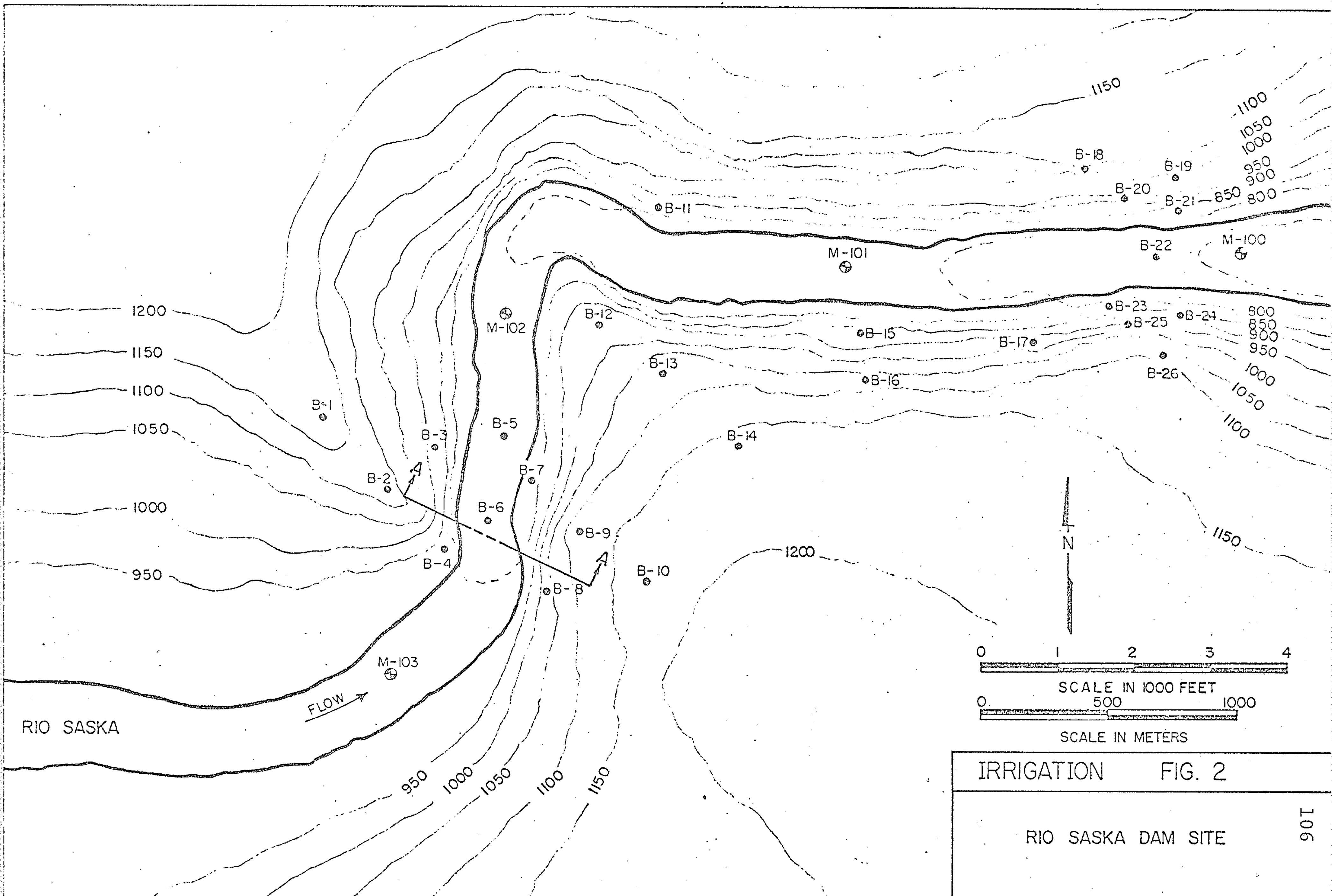
LEGEND

Settlements	0
Paved Roads	---
Railroads	++++++
Rain Gauging Stations	o^7
Control Structures	=
Contour Lines	−1000−
Major Canals	— — —
Distribution Canals	——
Drains	——
Tail Escapes	•••••••



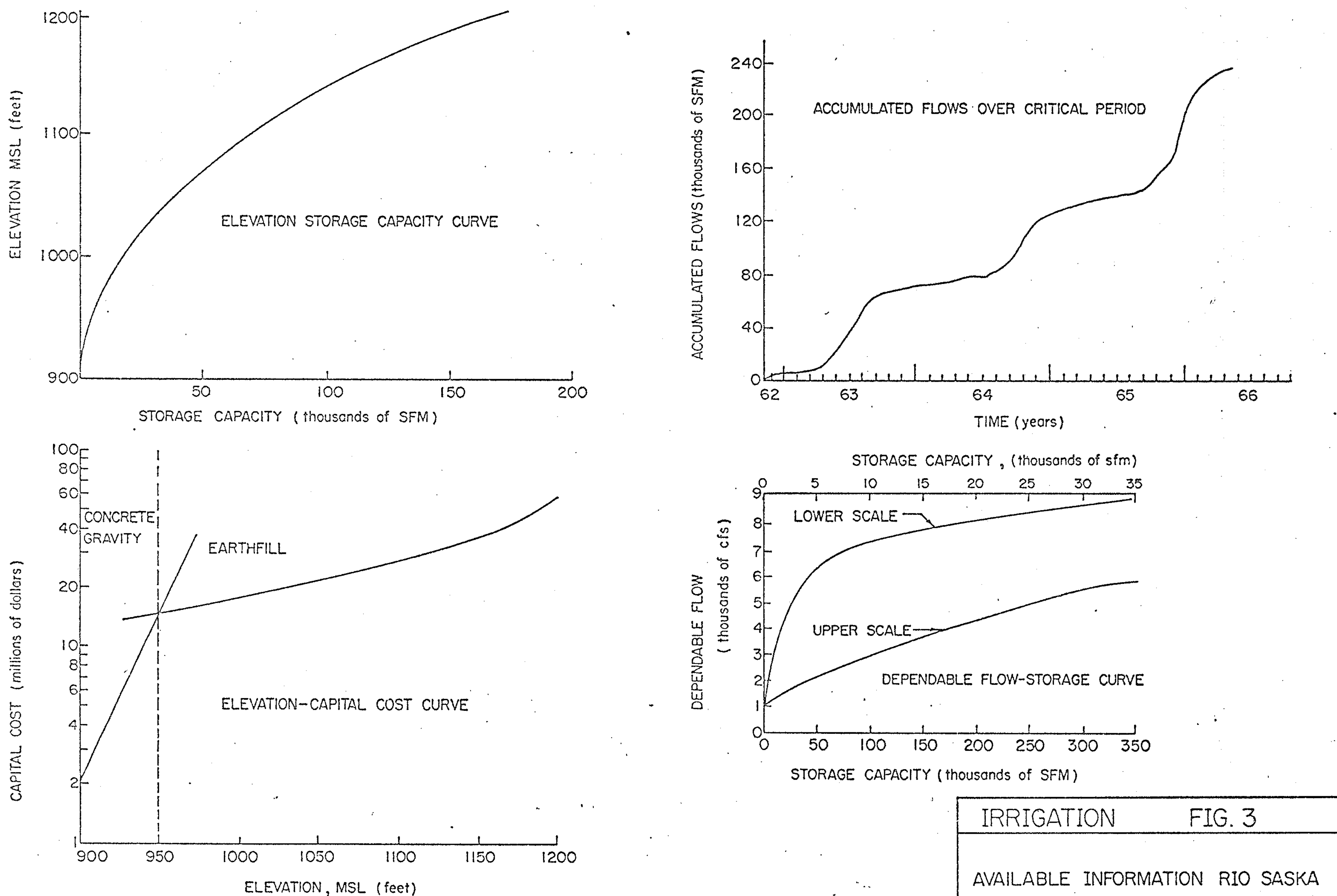
IRRIGATION FIG. I

KEY PLAN OF DEVELOPMENT AREA



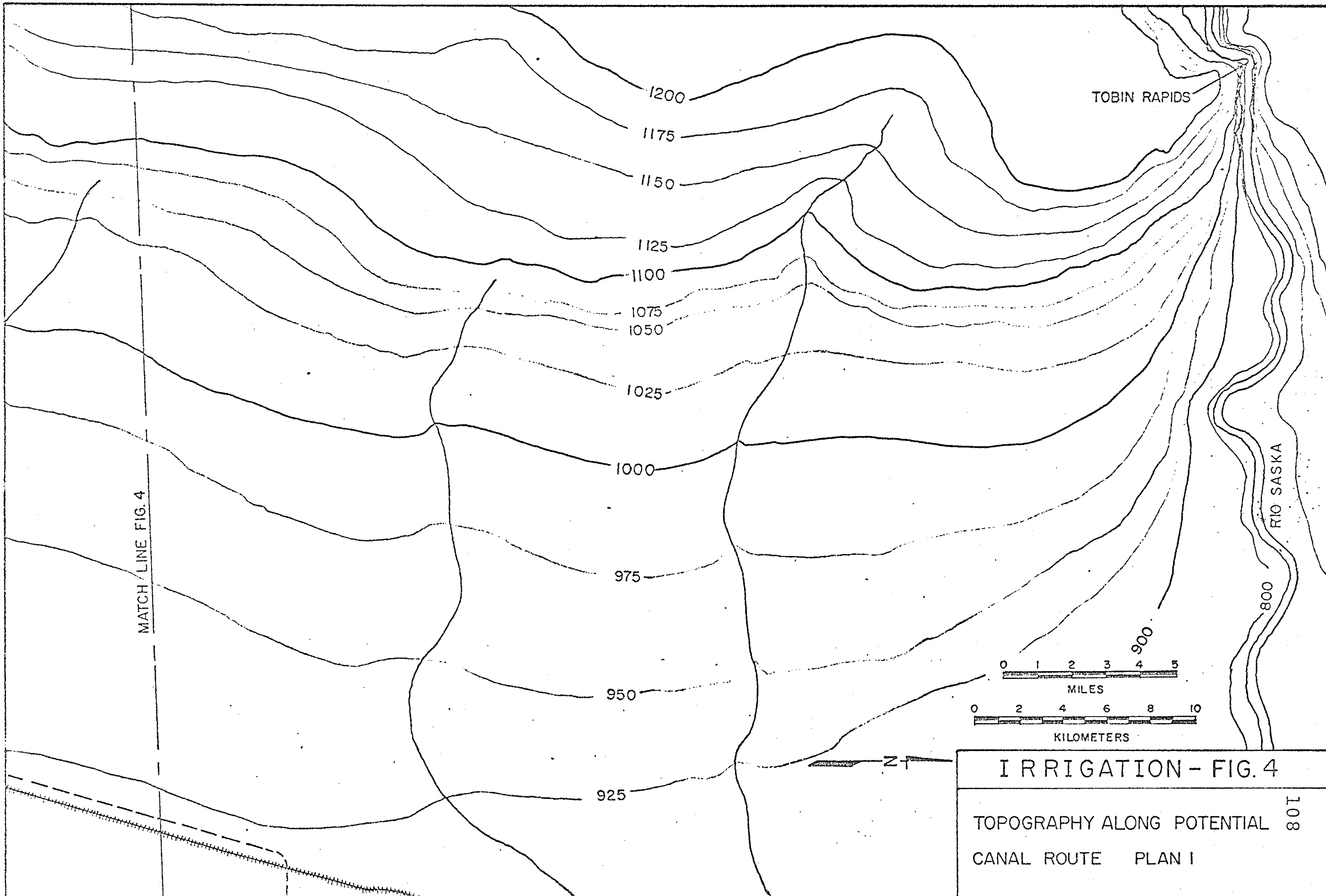
IRRIGATION FIG. 2

RIO SASKA DAM SITE



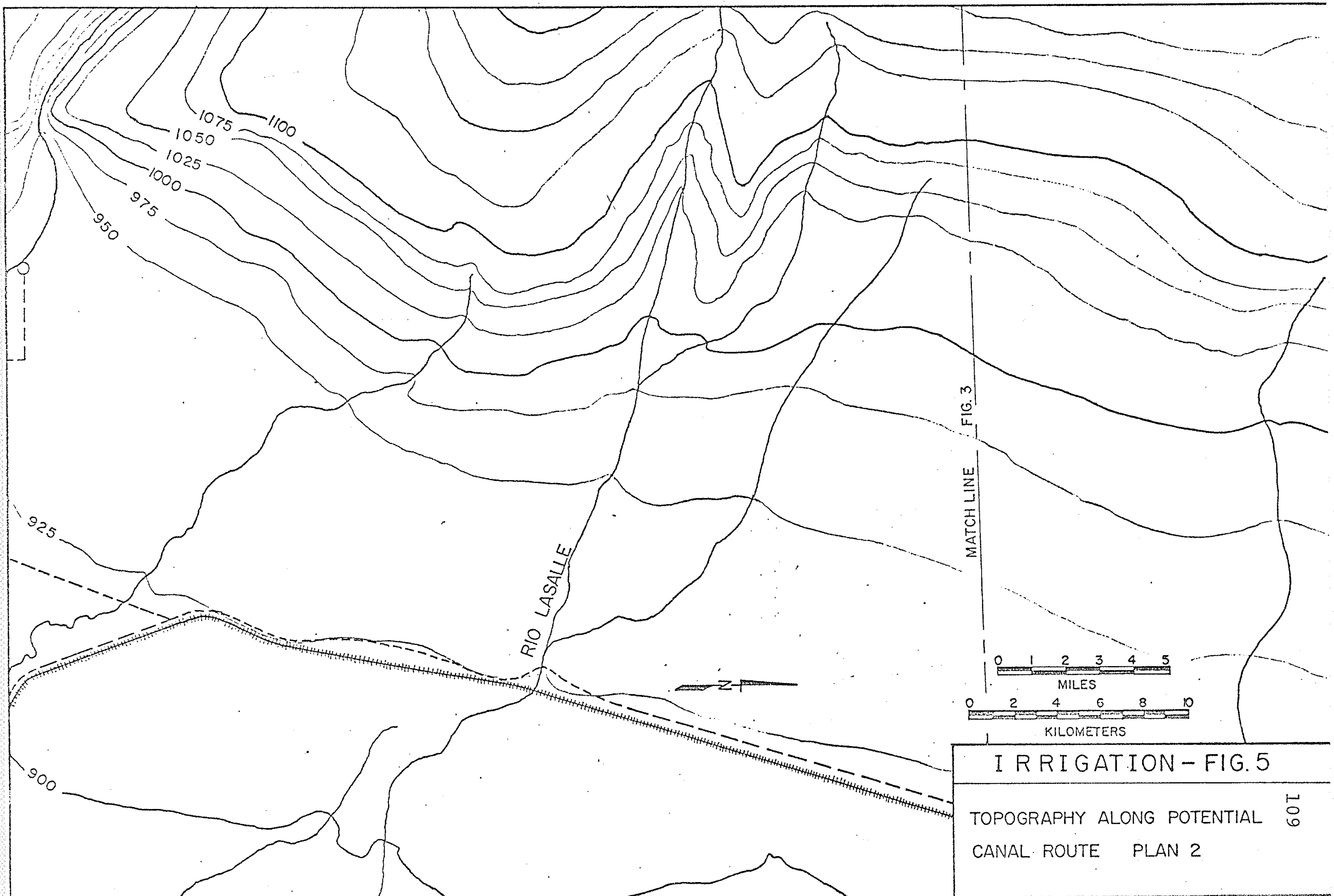
IRRIGATION FIG. 3

AVAILABLE INFORMATION RIO SASKA DAM SITE



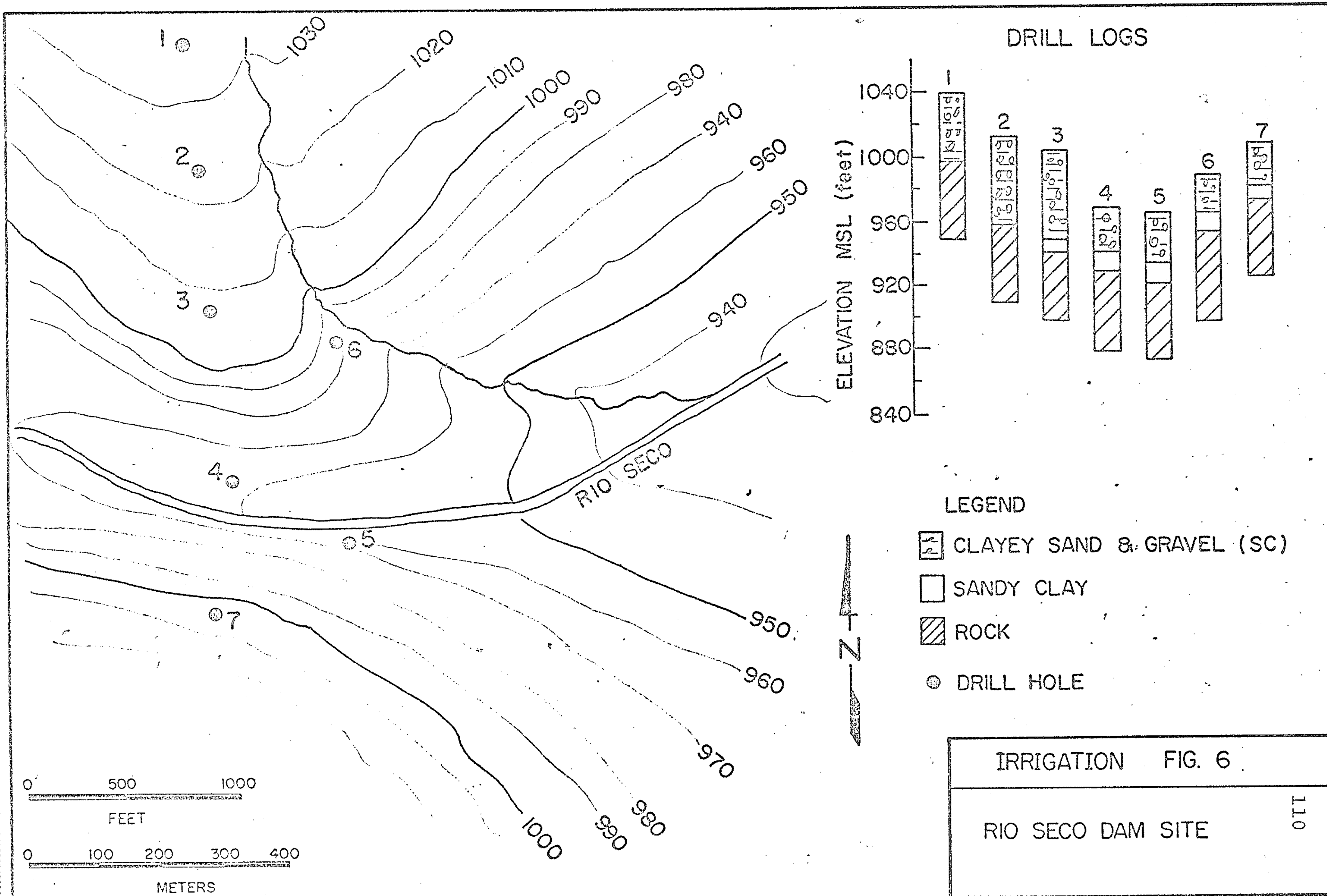
IRRIGATION - FIG. 4

TOPOGRAPHY ALONG POTENTIAL CANAL ROUTE PLAN I



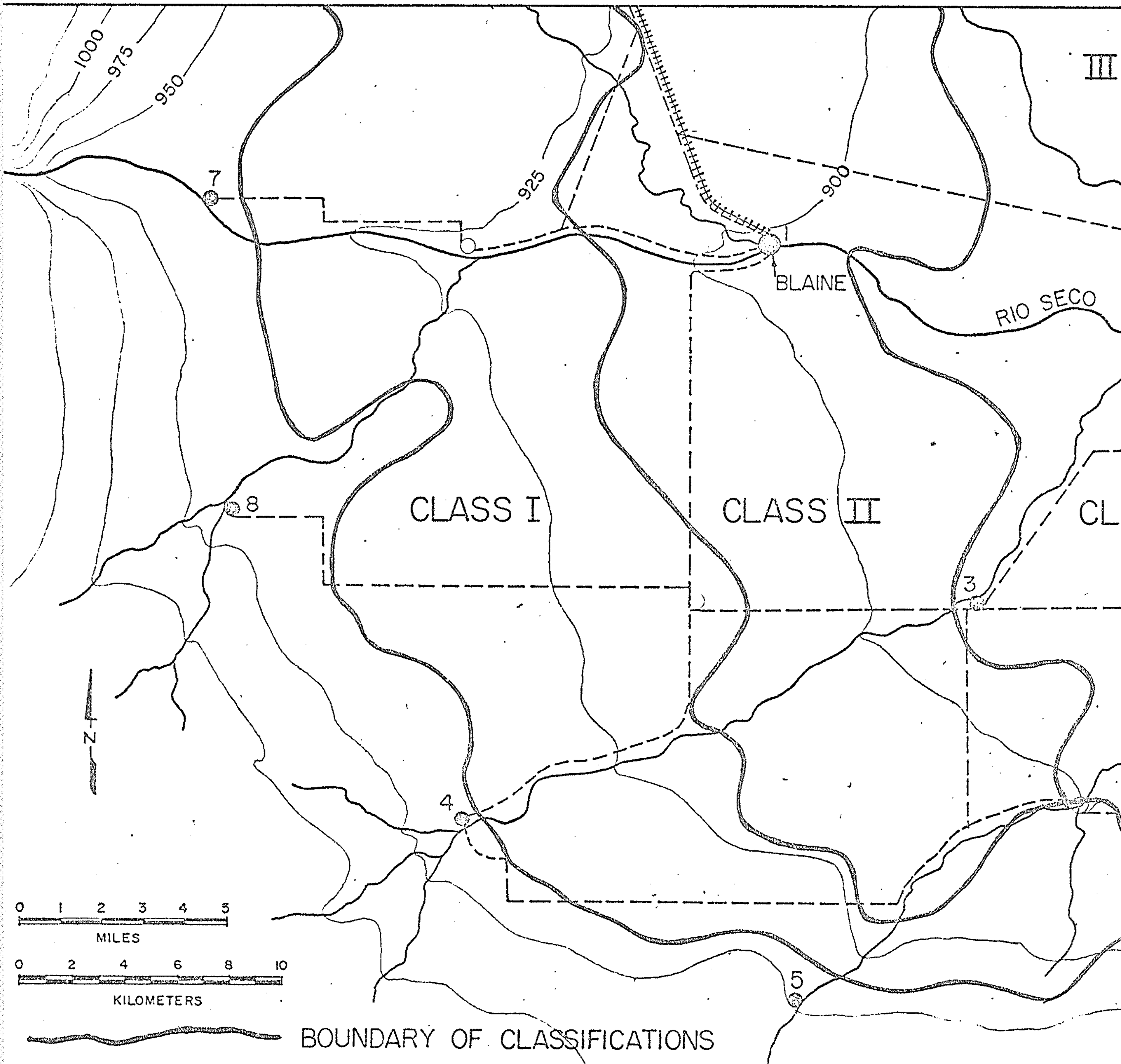
I R R I G A T I O N - FIG. 5

TOPOGRAPHY ALONG POTENTIAL CANAL ROUTE PLAN 2



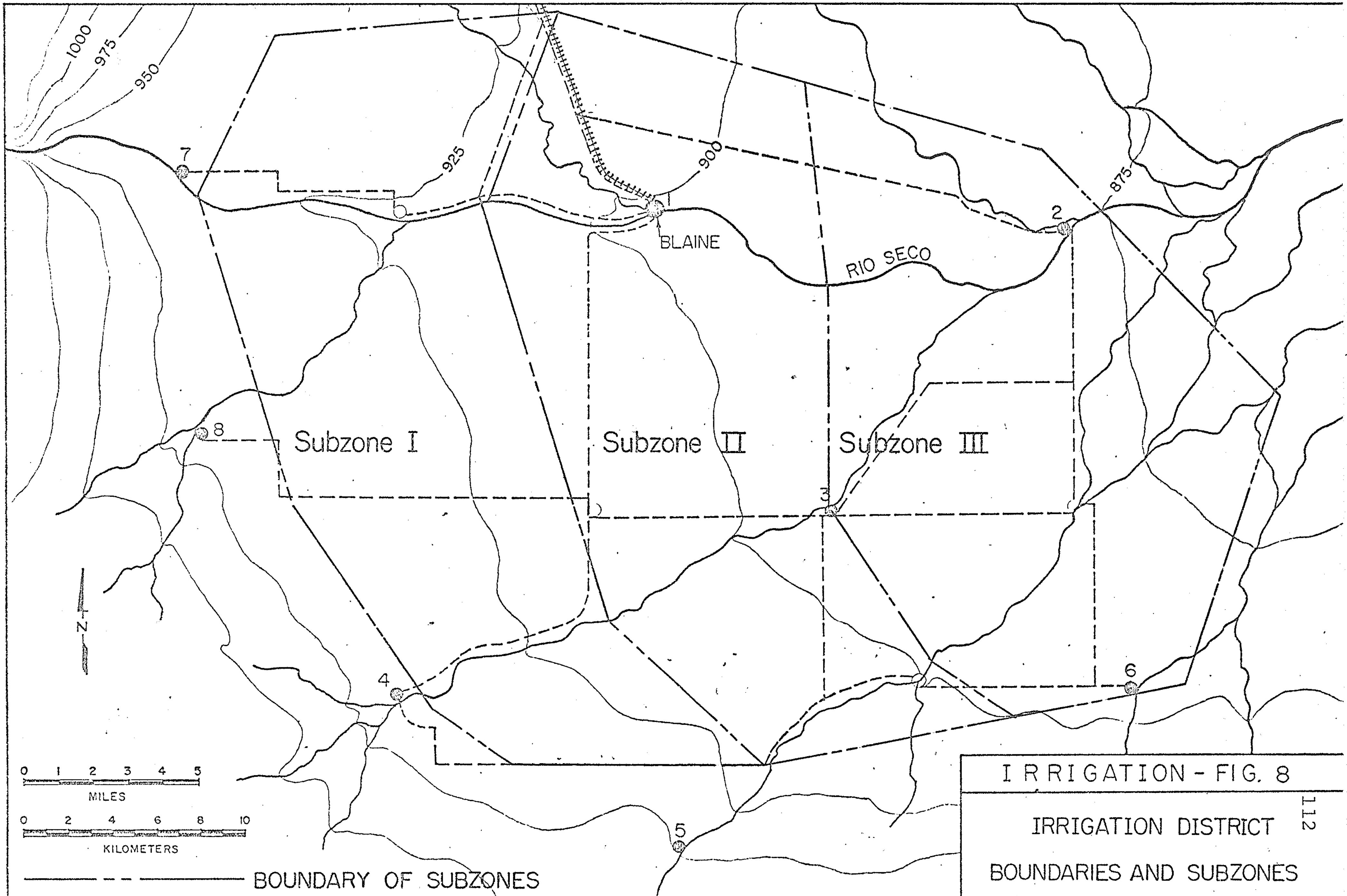
IRRIGATION FIG. 6

RIO SECO DAM SITE



IRRIGATION - FIG. 7

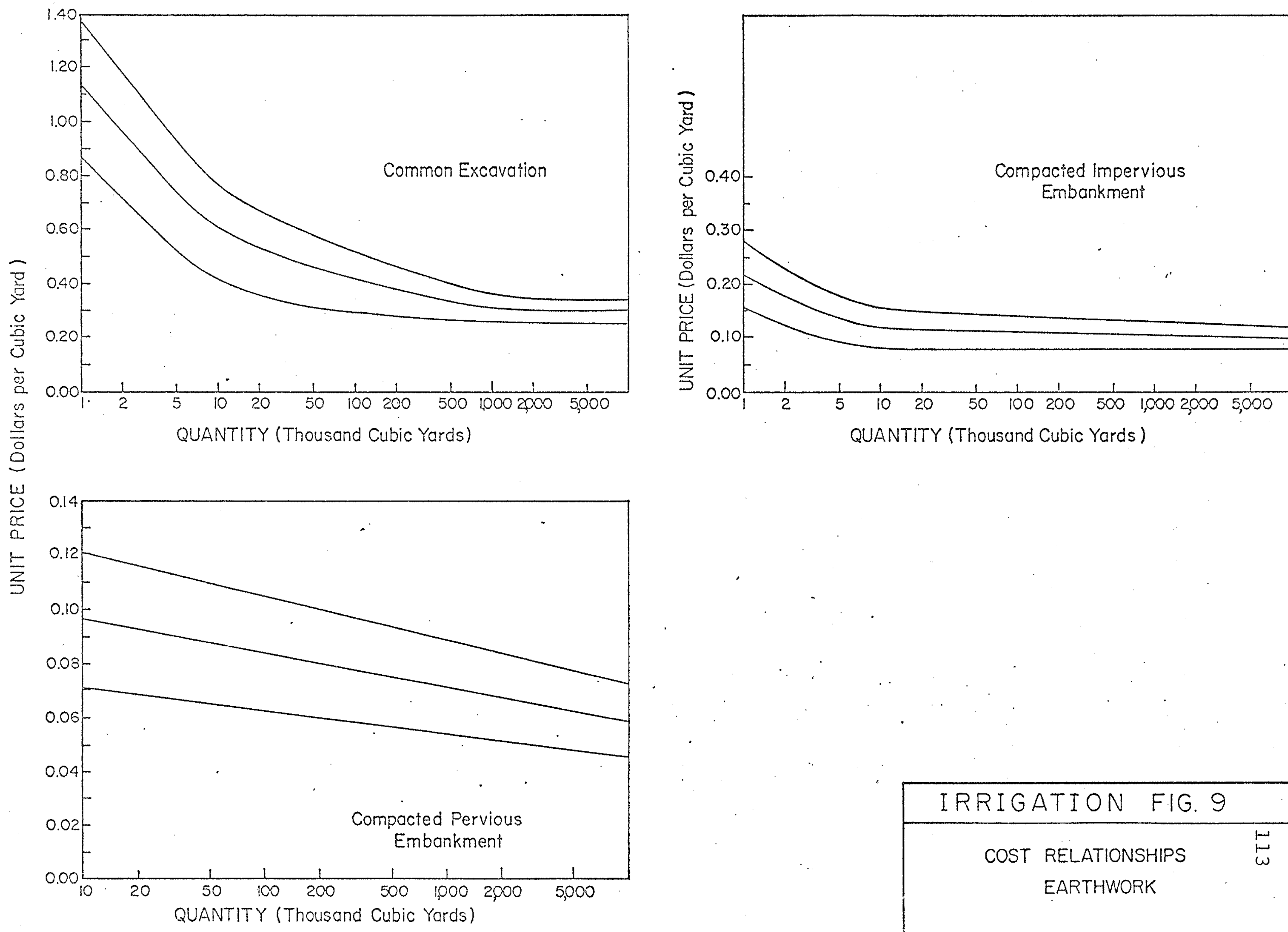
LAND CLASSIFICATION AND RAIN GAUGING STATIONS



IRRIGATION - FIG. 8

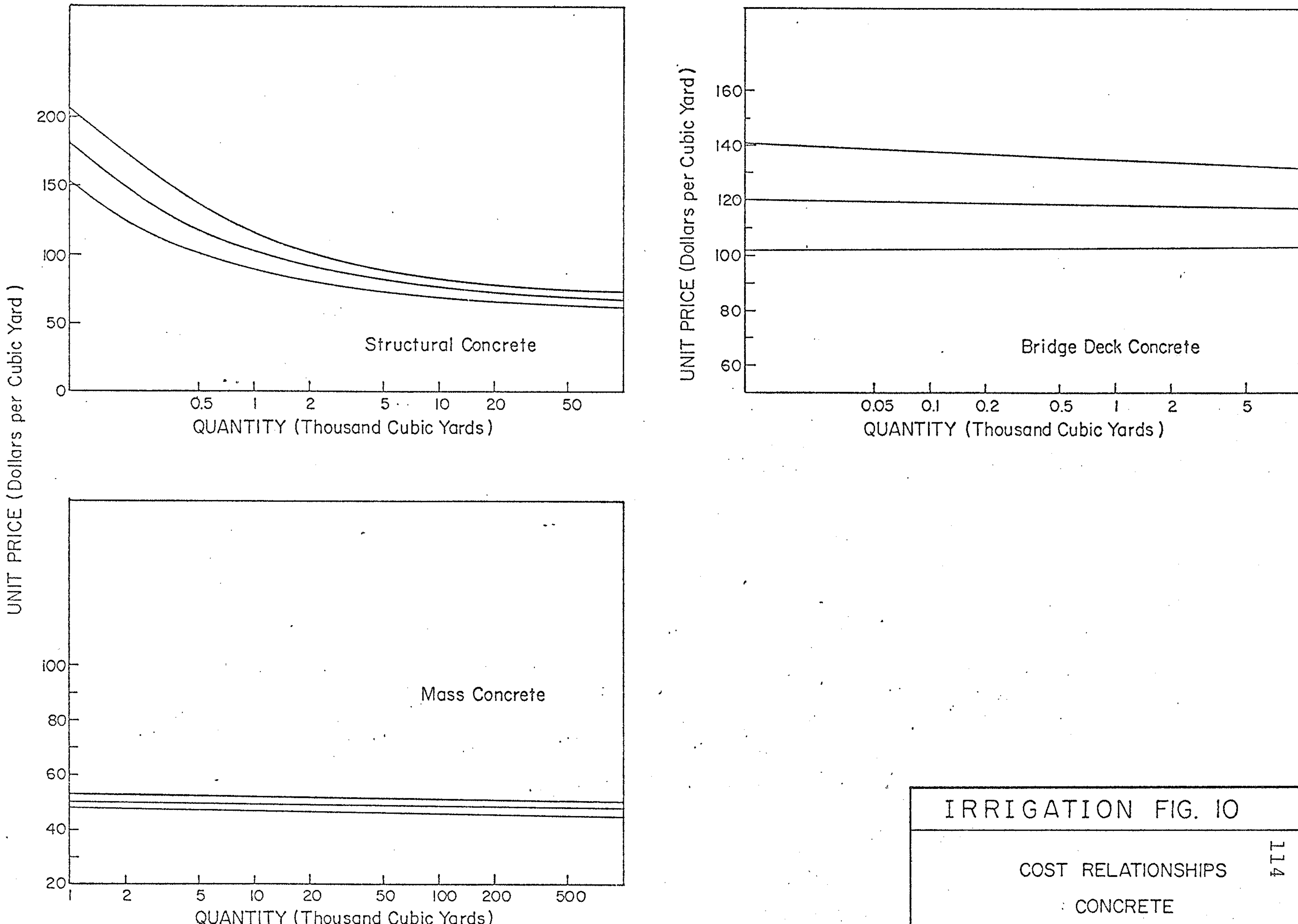
IRRIGATION DISTRICT

BOUNDARIES AND SUBZONES



IRRIGATION FIG. 9

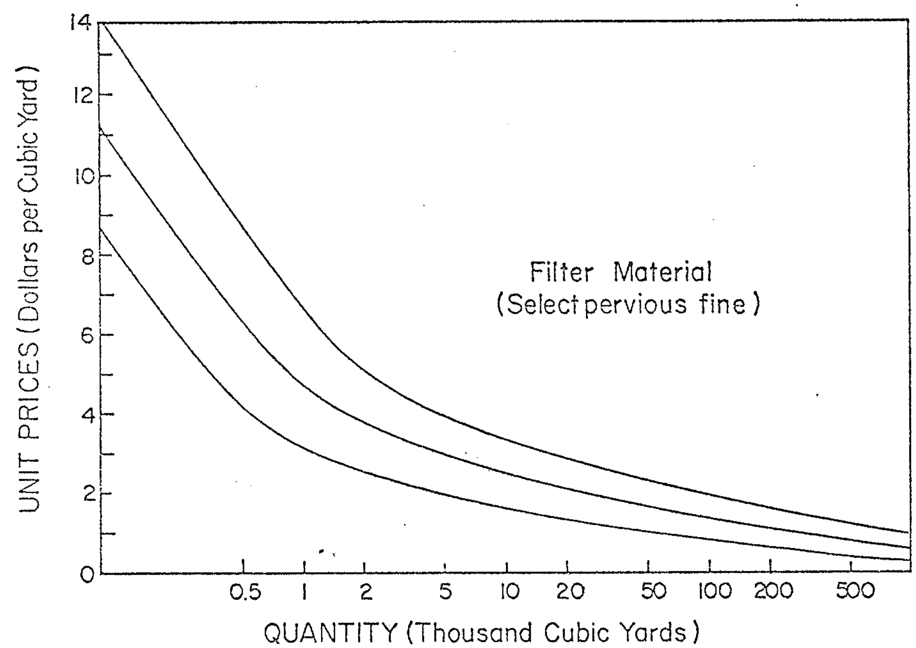
COST RELATIONSHIPS
EARTHWORK

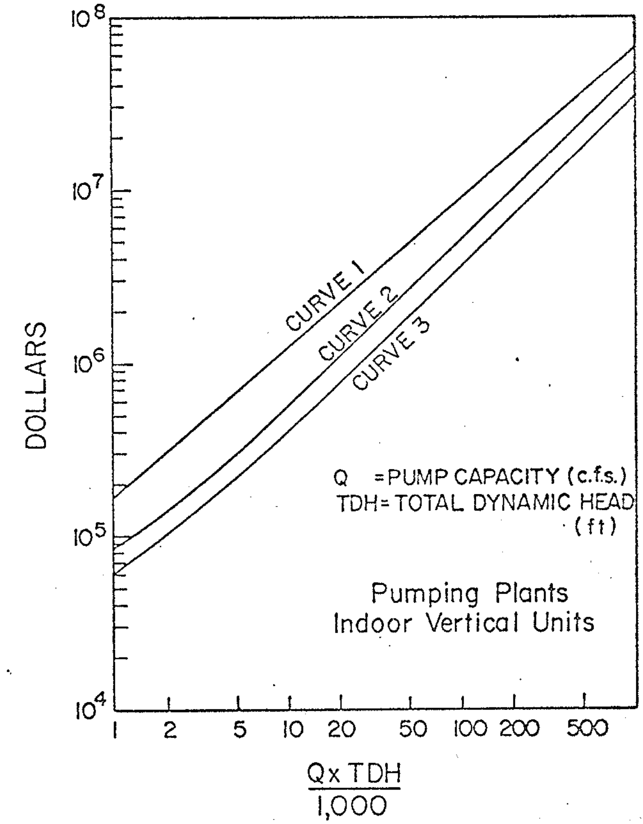


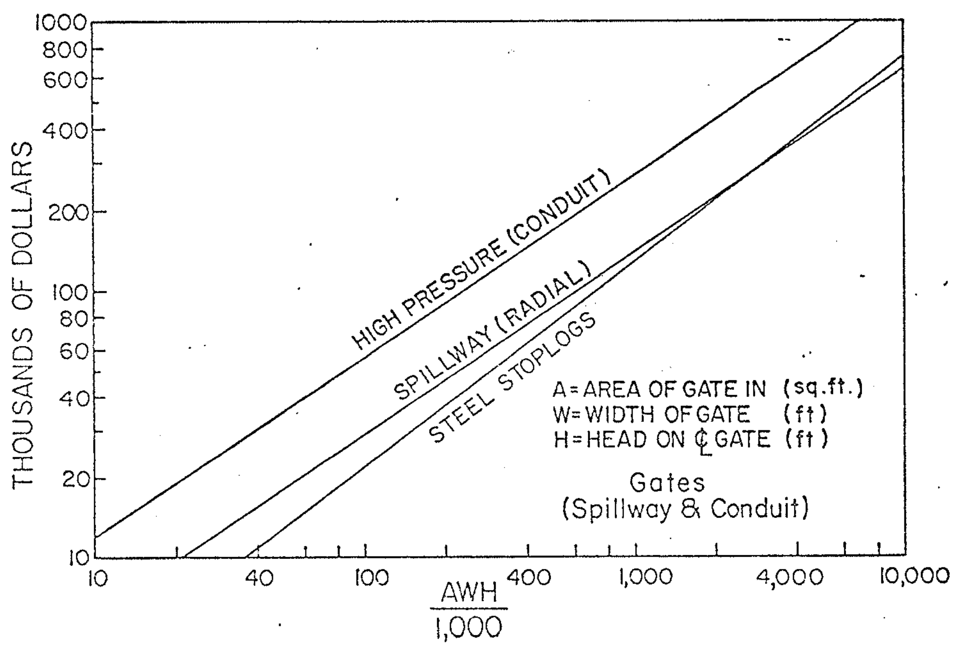
IRRIGATION FIG. 10

COST RELATIONSHIPS

CONCRETE







NOTE:

CURVE 1, 0-40 feet range of TDH
CURVE 2, 40-100 feet
CURVE 3, over 100 feet

IRRIGATION FIG. 11

COST RELATIONSHIPS

PUMPING GATES FILL

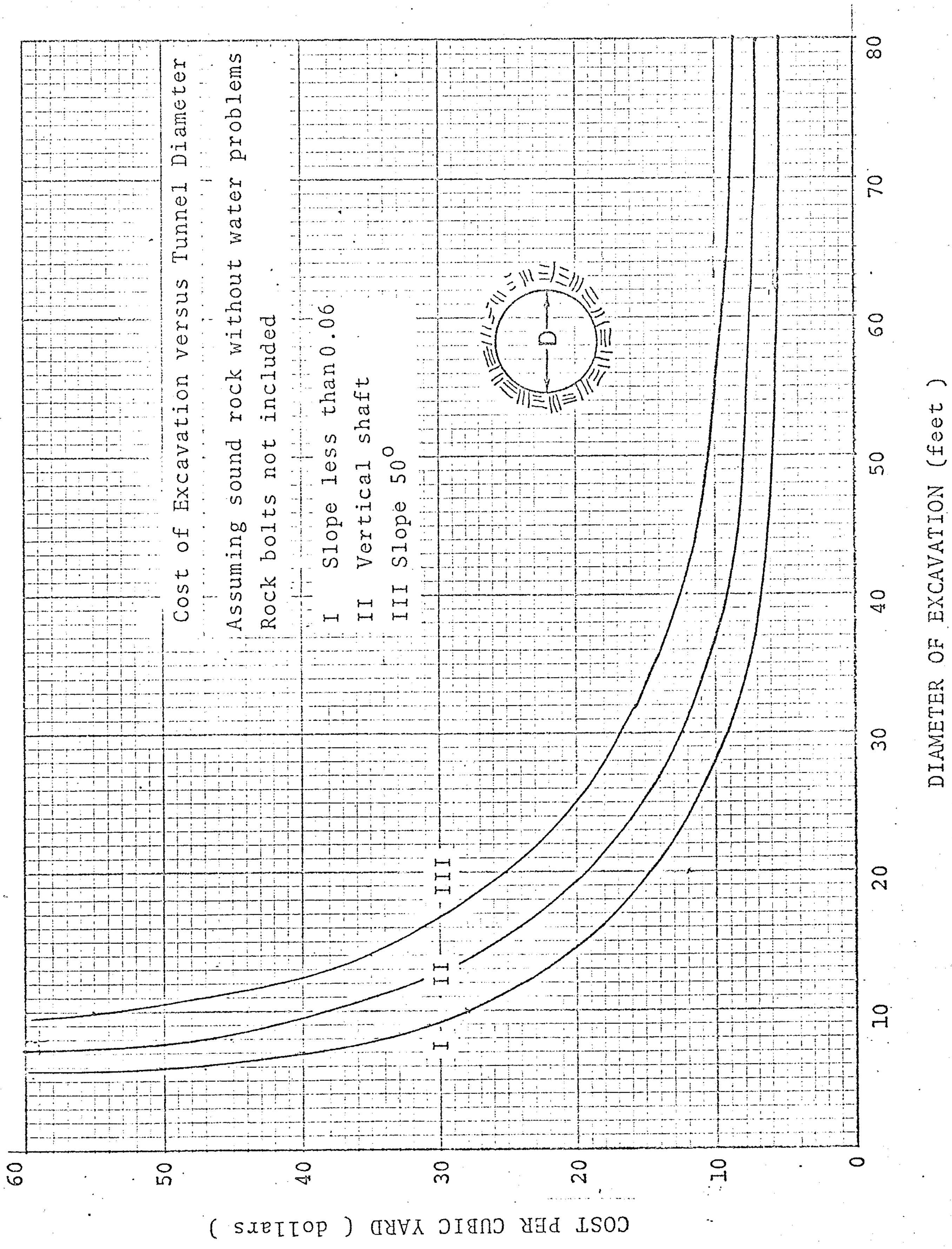

Cost of Excavation versus Tunnel Diameter
Assuming sound rock without water problems
Rock bolts not included
I Slope less than 0.06
II Vertical shaft
III Slope 50°
I
II
III
D
COST PER CUBIC YARD (dollars)
0
10
20
30
40
50
60
DIAMETER OF EXCAVATION (feet)
10
20
30
40
50
60
70
80

FIGURE 12

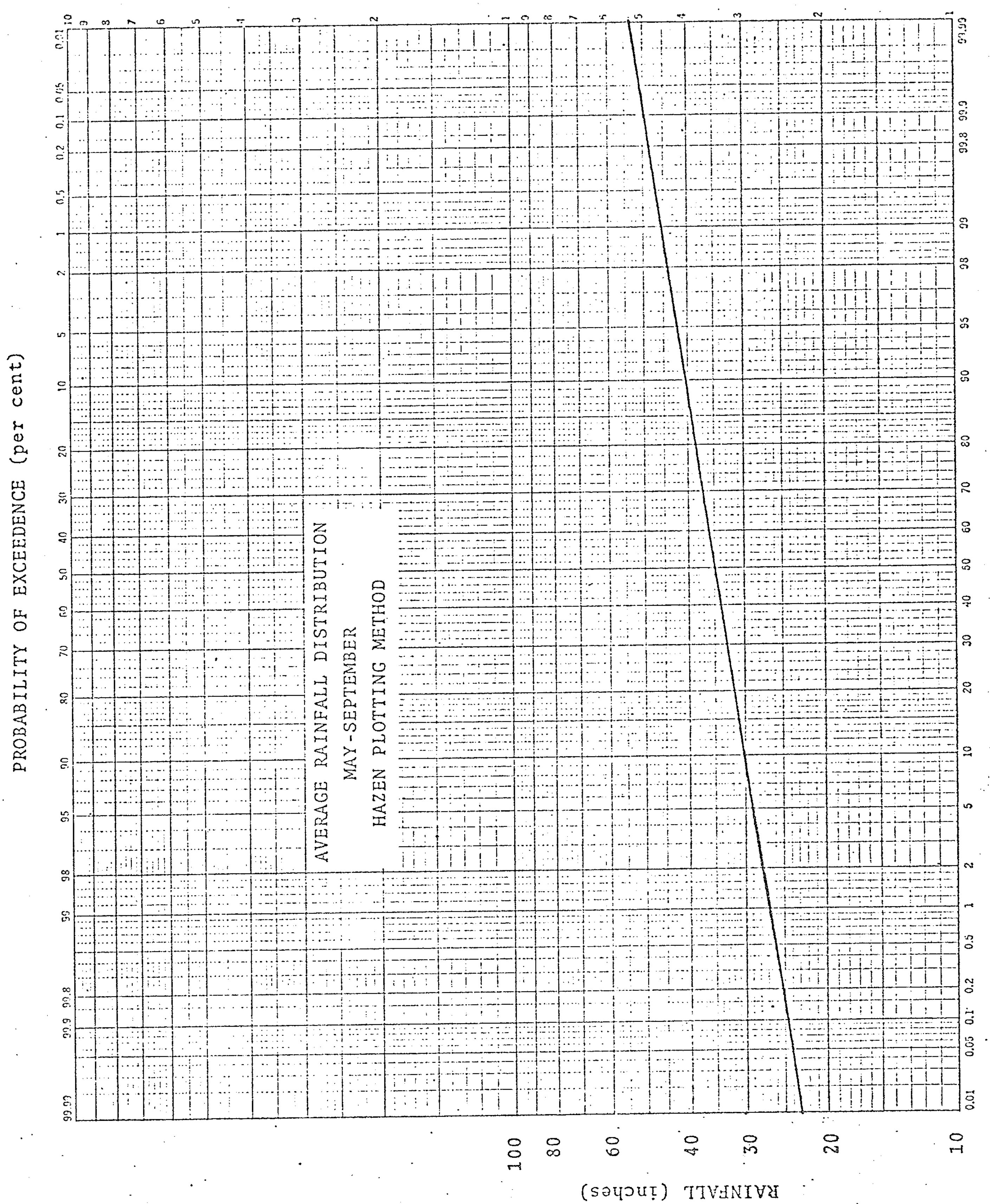
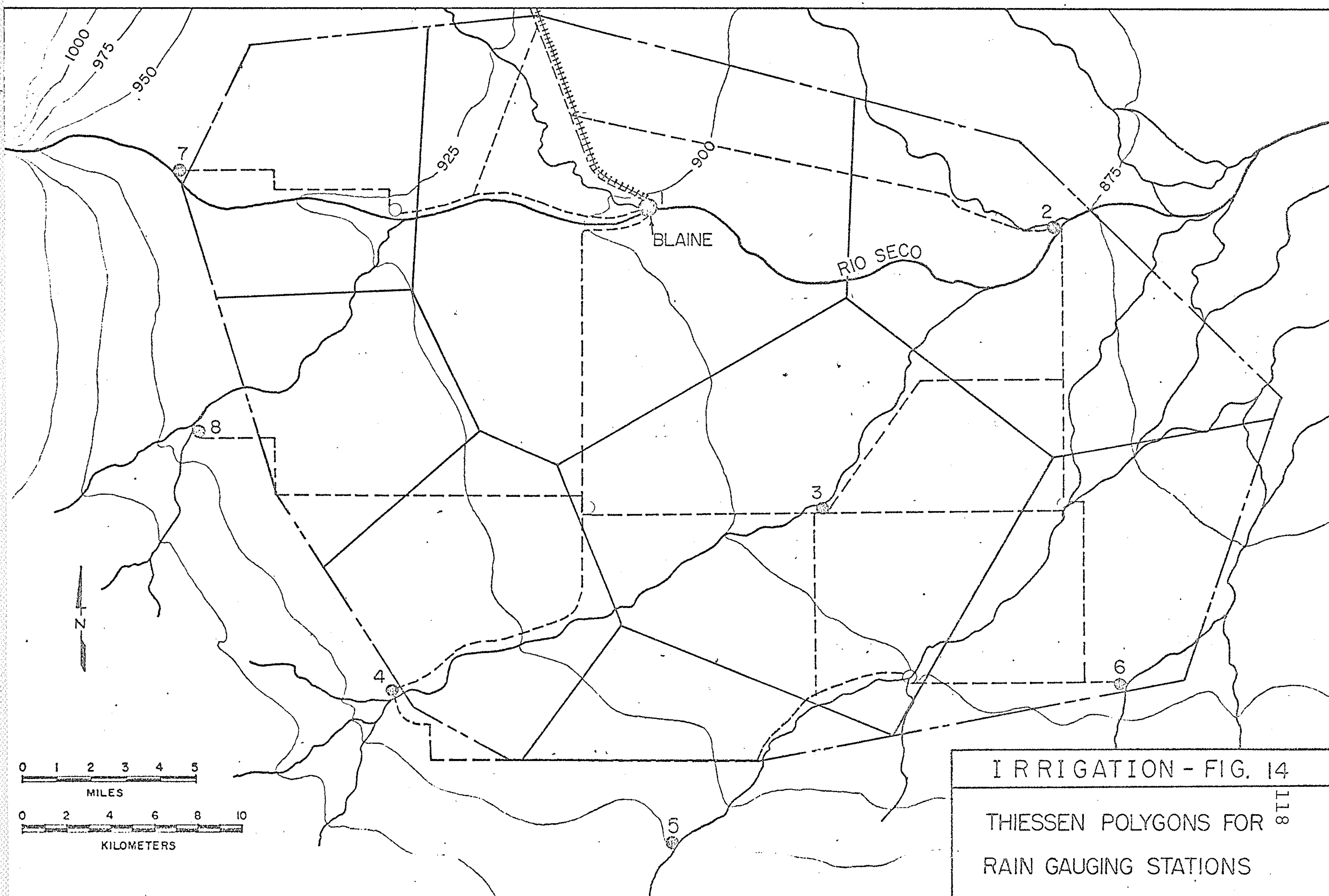

PROBABILITY OF EXCEEDENCE (per cent)
AVERAGE RAINFALL DISTRIBUTION
MAY-SEPTEMBER
HAZEN PLOTTING METHOD
RAINFALL (inches)
100
80
60
40
30
20
10

FIGURE 13



IRRIGATION - FIG. 14

THIESSEN POLYGONS FOR RAIN GAUGING STATIONS

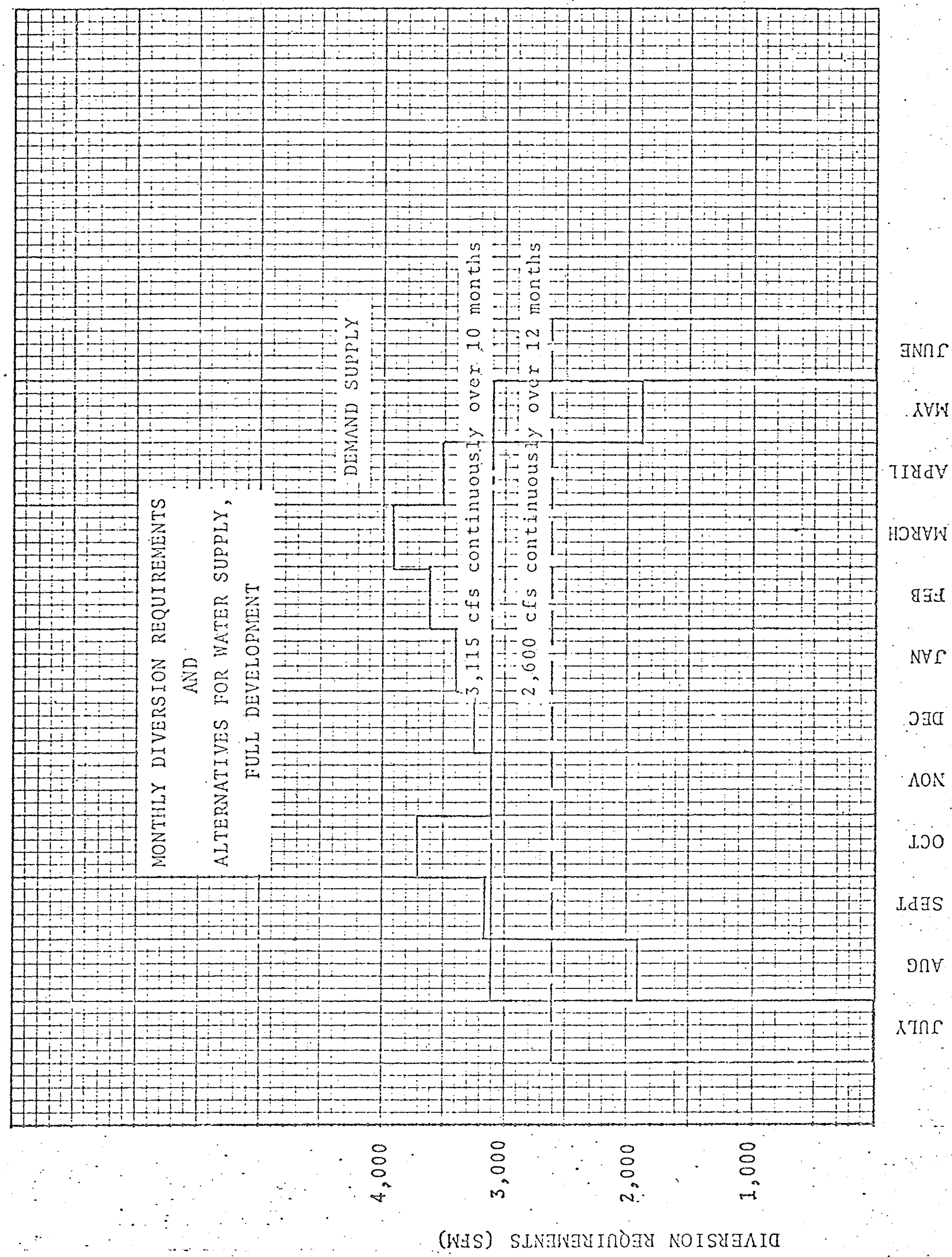

MONTHLY DIVERSION REQUIREMENTS
AND
ALTERNATIVES FOR WATER SUPPLY,
FULL DEVELOPMENT
DEMAND SUPPLY
3,115 cfs continuously over 10 months
2,600 cfs continuously over 12 months
JULY
AUG
SEPT
OCT
NOV
DEC
JAN
FEB
MARCH
APRIL
MAY
JUNE
MONTHS
4,000
3,000
2,000
1,000
DIVERSION REQUIREMENTS (SFM)

FIGURE 15

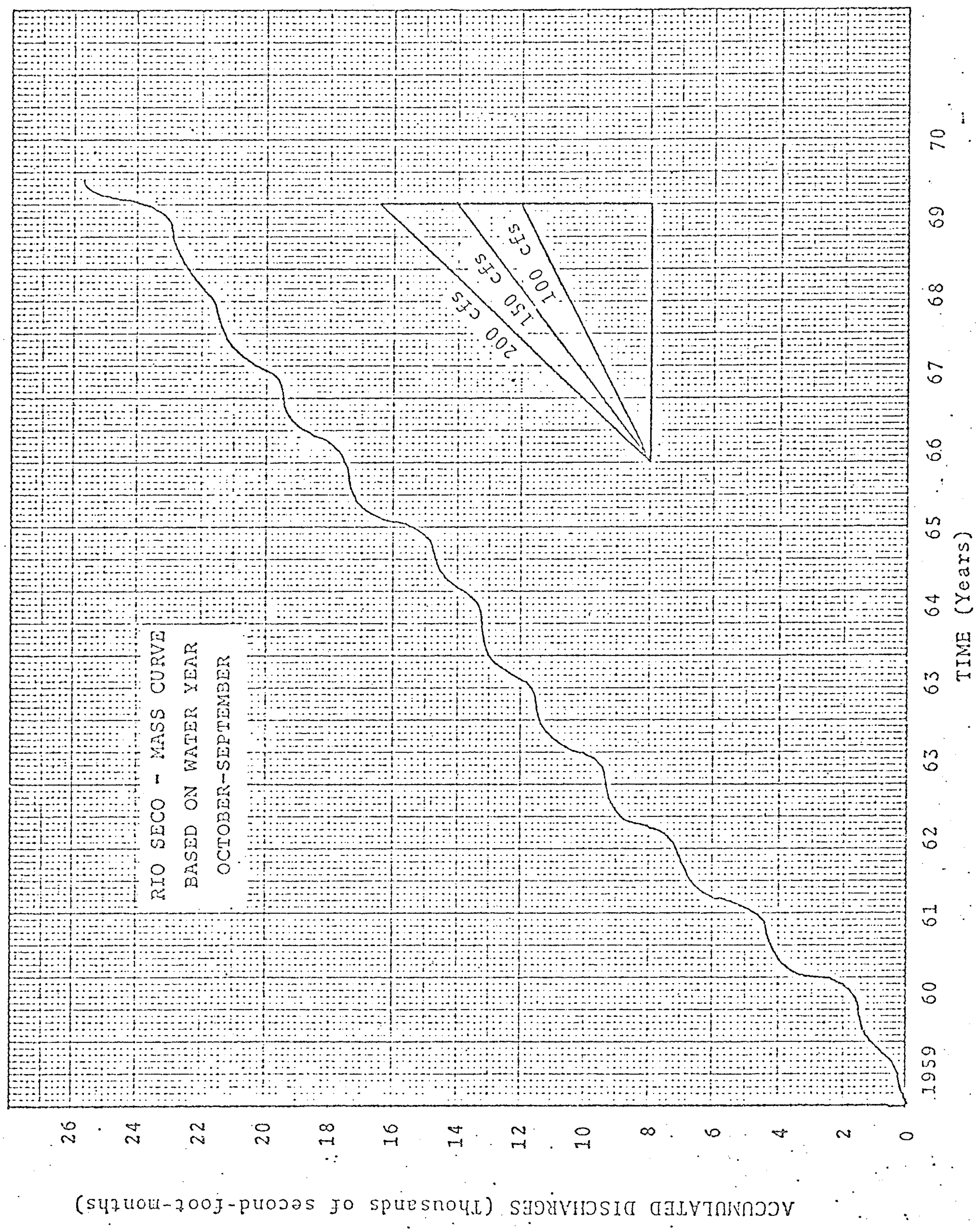

RIO SECO - MASS CURVE
BASED ON WATER YEAR
OCTOBER-SEPTEMBER
200 cfs
150 cfs
100 cfs
1959
60
61
62
63
63
64
65
66
67
68
69
70
TIME (Years)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
ACCUMULATED DISCHARGES (Thousands of second-foot-months)

FIGURE 16

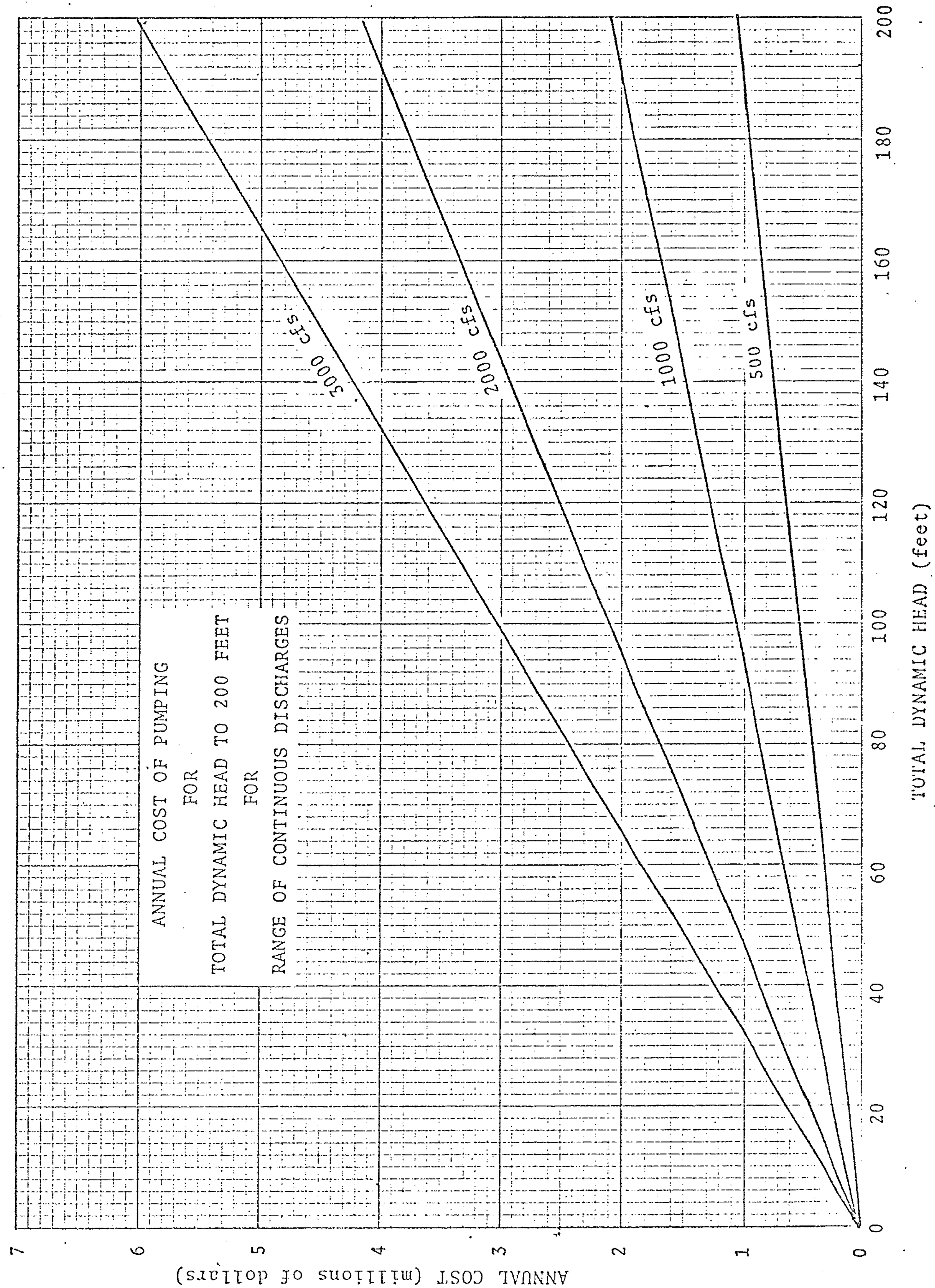
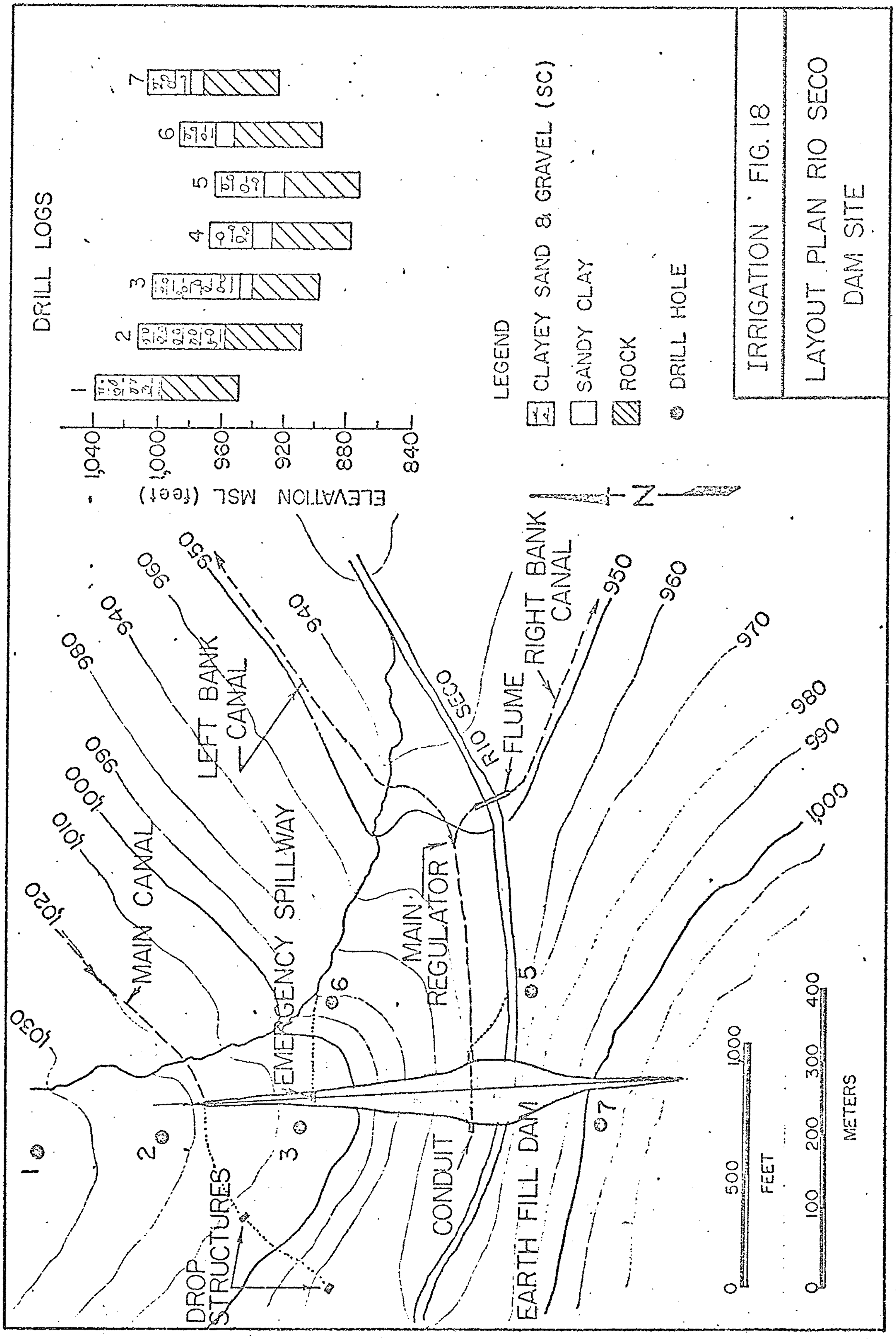
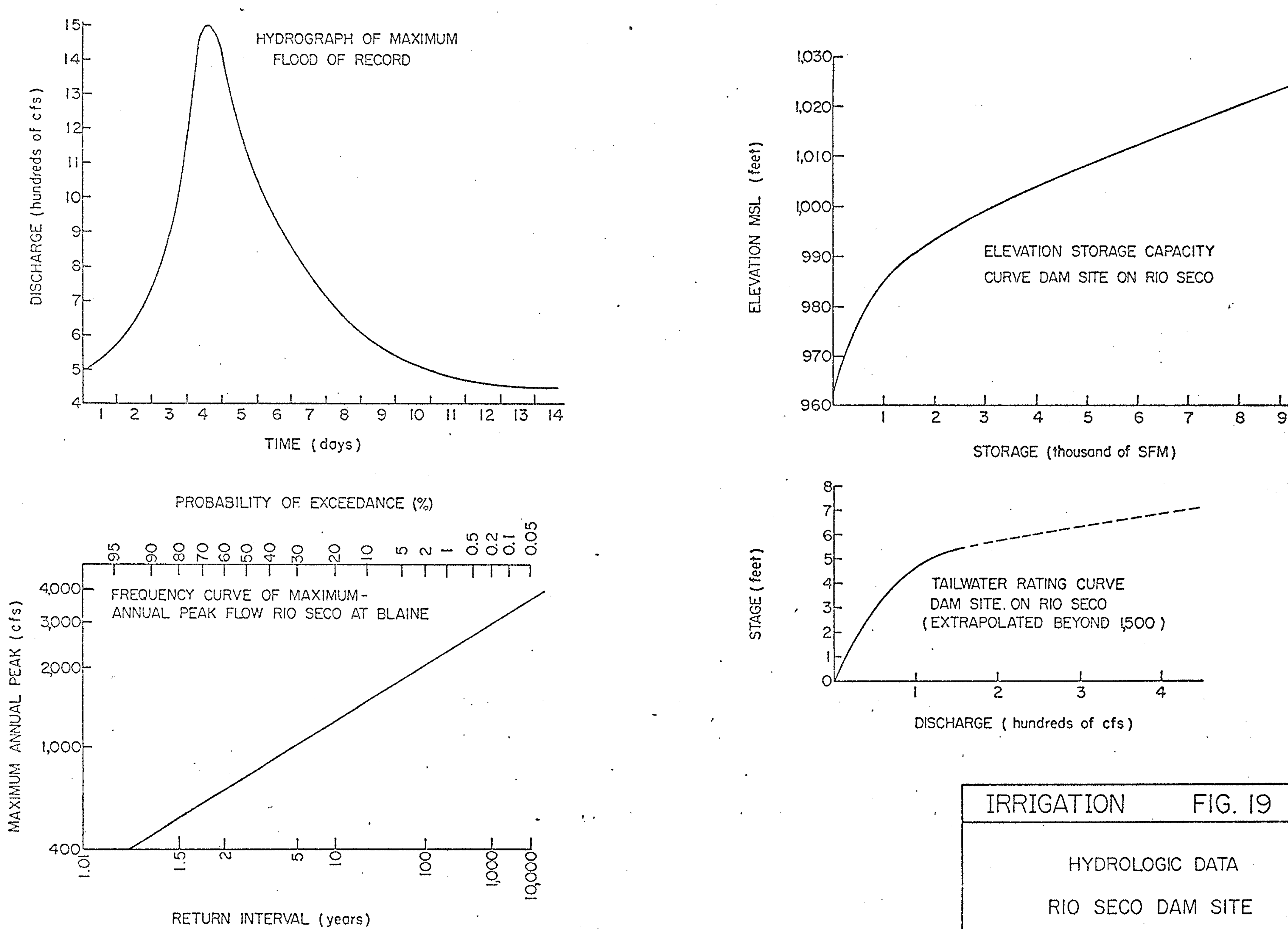

ANNUAL COST OF PUMPING
FOR
TOTAL DYNAMIC HEAD TO 200 FEET
FOR
RANGE OF CONTINUOUS DISCHARGES
3000 cfs
2000 cfs
1000 cfs
500 cfs
TOTAL DYNAMIC HEAD (feet)
0
20
40
60
80
100
120
140
160
180
200
ANNUAL COST (millions of dollars)
0
1
2
3
4
5
6
7

FIGURE 17



IRRIGATION FIG. 18

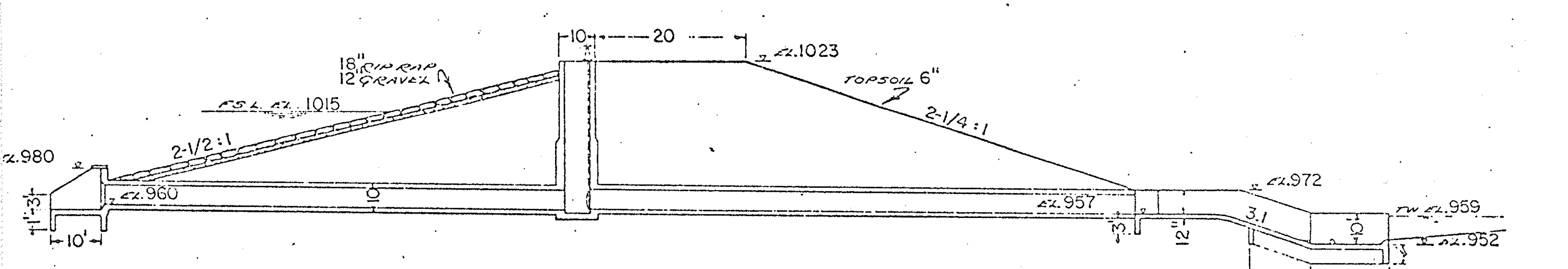
LAYOUT PLAN RIO SECO DAM SITE



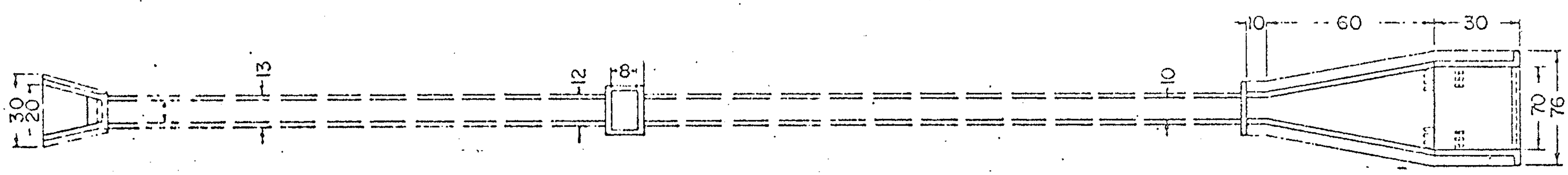
IRRIGATION FIG. 19

HYDROLOGIC DATA

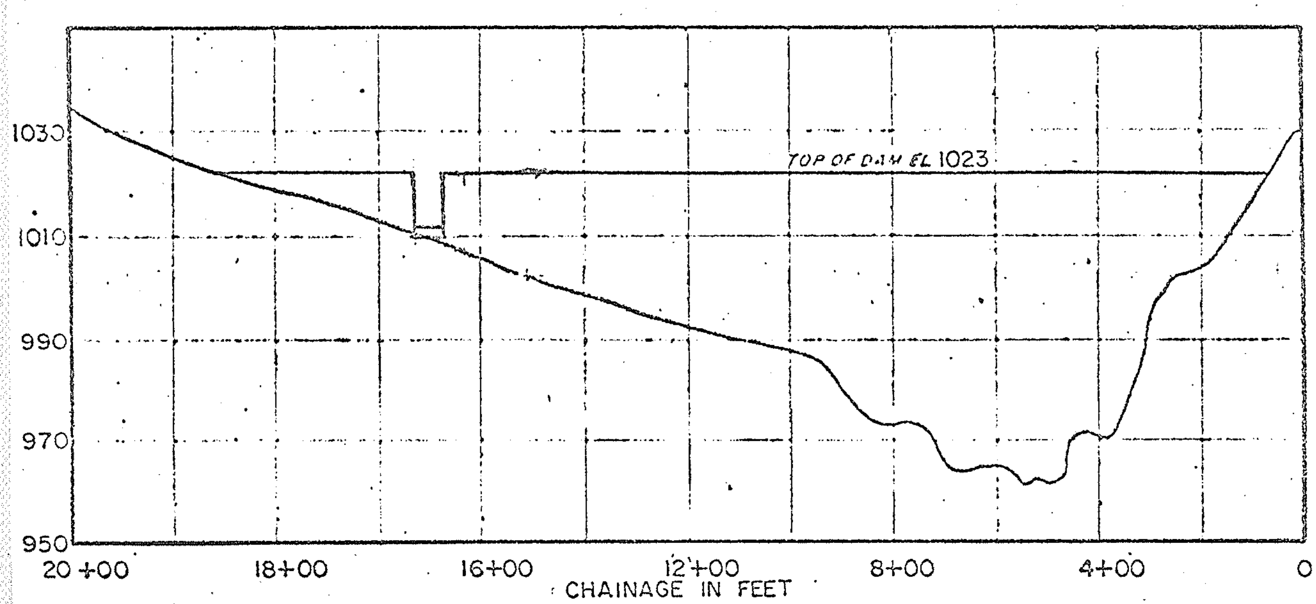
RIO SECO DAM SITE



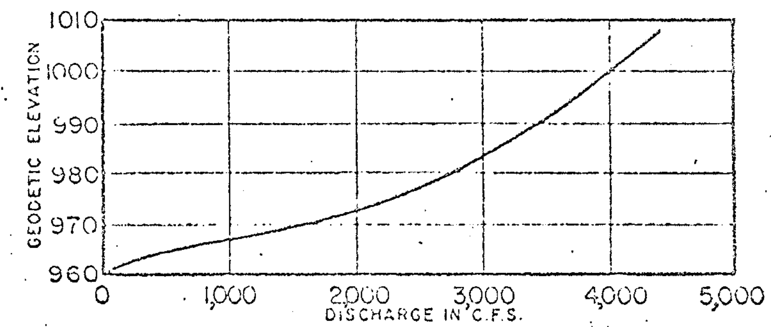
℄ SECTION



PLAN



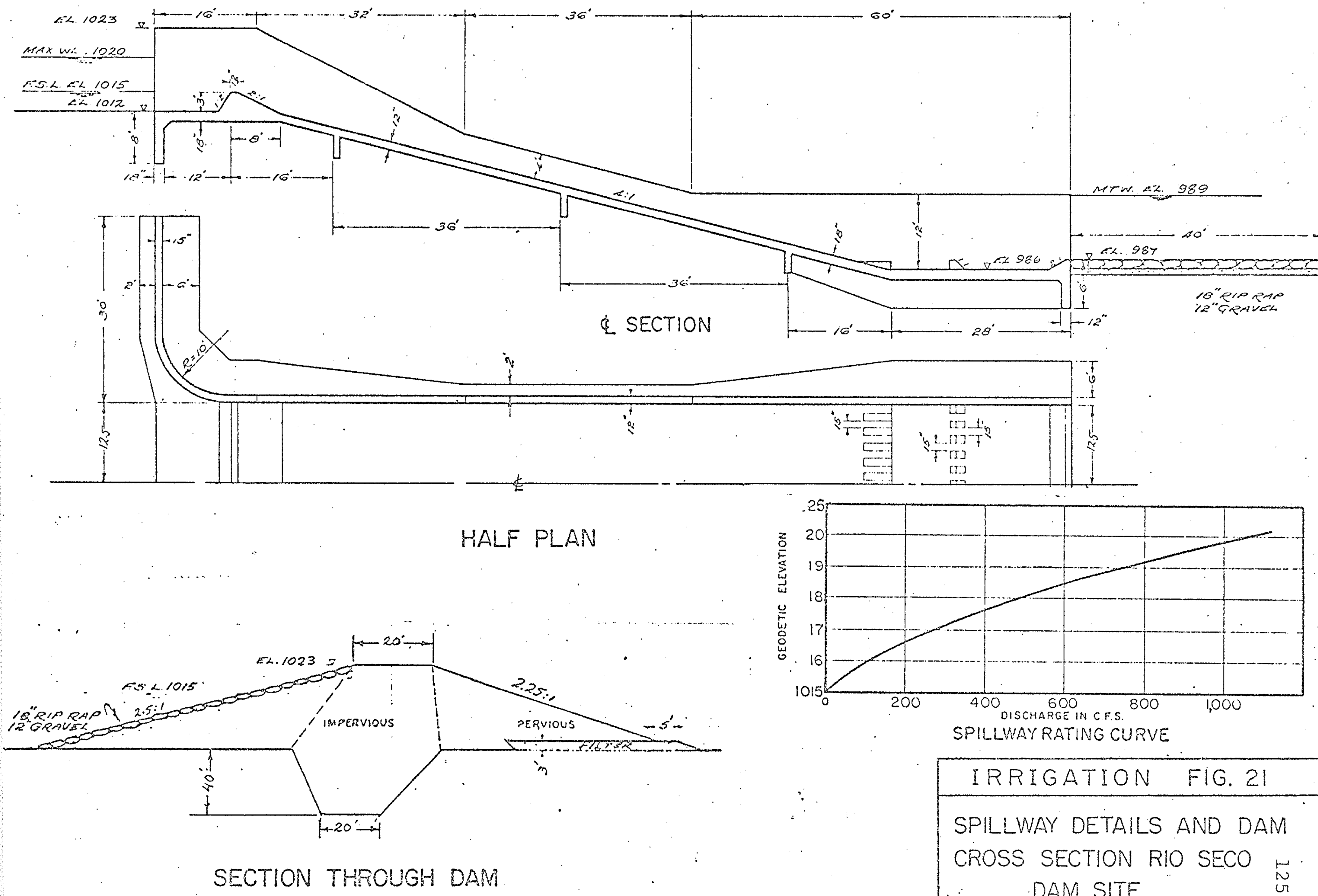
RIVER X-SEC AT ℄ DAM



CONDUIT RATING CURVE

IRRIGATION FIG. 20

CONDUIT DETAILS AND RIVER CROSS SECTION RIO SECO DAM SITE



IRRIGATION FIG. 21

SPILLWAY DETAILS AND DAM CROSS SECTION RIO SECO DAM SITE

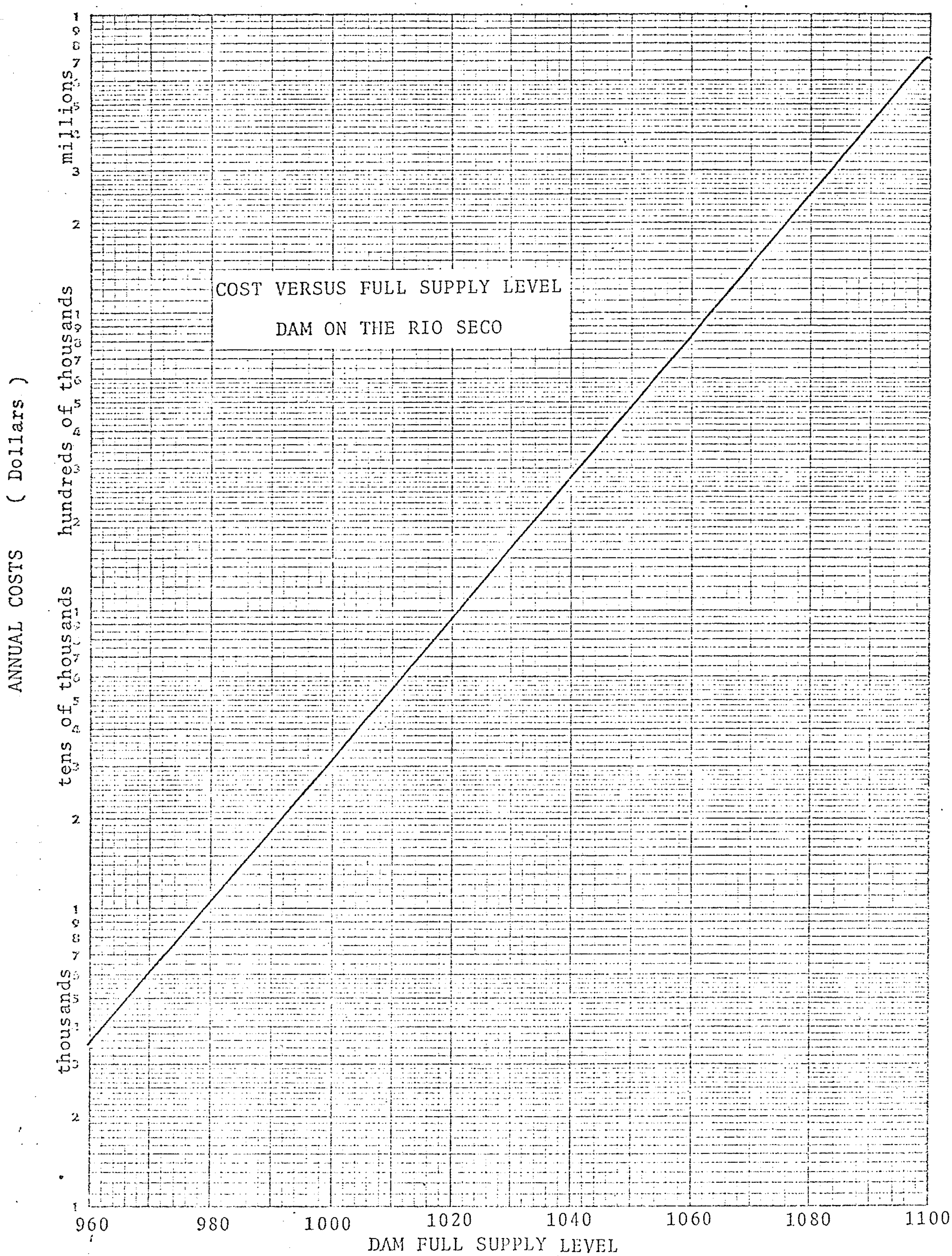
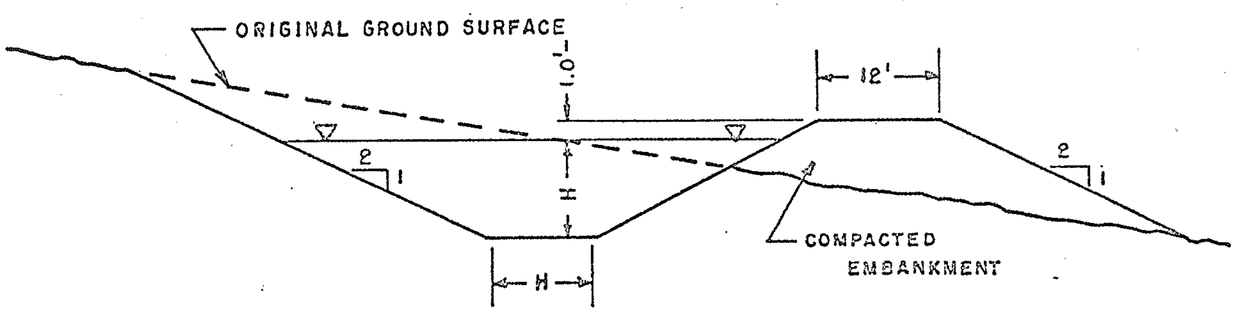

COST VERSUS FULL SUPPLY LEVEL
DAM ON THE RIO SECO
ANNUAL COSTS (Dollars)
millions
hundreds of thousands
tens of thousands
thousands
960
980
1000
1020
1040
1060
1080
1100
DAM FULL SUPPLY LEVEL

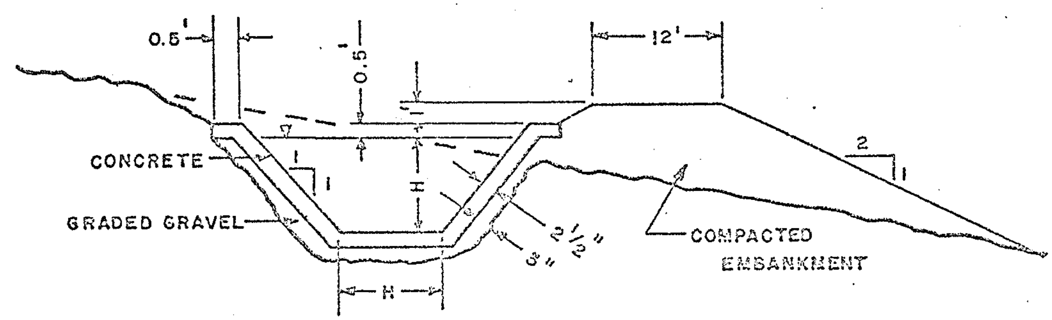
FIGURE 22

NOTES

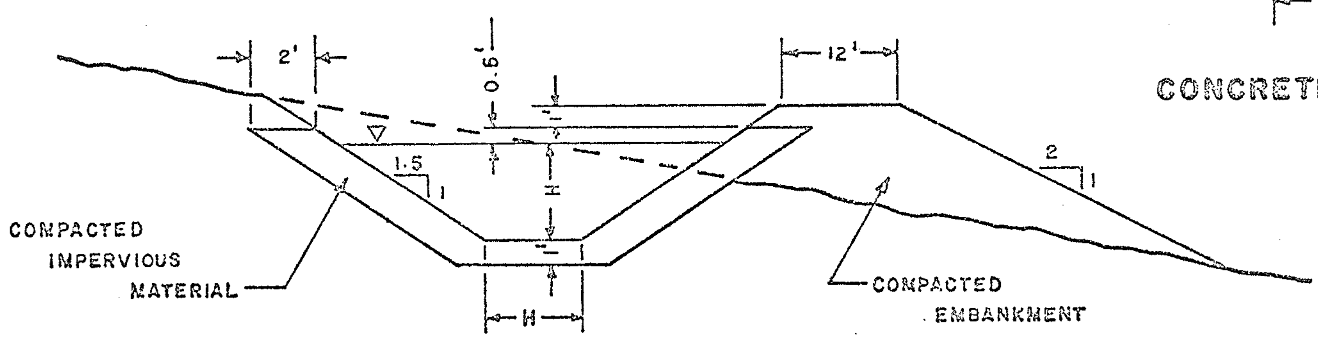
- FREEBOARD 0.5 ON LINED SURFACES
  1.0 ON UNLINED SURFACES
- OPERATION AND MAINTENANCE ROADWAY PROVIDED ON EMBANKMENT.
- H = CANAL DIMENSION IN FEET.
- ∇ = NORMAL WATER SURFACE
- SCALE = N.T.S.



EARTH SECTION



CONCRETE-LINED SECTION



EARTH-LINED SECTION

IRRIGATION - FIG. 23

TYPICAL MAIN CANAL
CROSS SECTIONS

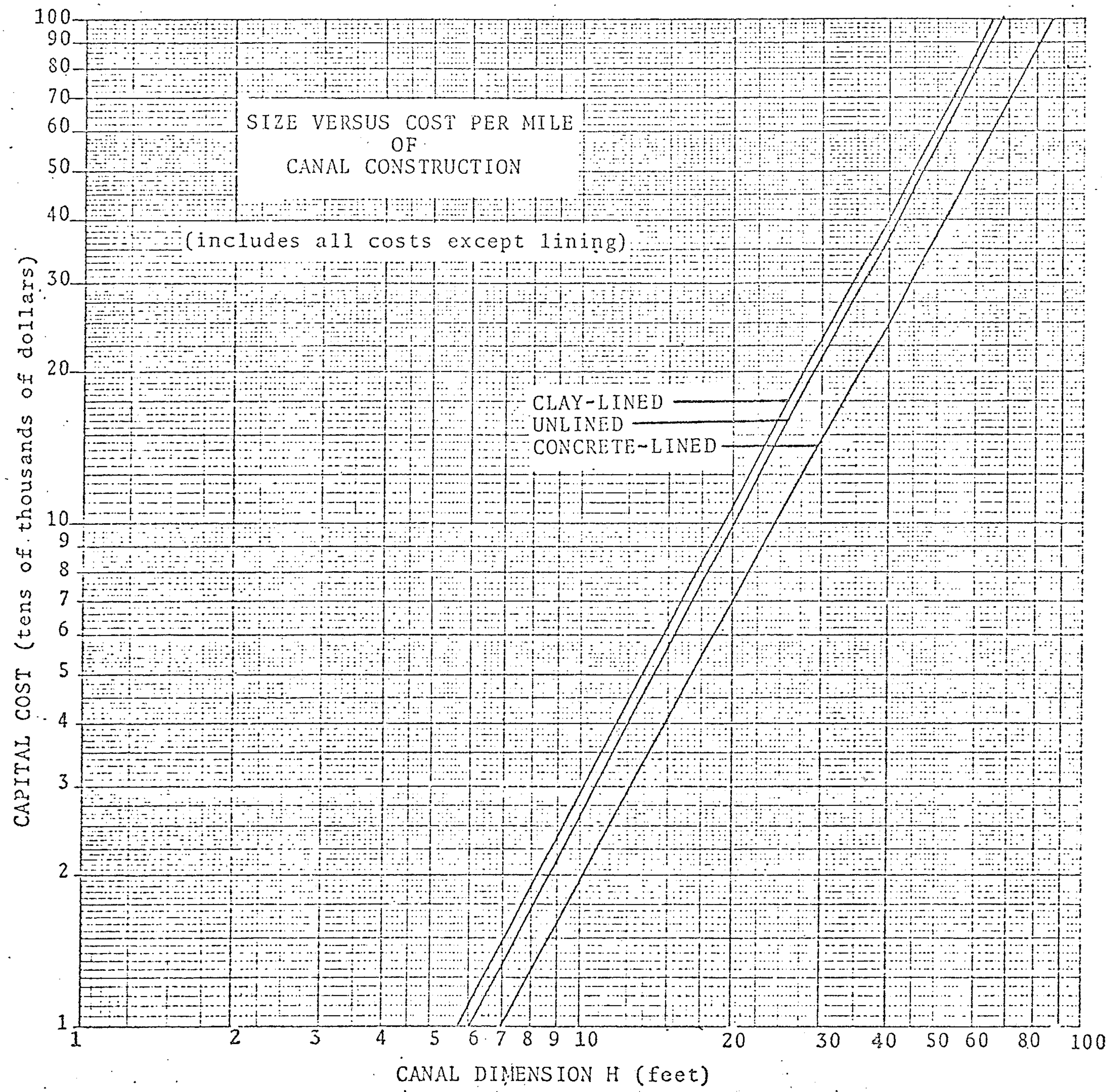

SIZE VERSUS COST PER MILE
OF
CANAL CONSTRUCTION
(includes all costs except lining)
CLAY-LINED
UNLINED
CONCRETE-LINED
CAPITAL COST (tens of thousands of dollars)
100
90
80
70
60
50
40
30
20
10
9
8
7
6
5
4
3
2
1
1
2
3
4
5
6
7
8
9
10
20
30
40
50
60
80
100
CANAL DIMENSION H (feet)

FIGURE 24

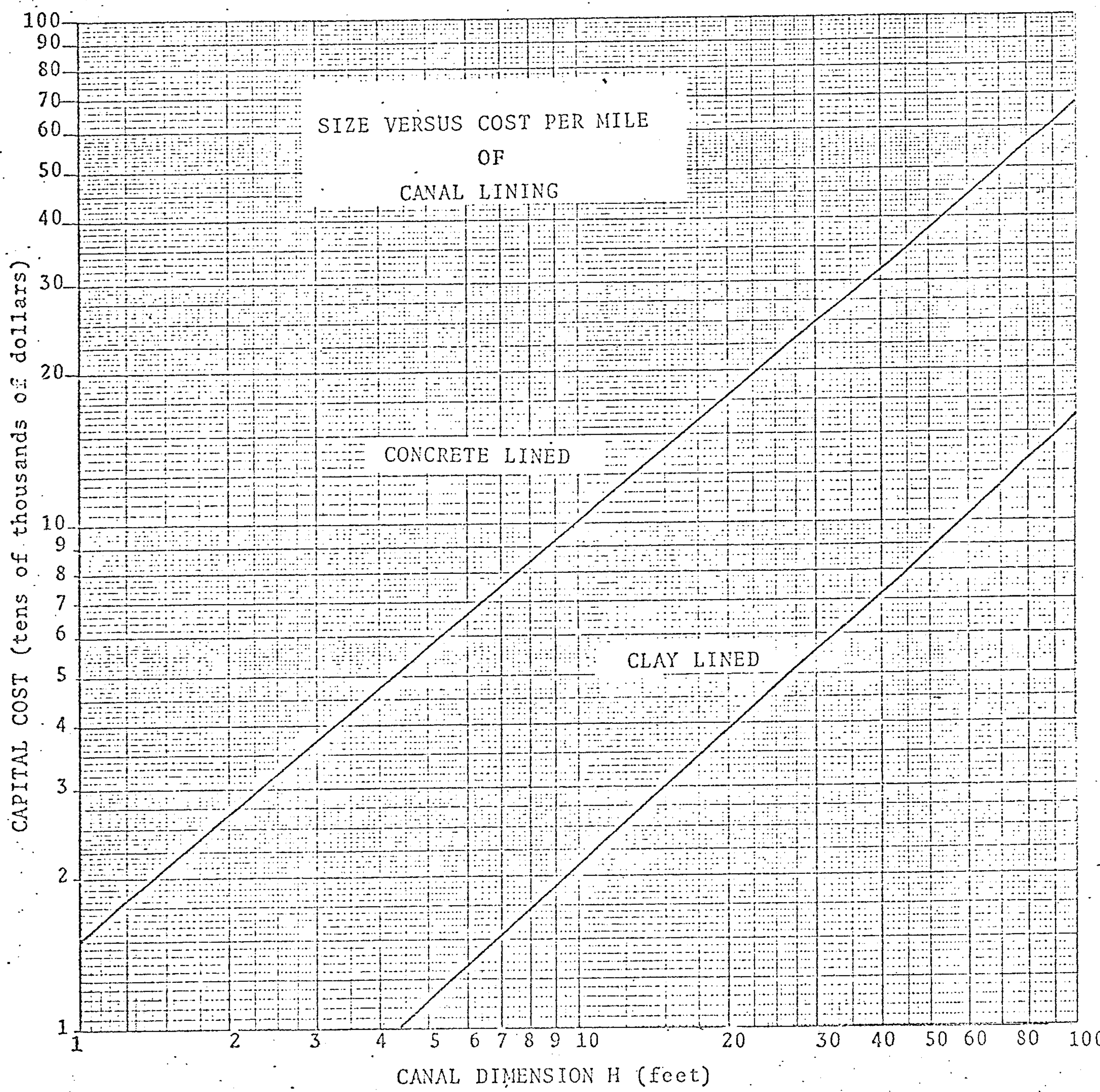

SIZE VERSUS COST PER MILE
OF
CANAL LINING
CONCRETE LINED
CLAY LINED
CAPITAL COST (tens of thousands of dollars)
CANAL DIMENSION H (feet)
100
90
80
70
60
50
40
30
20
10
9
8
7
6
5
4
3
2
1
1
2
3
4
5
6
7
8
9
10
20
30
40
50
60
80
100

FIGURE 25

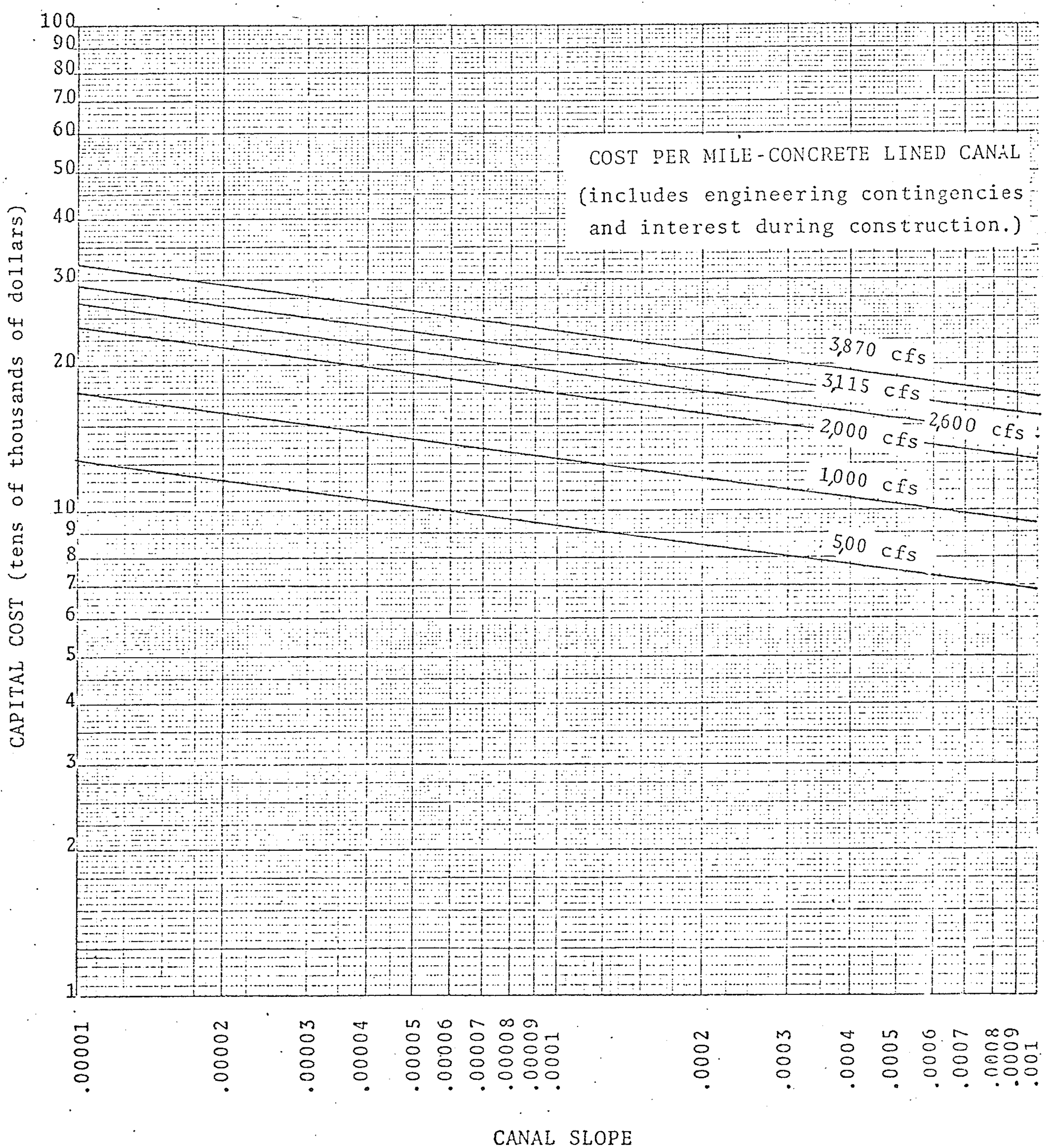

COST PER MILE-CONCRETE LINED CANAL
(includes engineering contingencies and interest during construction.)
CAPITAL COST (tens of thousands of dollars)
100
90
80
70
60
50
40
30
20
10
9
8
7
6
5
4
3
2
1
3,870 cfs
3,115 cfs
2,600 cfs
2,000 cfs
1,000 cfs
500 cfs
.00001
.00002
.00003
.00004
.00005
.00006
.00007
.00008
.00009
.0001
.0002
.0003
.0004
.0005
.0006
.0007
.0008
.0009
.001
CANAL SLOPE

FIGURE 26

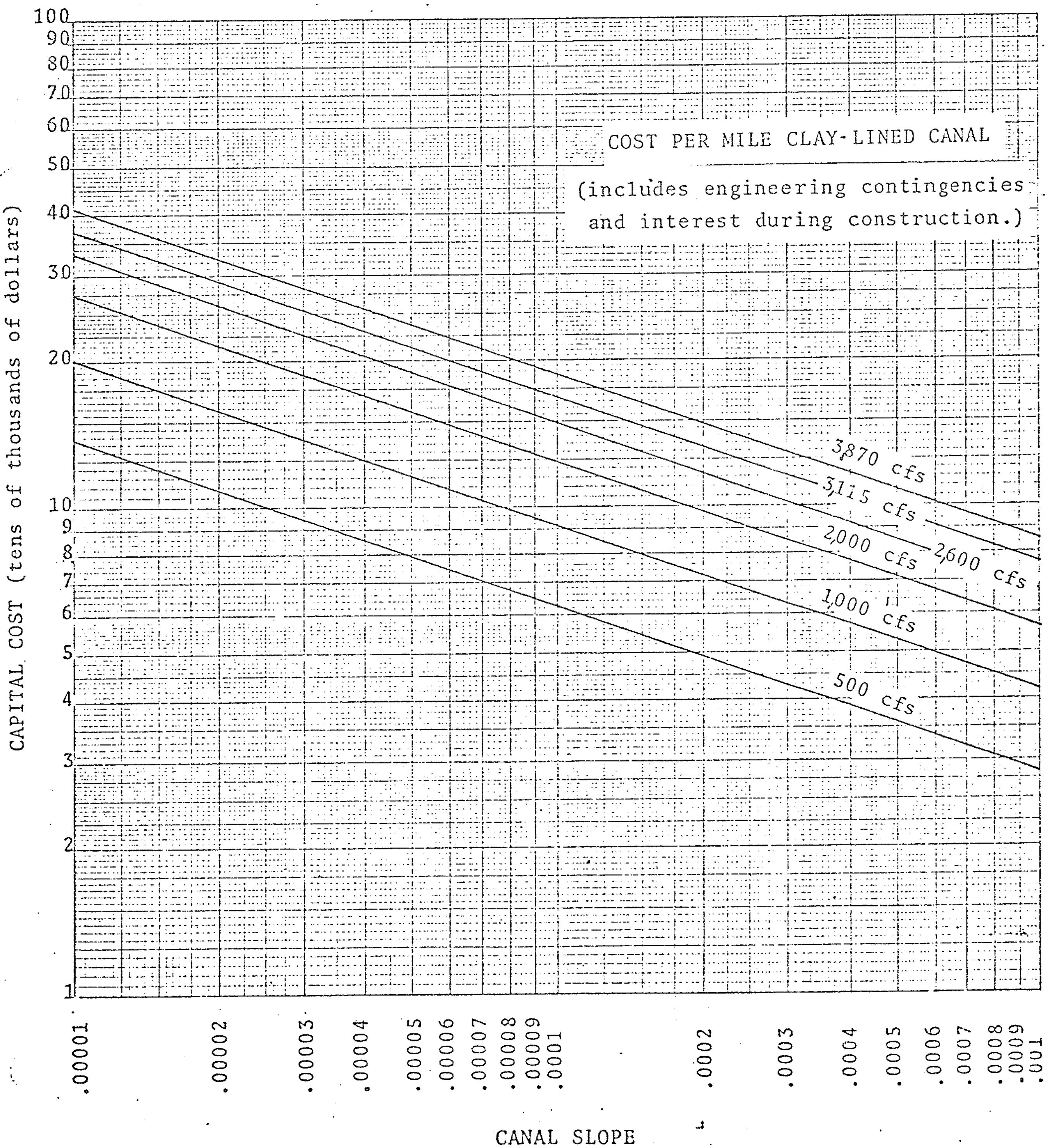

COST PER MILE CLAY-LINED CANAL
(includes engineering contingencies and interest during construction.)
3,870 cfs
3,115 cfs
2,600 cfs
2,000 cfs
1,000 cfs
500 cfs
CAPITAL COST (tens of thousands of dollars)
100
90
80
70
60
50
40
30
20
10
9
8
7
6
5
4
3
2
1
.00001
.00002
.00003
.00004
.00005
.00006
.00007
.00008
.00009
.0001
.0002
.0003
.0004
.0005
.0006
.0007
.0008
.0009
.001
CANAL SLOPE

FIGURE 27

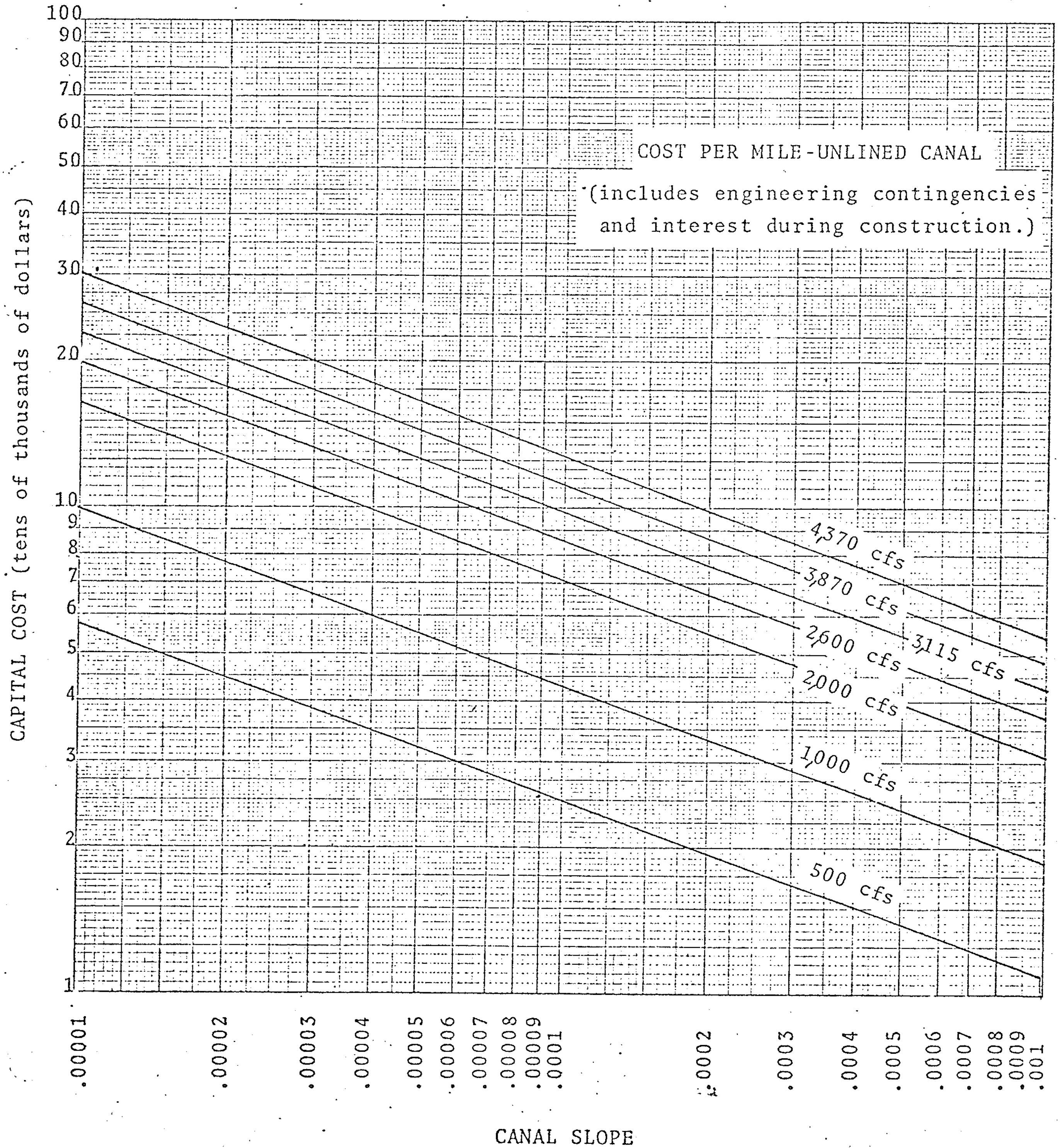
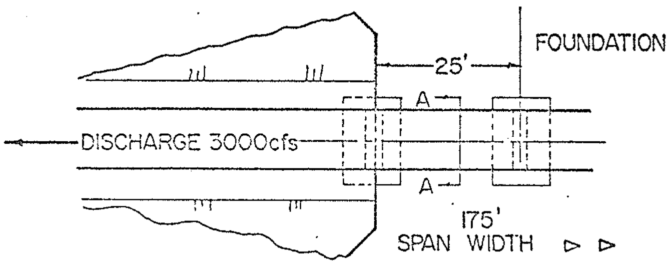
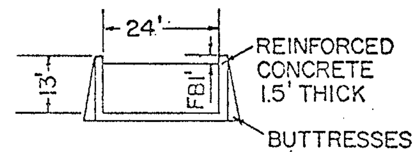

COST PER MILE-UNLINED CANAL
(includes engineering contingencies and interest during construction.)
CAPITAL COST (tens of thousands of dollars)
CANAL SLOPE
4,370 cfs
3,870 cfs
3,115 cfs
2,600 cfs
2,000 cfs
1,000 cfs
500 cfs
100
90
80
70
60
50
40
30
20
10
9
8
7
6
5
4
3
2
1
.00001
.00002
.00003
.00004
.00005
.00006
.00007
.00008
.00009
.0001
.0002
.0003
.0004
.0005
.0006
.0007
.0008
.0009
.001

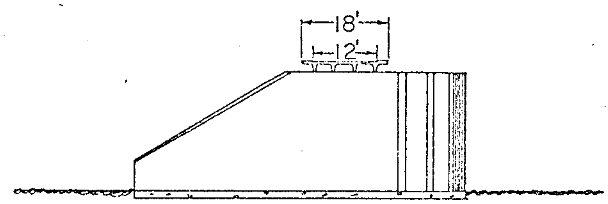
FIGURE 28



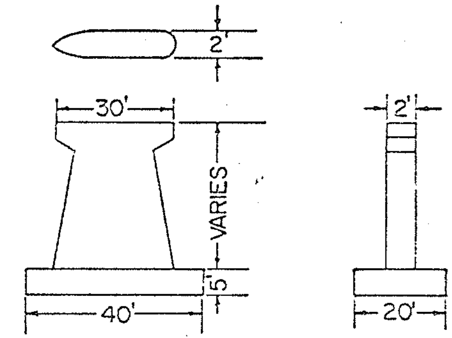
TYPICAL FLUME PLAN



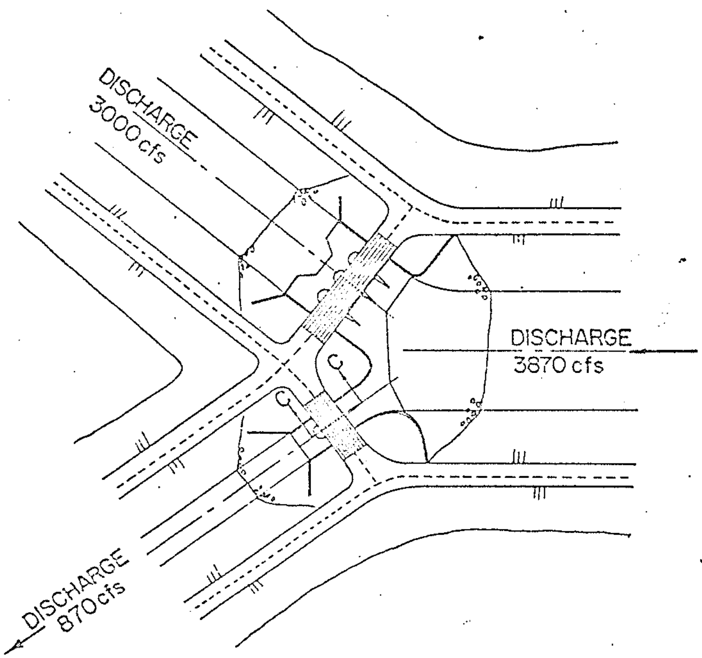
SECTION AA FLUME OVER RIO SECO



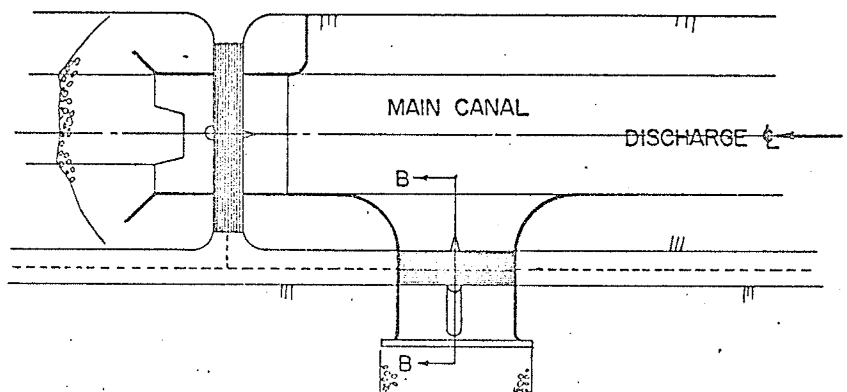
SECTION CC



TYPICAL BRIDGE PIER



REGULATORS AT MAIN CANAL BIFURCATION

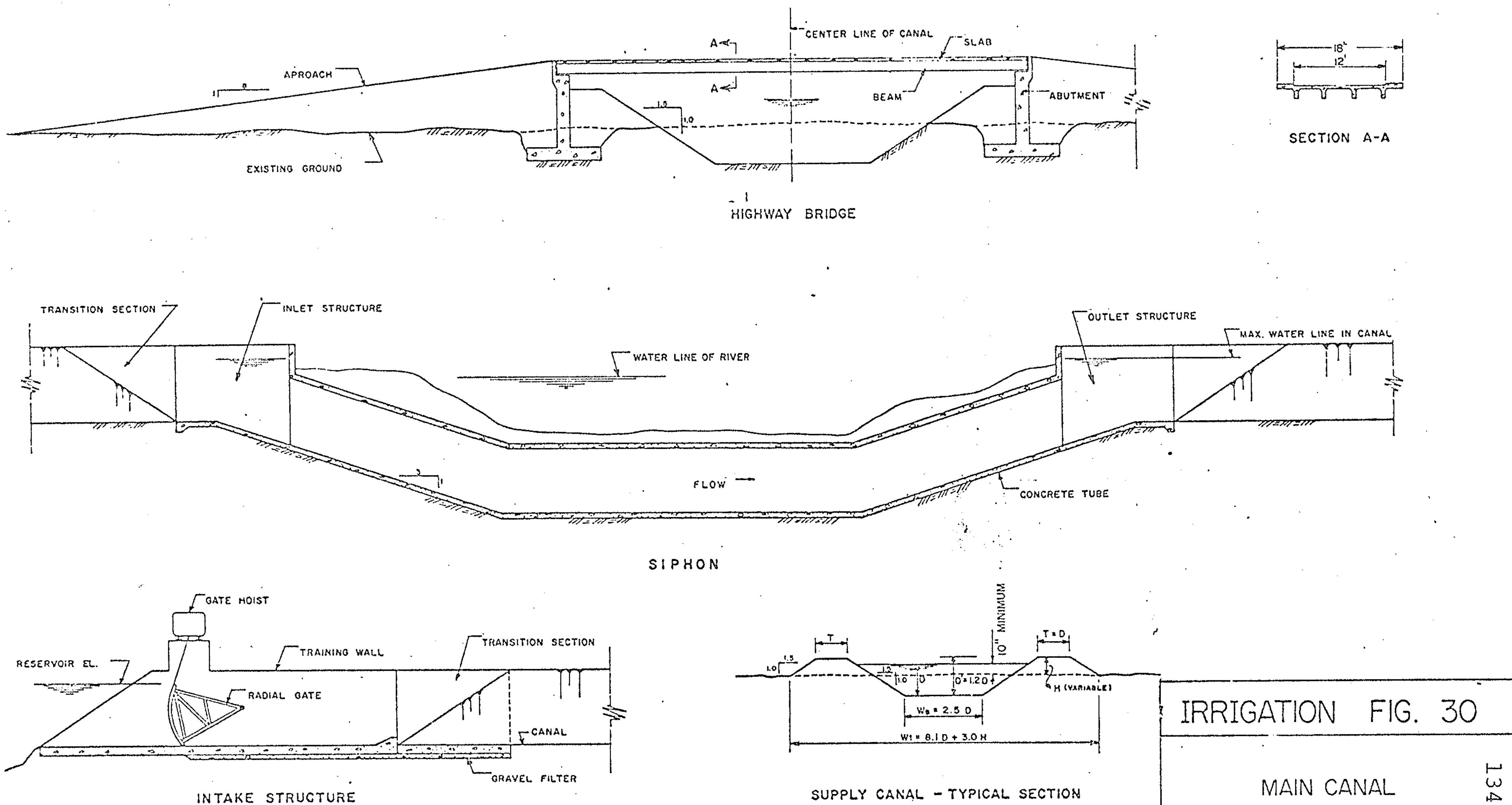


TYPICAL WASTE WAY AND DISTRIBUTION REGULATOR

- WING WALLS DOUBLE LINE
- RIP RAP
- CONCRETE BRIDGE
- ROADS

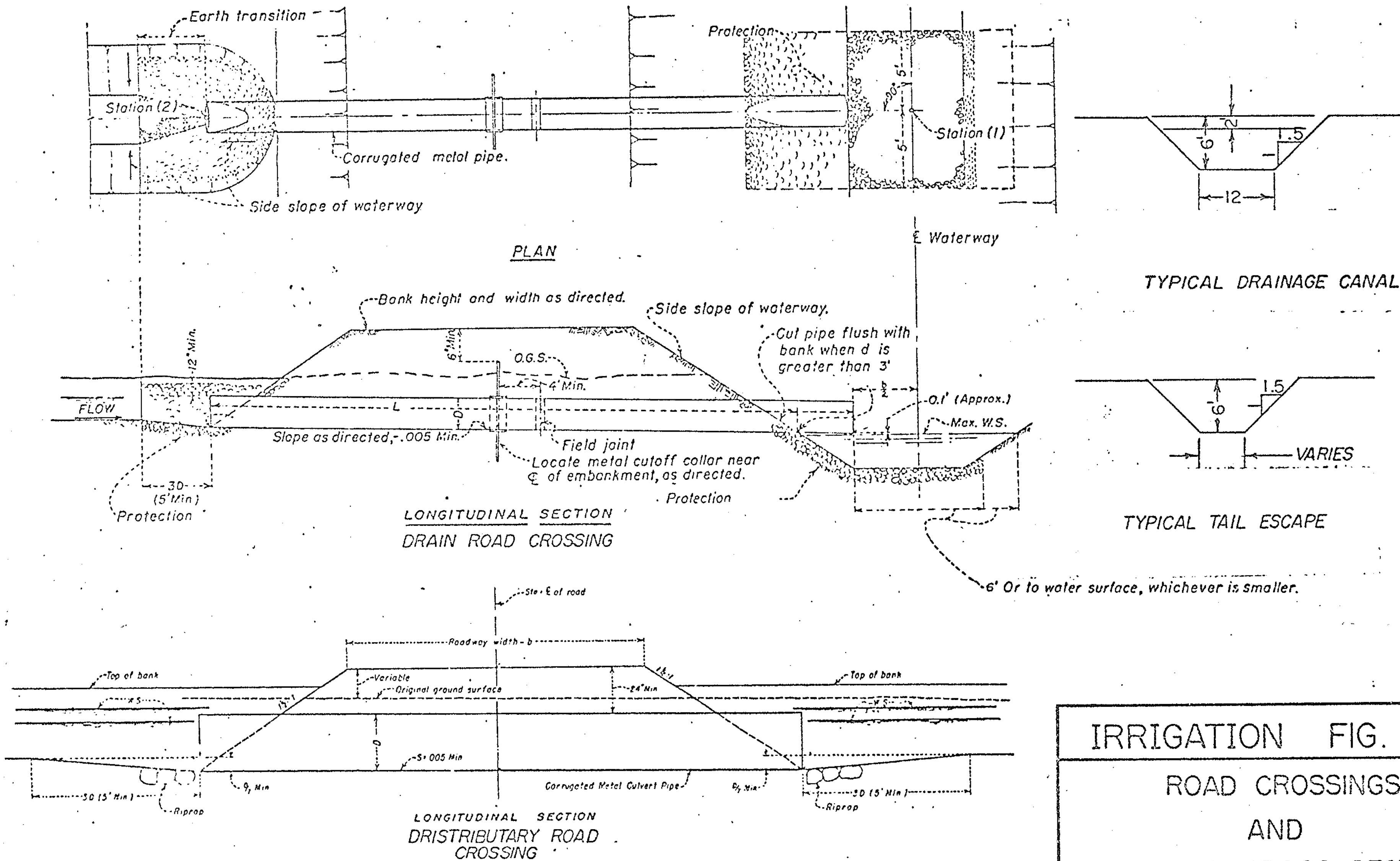
IRRIGATION FIG. 29

TYPICAL MAIN CANAL REGULATOR AND WASTE WAY STRUCTURES



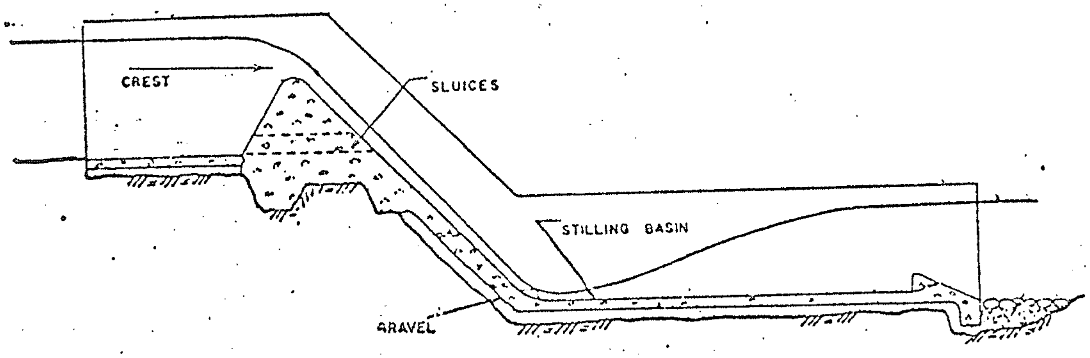
IRRIGATION FIG. 30

MAIN CANAL
TYPICAL STRUCTURES

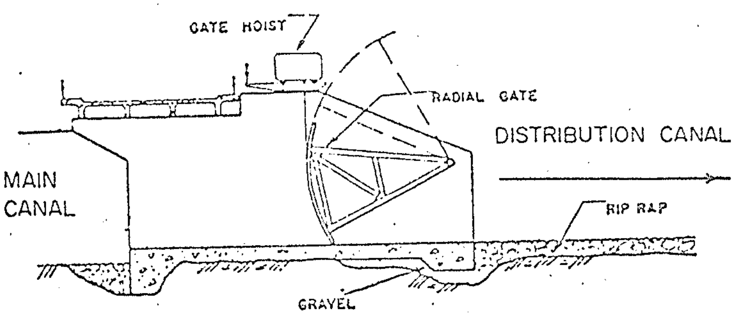


IRRIGATION FIG. 31

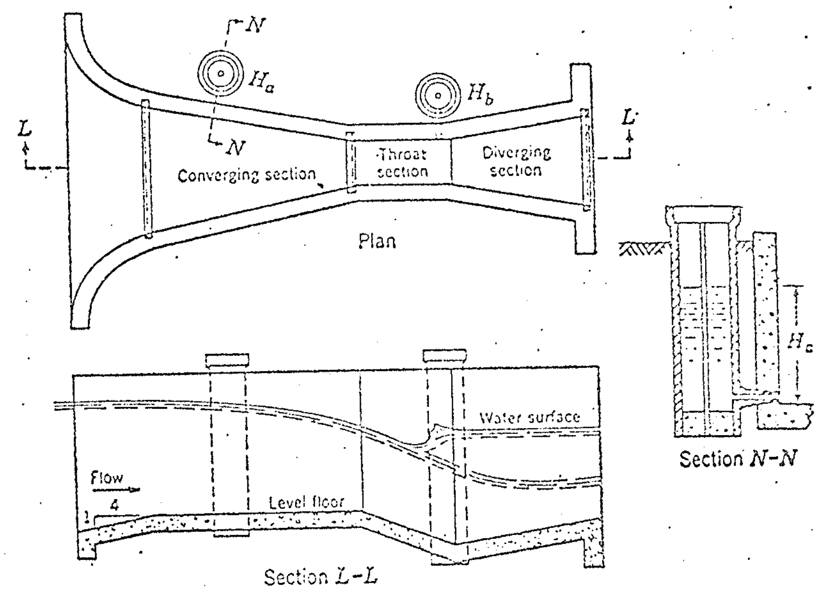
ROAD CROSSINGS
AND
DRAIN CROSS SECTIONS



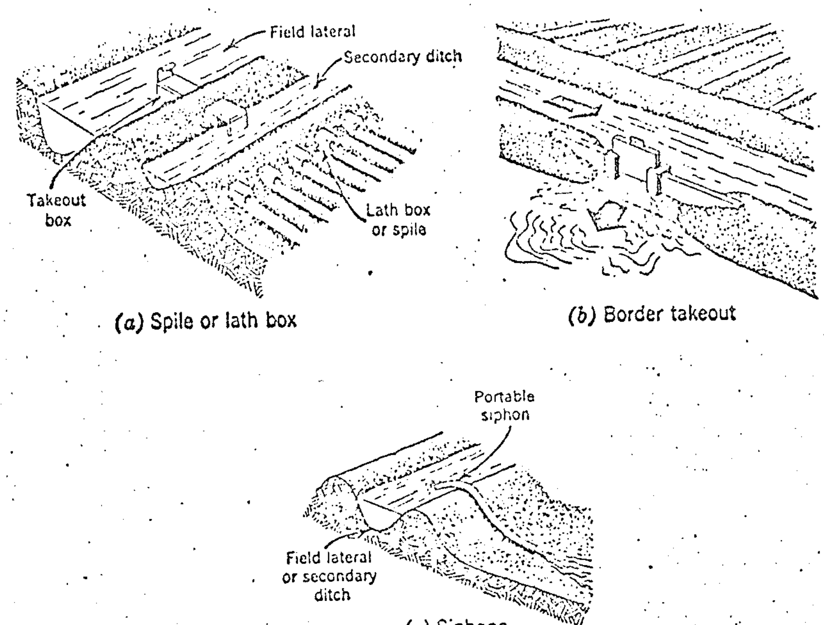
DROP STRUCTURE



GATE STRUCTURE

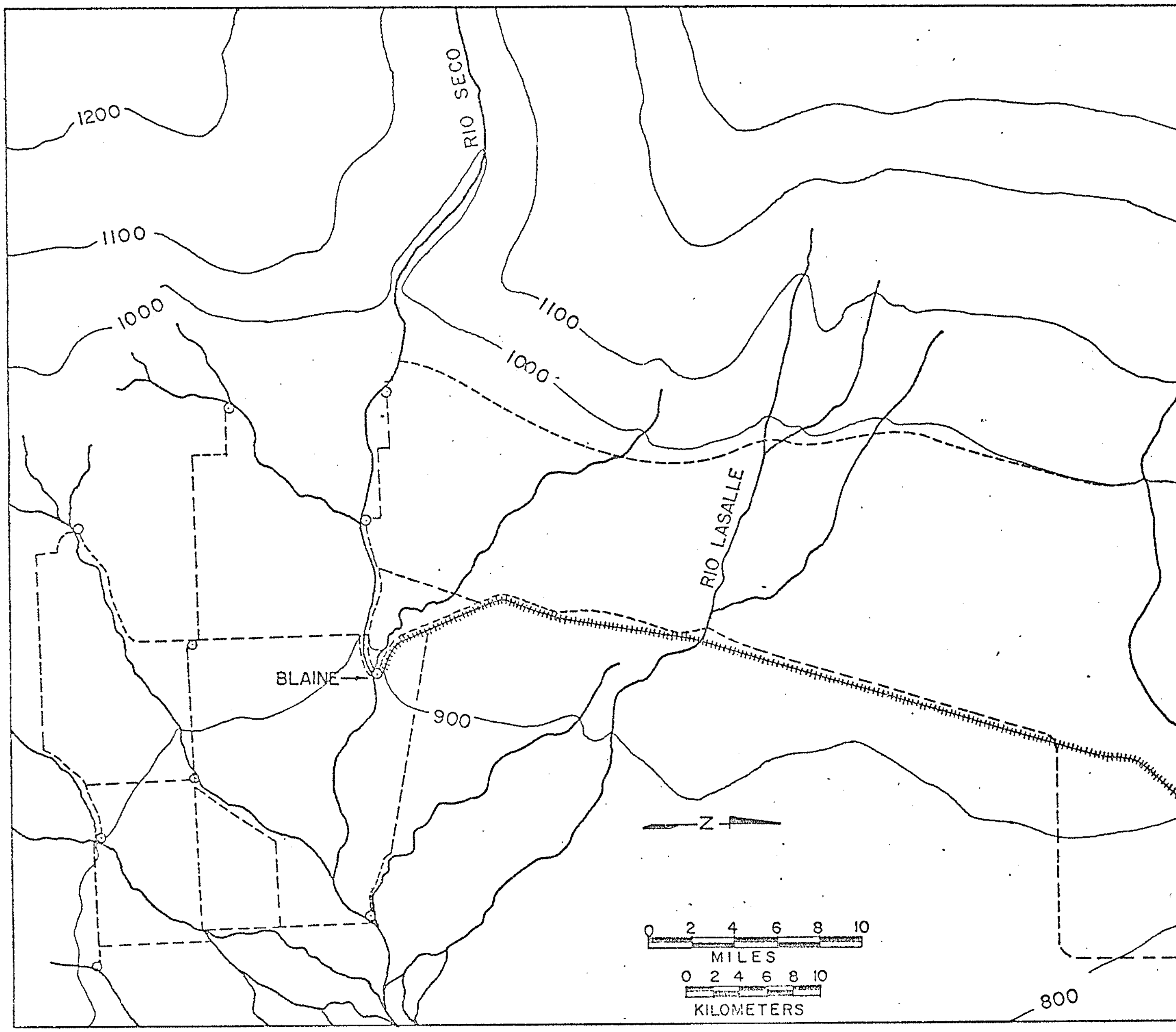


PARSHALL FLUME



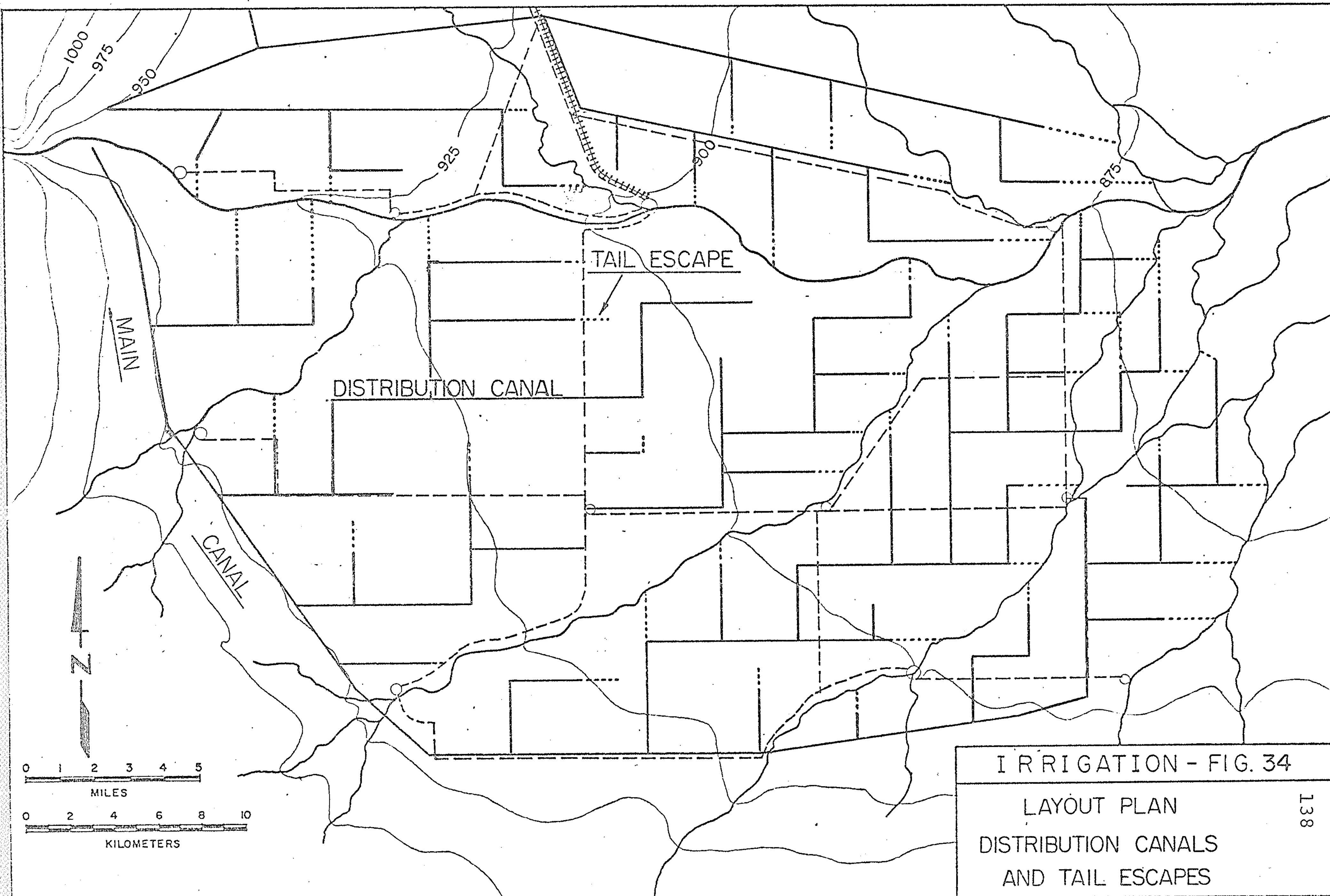
FARM TAKEOUTS

IRRIGATION	FIG. 32
TYPICAL DISTRIBUTION STRUCTURES	136



IRRIGATION FIG. 33

MAIN CANAL ROUTE

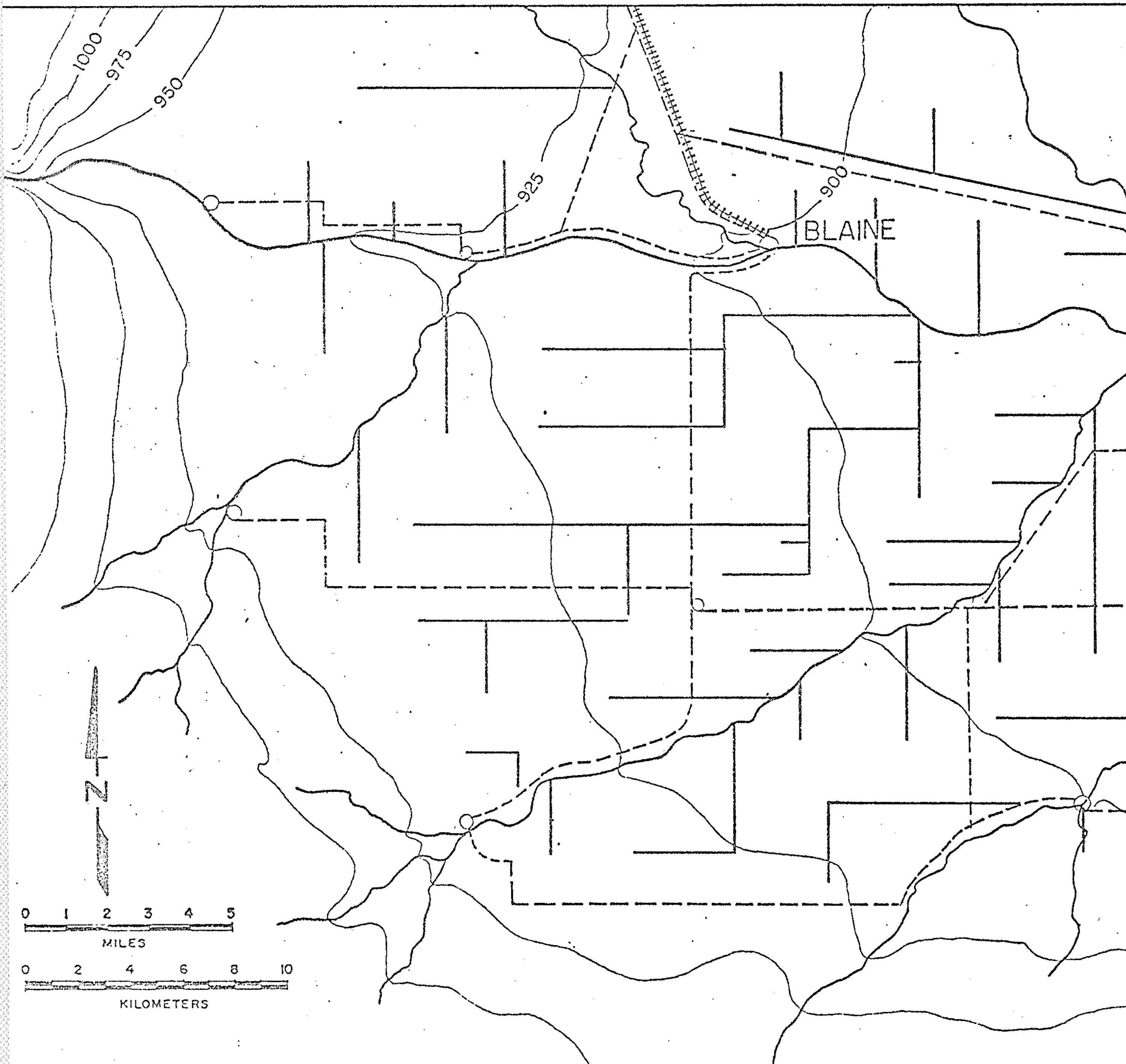


IRRIGATION - FIG. 34

LAYOUT PLAN

DISTRIBUTION CANALS

AND TAIL ESCAPES



IRRIGATION - FIG. 35

LAYOUT PLAN DRAINS

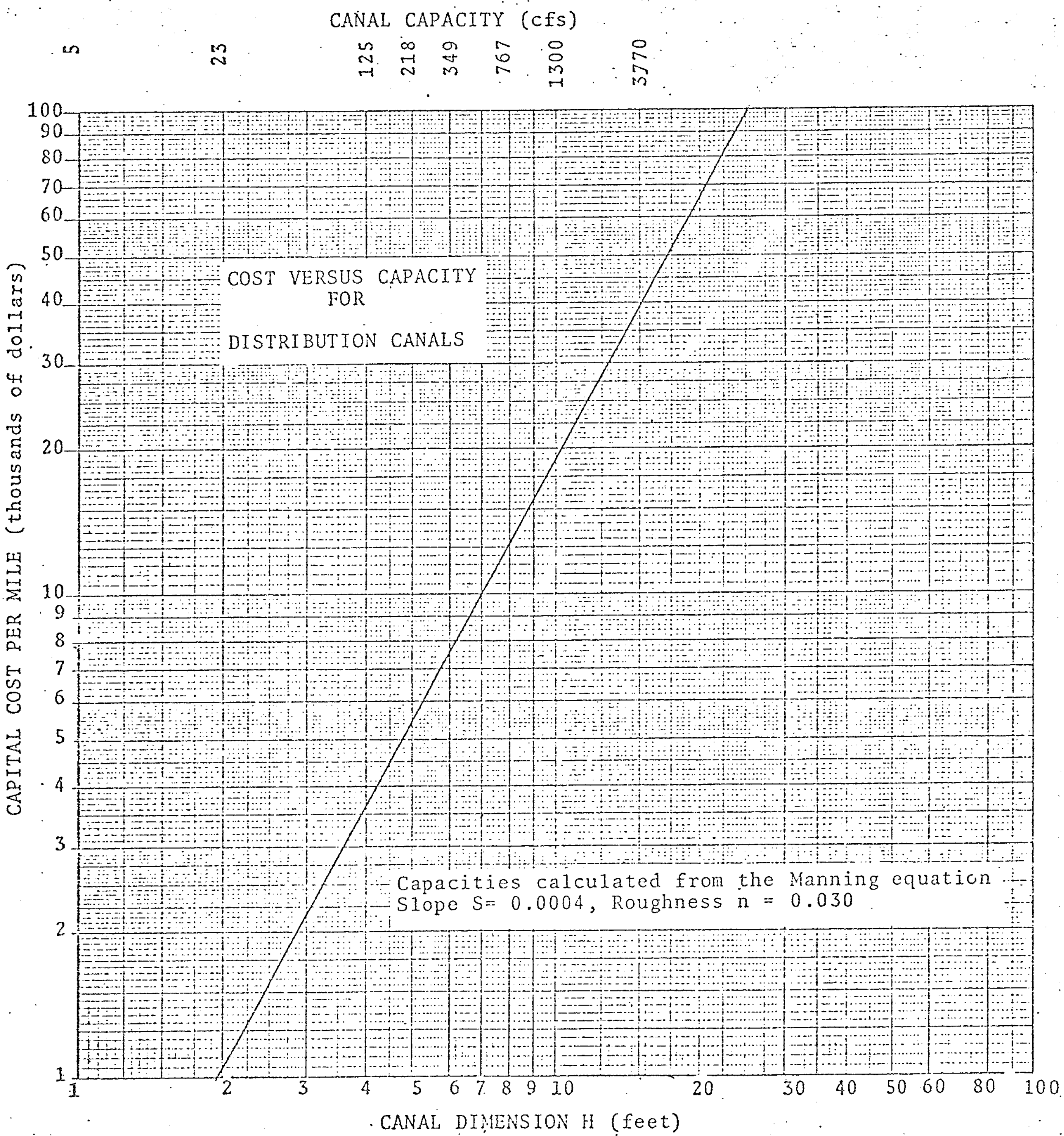

CANAL CAPACITY (cfs)
5
23
125
218
349
767
1300
3770
COST VERSUS CAPACITY
FOR
DISTRIBUTION CANALS
Capacities calculated from the Manning equation
Slope S= 0.0004, Roughness n = 0.030
CAPITAL COST PER MILE (thousands of dollars)
100
90
80
70
60
50
40
30
20
10
9
8
7
6
5
4
3
2
1
1
2
3
4
5
6
7
8
9
10
20
30
40
50
60
80
100
CANAL DIMENSION H (feet)

FIGURE 36

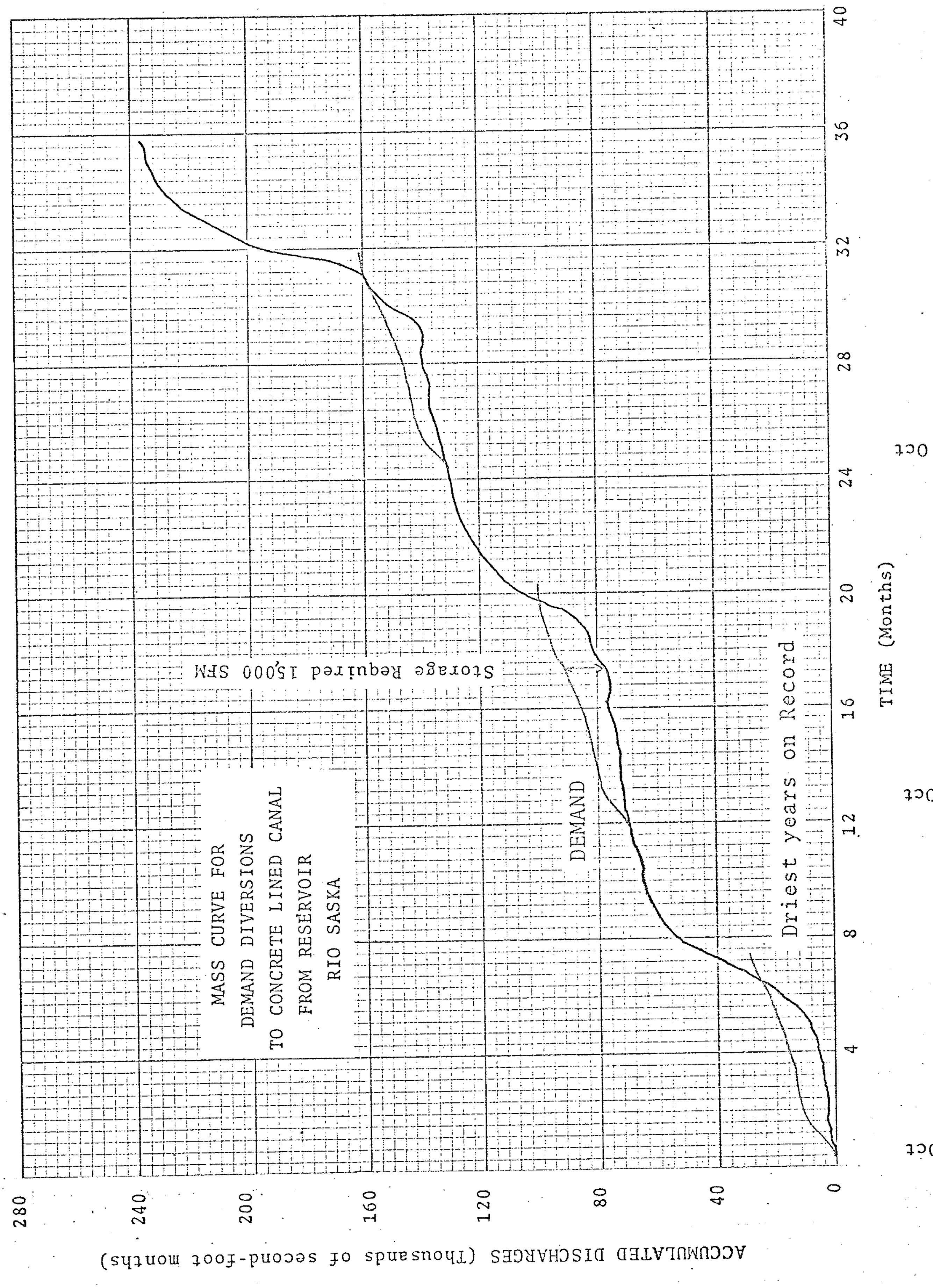

MASS CURVE FOR
DEMAND DIVERSIONS
TO CONCRETE LINED CANAL
FROM RESERVOIR
RIO SASKA
DEMAND
Storage Required 15,000 SFM
Driest years on Record
Oct
Oct
Oct
0
4
8
12
16
20
24
28
32
36
40
TIME (Months)
0
40
80
120
160
200
240
280
ACCUMULATED DISCHARGES (Thousands of second-foot months)

FIGURE 37

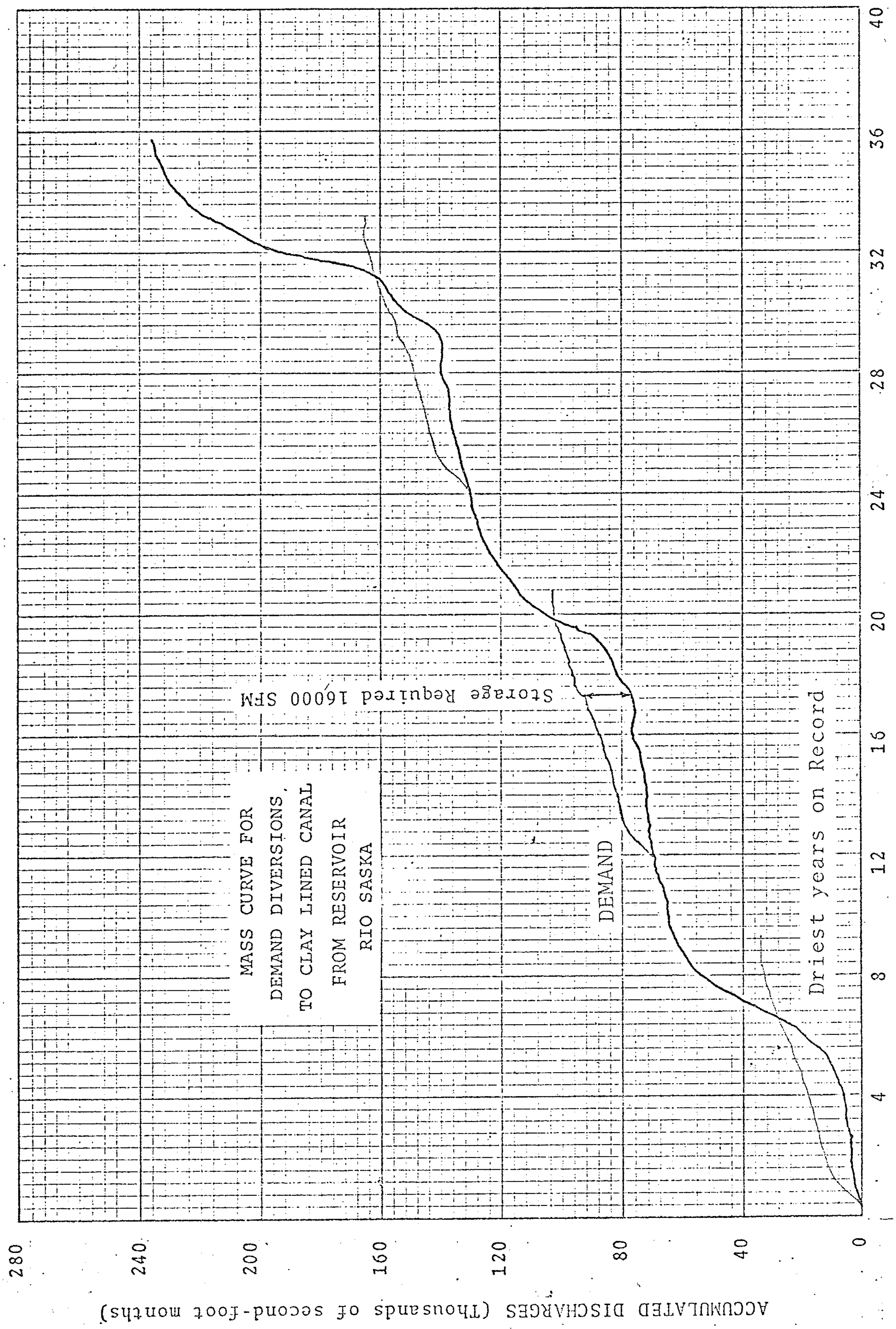

MASS CURVE FOR
DEMAND DIVERSIONS
TO CLAY LINED CANAL
FROM RESERVOIR
RIO SASKA
Storage Required 16000 SFM
DEMAND
Driest years on Record
ACCUMULATED DISCHARGES (Thousands of second-foot months)
0
40
80
120
160
200
240
280
4
8
12
16
20
24
28
32
36
40

FIGURE 38

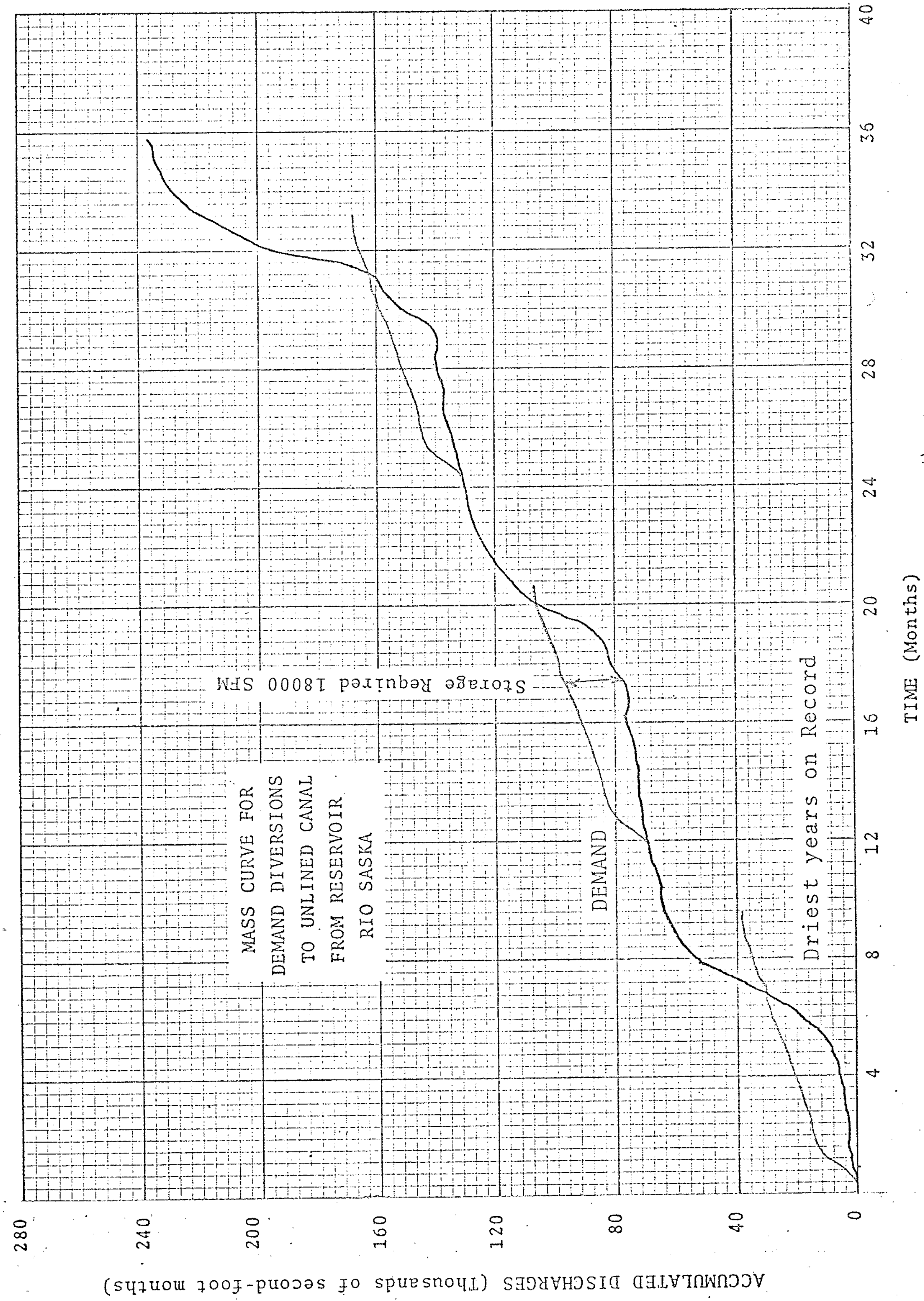

MASS CURVE FOR
DEMAND DIVERSIONS
TO UNLINED CANAL
FROM RESERVOIR
RIO SASKA
Storage Required 18000 SFM
DEMAND
Driest years on Record
Oct
Oct
Oct
0
4
8
12
16
20
24
28
32
36
40
TIME (Months)
0
40
80
120
160
200
240
280
ACCUMULATED DISCHARGES (Thousands of second-foot months)

FIGURE 39

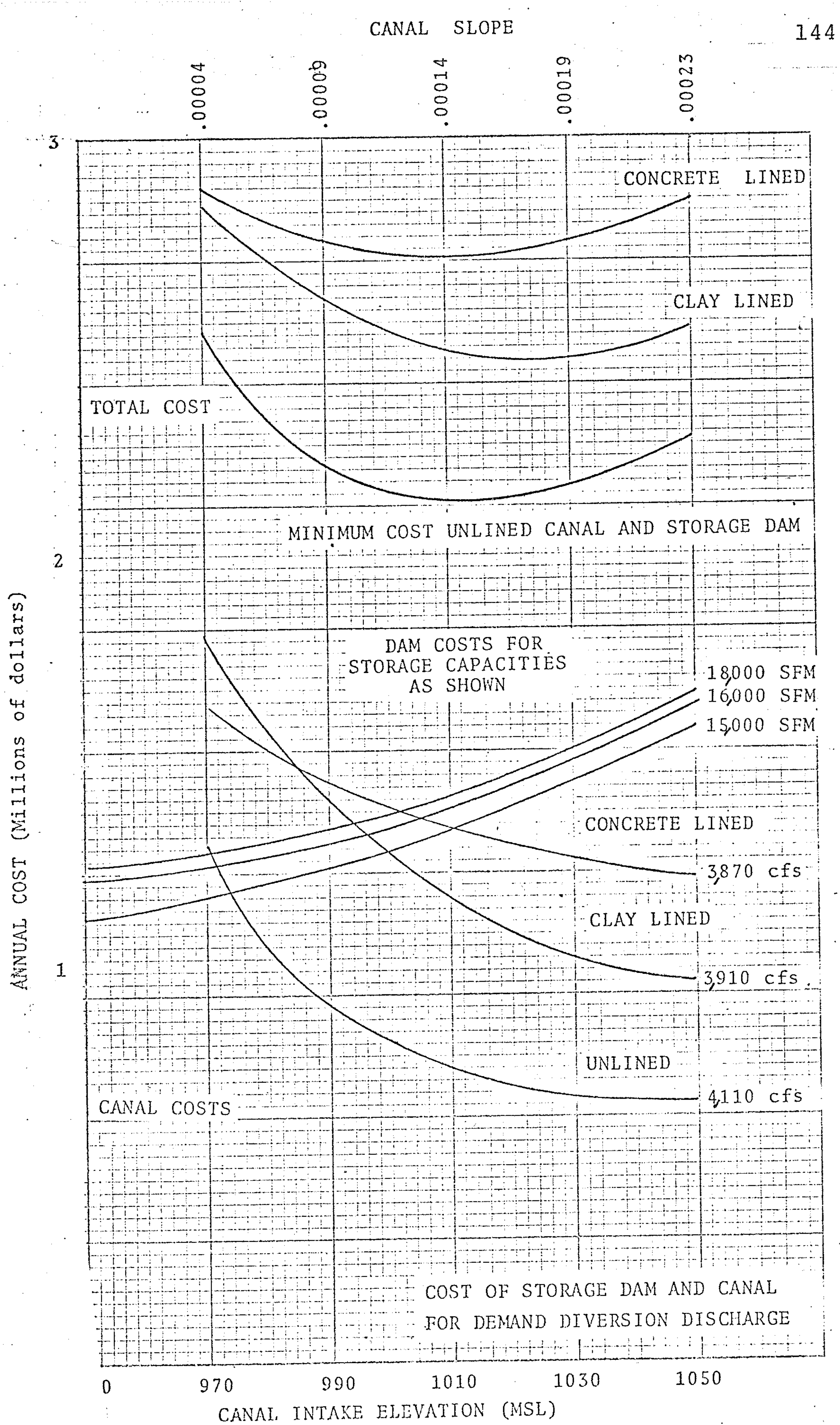

CANAL SLOPE
.00004
.00009
.00014
.00019
.00023
3
2
1
ANNUAL COST (Millions of dollars)
CONCRETE LINED
CLAY LINED
TOTAL COST
MINIMUM COST UNLINED CANAL AND STORAGE DAM
DAM COSTS FOR
STORAGE CAPACITIES
AS SHOWN
18,000 SFM
16,000 SFM
15,000 SFM
CONCRETE LINED
3,870 cfs
CLAY LINED
3,910 cfs
UNLINED
4,110 cfs
CANAL COSTS
COST OF STORAGE DAM AND CANAL
FOR DEMAND DIVERSION DISCHARGE
0
970
990
1010
1030
1050
CANAL INTAKE ELEVATION (MSL)

FIGURE 40

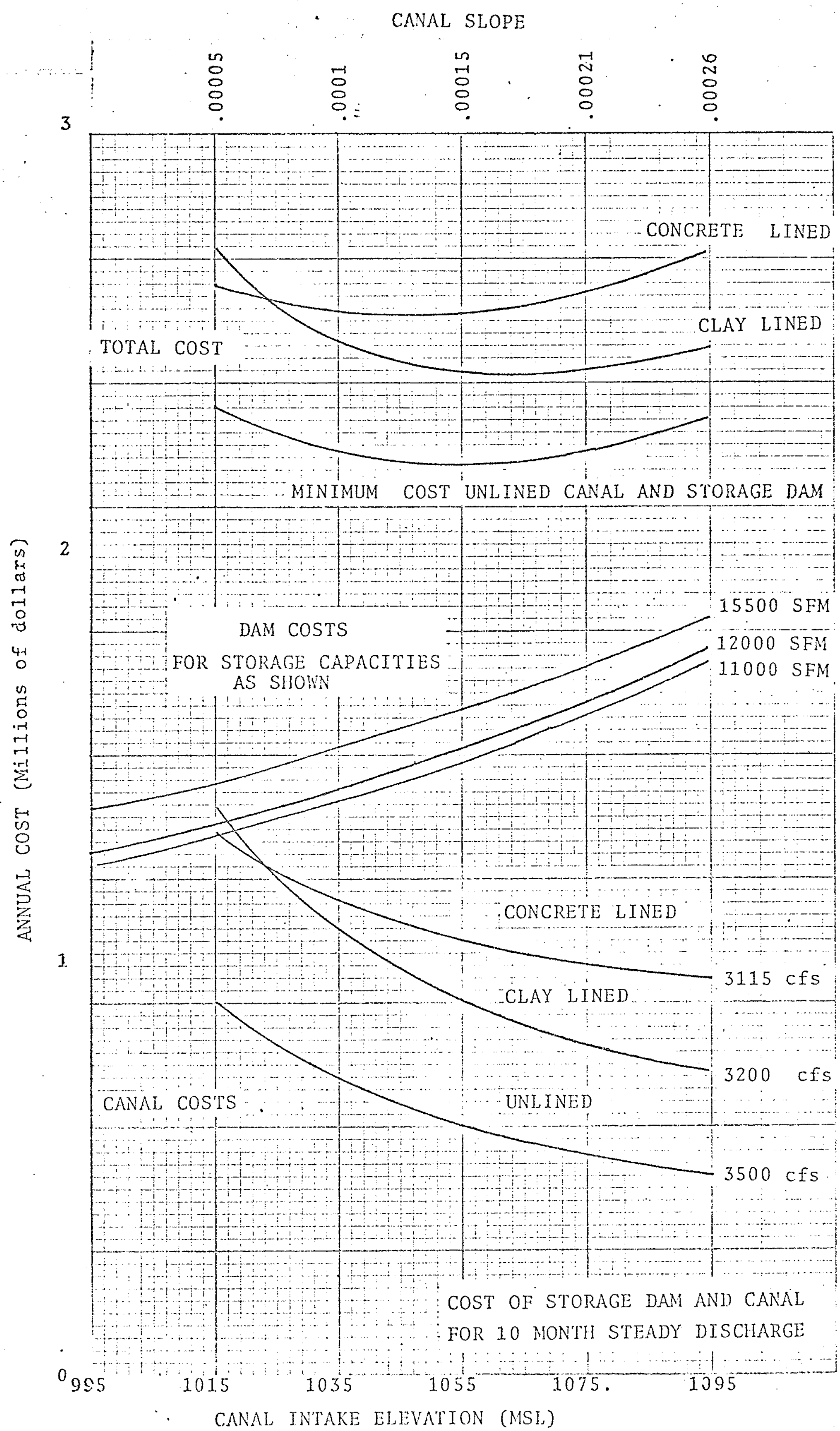

CANAL SLOPE
.00005
.0001
.00015
.00021
.00026
ANNUAL COST (Millions of dollars)
3
2
1
0
CONCRETE LINED
CLAY LINED
TOTAL COST
MINIMUM COST UNLINED CANAL AND STORAGE DAM
15500 SFM
12000 SFM
11000 SFM
DAM COSTS
FOR STORAGE CAPACITIES
AS SHOWN
CONCRETE LINED
3115 cfs
CLAY LINED
3200 cfs
CANAL COSTS
UNLINED
3500 cfs
COST OF STORAGE DAM AND CANAL
FOR 10 MONTH STEADY DISCHARGE
995
1015
1035
1055
1075.
1095
CANAL INTAKE ELEVATION (MSL)

FIGURE 41

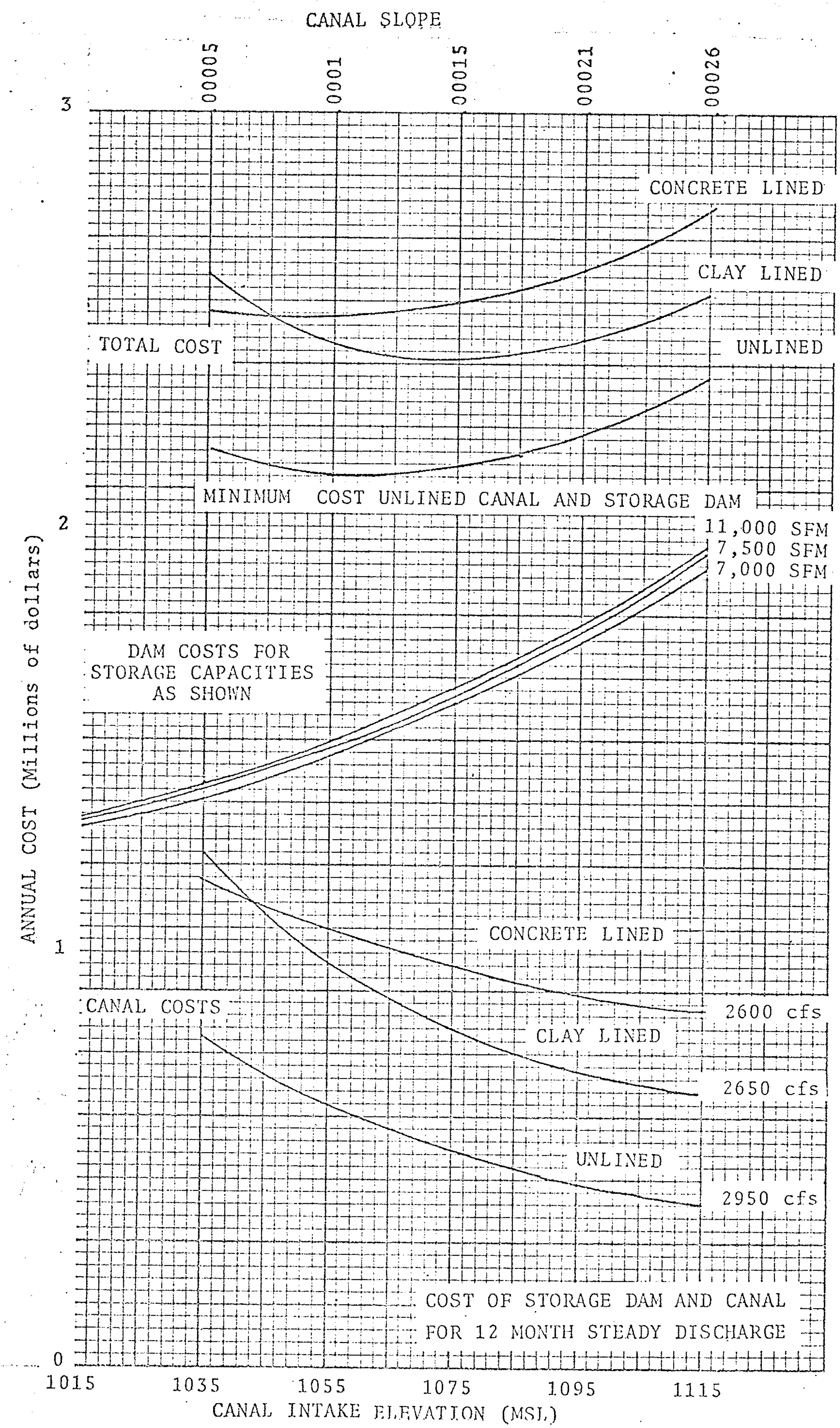

CANAL SLOPE
00005
0001
00015
00021
00026
CONCRETE LINED
CLAY LINED
TOTAL COST
UNLINED
MINIMUM COST UNLINED CANAL AND STORAGE DAM
11,000 SFM
7,500 SFM
7,000 SFM
DAM COSTS FOR STORAGE CAPACITIES AS SHOWN
CONCRETE LINED
CANAL COSTS
2600 cfs
CLAY LINED
2650 cfs
UNLINED
2950 cfs
COST OF STORAGE DAM AND CANAL FOR 12 MONTH STEADY DISCHARGE
ANNUAL COST (Millions of dollars)
0
1
2
3
1015
1035
1055
1075
1095
1115
CANAL INTAKE ELEVATION (MSL)

FIGURE 42

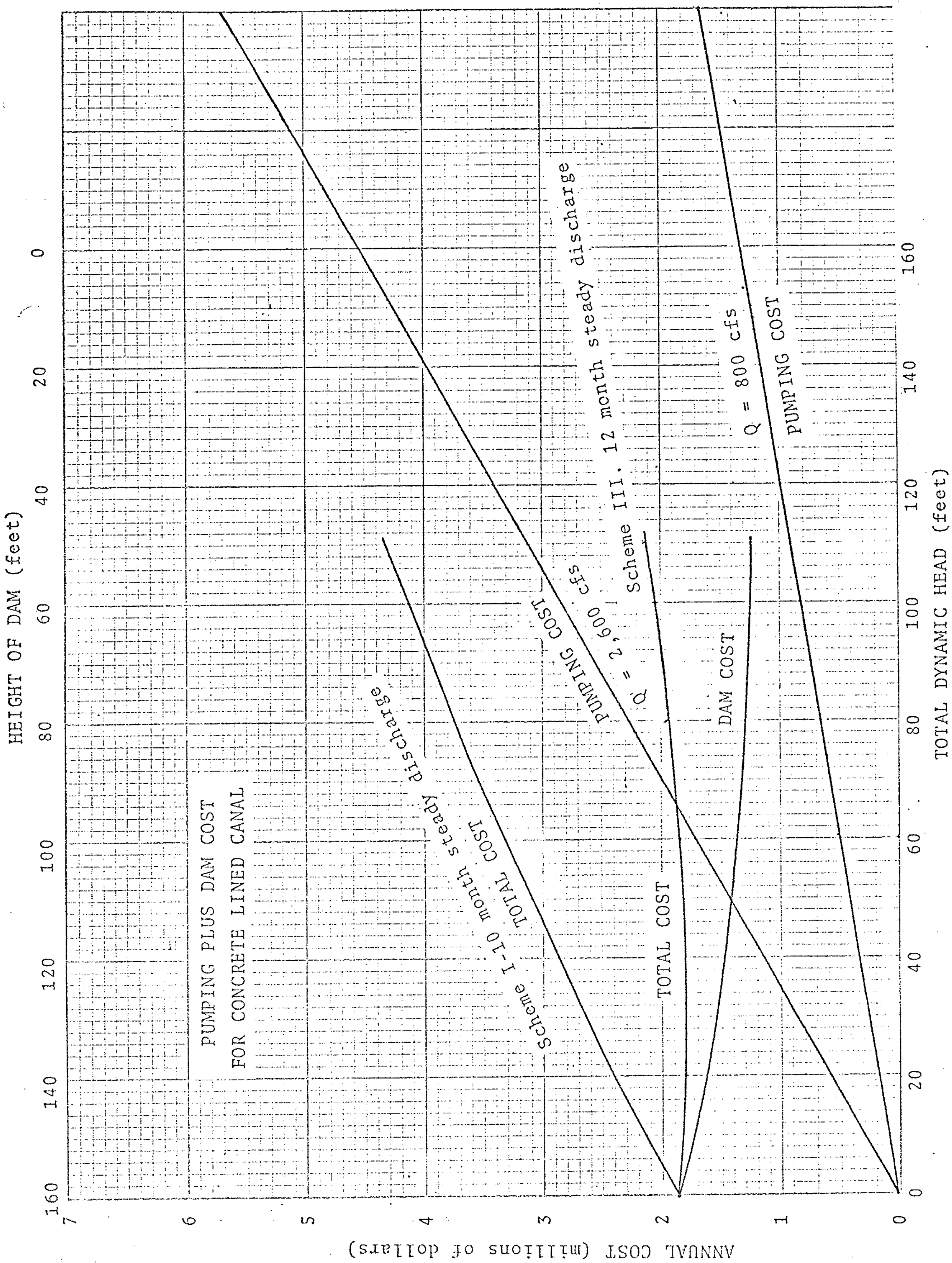

PUMPING PLUS DAM COST
FOR CONCRETE LINED CANAL
HEIGHT OF DAM (feet)
TOTAL DYNAMIC HEAD (feet)
ANNUAL COST (millions of dollars)
Scheme I-10 month steady discharge
TOTAL COST
Scheme III. 12 month steady discharge
Q = 2,600 cfs
PUMPING COST
Q = 800 cfs
PUMPING COST
DAM COST
TOTAL COST

FIGURE 43

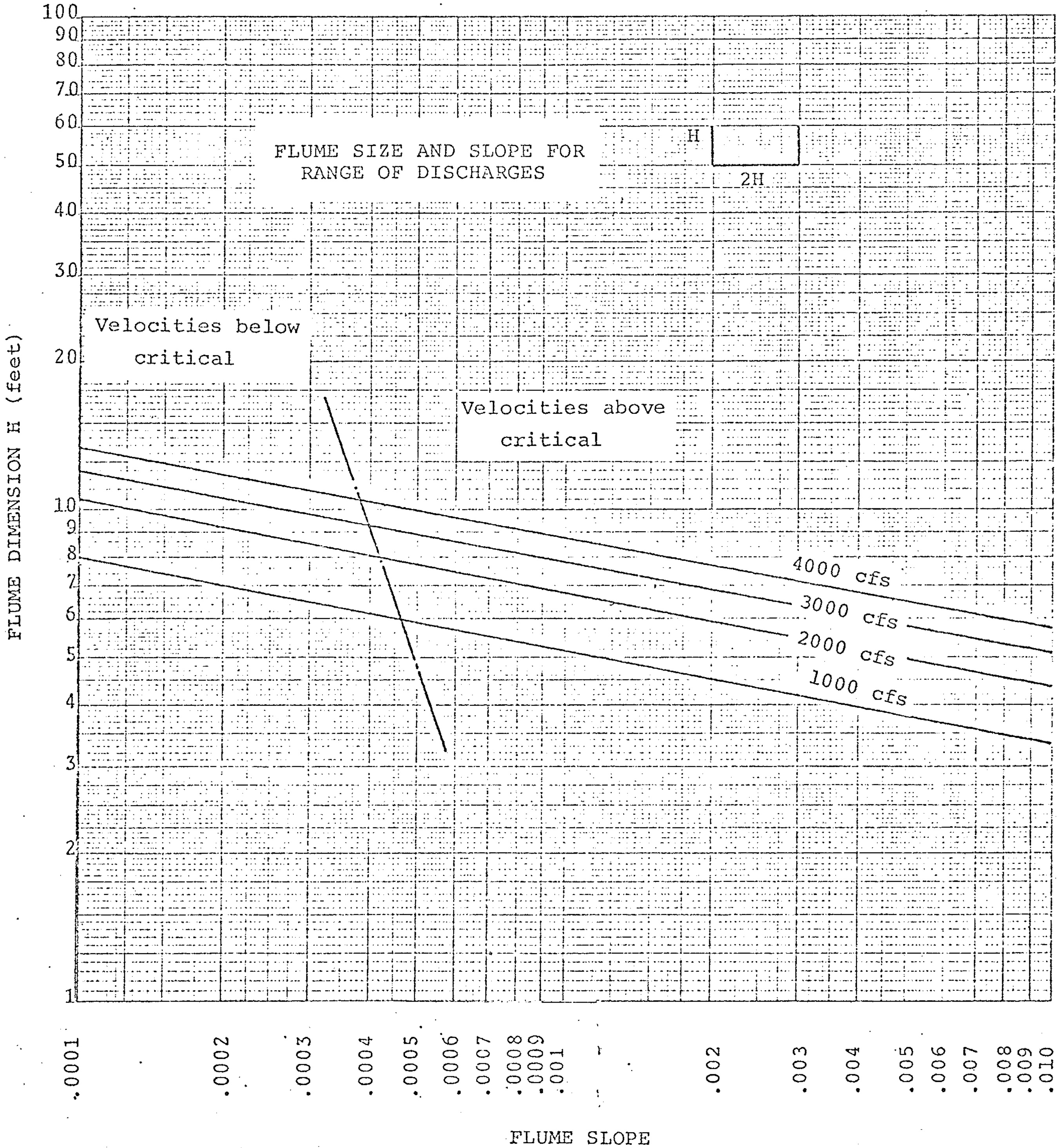

FLUME SIZE AND SLOPE FOR
RANGE OF DISCHARGES
H
2H
Velocities below
critical
Velocities above
critical
4000 cfs
3000 cfs
2000 cfs
1000 cfs
FLUME DIMENSION H (feet)
100
90
80
70
60
50
40
30
20
10
9
8
7
6
5
4
3
2
1
.0001
.0002
.0003
.0004
.0005
.0006
.0007
.0008
.0009
.001
.002
.003
.004
.005
.006
.007
.008
.009
.010
FLUME SLOPE

FIGURE 44