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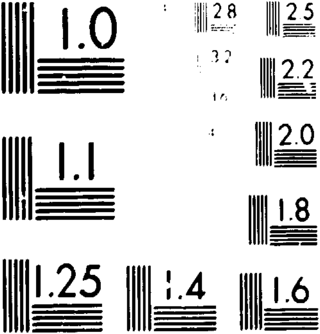

MICROCOPY RESOLUTION TEST CHART

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SYMPOSIUM ON PROJECT DESIGN AND INSTALLATION OF SMALL HYDRO POWER PLANTS

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Organized by Unido in Co-operation with the Austrian Federal Government the Federal Economic Chamber and Universities

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PREFACE

MANY DEVELOPING COUNTRIES HAVE AMPLE, AS YET UNTAPPED HYDROLOGICAL RESOURCES, SUCH AS SCATTERED STREAMS AND RIVULETS, THAT OFTEN COULD BE PROFITABLY EXPLOITED BY MEANS OF MINI-HYDROELECTRIC GENERATION PLANTS.

A NUMBER OF DEVELOPING COUNTRIES IN LATIN AMERICA HAVE ESTABLISHED SUCH PLANTS AND THUS HAVE BEEN ABLE TO IMPROVE THEIR ENERGY SUPPLY MAINLY IN RURAL AREAS. HOWEVER, SOME LATIN AMERICAN COUNTRIES WITH ABUNDANT POTENTIAL HAVE NOT YET BEGUN TO DEVELOP THEIR HYDROLOGICAL RESOURCES, OFTEN DUE TO THE LOCAL ABSENCE OF THE REQUIRED SPECIALIZED TECHNICAL KNOW-HOW. AUSTRIA HAS LONG EXPERIENCE IN THE DESIGN AND CONSTRUCTION OF SMALL HYDROELECTRIC POWER PLANTS. THEREFORE, AN EXCHANGE OF EXPERIENCE IN THIS FIELD BETWEEN LATIN AMERICAN COUNTRIES AND AUSTRIA WOULD BE MUTUALLY BENEFICIAL SINCE MUCH OF THE PRACTICAL KNOWLEDGE ON THIS SUBJECT BASED ON ACTUAL TECHNICAL DATA RELATED TO HYDROLOGY, DESIGN, CIVIL WORKS AND EQUIPMENT OF EXISTING SMALL HYDROPOWER PLANTS IN AUSTRIA COULD BE FRUITFULLY APPLIED ELSEWHERE. THE MAIN PURPOSE OF THIS PUBLICATION IS TO PROVIDE SUGGESTIONS ON HOW THE AUSTRIAN EXPERIENCE IN THE MINI-HYDRO POWER SECTOR COULD BE UTILIZED IN DEVELOPING COUNTRIES, PARTICULARLY IN LATIN AMERICA.

AS PART OF ITS PROGRAMME OF ACTION IN APPROPRIATE INDUSTRIAL TECHNOLOGY, UNIDO HAS BEEN ACTIVELY INVOLVED IN PROMOTING THE DEVELOPMENT AND APPLICATION OF MINI-HYDRO POWER TECHNOLOGY IN DEVELOPING COUNTRIES. SO FAR, UNIDO HAS ORGANIZED TWO MAJOR EVENTS RELATED TO THIS SUBJECT. THE FIRST ONE WAS THE SEMINAR-WORKSHOP ON THE EXCHANGE OF EXPERIENCES AND TECHNOLOGY TRANSFER ON MINI-HYDRO ELECTRIC GENERATION UNITS IN KATHMANDU, NEPAL, 1-14 SEPTEMBER 1979; IT WAS FOLLOWED BY THE SECOND SEMINAR-WORKSHOP/STUDY TOUR IN THE DEVELOPMENT AND APPLICATION OF TECHNOLOGY FOR MINI-HYDRO-POWER GENERATION, 17 OCTOBER - 2 NOVEMBER AT HANGZHOU, PEOPLE'S REPUBLIC OF CHINA, AND 3-7 NOVEMBER 1980 IN MANILA, PHILIPPINES. THE SYMPOSIUM ON PROJECT DESIGN AND INSTALLATION OF SMALL HYDRO POWER PLANTS, ORGANIZED BY UNIDO IN CO-OPERATION WITH THE FEDERAL GOVERNMENT OF AUSTRIA, THE FEDERAL ECONOMIC CHAMBER AND UNIVERSITIES, IS THE THIRD UNIDO-SUPPORTED ACTIVITY IN THIS FIELD.

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1 ELECTRICAL ENERGY FROM HYDROPOWER PLANTS: GLOBAL ANALYSIS

BAUER L.

ABSTRACT:

After having analysed the hydrological potential all over the world, the realistic data applicable to the production of electrical energy, i.e. data of past and future schemes will be presented. Energy consumption up to the present, especially regarding hydroelectric energy, future needs and the possibilities how to meet these needs will be discussed. Related to this, possibilities will be shown how hydropower plants, i.e. small-scale hydropower plants, can contribute to meet increasing demands for energy.

1.1 BASIC PRINCIPLES

When judging the contributions of one form of energy in regard to covering energy requirements, one has to state first of all which quality the energy possesses: whether this energy is consistently available for the needed production or whether it is dependent on variations of outflow. The hydrologically generated energy is by nature a form of the latter, but can up to a certain point be tranformed into the former (by the utilization of storage plants). Moreover, electrical energy generated from hydropower plants is a self-regenerating energy, a highly valuable energy source (contrary to non-self-regenerating energy, i.e. fuels, called resources). Furthermore the possible quantity of usable water-power which can be transformed into energy has to be determined. Finally the parameters must be considered regarding their economic profitability. It has, however, to be said that in the long run, because of the decreasing energy resources, all the different sources of energy will become more and more important in view of the future. Therefore it is necessary to change traditional thinking about economic profitableness. Small-scale hydroelectric plants are to be understood as those with a capacity up to 5000 kW (small-scale hydroelectric plants up to a capacity of 100 kW are called mini-plants; a maximum capacity of 50 000 kW is considered to be the upper limit for small-scale thermal plants).

First of all the most important global figures, data, etc. for energy supply will be given in order to draw the necessary conclusions for the contribution of hydrologically generated energy, especially regarding values relevant to small-scale hydropower plants.

1.2 HYDROLOGICAL ENERGY: ANALYSIS OF POSSIBLE WORLD-WIDE UTILIZATION (WEK 1974)

1.2.1 Facts and Figures

Annual sea-water evaporation is appr. 400 000 km^3, from this amount appr. 300 000 km³ returns as annual rainfall into the ocean and the remaining 100 000 km³ of sea-water evaporation is recycled to the earth as rainfall, ice or snow.

From these 100 000 km³, 63 000 km³ evaporate per year and the remaining 37 000 km³ flow annually back into the ocean with a mean runoff coefficient of 800 m.

1.2.2 Mathematical Determination of the Potential

$37\ 000\ km^3 \cdot 800\ m = 290 \cdot 10^6\ TJ/a = 80 \cdot 10^6\ GWh/a$

With consideration of a mean mechanical efficiency (appr. 85 %) and taking the topography into account, only 16 % in the average annual production (RAV) and 6 % during a dry year can be used: this results into a usable Primary Energy in the average annual production (RAV) of

$45 \cdot 10^6\ TJ/a = 12{,}5 \cdot 10^6\ GWh/a$

and a usable Primary Energy during a dry year of

$17 \cdot 10^6\ TJ/a = 4{,}6 \cdot 10^6\ GWh/a.$

Also the present status of technology and the presently existing economic conditions must be considered, which leads to certain restrictions and subsequently to changes in the calculated values as follows:

usable Primary Energy (RAV) in the average annual production

$37 \cdot 10^6\ TJ/a = 10{,}3^6\ GWh/a$ with $2{,}34 \cdot 10^6\ MW$,

usable Primary Energy during a dry year of

$15{,}6 \cdot 10^6\ TJ/a = 4{,}34 \cdot 10^6\ GWh/a$ with $0{,}545 \cdot 10^6\ MW$

1.2.3 Restrictions of Development owing to Topography

Twelve different types of climates exist throughout the world (different altitudes and subsequently deviating precipitations) moreover, the lengths of rivers from the sources to the mouths vary, for example, there are rivers which measure up to 6000 km.

1.2.4 Hydrological Utilization Possibilities according to Continents and Utilization Status in 1974 (WEK 1980)

area	possible potential average						present production				
	10^3 MW	10^6 GWh/a	10^6 TJ/a	10^3 MW	10^6 GWh/a	10^6 TJ/a	10^3 MW	10^6 GWh/a	10^6 TJ/a	RAV	95 %
Africa	437.1	2.020	1.272	145.2	1.162	4.182	8.154	0.030	0.109	1.5	2.6
Asia	684.3	2.638	9.497	139.3	1.114	4.011	47.118	0.198	0.714	7.5	17.7
Europe	215.4	0.722	2.601	53.0	0.428	1.540	103.998	0.382	1.376	52.9	89.4
USSR	269.0	1.095	3.942	50.0	0.400	1.440	31.500	0.123	0.443	11.2	30.8
North America	330.5	1.488	5.356	72.1	0.577	2.078	90.210	0.453	1.632	30.5	78.6
South America	288.3	1.637	5.893	81.2	0.650	2.339	18.773	0.091	0.329	5.6	14.1
South Pacific	36.5	0.202	0.727	13.0	0.104	0.374	7.609	0.029	0.104	14.3	27.8
total	2261.1	9.802	35.288	553.8	4.435	15.964	307.362	1.306	4.707	13.3	29.5
of that											
industrialized countries	870.7	3.449	12.415	183.1	1.469	5.288	249.320	1.052	3.789	30.5	71.6
developing countries	1390.4	6.354	22.874	370.7	2.966	10.676	58.402	0.255	0.918	4.0	8.6

Tab. 1: Waterpower sources according to Continents and its present utilization

1.2.5 Present Rate of Development

proportion: present utilization at possible RAV (GWh/a):

$$\frac{1.306}{9.802} = 13{,}4\ \%$$

proportion: present utilization of possible capacity (MW):

$$\frac{307.362}{2261.10} = 13.6\ \%$$

proportion: possible utilization of energy produced during a dry year (GWh/a):

$$\frac{4.435}{9.802} = 45.2\ \%$$

proportion: possible utilization of capacity at the beginning of a dry year (MW):

$$\frac{552.8}{2261.1} = 24.5\ \%$$

1.2.6 Hydropower Plants - Developments from 1925 - 1971

The following table 2 shows during a period of nearly 5 decades the global development of hydropower plant construction - that is to say, the increase world wide of potential utilization. (The total values are not exact due to rounding-off).

	1925			1971		
	installed capacity (MW)	GWh/a production	TJ/a production	installed capacity (MW)	GWh/a production	TJ/a production
Western Europe	10 000	30 833	110 000	97 822	366 533	1 319 518
USA	8 800	25 530	91 909	53 404	256 781	924 412
Canada	3 600	9 949	35 816	32 501	178 169	641 408
Japan	3 000	7 610	27 396	19 897	82 270	269 172
USSR		268	965	31 500	123 000	442 800
Eastern Europe		268	965	6 176	15 784	56 821
Eastern Asia		-	-	14 521	54 954	197 834
South Asia	1 000	368	1 325	12 469	60 902	219 247
Africa		65	234	8 154	30 168	108 594
Australia		343	1 235	4 310	12 353	44 471
New Zealand		306	1 102	3 200	16 000	17 600
South Pacific		-	-	99	544	1 959
Latin America		3 155	11 358	23 078	109 799	395 277
world total	26 400	78 678	283 241	307 131	1 307 257	4 706 113

Tab 2: Waterpower - Development

Comparison: development from 1925 up to 1971

GWh/a: 4 706 113 : 283 241 = 16.6 times greater 1971 than 1925
MW: 307 131 : 35 400 = 11.6 times greater 1971 than 1925

1.2.7 Planned Construction of Hydropower Plants

The following table 3 shows predictions of ECE (1980) about the estimated development in the production of electrical energy in hydropower plants distributed throughout groups of countries up to the year 2020 according to the plans of these different countries.

groups of countries	1976	1985	2000	2020
OECD	3.78	4.49	5.37	7.80
centralized economy countries	0.72	1.20	2.88	8.70
developing countries	1.17	1.97	4.49	11.80
world total	5.67	7.66	12.74	28.30

Tab. 3: Estimated production of electrical energy in hydropower plants in EJ

1.3 ENERGY PRODUCTION AND ENERGY CONSUMPTION THROUGHOUT THE WORLD

The charts presented below will help towards a neutral judgement of the importance of hydrological energy contribution in view of covering the present and future world energy needs (from BVL).

1.3.1 Energy Consumption throughout the World according to Energy Carriers

	coal		oil		natural gas		water power and atomic energy		total	
	10^{18} J	%	10^{18} J	%	10^{18} J	%	10^{18} J	%	10^{18} J	%
1975	73.7	32.2	103.2	45.1	45.3	19.8	6.6	2.9	228.8	100
1976	77.5	31.9	110.5	45.5	47.8	19.7	6.9	2.9	242.7	100
1977	80.4	32.1	114.2	45.6	48.4	19.3	7.3	3.0	250.3	100
1978	82.1	32.0	116.0	45.2	50.9	19.8	7.5	3.0	256.6	100

Tab 4: Classification of world energy consumption according to energy carriers

1.3.2 Energy Consumption throughout the World in 10^{18} J

	1975	1976	1977	1978
Western Europe	42.6	45.6	45.2	45.9
countries with centralized economy	72.1	75.5	79.8	84.0
North America	74.6	79.3	80.4	80.2
Central America	5.2	5.5	5.9	6.2
South America	4.4	4.6	4.9	5.1
Middle-East	3.5	3.9	4.3	4.8
Far East	19.1	20.4	21.5	22.0
Africa	4.4	4.8	5.1	5.2
South Pacific	2.9	3.1	3.2	3.2
world total	228.8	242.7	250.3	256.6

Tab. 5: Classification of the world energy consumption according to countries

1.3.3 Energy Production throughout the World according to Energy Carriers

	coal		oil		natural gas		water power and atomic energy		total	
	10^{18} J	%	10^{18} J	%	10^{18} J	%	10^{18} J	%	10^{18} J	%
1975	75.5	30.6	118.7	48.1	46.0	18.6	6.6	2.7	246.8	100
1976	77.6	29.8	128.4	49.2	47.9	18.4	6.8	2.6	260.7	100
1977	81.0	29.9	133.9	49.3	49.0	18.1	7.3	2.7	271.2	100
1978	81.6	29.8	133.6	48.9	50.8	18.6	7.5	2.7	273.5	100

Tab. 6: Classification of world energy production according to energy carriers

1.3.4 Energy Production throughout the World according to Countries in 10^{18} J

	1975	1976	1977	1978
Western Europe	18.5	19.2	20.4	21.3
countries with centralized economy	79.0	83.2	88.4	93.6
North America	67.8	67.6	68.3	66.9
Central America	9.5	9.6	10.1	10.1
South America	2.8	2.9	3.1	3.3
Middle East	44 0	49.4	50.1	47.8
Far East	8.5	9.5	10.5	10.5
Africa	13.3	15.5	16.5	16.1
South Pacific	3.4	3.8	3.8	3.9
world total	246.8	260.7	271.2	273.5

Tab. 7: Classification of world energy production according to countries

As almost all hydro energy is transformed into electrical energy the figures of the total production of electrical energy are stated below.

1.3.5 Production of Electrical Energy throughout the World

figures in TWh	1974	1975	1976	1977
Europe (without USSR)	1 768	1 785	1 913	1 973
USSR	976	1 039	1 111	1 150
North America	2 324	2 355	2 502	2 619
South America	155	168	181	198
Asia	824	877	957	1 014
Africa	122	131	142	148
South Pacific	92	98	102	108
world total	6 261	6 453	6908	7 210

Tab. 8: Annual production of electrical energy

1.4 ESTIMATION OF ENERGY NEEDS THROUGHOUT THE WORLD

Predictions for the needs in the year 2020 and assumptions of how to cover these needs according to WEK (1978).

coal	259
oil	106
natural gas	125
nuclear energy	314
water	56
non-conventional oil and gas	40
solar and geothermal energy, biomass	100
	1 000 . 10^{18} J

Tab. 9: How to meet world needs in the year 2020 in 10^{18} J

IIASA (1981) has analysed the needs of energy with similar data which correspond within acceptable tolerances - if the different initial criterias are compared and agreed with each other.

1.5 ENERGY RESOURCES THROUGHOUT THE WORLD

At the 11th plenary meeting of the World Energy Conference the Survey of Energy Resources 1980 was published: the results in WEK (1980) which is summarized in the following table shows - in comparison with the consumption development - the foreseeable difficulties regarding the liquid and gas fuels.

energy carriers	confirmed exploitable resources	estimated additional resources
coal	20 312.8	295 999.1
hydrogen	11 184.7	50 795.9
uranium *) **)	1 925.0	2 600.6
total	33 422.5	349 395.6

*) only the nuclear fission share of U^{235} (the other resources of uranium with usage in first nuclear reactor generation correspond to the resources of oil).

**) the thorium resources are being estimated at 4 (up to 10) times as much as the uranium resources.

Tab. 10: Energy resources throughout the world in EJ on basis of the results according to the questionnaires of the 11th Plenary Meeting of WEK - Survey of Energy Resources (SER) 1980

In the General Report 1 of this Plenary Meeting of WEK (1980/2) another summary of the energy resources was presented which - taking into account the deviating definitions above all with coal - again corresponded to the values of SER.

Energy Rawmaterial	Resources in 10^9 t - SKE		
	confirmed	geologically probable	usable now
oil	127	360	127
natural gas	79	276	79
oil in slates	50	720	30
oil in oily sands	50	360	30
pit coal	2 000	7 728	493
brown coal	1 000	2 399	144
total	3 306	11 843	903

Uranium at present: confirmed economically usable resources $2\ 200 \cdot 10^3$ t

assumed economically usable resources $4\ 000 \cdot 10^3$ t

This corresponds, when in the present nulear reactor generation, to appr. $120 \cdot 10^9$ t-SKE, which is also the value of the now confirmed oil resources. When in fast breeders, usage is 40 to 60 times higher.

It should also be mentioned that in slates with 25 - 80 g U_3O_8 /t there are, according to present knowledge, appr. $13\ 000 \cdot 10^3$ t U. In addition to that there are bigger resources in smaller concentrations and in sea-water.

Tab. 11: World resources of energy raw-materials according to the General Report of the 11th Plenary Meeting of WEK in 10^9 t-SKE (1 t-SKE = $29,3 \cdot 10^9$ J)

1.6 ENERGY SOURCES THROUGHOUT THE WORLD

IIASA has analysed in the previously mentioned study which was worked out during several years and which takes into account the growing development of technology, workers, raw-material and expenditures, the realistically usable potential of energy sources as shown in the following presentation. See also **IIASA** (1981) and Bauer et al. (1980).

	world-wide technical potential TWa/a	usable potential Secondary Energy TWa/a
biomass	6	5.1
solar energy for low temperature	0.9	0.9
waterpower	3	1.5
wind	3	1.0
waves	0.005	---
Ocean Thermal Electric Conversion	1	0.5
geothermal energy	2	0.6
tides	0.04	0.04
total	16 TWa/a	9.6 TWa/a

Tab. 12: Usable potential of energy sources throughout the world

1.7 CONCLUSIONS REGARDING GLOBAL SURVEY

The percentage of world-wide contribution of hydroelectrical energy is not higher than 2 % and is not expected to increase in the years to come. Hydroelectrical energy is, however, a source of self-regenerating energy and therefore the development of hydropower plants should be promoted. It goes without saying that together with hydropower plants the ecological factors must always be takenintoaccount. The apparently small percentage of global contribution is, however, essentially higher for single countries, as shown later, and is of great importance to their national economy. The reason for the discrepancy lies in the uneveness of energy resource distribution and likewise in the energy source distribution. Today hydropower plants are nearly always set up as multi-purpose plants and as such they fulfill additional functions for society. Moreover, one must not forget that the importance of small-scale hydropower plants had spread above all during the first decades of their usage for generated energy. Later on they decreased because of profitability considerations and trends towards centralization. But at present, with the formation of economic systems in developing countries and the danger of energy shortages in the developed countries, the importance of small-scale hydroelectric plants must be increasingly emphasized. It is very difficult to estimate in a global survey the potential of small-scale hydroelectric plants, i.e. its contribution to future oriented world-wide energy production. The percentage could be roughly estimated between 5 % - 10 % within the total electrical energy. (Percentage of the electrical energy in the total energy consumption of the world is at present appr. 9 %).

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2 HYDRO-ELECTRICAL ENERGY: FACTS AND FIGURES FOR AUSTRIA

BAUER L.

ABSTRACT:

For many years Austria has been one of the leading countries in the field of utilization its hydrological potentials for electrical energy. The percentage of hydroelectric power to cover the needs for electrical energy is very high and amounts to approximately two thirds of the total power consumption. The following report will present a number of details about the hydropower stations operating in Austria, that is, more than 1250 small scale hydroelectric plants, and will provide information about the intentions and the measures to be taken for intensifying the construction of further plants.

2.1 BASIC PRINCIPLES

In the field of power development Austria can look back at approximately one hundred years of experience. With the enterprises for power supply the Austrian industry concerned has received that basis upon which it could develop the utilization of hydrological power for electrical energy supply and could accomplish outstanding achievements in Austria itself and abroad. Together with the progress in the development of hydropower plants came simultaneously the better availability of data and values necessary for techno-economical usage (annual flow, floods and low flows, ice formation etc.) so that at present we posses long term statistical information about all the important catchments.

Small-scale hydropower plants, as mentioned before, are those with a capacity of 5000 KW.

2.2 POTENTIALS OF HYDROLOGICAL ENERGY

2.2.1 Basic Principles

Austria is a small inland country in Central Europe with an area of 83 813 km². In 1978 its population amounted to 7 508 400 (90 inhabitants per km²). The highest altitude in Austria is 3797 m and the lowest point is 140 m above sea-level (that is the Danube at the Hungarian border). An especially favourable characteristic of Austria is the fact that the Danube with a length of 2850 km (with a catchment of 817 000 km² = ten times as large as Austria) only on 1/10 of its length in Austria shows approximately half of the total head.

2.2.2 Investigation of the Potential of Hydrological Energy from BMH(198o/a)

The theoretical potential on the basis of precipitation (on the average 1190 mm

per year) is calculated with	250 000 GWh/a
after substraction of the quantities for evaporation	100 000 GWh/a
remain (mere surface potential)	150 000 GWh/a
The present status of technology permits according to SCHILLER (1980) the development of river systems of at least 1 m^3/s MQ with total of	74 000 GWh/a
The potential developed at present and stated in projects amounts to	49 200 GWh/a

2.2.3 Distribution of the Techno-economical Development Potential among the Federal States (VERBUNDGESELLSCHAFT, 1981)

Austria is politically divided into 9 federal states, the western part of the country is distinguished by high mountains and the eastern part by pre-alpine areas and plains. The distribution of the potential can be seen in table 1.

federal state	total	reservoir
Lower Austria	11 200	500
Upper Austria	9 900	500
Styria	3 500	900
Carinthia	7 500	2 300
Salzburg	4 500	2 700
Tirol	9 500	6 000
Vorarlberg	3 100	2 300
Total sum	49 200	16 200

Tab. 1 Possible development of the hydrological potential in Austria distributed according to federal states in GWh

2.2.4 Development of Hydroelectric Power Plants in Austria (BMH, 1980/b)

In 1925 the output of hydroelectric power plants (WKW) amounted to	1 330 GWh
In 1971 the output of WKW reached (= 12,6 times as much as 1925)	16 770 GWh
and in 1979 the output of WKW amounted to (= 21 times as much as 1925)	28 047 GWh

2.2.5 Present Development of Hydroelectric Power Plants in Austria

The standard annual energy potential (RAV) of the hydroelectric power-plants for general public supply in Austria in 1979 amounted to approximately 26 371 GWh/a, that means that about 41 % of the potential determined in different schemes are being already used.

2.2.6 Planned Development of Hydroelectric Power plants in Austria

It is planned with the development of new power plants up until 1995 to fully use the potential calculated in presently existing studies; the construction of small-scale hydropower plants will be specially promoted; the herewith related industry has made steps towards the possibility of standardization and typification and has achieved favourable results.

2.3 ENERGY PRODUCTION AND - CONSUMPTION IN AUSTRIA

2.3.1 Production and Consumption of Energy in Austria

The following data-sheet shows the energy which is consumed in Austria.

Year	Compact fuel		Oil products		Natural gas		Water power		Sum domestic consumption	Covered in form of electricity	
	10^{12} J	%	10^{12} J	%	10^{12} J	%	10^{12} J	%	10^{12} J	10^{12} J	%
1947	134165	82,9	15969	9,9	2168	1,3	9581	5,9	161883	12628	7,8
1955	237623	69,0	65310	19,0	16555	4,8	24670	7,2	333158	34926	10,1
1960	226899	53,3	113391	26,6	49458	11,6	35922	8,5	425670	50630	11,9
1970	197218	28,2	349666	50,0	94961	13,6	56989	8,2	698834	88633	12,7
1975	162469	20,9	404018	52,1	140258	18,1	69148	8,9	775893	110402	14,2
1979	168036	18,9	467423	52,7	165516	18,6	87138	9,8	888113	132516	14,9

Tab. 2 Primary energy and energy consumption in Austria

Table 3 states only the primary energy available in Austria, that means, that at present 2/3 of the required energy are covered by energy imports - they strongly debit the balance of payment and the current accounts - and yet in 1955 Austria was nearly self-supporting in energy. The rate of contribution of hydrological power is appr. 10 % of the total energy consumption, that is far above the world average (2 %).

Year	Coal		Oil		Natural gas		Water power according to the method of equivalence		Domestic availability
	10^{12} J	%	10^{12} J	%	10^{12} J	%	10^{12} J	%	10^{12} J
1947	46792	43,3	40024	37,0	6739	6,2	14591	13,5	108146
1955	101964	31,2	161121	49,3	27835	8,6	35746	10,9	326666
1960	91387	29,9	107618	35,2	53385	17.4	53707	17,5	306097
1970	53766	15,7	122972	36,0	68943	20,2	96016	28,1	341697
1975	49780	15,0	89512	26,9	85703	25,8	107355	32,3	332350
1979	40141	12,3	75975	23,2	84003	25,7	126810	38,8	325929

Tab. 3 Primary energy in Austria

2.3.2 Production of Electrical Energy

Year	Production				Import	Production and import	Export	Consumption including losses	
	Public supply	Austrian Rail Roads	Industrial self-supply	Total				with pumped storage	without pumped storage
	1	2	3	4=1 2 3	5	6= 4 5	7	8=6-7	8a
1918				1 765	–	1 765	6	1 760	1 760
1933	1 434	110	640	2 272	1	2 273	302	1 971	1 954
1945	2 400	130	650	3 180	190	3 370	578	2 792	2 785
1947	3 234	235	600	4 069	80	4 149	644	3 505	3 491
1950	4 911	333	1 107	6 351	29	6 380	720	5 660	5 641
1955	8 417	490	1 844	10 751	446	11 197	1 498	9 699	9 594
1960	13 184	489	2 292	15 965	641	16 606	2 544	14 062	13 315
1970	25 818	855	3 363	30 036	1 371	31 407	6 785	24 622	23 906
1979	35 232	949	4 464	40 645	2 854	43 499	6 689	36 810	36 170

Tab. 4 The total supply of electricity in Austria in GWh

2.4 ESTIMATION OF THE REQUIRED ENERGY IN AUSTRIA (WIFO, 1980)

	1985		1990	
	10^{15} J	%	10^{15} J	%
Net domestic consumption	983	100	1 147	100
there:				
compact fuel	85	8,5	81	7,5
oil products	498	50,7	587	51,2
natural gas	206	20,9	240	20,9
electrical energy	178	18,1	220	19,2
long-distance heat	16	1,6	19	1,7
average increase in percentage		3,95		3,10

Tab. 5

2.5 RESOURCES IN AUSTRIA

The following table shows clearly that the resources of energy carriers will not be sufficient for more than a few decades at today's rate of exploitation.

	assured and probable reserves	possible and hypothetical reserves	production 1977	period of utilization with consistent production
brown coal million t	116 x)	220	3,2	38-57 years

	assured and probable reserves	possible and hypothetical reserves	production 1977	period of utilization with consistent production
oil	22	33 - 39	1,8	13 - 19 years
natural gas	14	61 - 64	1,985	10 - 32 years
uranium	1800 t-U-metall			

x) 50 % possible according to present conditions

Tab. 6 Energy resources (BMH, 198o/a)

2.6 ENERGY RESOURCES IN AUSTRIA (BAUER(ed.), 198o)

Austria is not rich in resources and is not favoured with important self-regenerating energies - with the exception of hydrological energy. The following illustration describes the situation.

2.6.1 Households with Energy

2.6.1.1 Effective energy utilisation in the field of "Construction and Living" could amount to about 27 TWh/a in Austria (present consumption of electrical energy per year is about 40 TWh).

2.6.1.2 Long-distance Heating Systems

additional 2 900 MW with 40,2 GJ/h possible
investments 30 - 40 milliards AS
with that 19 % of the needs for low-temperature heat can be covered. One half of all oil-fired central heating systems are equipped with heat pumps: 54 % of this required heat for apartments = 5 % of need of primary energy.

2.6.2 Wind

Only a fraction of a percentage of the total energy consumption could be covered by wind (seen realistically as to the present situation).

2.6.3 Fire - Wood

About 40 % of the area of Austria are covered with woods. About 20 million m^3 are growing per year, 14 million of which are being used for industrial or other purposes. The remaining 6 million m^3 = 3 million t-SKE could be used for the production of energy = 1/10 of the total energy consumption.

2.6.4 Straw

Small-scale	power stations	500 - 1200 MW, 4000 h/a
Savings of		1 million t crude oil/a

2.6.5 Sun

All roof panes:	10 % of the final energy consumption in the lower temperatures
	2500 km² surface area of Austria
Collectors	theoretically 2x final energy consumption actually
Solarthermal plants	15 % of the final energy consumption, if with 30 TWh storage capacity (44x Malta)
Photovoltaic and electrolysis	hydrogen - about 40 % of the final energy consumption

2.7 SMALL-SCALE HYDROPOWER PLANTS IN AUSTRIA (PARTL, 1979)

2.7.1 Development up to the Present

Table 7 shows the status 1974 (last official statistical investigation) of small-scale power plants in Austria as distributed through the 8 federal states (the ninth is Vienna and has no small-scale power-plants); in 1974 there were a total of 1254 hydropower plants (capacity up to 5000 kW) in operation, with a total capacity of 355 415 kW (an average of 283 kW) and a total standard annual energy potential of 1928,8 GWh (an average of 1,54 GWh and an average annual number of hours of utilization of 5442).

Contributing to the total production of electrical energy were those small-scale hydropower plants which are listed in table 8.

Contributions in Percentage (%)			
capacity kW	standard annual energy potential	production	electrical energy domestic consumption
up to 1000	4,1	2,6	3,0
up to 2000	5,8	3,7	4,3
up to 5000	8,0	5,1	5,8

Tab. 8 Small-scale hydropower plants, contribution to the electrical production

The growing number of large-scale hydropower plants, the issues of profitability of small-scale plants and also the shortage of workers at that time resulted in the fact, that between 1959 - 1974 appr. 400 small-scale plants had to be abandoned (all of them below 100 kW).

Tab. 9 shows the respective figures for the whole of Austria.

	BIG PLANTS			SMALL PLANTS			DOMESTIC PLANTS			TOTAL		
	fig.	kW	GWh	fig.	kW	GWh	fig.	kW	GWh	fig.	kW	GWh
BURGENLAND												
10 — 100 kW		—		1	75	0,4	4	145	0,8	5	220	1,2
101 — 1000 kW		—		3	640	3,0	2	520	2,6	5	1 160	5,6
1001 — 2000 kW		—			—			—			—	
2001 — 5000 kW		—			—			—			—	
Σ Burgenland		—		4	715	3,4	6	665	3,4	10	1 380	6,8
NIEDERÖSTERREICH												
10 — 100 kW	3	270	1,5	12	585	3,3	133	5 515	27,1	148	6 370	31,9
101 — 1000 kW	20	9 660	52,3	10	3 040	14,5	79	20 300	113,4	109	33 000	180,2
1001 — 2000 kW		—			—			—			—	
2001 — 5000 kW	3	9 600	59,4	1	2 640	15,6		—		4	12 240	75,0
Σ Niederösterreich	26	19 530	113,2	23	6 265	33,4	212	25 815	140,5	261	51 610	287,1
OBERÖsterreich												
10 — 100 kW		—		28	1 450	8,5	150	5 685	31,9	178	7 135	40,4
101 — 1000 kW	7	2 830	15,6	9	2 360	15,6	43	13 255	75,3	59	18 445	106,5
1001 — 2000 kW	2	2 200	9,1	5	5 550	33,3	4	6 250	33,8	11	14 010	76,2
2001 — 5000 kW	2	5 700	22,9		—		1	3 900	19,6	3	9 600	42,5
Σ Oberösterreich	11	10 730	47,6	42	9 370	57,4	198	29 090	160,6	251	49 190	265,6
SALZBURG												
10 — 100 kW	5	440	2,4	8	375	2,4	62	2 375	13,8	75	3 190	16,6
101 — 1000 kW	6	2 880	17,9	6	1 055	7,2	12	2 140	12,9	24	6 075	38,0
1001 — 2000 kW	2	3 000	17,9	3	5 700	32,2		—		5	8 700	50,1
2001 — 5000 kW	2	7 700	35,6		—		3	10 500	53,1	5	18 200	88,7
Σ Salzburg	15	14 020	73,8	17	7 130	41,8	77	15 015	79,8	109	36 165	195,4
TIROL												
10 — 100 kW	10	730	5,1	28	1 300	7,7	65	1 900	16,6	103	3 930	29,4
101 — 1000 kW	22	6 600	39,8	37	11 700	71,1	26	8 285	46,1	85	26 585	157,0
1001 — 2000 kW	5	7 170	40,6	10	15 210	82,0	2	3 885	18,8	17	26265	141,4
2001 — 5000 kW	1	2 280	15,3	5	11 660	55,8	2	7 450	38,9	8	21 390	110,0
Σ Tirol	38	16 780	100,8	80	39 870	216,6	95	21 520	120,4	213	78 170	437,8
VORARLBERG												
10 — 100 kW	3	145	1,0	2	155	1,3	17	830	4,0	22	1 130	6,3
101 — 1000 kW	4	1 380	8,9	2	680	2,5	13	4 380	27,3	19	6 440	38,7
1001 — 2000 kW	2	2 660	14,9	1	1 600	6,0		3 740	27,6	9	8 000	48,5
2001 — 5000 kW		—		1	2 240	17,1	2	5 370	41,3	3	8 610	56,4
Σ Vorarlberg	9	4 185	24,8	6	4 675	26,9	35	15 320	100,2	50	24 180	151,9
STEIERMARK												
10 — 100 kW		—		26	1 750	9,0	76	3 730	20,8	104	5 480	29,8
101 — 1000 kW	4	1 420	7,1	38	11 165	64,2	68	17 970	101,9	110	30 555	173,2
1001 — 2000 kW	2	2 400	10,8	2	2 100	10,4	5	7 600	47,1	9	12 330	58,3
2001 — 5000 kW		—		3	9 550	71,8		—		3	9 550	71,8
Σ Steiermark	6	3 870	17,9	71	24 505	155,4	149	29 500	169,8	226	57 915	313,1
KÄRNTEN												
10 — 100 kW	4	355	1,9	9	565	3,2	44	2 030	10,9	57	2 950	16,0
101 — 1000 kW	15	6 100	26,5	5	1 255	7,8	44	15 355	80,4	64	22 710	116,7
1001 — 2000 kW	2	2 950	18,0				3	4 500	24,3	5	7 450	42,3
2001 — 5000 kW	6	17 650	43,3				2	6 035	22,8	8	23 685	65,1
Σ Kärnten	27	27 055	91,7	14	1 820	11,0	93	27 930	138,4	134	56 805	[illegible]
GANZ ÖSTERREICH												
10 — 100 kW	25	1 940	11,9	116	6 255	35,8	551	22 220	125,9	692	30 415	173,6
101 — 1000 kW	78	30 870	170,1	110	31 895	195,9	287	82 205	459,9	475	144 970	815,9
1001 — 2000 kW	15	20 380	111,3	21	30 200	163,9	17	26 175	151,6	53	76 755	426,8
2001 — 5000 kW	14	42 930	176,8	10	26 090	160,3	10	34 255	175,7	34	103 275	512,5
TOTAL SUM	132	95 120	469,8	257	94 440	545,9	865	164 855	913,1	[illegible]	355 415	1928,8

Tab. 7 Hydropower plants in Austria up to 5000 kW

Capacity in kW	Number	Capacity in kW	RAV in GWh
10 - 100	340	12 680	62,4
101 - 1000	63	13 625	74,2
Total	403	26 305	136,6

Tab. 9 Abandoned plants 1959 - 1974

On the other hand 173 small-scale hydropower plants were enlarged (i.e. transformed or new installations added) and 274 entirely new plants were constructed during the above mentioned period.

Capacity in kW	Total		
	Number	Capacity in kW	RAV in GWh
10 - 100	66	1 770	10,1
101 - 1000	83	10 955	52,5
1001 - 2000	15	9 450	52,9
2001 - 5000	9	11 170	49,5
Total	173	33 745	165,0

Tab. 10 Enlarged small-scale power plants between 1959 and 1974

Capacity in kW	Total		
	Number	Capacity in kW	RAV in GWh
10 - 100	189	7 345	39,8
101 - 1000	70	18 195	97,3
1001 - 2000	12	15 090	88,4
2001 - 5000	3	7 020	34,7
Total	274	48 650	260,2

Tab. 11 Construction of new plants between 1959 and 1974

2.7.2 Conclusion

It is true that the number of small-scale power plants during the 15 years under consideration has decreased from 1360 to 1220, but their capacity has risen to 28 MW = 12 % and to the standard annual energy potential of 176 GWh = 14 %. Austria, as already mentioned, must exploit every opportunity to utilize domestic sources of energy. How much can the development of small-scale hydroelectric plants be intensified? In terms of output, experts estimate, that with the existing total hydroelectric production potential of 49,2 GWh (RAV) an additional capacity of 200 - 400 MW and appr. 2200 GWh of additional power could be gained. The time needed for the development of small-scale hydropower plants - it would have to go hand in hand with the development of large-scale hydropower plants in order to keep the share of contribution consistent - could be estimated at 20 - 25 years.

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3 SPECIAL REMARKS ON HYDRO POTENTIAL FOR MINI POWER DEVELOPMENTS

RADLER S.

ABSTRACT:

Efficient management should be based on an inventory of the available sources and resources. There are various possibilities of determining hydro potential, ranging from estimations covering large regions to the detailed study of catchments limited as desired. Depending on the surface configuration, the altitude and the hydrological features of the area under investigation, relationships can be established between different methods and information can be obtained on the developable potential.

3.1 INTRODUCTION

A systematic survey of the hydro power potential is a fundamental condition of a sound power economy in a country like Austria, which meets more than two thirds of its energy requirements from hydro generation.

In due recognition of the importance of hydro power in Austria, especially after the Second World War, government authorities conducted a detailed survey of the country's hydro potential. This resulted in the publication of the ÖSTERREICHISCHE WASSERKRAFTKATASTER (Austrian hydro register) (1948 - 1964) supplemented by contributions to the above register: ENERGIEPOTENTIAL DES NIEDERSCHLAGES IM ÖSTERREICHISCHEN BUNDESGEBIET (energy potential of precipitation on the federal territory) (1956), and in the periodic publication of surveys of the developable hydro potential by the ÖSTERREICHISCHE ELEKTRIZITÄTSWIRTSCHAFTS-AG. (LANG and PARTL, 1965; PARTL and KNAUER, 1970 and 1975; KNAUER and GÖTZ, 1979). Apart from these general investigations, detailed surveys of the developed and developable potentials have been performed for a great number of river and stream basins as well as for individual federal provinces. In the following chapters, the different methods and procedures used for determining hydro potential will be presented and discussed.

3.2 MEAN-RUNOFF ENERGY POTENTIAL (Line Potential)

The hydro power register indicates a mean-runoff energy potential for each river basin. This is obtained by multiplying the mean runoff (MQ) from a sub-catchment (e.g. between two tributaries) by the respective available head, the specific density and the acceleration g, and integrating it over the period of investigation (one year).

$$A_r = MQ \cdot h \cdot \rho \cdot g \cdot \frac{31,5 \cdot 10^6}{3.600} \text{ in (Wh) or}$$

$$A_r = \frac{I \cdot h}{367} \text{ in (GWh)}$$

where I .... annual volume of water yield in terms of millions of m³.

Efficiences are ignored.

The result is usually plotted as a longitudinal section and corresponds to the energy generated by a continuous chain of run of river schemes (without impounding and flow limitation). This potential normally only relates to the main stream (hence line potential). Tributaries are accounted for only by increasing the total volume of water yield. Fig. 3-1 shows the potential of a main stream including the potential of the tributaries.

As to the utilisation of the available water resources, the line potential represents optimal development. Any increase in energy potential can only result from an increase in head. Therefore, by constructing reservoirs, the developable energy potential can be raised above the theoretical value of the line potential. An example is shown on fig. 4-5. This represents the result from a study of alternative development possibilities for a stream in Greece. Fig. 3-1 is a longitudinal section of this stream. This example demonstrates that partial development of the stream by means of reservoirs (Alternative A) utilises as much as 93 percent of an energy potential of 1140 GWh although developing only about 80 percent of the available head.

3.3 AREA POTENTIAL OF PRECIPITATION

Systematic determination of the energy potential of precipitation has been based on the following basic assumptions:

- division into summer and winter precipitation depths
- allowance for retention effects (snow)
- assumption of different runoff coefficients for summer and winter half-years
- subdivision into natural catchments
- assumption of an average relationship of precipitation to altitude - uniform for the summer, graded according to altitudes for the winter.

The above study differentiates among 3 possibilities of occurrence (fig. 3-2):

- o complete catchment
- o catchment transversely divided by national border
- o partial catchment on a stream divided lengthwise by national border

The energy potential of annual precipitation, or half-year precipitation, can be regarded as the upper limit of the gross hydro potential. This is particularly useful as a reference in relation to line potential and developability for different stream types.

The above study has yielded the following result:

- energy potential of precipitation 252.3 TWh/a
- energy potential of runoff from winter precipitation 42.5 TWh/a

3.4 COMPARATIVE STUDIES

It is primarily the topography of a catchment that decides which proportion of the runoff from precipitation can be translated into line potential or developable potential. On the basis of several studies made in different types of catchments and stream reaches, area potential, line potential and actually developed or developable potential were compared.

3.4.1 High-Altitude Catchment

The Kapruner Ache was used as a typical example of the development possibility of a high-mountain valley. In an area less than 100 km^2, two large reservoirs were constructed. Deducting the proportion of generation from trans-mountain diversions, the following situation was obtained:

	Catchment (km^2)	Range of altitude (m)	Energy (GWh)
Precipitation potential	89	3,564-753 (2,811)	620
Mean-flow potential	89	2,380-757 (1,623)	312
Developed potential	80	2,036-781 (1,255)	291

It is seen from the above table that the area potential, which allows comparison with the line potential only after deduction of evaporation, is not more than twice as large as the line potential, and furthermore, that the developed potential, although only 77 percent of the available head is utilised, accounts for 93 percent of the line potential. It need not be emphasised that the developable potential has been attained in every respect.

3.4.2 Stream in the Alpine Foreland

The comparative figures obtained from the study of the largest river entirely on Austrian territory are completely different. In this case, the area potential can in no way be compared with the line potential, even after deduction of evaporation. This is due to the great proportion of tributary catchments.

	Catchment (km^2)	Range of altitude (m)	Energy (GWh)
Precipitation potential	6080	2.966-240 (2.726)	23.400
Mean-flow potential	6080	1.009-240 (769)	4.851
Developed potential	6057	564-240 (324)	2.785
Developable potential	6057		3.483

Although the river is developed by a continuous chain of power schemes only over its lower course, which is interesting from the energy point of view (fig. 4-2), utilising not more than 42 percent of the available head, 57 percent of the line potential has so far been developed. If the planned projects totalling approx. 700 GWh are realised, utilisation will rise to 72 percent, by which the limit of developability will probably be reached.

3.4.3 Small Catchments

The substantial difference between area potential and line potential is due to the large proportion of small catchments which are not allowed for in the line potential of the main stream. It is, however, the small stream basins that represent ideal locations for mini power schemes. Their energy potential as well as the selection of the suitable power plant locations by the study of potentials will be discussed in the following.

At the Institut für Wasserwirtschaft of the Universität für Bodenkultur, Vienna, a number of different small stream basins were investigated for diploma theses and research studies. The hydrological, geological and topographical data obtained will be used for developing a procedure of estimating the potential. Some of these results will be presented in the following illustration:

- Elongated catchment in the Alpine foreland (Ybbs)

	Catchment (km^2)	Range of altitude m	Energy (GWh)
Precipitation potential	1293	1892-215 (1677)	2218
Line potential	1293	1380-215 (1165)	743
Developed potential	1293	between 590 and 215 (375 m)	130
Developable potential			350

- Concentrated catchment in the Alpine foreland (Schwarza)

	Catchment (km^2)	Range of altitude (m)	Energy (GWh)
Precipitation potential	735	2076-310 (1766)	850
Line potential	735	580-310 (270)	220
Developed potential			40
Developable potential			80

The relatively small proportion of developable potential is due to the fact that a large proportion of the catchment is an official water protection zone.

- Small concentrated catchment in the Alpine foreland (Kleine Ybbs)

	Catchment (km^2)	Range of altitude (m)	Energy (GWh)
Precipitation potential	113	1328-360 (968)	86
Line potential	113	990-360 (630)	35
Developed potential			4,2
Developable potential (especially efficiency improvement)			10

3.5 CONCLUSION

The above mentioned examples have shown that the line potential can almost always be used as the upper limit of the developable potential.

In many cases, especially in catchments on which little information is available, apart from a general idea of the precipitation and evaporation behaviour, the precipitation potential will give the better results in the absence of flow data. However, the precipitation potential will not allow conclusions regarding line potential and developable potential unless under certain circumstances as listed below:

- Where elongated catchments with minor tributaries are concerned,
- or where major tributaries are treated separately.

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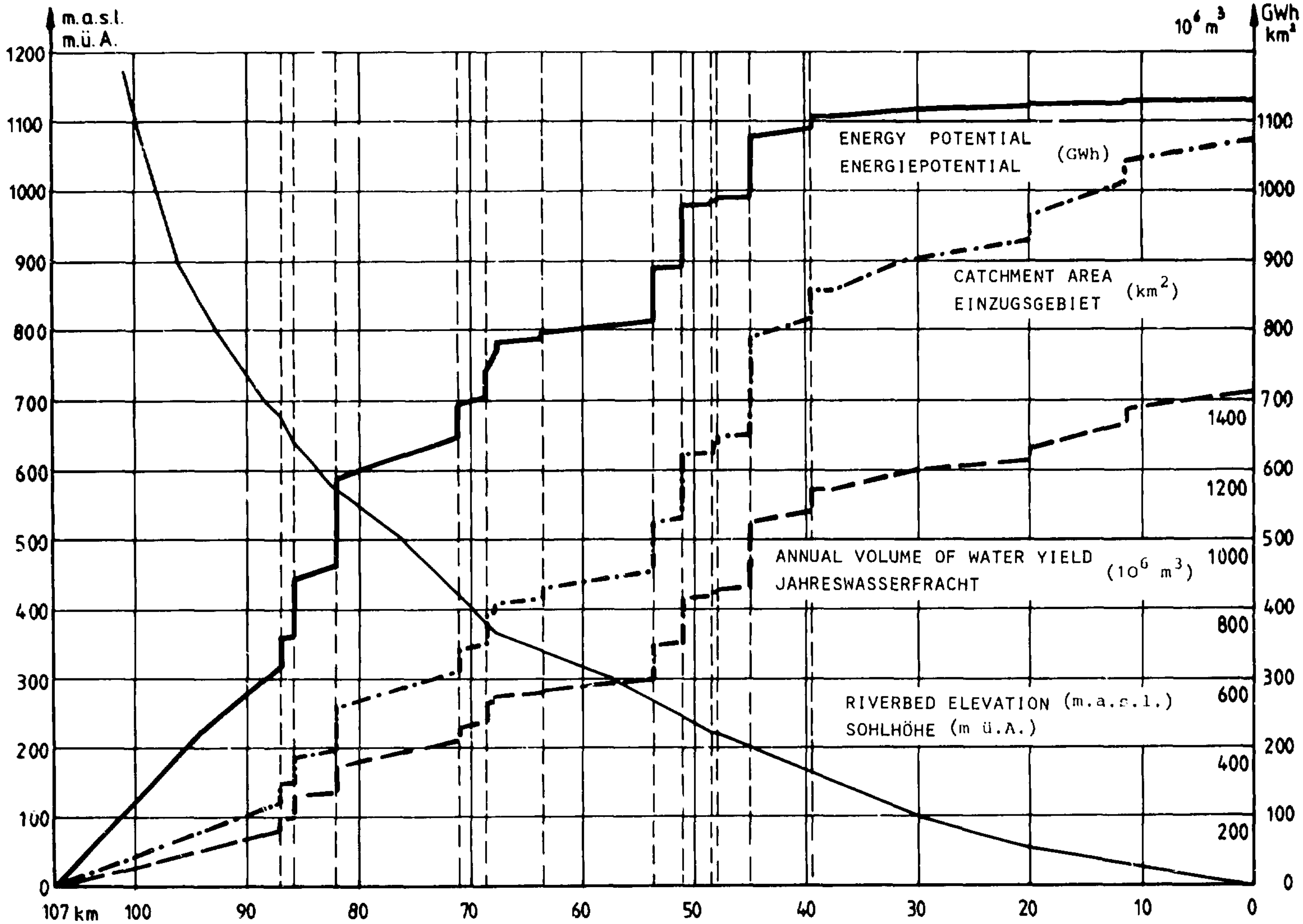
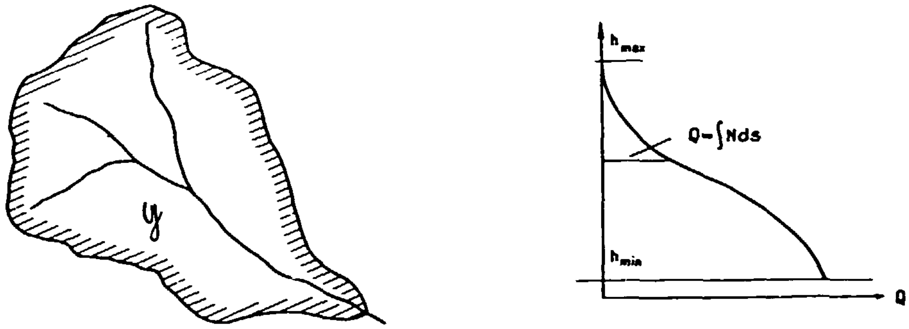
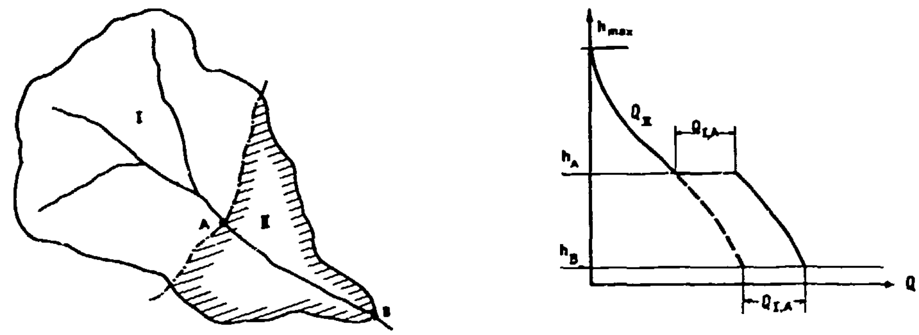


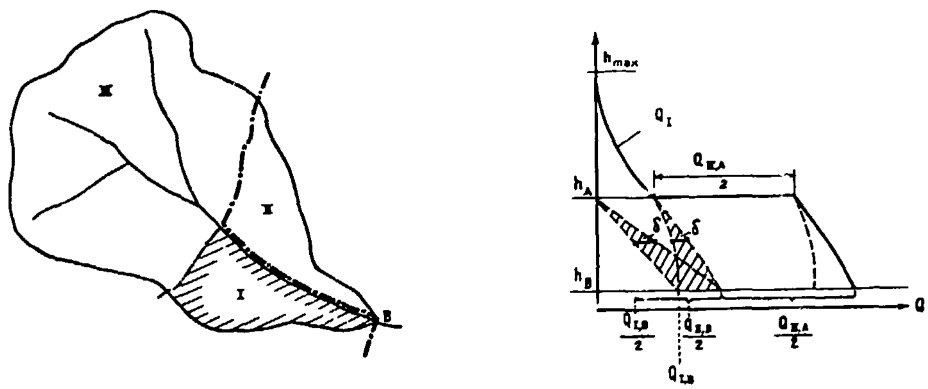
FIG 3 - 1 POWER ECONOMY-LONGITUDINAL SECTION
ENERGIEWIRTSCHAFTLICHER LÄNGSSCHNITT



COMPLETE CATCHMENT AREA
GESCHLOSSENES EINZUGSGEBIET



DIVIDED CROSSWISE BY ORDER
DURCH STAATSGRENZE QUERGETEILTES EINZUGSGEBIET



DIVIDED LENGTHWISE BY BORDER
DURCH STAATSGRENZE LÄNGSGETEILTES EINZUGSGEBIET

FIG 3 - 2 ENERGY POTENTIAL OF PRECIPITATION
ENERGIEPOTENTIAL DES NIEDERSCHLAGES

4 MASTER PLAN FOR HYDRO POWER DEVELOPMENT

RADLER S.

ABSTRACT:

A power development master plan forms the basis for the utilisation of a water course or river basin for power generation. However,where utilisation of the hydro potential of a stream is envisaged, allowances must at the same time be made for all the other requirements and uses connected with water resources such as drinking and industrial water, irrigation, flood control and pollution control, fishery, provision of recreation areas as well as protection of landscape and environment.

Preparation of a master plan requires knowledge about the physical features of the area under consideration. Minimum requirements regarding topographical, geological and hydrological information should be satisfied. Development planning is governed by structural, administrative, power and finally ecological and economic aspects. Recommendations will be made in this Report regarding the representation of the plan on drawings.

In consistent pursuit of their intended purposes and objectives, master plans for hydro development should form the basis for legislative measures.

4.1 INTRODUCTION

In accordance with the UN Manual of Standards and Criteria for Planning Water Resource Projects (UNITED NATIONS,1964), master plans should include a survey of the requirements and resources of a river basin and a comprehensive proposal regarding the orderly provision and development of the water resources for power production. The underlying studies and investigations should be of an informative nature. The report should offer sufficient bases, descriptive information and analyses to allow clear appraisal and knowledge of the comprehensive project design. This report should demonstrate what possible units will be selected for a feasibility study and should state the sequence of investigations to be conducted.

Sections 53 to 55 (regarding water resource master plans and water resource planning) of the Austrian water right law provide rules for general planning. These relate to the water resource management desirable in the interest of the development of life and economy in a certain region, with due allowances being made for the different uses and requirements involved, provided that these are in the public interest.

Master plans for hydro development may constitute a certain restriction to the above mentioned general and special rules in that mini hydro schemes usually involve minor catchments so that their implications in respect of regional planning are of secondary importance. Even in this case, the planned water resource measures should on principle be designed so as to allow for all the superior uses (provision of drinking and industrial water) as well as other aspects (flood control, fisheries, tourist trade, provision of recreation space).

4.2 BASES

As in any type of planning, master plans should be based on general and special information on the physical features of a site or area. Topographical, geological, morphological and meteorological data should provide the basis for investigations in respect of regional planning, structural design and hydrology required for the preparation of a master plan.

4.2.1 Topography

The basis for planning studies is a map to scale 1:50,000 if possible. Landforms are represented by contours (with at least 2 to 10-metre contour intervals). Altitudes are indicated for prominent landmarks, buildings and bridges. Besides geographical situations, this map will show engineering structures, transport routes, vegetation forms and boundaries, bodies of surface water and other important artificial and natural features.

The surveying scale, which is normally 2 to 2.5 times the scale of the map as published and will in most cases be separated according to groups of subjects (vegetation boundaries, water courses, transportation routes etc.), is particularly suited for preliminary hydrological investigations (catchment, distribution of altitudes etc.) and preliminary project studies. The general project studies for engineering structures will preferably be based on location plans drawn to scales not smaller than 1:1,000.

4.2.2 Geology

On the basis of geological key maps covering major regions (in Austria e.g. available to scales 1:75,000 and 1:50,000 for selected regions), aerial photographs with stereoscopic coverage and satisfactory topographical documentation, an experienced engineering geologists, upon thorough site reconnaissance, will be capable of making a general statement on the geological suitability of an area for water resource projects. The systematic geological investigations necessary for preliminary and detailed project studies are the subject of Paper 5.

The engineering geologist's contribution to a master plan should include, apart from general geological forecasts, information on the expected reaction to precipitation of the ground surface. This information is an essential aid for the hydrologist in his investigation of run-off coefficients and concentration times. During this stage of the project studies, close contacts between the project engineer, who should possess knowledge in the fields of structural design, hydrology as well as construction, and the engineering geologist are imperative.

4.2.3 Hydrology

Project studies for hydro development master plans in developing countries must usually rely on an minimal amount of hydrological data. Consequently, in setting up a network of hydrological stations the hydrologist is faced with the task of providing hydrological records as fast as possible. On the other hand, however, he should attempt at the same time to design this network such as to allow integration into a potential supra-regional system to be constructed subsequently.

Determination of the energy potential of a catchment will be based not only on the morphology of the area under study but primarily on information about the flow pattern. Therefore, the planning engineer will in the first place tackle the problem of determining systematically the run-off behaviour of the river basin concerned. Following well-organised site reconnaissance carried out at different seasons characteristic of run-off, he will have gauging stations installed at suitable locations along the river bed, in particular above and below junctions with tributary streams as well as in the tributaries themselves. Stages will be recorded either manually (in the case of staff gauges or dip rods) or automatically - in situ or by telemetering (limnigraphs). In addition, flow measurements should be made by means of screw current meters and these used for establishing stage-discharge relations in the form of rating curves. Where the gauging profiles are stable and the morphological behaviour of the stream bed involves neither sedimentation nor erosion, even major floods or long low-water periods will not involve any appreciable variations of the gauging profile. Then checking of the rating curve will be required only at major time intervals. For gauging profiles situated in overburden material, however, rating curves will have to be corrected after each flood on the basis of new flow measurements in order to allow for sediment transport in the stream. This is particularly important where flows are small.

Suspension load transport has to be determined in conjunction with stage and flow measurements. The equipment used for measuring bed and suspension loads will be selected in each case to suit the specific features of the river basin. Normally, the structural equipment provided for flow measurement is also used for bed and suspension loads (as e.g.blondin installations).

In order to furnish well-founded hydrological information, the network of hydrological stations should extend beyond the stream bed proper to cover the whole catchment area. Any water resource study will be particularly concerned with each of the components making up the water balance equation

$$R = P - E - R + S$$

where:

R ... runoff
P ... precipitation
E ... evaporation
R ... retention
S ... supplemental flow.

Besides runoff,precipitation is the second easily determined component of the water regime. Precipitation is measured at representative locations of the catchment. In mountainous or hilly country, allowance will have to be made for the altitude component. Provision of totalisers protected from evaporation and frost by adequate means will meet the requirements of basic investigations. Inspections will be made at long intervals, probably in summer and winter, or during rainy and dry seasons. Greater accuracy is obtained from rain gauges (ombrometers), which,however,call for maintenance and servicing. Recording rain gauges (ombrographs), which in turn can be equipped so as to be read at a central station will naturally afford maximum accuracy and an optimal basis for interpretation.

The areal evaporation component (including plant transpiration and interception) can be determined from the difference between precipitation and runoff where long records are available. Short-term studies should make allowance for retention components (glacial ice, ground water, retention effect, water storage in the soil etc.).

The hydrological data so obtained forms the basis for evaluation in the form of duration hydrographs, frequency curves and summation hydrographs and flood and low-water flow statistics by means of various methods of hydrological analysis. Paper 6 "Hydrological Analysis for the Planning of Small Hydropower Plants", states several examples of hydrological analysis.

4.3 BASIC DEVELOPMENT PLANNING

Design studies in a hydro development master plan should be based on various considerations.

4.3.2 Design Considerations

The great variety of development possibilities resulting from function, magnitude, head-flow relation, turbine type and topography govern the division into individual development units. In addition, the nature of a water course and the topography will call for a different type of structure in each particular case.

Criteria for storage schemes generally result from the water resource requirements, as e.g. insufficient gradient of a river rendering a diversion by-passing the river prohibitive, or cascade-development with daily-cycle pondage being desired, or aspects of environmental protection.

Storage and diversion type power developments will preferably be realised on streams with alternating flat and steep reaches, which are usually associated with gorges and basins. Whereas these considerations can be made in the office on the basis of topographical documentation, site reconnaissance offers an opportunity to perform a general investigation of potential reservoir sites. In this context, it should be emphasised again what has been mentioned above regarding the advantage of contacts with the engineering geologist. Fig. 4 - 1 (according to DEMMER, 1976) shows a geotechnical longitudinal section through a river course prepared as a basis for a master plan. This can be regarded as an example of a basic possibility of a time saving and efficient procedure.

The siting of storage units and in particular decisions regarding the development sequence are essentially aided by a simple economic consideration: Plotting of the product MQ . I (mean flow x bottom slope) on the longitudinal section gives a line corresponding to the specific output (related to the length of the stream) (fig. 4 - 2). Therefore, pronounced falls along a stream course suggest favourable development possibilities.

Where potential reservoir sites result from the topographical features of a stream contemplated for power development, there will be advantage in investigating dam sites not only for geotechnical suitability but also from the economic point of view. A simple comparative aid is found by plotting the reservoir storage curves of all the potential reservoir sites (fig. 4-3) over the whole

head range of the reach considered for development, or the quality coefficient of the dam relating, the quotient of storage volume divided by structural volume to the head of water impounded by the dam (fig. 4 - 4).
Other indices are:

$$\text{the retention capacity} = \frac{\text{storage } (hm^3)}{\text{annual volume of water yield } (hm^3)} \quad \text{and}$$

$$\text{the consumption time (h)} = \frac{\text{storage } (hm^3)}{3{,}600 \cdot Q_r (m^3/s)}$$

where:

Q_r ... rated discharge.

4.3.2 Multi-Purpose Utilisation, Regional Development

Like all water resource measures, development of small streams for power production should allow for general aspects of regional planning aiming at improving the infrastructure. In developing countries, the connexion of hydro power with any type of water supply and irrigation project will be a prime consideration, whereas in the highly industrialised countries, there will be advantage in combining mini hydro stations with measures of stream regulation and flood control. In basins with very irregular flow, any water resource project will necessarily involve the construction of dams and reservoirs, that is to say, water resource management has priority. This calls for certain layouts, that is, reservoir with or without diversion. However, besides the desirable effects, there are also a number of disadvantages to water resource management. Impounding of stream water disturbs the equilibrium of the river basin, firstly due to the reservoir space subject to water level fluctuations, which impairs the aesthetics of the landscape and the ground water balance. Whereas the transport of suspension loads stops in the reservoir, the reach downstream of a reservoir is subject to increased erosion. Where power stations are situated on a diversion from a reservoir, the problem of compensation water to be provided to the original stream bed must be considered.

4.3.3 Intermediate Uses

A problem of a special nature is the intermediate use of streams which, although intended for large-scale development for power production, are not planned to be developed until later. As the period of depreciation of installations for such intermediate uses must be reduced to allow for the shorter utilisation period, the economy involved will generally be lower. This disadvantage can be offset at least in part in various manners: In respect of electrical and mechanical equipment, preference will be given to turbine runners with a minor resistance to wear, e.g. to cavitation, and hence lower in cost, although this will give lower efficiencies. In the structural respect, too, greater risks will be accepted as to the return periods of floods, in view of the shorter service life.

4.3.4 Efficiency

Efficiencies in hydro power production have attained optimal values due to the development of technology over the last decades, by model testing and by the use of electronic data processing. Approaching the 100 percent limit would involve a substantial increase in cost. The maximum value and the degree of utilisation of the efficiency should exhibit a reasonable relationship with economy. A simple example is the cost increase of a turbine which is to be adapted to the fluctuations of stream flow and power demand rather than being evenly supplied, that is to say provision of a double control mechanism (Kaplan turbine) rather than a single control (propeller-type turbine). Considerations regarding the efficiency level to be aimed at could also be based on the following question: what is the relationship of the energy necessary for accomplishing this high efficiency to the actual energy output with account being taken of the life of the electrical and mechanical equipment. A ratio of 1:3 will probably meet the ordinary efficiency requirements of a good hydro power installations.

Small run of river stations equipped with say two power units and designed for a certain period of falling below the rated discharge must develop the flows smaller than the rated discharge at partial load. They will be operated so as to achieve optimal efficiency under the given circumstances. It may be good practice under certain conditions to provide only one double-control turbine, whereas the second will be of the single-control type. Whereas the latter turbine can almost always run at its optimal efficiency level, the Kaplan turbine, which attains good efficiencies even under partial load, will handle the balance of flows. A disadvantage of this policy is probably the greater amount of spare parts storage involved. Structural deviations can, however, be reduced to a minimum.

4.3.5 Aspects of Power Economy

Depending on the power situation of the area to be supplied, allowance should be made for the availability of energy. In mini power plants whose output will be fed to a large transmission system, availability will be less important than in the case of a power station intended for isolated operation for regional supply. In the latter case, provisions should be made both in the structural and the electrical/mechanical respects for facilities allowing fluctuations in the demand to be compensated both from the turbine supply and by the turbine control mechanism.

Essential factors in terms of power economy are the rated discharge, the guaranteed capacity, the long-term average power generation separated according to winter and summer (or wet and dry periods), as well as day and night availabilities. The relation of rated capacity to the time pattern of water yield in a river basin is expressed by the development factor

$$n = \frac{\text{number of hours of the year (8.760)}}{\text{utilisation period}}$$

where the utilisation period is defined by the quotient of annual energy divided by the installed capacity. Run of river stations have a development factor varying between 1.0 and 2.0 corresponding to a utilisation time of more than 4000 h/a.

Storage plants can reach a development factor exceeding 10, depending on the number of operating hours per year.

4.4 ECONOMY

The development characteristics adopted for of an installation (discharge, head, efficiency) should make allowance not only for the actual power requirements but primarily for economic considerations. Factors characterising the respective energy situation as e.g. oil crises, difficulties in the construction and operation of nuclear power stations,the fossil fuel shortages and the resulting electricity tariffs as well as the capital structure and primarily the construction cost, are of decisive importance.

Besides discussing construction cost, mention will only be made of the three most important groups among the great number of investigation methods based on the mathematics of finance, which are commonly applied for master plans.

4.4.1 Construction Cost

In determining the construction cost, division of cost items into three groups, namely

- lands and rights of way
- engineering structures
- electrical and mechanical equipment

to allow for the differences in depreciation periods has proved very useful. Cost of lands and rights of way are normally depreciated over a period of 90 years, engineering structures over 50 years and electrical and mechanical equipment over 25 years.

As to the cost of lands and rights of way, it is difficult to state general principles. Prices vary from region to region and are also subject to substantial instantaneous fluctuations.

The rough estimation of construction cost should be based on unit prices of construction materials (concrete, embankment dam, earth and rock excavation etc. in terms of cubic metres, impervious diaphragm, grout curtain etc. in terms of square metres) as well as on cost functions for total structural components (power intake, surge tank, stilling basin etc.) which, adjusted to the loading of the structural component by discharge, head, flow velocity etc., yield informative cost relations.

For the electrical and mechanical equipment (turbine and generator) cost diagrams showing the specific construction costs of such installations (installed capacity as a function of rated discharge and head, see Paper 13, figs. 13 - 4 and 13 - 5).

The study of different development alternatives should be based on cost relationship between the individual units rather than absolute costs, which are subordinate in importance.

4.4.2 Statistical Methods

The term "statistical method" applies to those economic analyses which assume both the capital charges and the electricity proceeds from power and energy to be constant over the depreciation period of the installation under study. These methods include e.g. the capital value method, the method of the internal rate of return, the annuity method and various cost-benefit analyses. The latter are often based on the power production cost of thermal power stations as reference.

4.4.3 Dynamic Methods

Dynamic methods consider not only the present status of the power and finance situation but are better adapted to the actual facts by means of trend analyses of inflation and electricity tariffs. As in all methods of forecasting, long-term forecasts should be corrected constantly to allow for newly arriving data so as to remain up to date. Consequently, the applications of this method are limited in the case of hydro installations as only present forecast data are available at the time of the project studies and, therefore, such data should not be used before the design stage.

4.4.4 Specific Construction Costs

Where hydro schemes are largely similar in type, comparison based on specific construction costs will give valuable results. A condition of the use of this method is, however, the possibility of characterising the economic importance of a project by means of the data of an average or characteristic year.

Specific costs are related both to energy and power and, in view of the great differences in the development factors of power schemes, are useful only where both values are available. The specific cost is obtained by dividing the total construction cost by the annual energy (kWh) or the installed capacity (kW):

$$\text{specific cost of energy} = \frac{\text{construction cost (AS)}}{\text{average annual energy (kWh)}} \quad \text{in AS/kWh}$$

$$\text{specific cost of power} = \frac{\text{construction cost (AS)}}{\text{installed capacity (kW)}} \quad \text{in AS/kW}$$

4.5 REPORT

The report should present all the documents and data made available as well as the information gathered in the course of the project investigations.

This mainly includes, besides the general geological report (fig. 4 - 1), the results of hydrological evaluations such as flow hydrographs, flow durations curves, frequency and summation curves.

In many cases, an argumentation in favour of the project proposal finally submitted will best be based on a study of project alternatives. Fig. 4 - 5 is a schematic representation of the result of such a study of alternatives. The upper part shows possible combinations of different project units, whereas the lower part indicates data on energy and power with their specific construction costs. Naturally this calls for a rough but uniform determination of the cost of the individual project alternatives already at the stage of alternative study.

Presentation of the project alternative suggested for construction should be in the form of a set of drawings which should be clear and easily understandable and include at least

- vicinity map
- general longitudinal section
- location plans of the individual units
- longitudinal sections of the individual installations
- characteristic section through the structure
  (basis for calculation of quantities)
- cost calculation
- presentation of water and power management

As to the development sequence for the individual units, recommendations should allow for the requirements of the region, the economy of the individual installations and the financial possibilities.

4.6 BIBLIOGRAPHY

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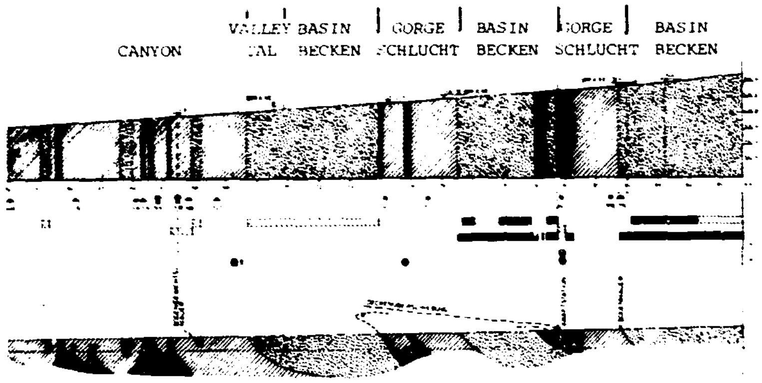
4 MASTERPLAN FOR MHP
RAHMENPLAN FÜR KWKW



1 POSSIBLE DAM SITES
MÖGLICHE SPERRENSTELLEN
▲ FILL DAM/SCHÜTTDAMM
● GRAVITY DAM/GEWICHTSMAUER
▲ ARCH DAM/BOGENMAUER

2 FEASIBLE TOP WATER LEVEL
MÖGLICHE KRONENHÖHE

3 DANGER OF SLIDES
HANGRUTSCHGEFAHR
OBSERVED/BEOBACHTET
SUPPOSED/VERMUTET

4 PERMEABLE
WASSERDURCHLÄSSIG
OBSERVED/BEOBACHTET
SUPPOSED/VERMUTET

5 IMPERMEABLE
WASSERUNDURCHLÄSSIG

6 MATERIAL FOR GRAVEL FILL DAM
MATERIAL FÜR KIESSCHÜTTDAMM

7 QUARRY FOR ROCK-FILL MATERIAL
STEINBRUCH FÜR STEINBROCKENDAMM

8 QUARRY FOR AGGREGATE
STEINBRUCH FÜR BETONZUSCHLAGSTOFFE

9 FLYSCH TRANSITION ZONE
FLYSCH ÜBERGANGSZONE

10 THIN BEDDED LIMESTONE
DÜNNBANKIGER KALK

11 RADIOLARITE/RADIOLARIT

12 THICK BEDDED LIMESTONE
DICKBANKIGER KALK

FIG 4 - 1 GEOTECHNICAL LONGITUDINAL SECTION
GEOTECHNISCHES LÄNGENPROFIL

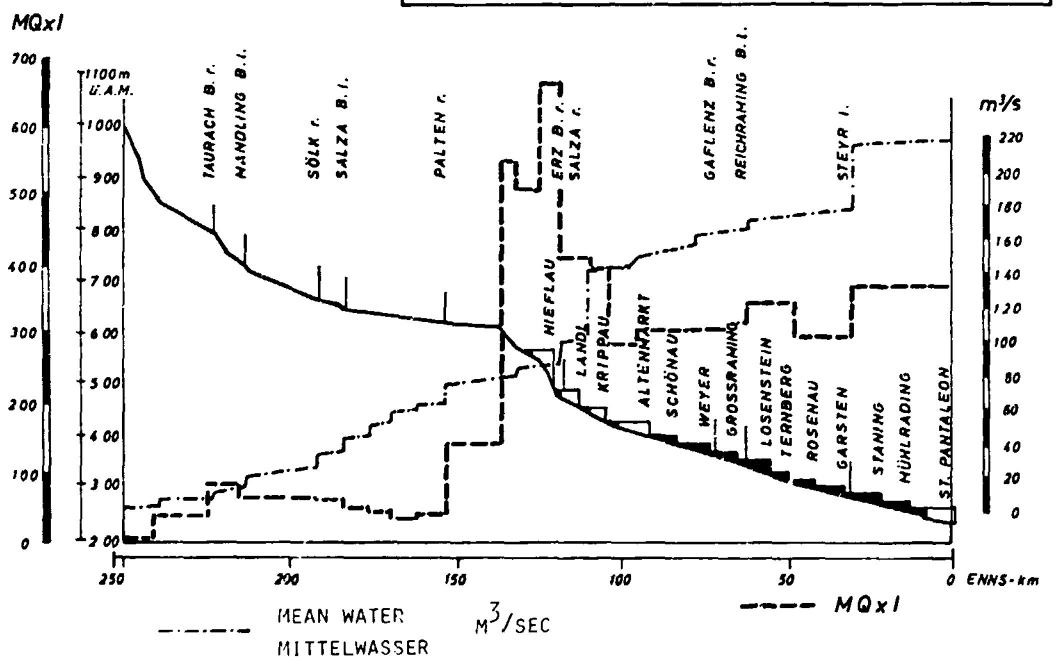


FIG 4 - 2 HYDROLOGICAL LONGITUDINAL SECTION
HYDROLOGISCHES LÄNGENPROFIL

4 MASTERPLAN FOR MHP
RAHMENPLAN FÜR KWKW

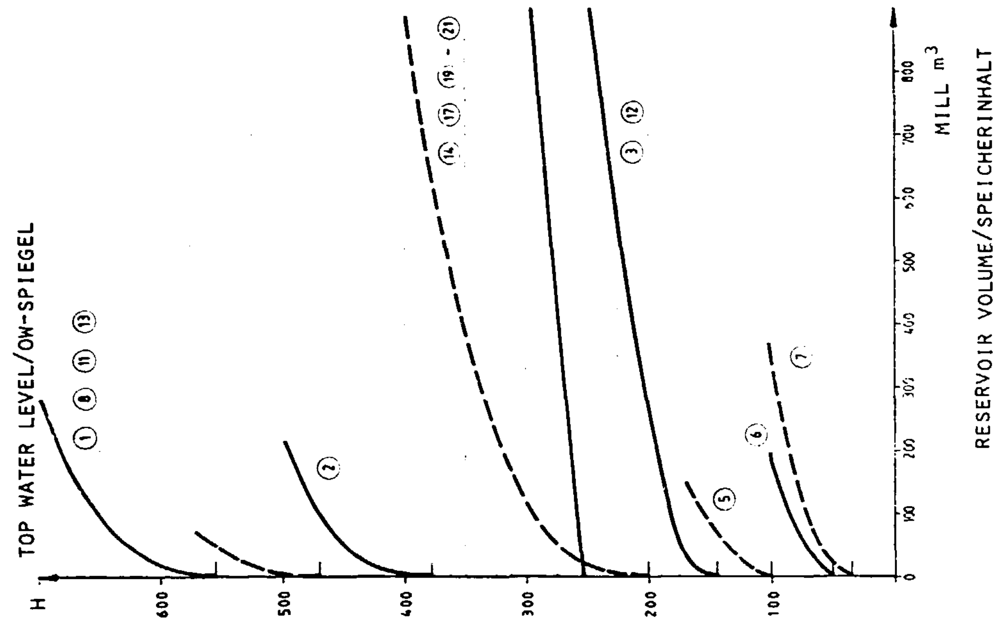


FIG 4 - 3 STORAGE CURVES
SPEICHERINHALTSLINIEN

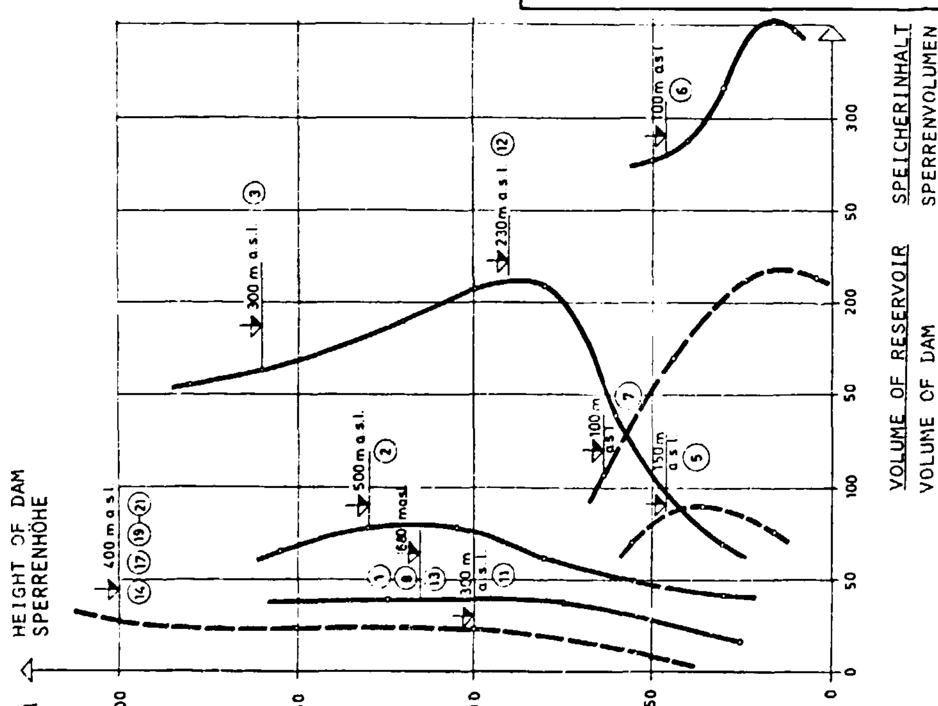
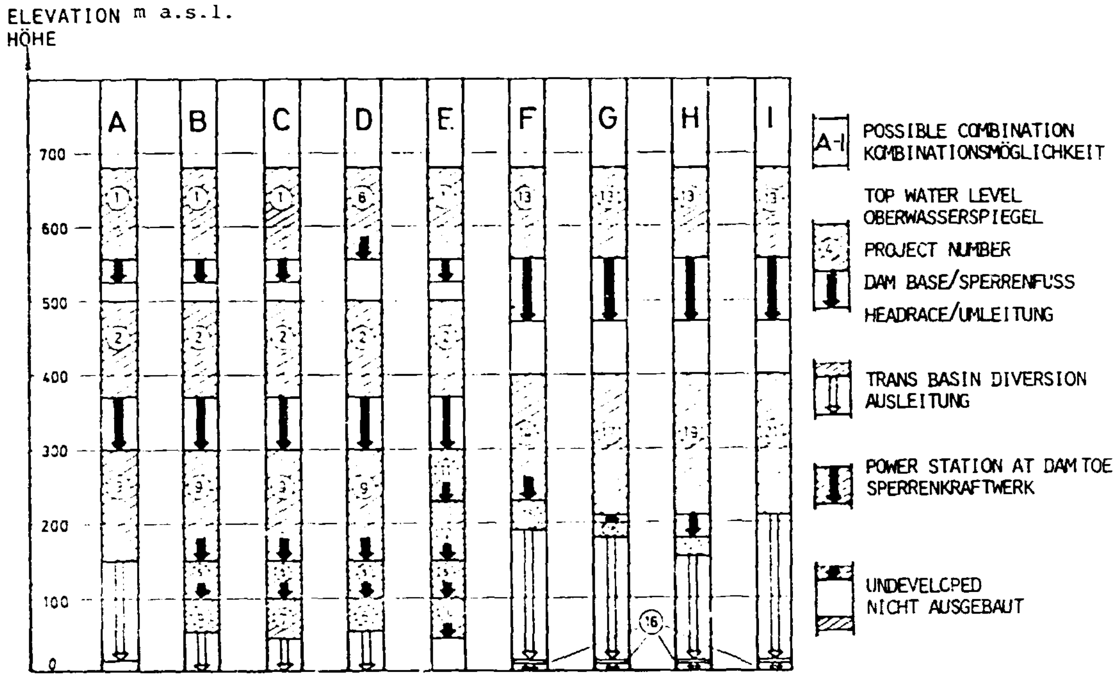


FIG 4 - 4 QUALITY COEFFICIENT
WW-AUSNUTZUNGSGRAD

4 MASTERPLAN FÜR MHP
RAHMENPLAN FÜR KWKW



POSSIBLE COMBINATION KOMBINATIONSMÖGLICHKEIT		TOTAL COSTS GESAMTKOSTEN (10^6 \$)	ENERGY ARBEIT (GWh)	POWER LEISTUNG (MW)	SPECIFIC COSTS SPEZIFISCHE KOSTEN ENERGY ARBEIT (\$ / kWh)	SPECIFIC COSTS SPEZIFISCHE KOSTEN POWER LEISTUNG (\$ / kW)
A	1+2+3	248.337	1056	888	0.235	280
B	1+2+5+6+9	321 383	1 054	880	0.305	355
C	+2+5+7+9	319 222	1 054	880	0 303	353
D	2+5+6+8+9	318 472	1 031	867	0 309	367
E	1+2+5+10+11+12	255 594	907	733	0 287	349
F	13+14+15+15	393 798	961	860	0 410	458
G	13+16+17+18	408 508	971	870	0 421	470
H	13+16+19+20	374 375	981	890	0 382	421
I	13+16+21	373 163	981	890	0 380	419

FIG 4 - 5 COMPARISON OF ALTERNATIVE SCHEMES
VARIANTENUNTERSUCHUNG

5 GEOLOGICAL AND GEOTECHNICAL STUDIES IN SMALL HYDRO PROJECTS

RIEDMÜLLER G.

ABSTRACT:

The geological and geotechnical investigations for mini hydro projects are composed of three phases, i.e. preliminary information, surface investigations and subsurface investigations.

Preliminary information refers to the evaluation of available geological documentation, topographical maps, air pictures and satellite photographs. Knowledge is thus gained on the main features of regional geology, photogeological interpretation is carried out, and seismic data allow conclusions regarding potential earthquake risks.

The ensuing surface investigation represents the main phase of the engineering geological studies. This concentrates on the geology of the foundations of dam or weir sites, the lithological composition as well as the effects of weathering, erosion and tectonics.

The geological assessment of planned reservoir areas with respect to slope stability, watertightness of the basin and sedimentation calls for comprehensive mapping works.

Detailed surface investigations also constitute the chief basis for the preparation of longitudinal-section forecasts for turbine-supply waterways and diversion tunnels.

It is on the basis of the results from the surface investigations that exploratory work, geophysical measurements and geotechnical tests are planned.

Subsurface investigation is primarily concerned with the area of the planned dam site and the foundations of surge tank, penstock thrust blocks and powerhouse.

The geotechnical tests for mini hydro projects are mainly confined to the study of permeability of rock mass (water pressure tests, seepage tests, piezometer measurements).

Large-scale testing to explore the strength of the rock mass is of secondary importance.

By way of summary, the geologist's function in the design of dams, in the clarification of slope stability problems and in the forecast of deep tunnels is demonstrated,using as examples several case studies from Africa, Greece, Indonesia and Austria.

5.1 INTRODUCTION

The construction of hydro projects usually constitutes a severe disturbance to the state of equilibrium geological complexes have attained over very long periods of time. The results are changes in natural drainage systems and erosion processes, long-term redistribution of stresses rock deformation and disturbance of slope stability (STINI 1942, 1956, JAEGER 1963, 1965, KERSCH 1964, FRY 1950, BELL 1980).

For appraising these changes and the effects of engineering structures on natural geological systems, the geologist's help is indispensable. The engineering geologist has important functions to fulfil during the whole development of a hydro project from the preliminary study to the detailed project. This also applies to project studies for mini power schemes, where geology may often make decisive contributions to feasibility studies and cost estimates. The main functions and work phases of the engineering geologist in the development of hydro project studies will be presented in the following chapters. This will be followed by some cas- -dies demonstrating the importance of geological results as a basis the design of hydro power schemes.

5.2 THE MAIN FUNCTIONS OF GEOLOGY

The great number of questions to be answered by the geologist in hydro projects result in a series of chief functions,which extend through the whole development of the project studies and well into the construction phase.

Already at the stage of the preliminary project studies, the geologist will have to answer questions regarding possible dam locations, reservoir sites, stream intakes as well as potential earthquake risks to engineering structures. It will be his main task to find and appraise the most favourable tunnel and pipe line alignments for waterways and diversion systems.

During ensuing stages of power project studies, the engineering geologist, in conjunction with the geotechnical expert, will appraise the stability and permeability of rock mass at envisaged dam and weir sites and thoroughly investigate the foundations of the powerhouse and of penstock thrust blocks subject to high loadings.

Examination of contemplated reservoir areas is another typical item of geological work It is primarily potential disturbance of slope stability from reservoir filling, the waterthightness of the basin and expected sedimentation that call for investigation.

Finally, the search for potential construction materials in the vicinity of the construction site is a task entirely within the geologist's responsibility.

The presence of raw material within economical haul distance from the site - concrete aggregates, materials for dam fill and impervious core - has often been an essential factor in the design of dams - whether concrete structures or embankments - and of upstream or internal impervious elements.

Plate 5-1 is a schematic representation illustrating the geological tasks and investigation items for the design of a hydro project.

5.3 DEVELOPMENT OF PROJECT STUDIES

The geological and geotechnical investigations in hydro projects are generally subdivided into three phases, i.e. preliminary information, surface investigations and subsurface investigations.

These phases of geological investigation are coordinated with the corresponding phases of the engineering studies, i.e. preliminary project, general project, detailed project (Plate 5-2).

5.3.1 Preliminary Information

Preliminary information is secured by a thorough study of existing geological maps and relevant literature. Coverage should be ample and extend beyond the construction engineering and engineering geological questions properly. It should be the objective of these studies to obtain a comprehensive knowledge of the regional geology of the area under investigation. Preliminary studies should not be confined to geological documentation. Interpretation of topographical maps, aerial pictures and satellite photographs provide information of great relevance to questions of engineering geology.

Topographical maps furnish data on the relief, the drainage system, reception basins and major landforms. Convex bulging of slope portions as recognised on maps may be an indication of large-scale landslides.

The information obtained from a topographical map is not restricted to the identification of landforms. Thus, a close relationship has been found to exist between the internal structure of a rock mass and its surface configuration. E.g. STINI (1925) and FLÜGEL (1951) have pointed out relationships between joint systems and the formation of the drainage systems. Very recently, SCHEIDEGGER (1979) has evaluated location patterns of Alpine valleys, which has allowed conclusion regarding the tectonic stress field. From a statistical analysis of valley directions in a hydrographical network, SCHEIDEGGER has obtained histographs from which the directions of the main normal stresses can be derived.

A valuable contribution to preliminary engineering geological information is obtained by the study of aerial photographs, the so-called air photo geology (ALLUM 1966). Exposed rock surfaces, outcropping rock strata, fault lines, unstable valley slopes, slides, effects of soil erosion and emerging springs can directly be seen from air photos, in particular from stereo pairs. In barren areas, fold structures, talus slopes, alluvial fans, rock fall material and karst phenomena are clearly recognised. By use of stereoscopic interpretation techniques (stereometer, parallax micrometer) it is even possible to obtain information on dip of rock strata and orientation of fault planes.

Besides air photos, multispectral satellite pictures of the LANDSAT (-ERTS) series, taken from an altitude of about 915 km, have lately been interpreted for use in geological investigations (BECKEL 1976, JASKOLLA 1978, TOLLMANN 1977). Satellite photos from the infrared region of the spectrum enable the resolution of rock mass into elements resulting from fault tectonics, inspite of cloud and vegetation cover. Structural details may stand out which even accurate investigation of the terrain would not reveal.

A main concern of preliminary information is the question of earthquake risk. All the seismic observations are collected and the extent and intensities of epicentral regions as well as their distance from the project area are recorded.

5.3.2 Surface Investigations

Surface investigations constitute the second and most important phase of the engineering geological studies.

Unfortunately enough, the importance of engineering geological reconnaissance is underrated in many construction projects. Careful surface investigations help to save expensive exploratory works and facilitate substantially the interpretation of geological and geotechnical data.

The engineering geological surface investigations can be subdivided into two parts, i.e. general site reconnaissance and the more time-consuming mapping.

The general site reconnaissance consists in fact of surveying geological sections to establish the fundamental features of the geological make-up, the suitability of reservoir sites and the best dam or weir locations. First indications of potential difficulties are obtained. Comparison of project alternatives and rough cost estimates are possible at this stage.

Geological detailed mapping provides the essential basis for the ensuing project studies. They afford representative information on the topography of the area under study. All the geomorphological and geological details recognised from surface inspection should if possible be represented on maps.

Geological detailed mapping is necessary for preparing longitudinal section forecasts for tunnels (transbasin diversions, waterways for turbine supply) and penstocks, for appraising envisaged reservoir basins as well as for a thorough investigation of subsurface conditions at dam and weir sites, water intake location and points from which tunnel driving is to be started.

The minimum scale allowed for engineering geological detail mapping is 1:10,000. This scale is suited for the geological mapping of deep tunnels. In mapping alignments, rock types should be classified according to aspects of rock engineering with particular attention to strength properties and texture (e.g. massive, plane schistosity, or gneisses poor or rich in mica).

A chief mapping item refers to the structural features. This should include, besides bedding and schistosity planes, faults, rupture zones as well as the strike directions of main joint sets. The mapping of structural features enables homogeneous zones to be identified in the structural pattern.

Structural statistics provide information on the pattern of tectonic stress fields, the primary state of stress and the anticipated degree of fracturing of the rock mass along the tunnel alignment.

Mapping should include details of wet zones, springs or spring horizons. This is done not only to secure evidence. The observation of ground water will allow conclusions regarding potential seepage paths and water conditions in the planned tunnel.

As to the detailed geological mapping of planned reservoir sites, a topographical map to scale 1:5,000 provides a very satisfactory basis. Clarification of the main questions, i.e. slope stability and watertightness of the basin, calls for a detailed study of the make-up of the rock mass, the structural features and the degree of weathering of exposed rock portions. Furthermore, the thickness of the waste mantle on valley slopes should be established. In connexion with a thorough study of landforms, it is necessary to map processes of slope erosion and denudation, the diverse forms of mass movement and mass transport ranging from talus creep, through slides, mud flows and rock slips to deep large-scale landslides.

For the investigation of reservoir watertightness, geological and morphological field work should be supplemented by hydrogeologicalmethods. E.g. aboveground and underground flow paths, potential percolation zones and karst phenomena should be studied. All ground-water levels as revealed by wells should be recorded and represented on maps together with springs and spring horizons. Data on yields and chemical properties, temperature as well as conductivity of waters should be gathered.

In appraising ground-water flow and the potential formation of new seepage paths in the planned reservoir basin, tracer tests with dyes or isotopes may be indispensible.

Geological detailed mapping of dam and weir foundations will usually call for topographical maps drawn to scales of 1:2,000 to 1:500. In conjunction with the surveying engineer, the geologist can substantially improve the contents of the map by use of the so-called geological-topographical surveying method, by setting out observation points for geological features as e.g. relevant exposed rock surfaces, ruptured zones, fault lines, evidence of failure surfaces, etc., and having these points surveyed by tacheometric or photogrammetric means by the surveyor (L. MÜLLER 1971).

The main purpose of detailed geological mapping of the planned dam or weir sites is obtaining information on the stability and permeability of the rock mass.

The engineering geological map should mark off soft from solid rocks and differentiate rock types and status of rock mass and should show the pattern of structural features in the greatest possible detail. Where fault systems are present, particular consideration must be given to the relative ages of the individual faults and to the establishment of young active movement.

An important factor is the study of the hydrogeological conditions within a major radius of the planned dam site. The question of permeability of rock mass and potential seepage around and under the dam calls for careful examination.

A diagrammatic representation of geological mapping as a basis for engineering geological investigations is given in Plate 5-3.

5.3.3 Subsurface Investigations

Careful investigation of surface features by the geologist provides the basis for the preparation of a subsurface investigation programme by exploratory methods, geophysical measuring methods and geotechnical testing.

Common exploration techniques are uncovering in-situ rock, trenches, drillings and adits. Exploration measures should be supplemented by geophysical investigations (GRIFF and KING 1969). These are primarily seismic refraction and resistivity techniques.

Over the last few years, gas measurements have been used successfully for tracing faults under thick masses of detrital material (ERNST 1971).

In mini hydro projects, geotechnical in situ tests to determine strength of rock mass (*e.g.* rock deformation tests, large-scale shear tests etc.) will be of minor importance, as loading from structures and impounded water will be moderate.

In situ tests will mainly be confined to the determination of rock permeabilities. These will primarily consist of tests carried out in drill holes (water pressure tests, seepage tests and piezometer measurements).

Where structures have to be founded on soft rocks and in waste mantles overlying bedrock, standard penetrometer tests should be performed to evaluate bearing capacity and settlement behaviour.

Subsurface investigation will mainly concentrate on the area of the planned dam site including ancilliary structures such as power intake, cofferdam and diversion tunnel.

In order to establish the depth to bedrock and the status of the rock mass in the foundation of the planned dam, it is recommended that a row of core drillings be sunk along the dam axis and these should extend up to the abutment slopes at least to the maximum operating level. Additional core drillings should be placed at the heel and toe of the future dam. The drilling depth is dependent on the height of impounded water and on the foundation conditions. It is a general rule that bore hole depths in the valley bottom should correspond to the planned height of impounded water. Depending on the orientation of discontinuities (faults, schistosity and stratification planes, main joints), abligne drill holes should be sunk besides the vertical drillings for a better exploration of the structure of the rock mass.

The lifted cores should be submitted to careful geological and petrographical investigation to determine the structure of the rock mass, overburden thickness, degree of weathering and fracturing as well as the mineralogical composition of clayey fillings of joints.

The permeability of the rock mass to water is appraised by means of water pressure tests. These are best performed during the process of vertical drilling by means of simple packings in sections of 5 m or 2 m each. The permeability of the rock mass is expressed as water loss in terms of 1/min/m under a maximum pressure acting in the test length of 100 N/cm². The result is then converted to a length of 1 m for a 10-minute test duration, in terms of Lugeon units (LUGEON 1933).

For the purpose of planning foundation grouting and treatment at the envisaged dam site, water pressure tests should be supplemented by trial injections. These will be located with allowance being made for the geological make-up and the structural features of the dam foundation as well as on the basis of water pressure test results.

Piezometers should be installed in some of the vertical drill holes for the observation of water table and water pressure fluctuations in the ground-water reservoir over a major period.

In some instances exploratory work at the dam site will have to include adits, even in the case of low dams for mini power projects. Adits are particularly suited for the investigation of the abutments, particularly where steeply dipping systems of joints, faults, and karstic zones are present.

Exploratory work at the dam site, including not only core drillings and adits but also trenches and artificial exposure of rock surfaces, should by all means be supplemented by seismic refraction profiles places along the future dam axis. Seismic refraction measurements enable interpolation of the boundaries of strata between the drill holes and are useful in discovering buried erosion channels not revealed by the drillings.

Subsurface investigations in the reservoir areas are normally moderate in extent. Exploratory work is limited to several core drillings sunk in the valley floor and slopes and located according to the geological requirements.

Water pressure and seepage tests should be carried out to investigate permeability of rock mass, flows of water, seepage paths and karst phenomena. It is recommended that some of the drillings in the slopes be extended to the level of the valley floor. All the drill holes should be equipped with piezometers, and measurements should be continued for the period of at least one year.

For the investigation of buried erosion channels and slope instabilities, seismic refraction profiles should be placed both lengthwise and across the valley axis.

In some cases, exploratory adits will even have to be provided to allow better appraisal and location of deep-reaching landslides. The adits can subsequently be used for long-term measurement of seismicity and slope deformation (precision levelling, extensometers, rubber tube levels).

For investigating the foundations of surge tank, penstock thrust blocks and powerhouse, trenches and several short core drillings will normally suffice.

Appraisal of power and diversion tunnel alignments will in most cases be based on surface investigations alone and require no additional subsurface investigation.

The results of geological and geotechnical explorations should be plotted on geological maps and sections, structural diagrams, or drill hole logs in connexion with results from water pressure measurements, seepage and piezometer measurements as well as trial injections. These documents should be accompanied by a report comprising general geological explanations and a description of the rock materials and the make-up of rock mass found in the project area as well as a detailed critical appraisal of all the geological and geotechnical questions. The organisation of project site investigations is schematically shown in Plate 5-4.

5.4 CASE STUDIES

In the following paragraphs, several brief case studies will be presented to illustrate the geologist's work phases and functions.

5.4.1 Dams

The geological problems involved in design studies for dam construction will be illustrated by the example of the planning and design studies for small dams in Dogonland (Republic of Mali, Africa) (WEISS 1979; STEINGRUBER 1978). The planned dam sites are located in a thick sandstone complex of Cambrian age. The structural features consist of near horizontal bedding joints and steeply dipping, NNE to SSW and ENE to WSW striking main joint sets.

Owing to the poor vegetation cover, it was possible for WEISS to appraise the stability and permeability properties of the rock mass on the basis of a detailed surface reconnaissance of the planned dam sites. It followed from geological field work that the valley floor was characterised by pronounced bedding planes and was largely impermeable to water. Zones of potential seepage paths had mainly formed as a result of stress relief towards the surface, which had caused fissuring and separation of blocks in zones above the level of the valley floor. Thus careful geological investigation of the terrain even without artificially exposed rock areas yielded indications of potential paths of water flow and seepage under and around the planned dams with heights ranging from 8 to 20 metres. This information allowed definite proposals to be derived regarding required rock excavation and grouting depths (Fig. 5-1).

5.4.2 Reservoir Area - Slope Stability

An example of a very comprehensive appraisal of slope stability in a reservoir area on the basis of detailed geological surface and subsurface investigations is afforded by the project studies for a hydro power scheme in north-western Greece (MACOVEC, HORNINGER and RIEDMÜLLER 1972). The reservoir basin is entirely composed of sandy-silty flysch series of Early Tertiary age. Consequently, problems of seepage were of minor importance from the outset. Geological exploratory work chiefly concentrated on questions of slope stability.

Following detailed geological and geomorphological mapping to scale 1:10,000 of the whole reservoir area, a slope portion in the vicinity of the dam site was first selected for detailed study as its stability appeared doubtful. Surface evidence had suggested that reservoir filling might trigger large-scale sliding. This was suspected on the basis of outcropping failure planes and structural features as e.g. the bending of strata near the surface.

Detailed geological and geomorphological mapping to scale 1:5000 of the critical zone by HORNINGER proved that the slip surfaces seen in the field were not an indication of a deep failure surface on which large-scale sliding was taking place but were associated with a great number of isolated shallow slides. The structural analysis of the rock mass allowed the conclusion that the near surface bending of strata should not be interpreted as a result of the action of slope movements but was due to folding along with the process of primary tectonics.

To support surface evidence, two core drillings and two exploratory adits were placed.

One adit was intended to provide information on the foot of the slope. The other one was driven in a shallow depression resulting from sliding, in the upper portion of the slope. Detailed geological investigations performed by RIEDMÜLLER yielded the following result: The adit at the foot of the slope first crossed a thick mass of mudflow debris and then reached solid rock. The mudflow deposits exhibited cross-bedding and a preferred orientation of the pebbles. Evidence of displacement of the primary sedimentary structures as a result of potential slope movement was found in no place.

Underlying the mudflow debris, and immediately above the slightly weathered relief of base rock, a thick layer of silts with fossil wood particles was encountered. Geochronological analysis of this fossil wood gave an age of 27,000 ± 800 years. This proved that undisturbed fossil mudflow debris had sealed the foot of the slope like a plug and that at least since that time slope movement had not taken place.

Investigation of the structural features of the siltstone and sandstone sequence in the upper adit revealed a pattern of mechanical stresses that was clearly a result of primary tectonics and showed good agreement with the data determined from surface evidence.

The rock mass exhibited no indication of deep loosening or deep-reaching mass movement. The results of structural geology were corroborated by the results of mineralogical analyses, which suggested that there was an undisturbed sequence of fossil zones of weathering from the surface towards the interior of the rock mass (RIEDMÜLLER 1972).

In the light of the sum of the different results of detailed geological investigations, it appeared safe to preclude entirely the potential risk of large-scale failure.

5.4.3 Slope Stability - Penstock

In the appraisal of a slope considered as a potential penstock location, the use of mineralogical methods furnished a substantial contribution to the geological information.

This example has been taken from the project studies for a mini hydro scheme in Minahasa, the northernmost peninsula of Sulawesi (Indonesia).

The geological make-up of the site is simple. In situ rock is composed of mostly near-horizontal beds of andesites and tuffites. The overburden consists of generally substantial thickness of brown latosoles which cover the base rock partly as residual deposits, mainly, however, as transported soils.

The project provided for a penstock of 300-metre length leading from a valve chamber down a steep slope and ending in the powerhouse located 100 metres below, at the level of the stream.

Appraisal of slope stability on the basis of geomorphological features did not yield any indication of mass movement. Apart from a few small erosional gullies, no evidence of slope denudation or major mass movement was found.

For the purpose of subsurface investigation, trenches and drillings were placed along the penstock alignment. These revealed that andesites weathered to major depth were covered with brown loams ranging from 4 to 7 metres in thickness.

The detailed geological surveys made in the trenches allowed the brown loams making up the overburden to be clearly identified as soils transported parallel to the slope surface. Rows of oriented andesite blocks gave evidence of talus creep (fig. 5-2a).

For the design of the penstock foundation it was necessary to determine the age of the movements in the overburden. It was essential to know whether these were evidence of recent or fossil soil creep.

The answer to this question was found by systematic sampling and soil chemistry and clay mineralogy analyses (MÜLLER, RIEDMÜLLER and SCHWAIGHOFER, printing).

Throughout the section, from the surface down to the weathered andesite, the clay mineral halloysite was found to be almost the only crystalline constituent. Halloysite was encountered in two forms: in a hydrated form as so-called 10 A halloysite, and in a dehydrated form as 7A halloysite. The hydration status of the halloysite is governed by the moisture content of the soil. Prolonged drying leads to irreversible dehydration of the 10A halloysite.

The result of x-ray diffraction analyses showed that only 7A halloysite was present in the weathered andesitic base rock, whereas the overburden exhibited a gradual dehydration of the halloysite from the depth towards the surface. That is to say, the highest concentration of hydrated 10A halloysite was found at greater depths in the overburden, whereas the 10A halloysite gradually decreased for the benefit of 7A halloysite along with decreasing depth. The relationship between the two types of halloysite showed a clear linear gradient along with section depth (fig. 5-2b).

The mineralogical result allowed important conclusions for the appraisal of slope stability at the penstock site: Dehydration of 10A halloysite was younger than talus creep. Recent tropical climate was not a satisfactory explanation of the dehydration process. This had to be due to a fossil climate with more pronounced dry seasons. This means that the soil creep processes had to be of a fossil nature and that no slope movement had taken place after the formation of the dehydration gradient.

5.4.4 Tunnel Forecast

The trans-mountain diversion system forming part of a hydro project constructed in the central zone of the eastern Alps (Hohe Tauern) will be used as an example (KIESSLING 1969, 1979, WEISS 1969) to illustrate the engineering geological studies for tunnel. With the help of the geological information available (EXNER 1964), it was possible already during the preliminary stage of project investigations to determine the fundamental features of water intakes and tunnel alignments for an extensive transbasin diversion system.

Alignments traced on a map to scale 1:10,000 then provided the basis for making the tunnel forecasts for the detailed project. The conditions actually encountered during tunnel driving showed very good agreement with the forecasts, which had exclusively relied on the results of geological surface investigations (RIEDMÜLLER and WEISS 1967, RIEDMÜLLER 1969, LITSCHER 1974).

One of the planned tunnel alignments of the diversion system will be described in greater detail to illustrate engineering geological forecasting for a deep tunnel (G. RIEDMÜLLER 1979).

The tunnel alignment crosses a gneiss complex ("Zentralgneiss" central gneiss formation) and a thick series of limestone phyllites, calcareous phyllites, greenschists and quartzites etc. ("Schieferhülle"). The last one-third of the tunnel alignment was classified as particularly unfavourable from the tunnelling point of view. This is a zone of complex geological make-up with severe shearing of gypsum-bearing phyllites ("Matreier Zone"). In this zone, a large-scale landslide had developped so that the diversion tunnel had to pass underneath. In order to explore the depth of the basal slide planes, seismic refraction measurements were taken in the front zone. In conjunction with the geological surface investigations, it was finally possible to determine the boundaries of the slide mass.

The factors to be allowed for in geological tunnel forecasting are mainly rock, structural tectonics, overburden and pore and joint water.

These factors are established by geological surface mapping to scale 1:10000 of the area traversed by the alignment.

Arrangement of the great variety of rock types in series was based on engineering geological aspects rather than a classification according to regional geology. From laboratory analyses of the mineral constituents and microstructure as well as determination of uniaxial strengths, rock characteristics were derived which were intended as a basis for preparing the tender documents for heading driving by means of tunnelling machines.

Zones homogeneous in respect of the orientation of structural features and the degree of disintegration of the rock mass were defined on the basis of structural statistics. Extrapolation of rock boundaries and structural features from surface evidence to deep zones for the design of deep underground structures is extremely doubtful. With allowance being made for the tectonic pattern, only general information was derived on the orientations of structural features in relation to the centreline of the excavation. The same is true of forecasting rock disintegration. It was possible only to estimate certain zones with different types of schistosity and rough values of joint density.

All the faults and disintegration zones were projected onto the tunnel level. From empirical values it was possible to forecast rock decomposition and the secondary formation of clay minerals with a prevalence of smectite.

Information on the make-up of the rock mass, structural data, the positions of structural features relative to the tunnel centreline and the overburden pressure combined conveyed an approximate idea of the state of stress of the rock mass.

Forecasts of pore and joint water conditions had to remain general in nature and provided no information on individual points of ingress or quantities. The permeability of the rock mass was roughly estimated from springs and wet zones established during the surface investigations.

All the predicted geological factors finally enabled a forecast of rock classes according to aspects of heading driving and the classification of rock masses from the tunnelling point of view.

5.5 CONCLUSION

Geological and geotechnical studies for mini power projects should mainly consist of extensive surface investigations which should allow for all details of surface evidence ranging from structural geology to morphology and Quarternay geology.

It is only following geological surface investigations that subsurface investigations in the form of exploration techniques, geophysical methods and geotechnical in situ tests will give satisfactory results.

The engineering geologist should act as a coordinator and, in cooperation with the construction engineer, should draw conclusions from the great number of geological results obtained for use in structural design.

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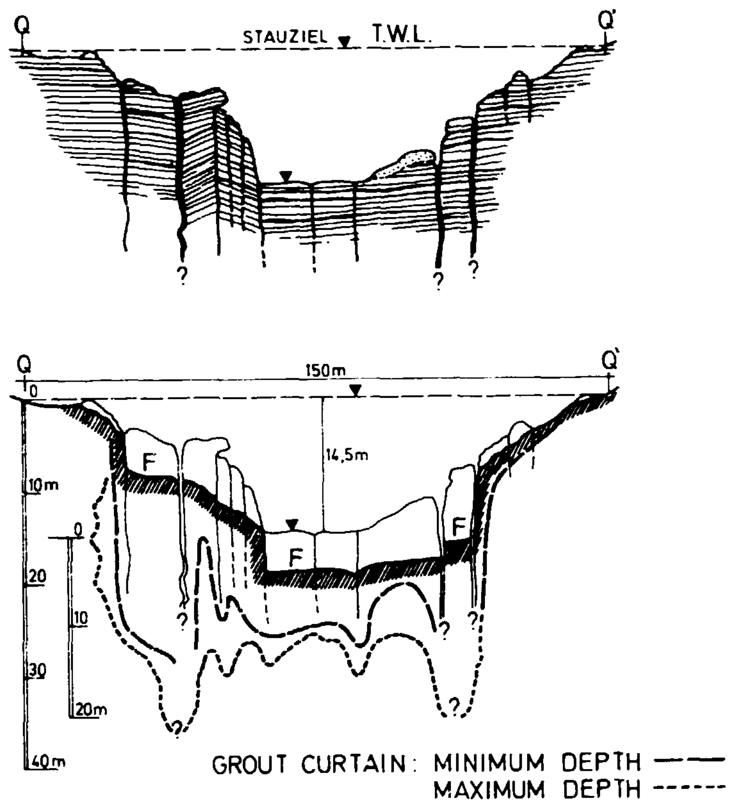

FIG 5-1 SECTION THROUGH DAM FOUNDATION
PROPOSED DEPTH OF ROCK EXCAVATION (F)
AND GROUT CURTAIN (E.H.WEISS, 1979)

5 GEOLOGICAL AND GEOTECHNICAL STUDIES
GEOLOGISCHE UNTERSUCHUNGEN

STRUCTURES	MAIN TOPICS OF GEOLOGICAL INVESTIGATION 1	2	3	4	5	6	7	8	9	1o
WATER INTAKE, WEIR	●	●	●		●					
EMBANKMENT DAM	●	●	●		○	●	○	○	○	
CONCRETE DAM	●	●	●		○	●	○	○	○	○
RESERVOIR BASIN	●	●	●	●	●					
TUNNEL, SHAFT		●	●		●	●	○		○	○
PENSTOCK	●	●	●	●	●		○		○	○
POWERHOUSE	●	●	●		●				○	
CHANNEL	●	●	●	●	●					

CONSTRUCTION MATERIALS	1	2	3	4	5	6	7	8	9	1o
SOFT ROCKS (CLAYS, SANDS, GRAVELS)	●	●				●	○			
HARD ROCKS	●	●	●			●	○			

● MAIN FUNCTION ○ ADVISORY FUNCTION

1 SPECIAL GEOMORPHOLOGICAL FEATURES
2 GEOLOGICAL STRUCTURE
3 STRUCTURAL FEATURES
4 SLOPE STABILITY
5 HYDROGEOLOGICAL FEATURES (GROUND WATER, PERMEABILITY, UNDERSEEPAGE)
6 PETROGRAPHICAL-MINERALOGICAL DATA
7 GEOTECHNICAL DATA
8 BEARING CAPACITY
9 DEFORMATION AND SETTLEMENT BEHAVIOUR
1o ANISOTROPY OF LOADING CAPACITY

PLATE 5-1 MAIN TOPICS OF GEOLOGICAL INVESTIGATION

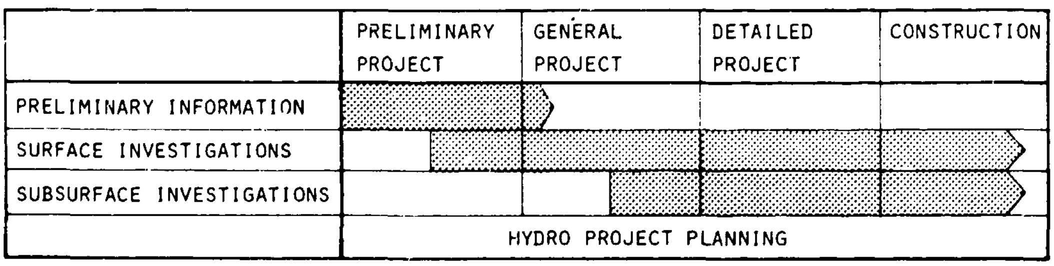

PLATE 5-2 DEVELOPMENT OF PROJECT STUDIES

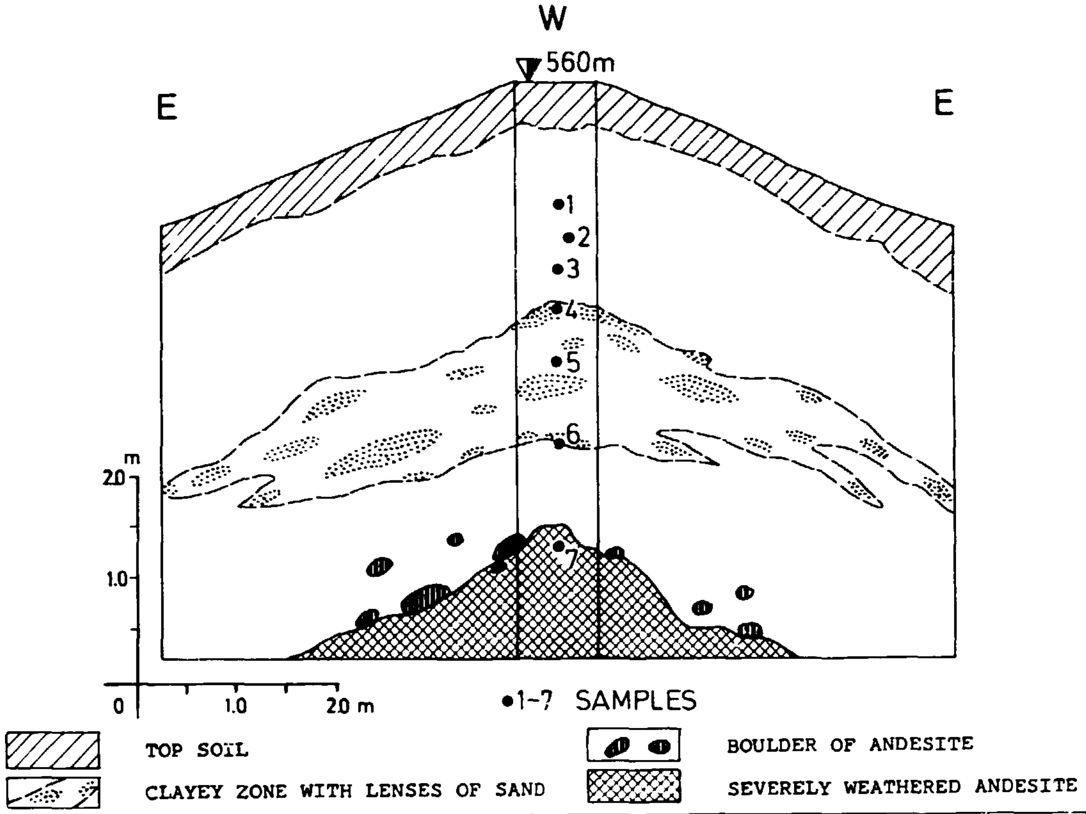

FIG 5-2A TRENCH SHOWING SECTION THROUGH OVERBURDEN

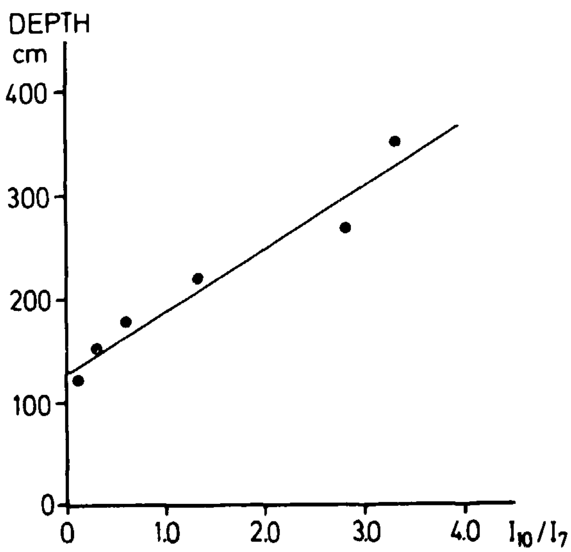

FIG 5-2B DIAGRAM SHOWING DEPTH PLOTTED AGAINST INTENSITY RATIO OF BASAL REFLECTIONS (10 Å-HALLOYSITE TO 7 Å-HALLOYSITE)

5	GEOLOGICAL AND GEOTECHNICAL STUDIES GEOLOGISCHE UNTERSUCHUNGEN

	FEATURES SHOWN ON MAPS	SCHEMATIC MAPS NOT LARGER THAN 1 : 500 000	SMALL SCALE MAPS 1 : 50 000 TO 1 : 200 000	MEDIUM SCALE MAPS 1 : 10 000 TO 1 : 25 000	LARGE SCALE MAPS NOT SMALLER THAN 1 : 5 000
ROCKS	FORMATIONS				
	STRATIGRAPHIC-GENETIC UNITS				
	LITHOLOGICAL SERIES				
	PETROGRAPHIC ROCK TYPES				
	ENGENEERING PROPERTIES OF ROCKS				
	OVERBURDEN IN GENERAL				
	OVERBURDEN, DIFFERENTATION				
DEGREE OF WEATHERING	DECOMPOSITION RESIDUAL SOILS				
	WEATHERED				
	NON-WEATHERED				
STRUCTURES	MAJOR TECTONIC UNITS				
	NAPPE BOUNDARIES				
	MAJOR FAULTS				
	STRATIFICATION AND SCHISTOSITY PLANES				
	FAULTS				
	MAIN JOINT SETS				
	SLICKENSIDES				
	JOINT DENSITY				
	ZONES OF HOMOGENEOUS STRUCTURE				
	FAULT AXIS (b-AXIS)				
	LINEATIONS				
GEOMORPHOLOGY	TERRACES				
	TALUS SLOPES, ALLUVIAL FANS				
	TALUS SLOPES, ALLUVIAL FANS, IN DETAIL				
	ROCKFALL DEBRIS				
	MORAINES IN GENERAL				
	MORAINES IN DETAIL				
	SLIDES				
	LARGE-SCALE LANDSLIDES				
	KARST PHENOMENA				
HYDROGEOLOGY	DRAINAGE SYSTEM				
	SPRINGS				
	WET AREAS				
	DEPTH OF WATER TABLE				
	ESTIMATED ROCK PERMEABILITIES - K_F (cm/s)				

PRELIMINERY INFORMATION GENERAL INFORMATION ON REGIONAL GEOLOGY	GENERAL KNOWLEDGE OF ROCK SEQUENCE AND TECTONIC FEATURES FIRST IDEAS FOR THE STUDY OF PROJECT ALTERNATIVES	DETAILED INFORMATION ON GEOLOGICAL GEOMORPHOLOGICAL AND HYDROGEOLOGICAL CONDITIONS BASIS FOR STUDY OF ALTERNATIVES SUBSURFACE INVESTIGATIONS AND FORECAST REGARDING DEEP TUNNELS GENERAL MAPPING OF RESERVOIR AREA	DETAILED SURFACE INVESTIGATION OF DAM SITES RESERVOIRS, PENSTOCK ALIGNMENT AND POWERHOUSE FOUNDATION AREA

PLATE 5-3 GEOLOGICAL MAPPING AS A BASIS FOR ENGINEERING GEOLOGICAL INVESTIGATIONS

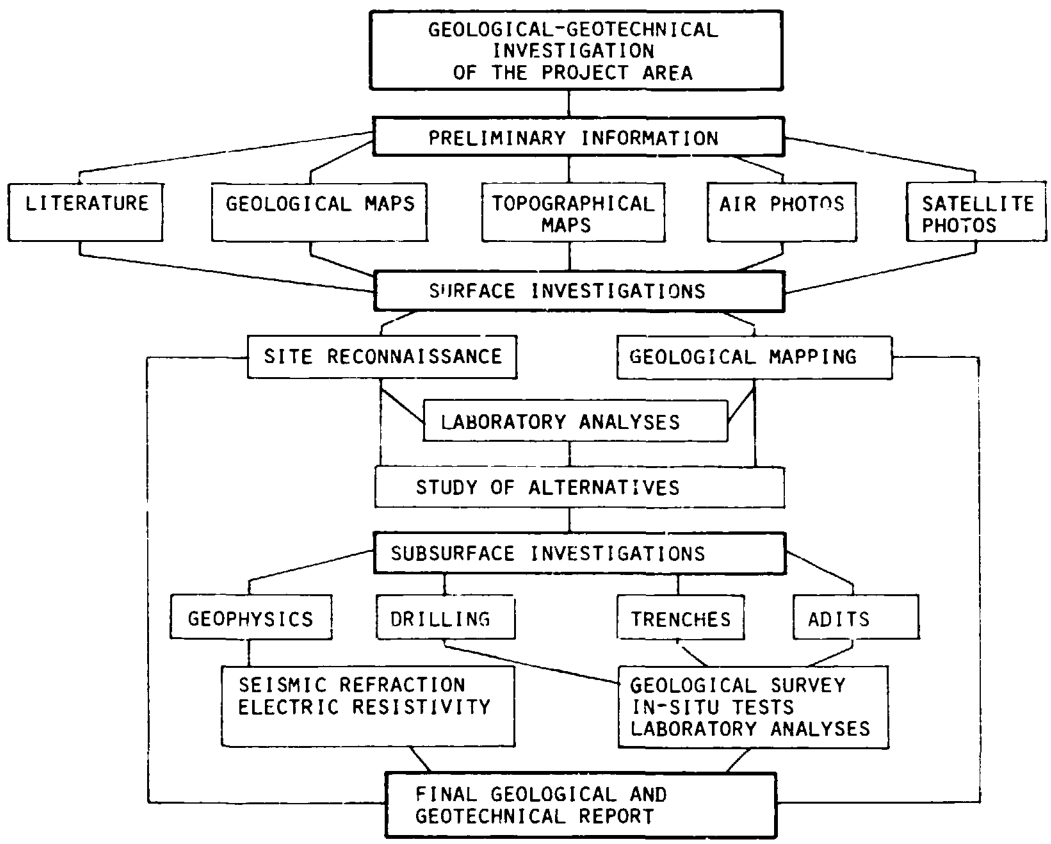

PLATE 5-4 ORGANISATION OF PROJECT SITE INVESTIGATIONS

6 HYDROLOGICAL ANALYSIS FOR THE PLANNING OF SMALL HYDROPOWER PLANTS

NACHTNEBEL H.P.

ABSTRACT:

For the planning of small hydropower plants as well as for their operation basic hydrological values have to be determined. These basic design data contain the determination of the average annual runoff as well as of the rated discharge, the high- and low flow statistics and the plotting of duration curves. According to the availability of runoff and precipitation observations different methods can be applied, each of which will be explained with examples. In each chapter, first, appropriate methods for analysis will be described when sufficient data from the project site are available. Following that, special attention is given to the transposition of data and regionalisation, if documentation is scarce or exists only for other measuring stations.

6.1 INTRODUCTION

In general it may be said that the quality of the necessary hydrological values is dependent on the quality of the observation material. The quality of data depends upon the frequency of measuring stations, the time scale of resolution of the measuring instruments, the length of observation and the accuracy of observation. The better the data material the more simple the evaluation methods. Consequently, in cases of limited information specific methods and more extensive investigations are necessary. In the following presentation, first the selection of interesting river sections will be shown. Then, design units will be determined and operation criteria discussed.

6.2 DETERMINATION OF RAW ENERGY POTENTIALS

The raw energy potential A of a river section results from (1) the difference of level H_S; the discharge Q; the time t in which discharge Q is available, and a constant C which together with the gravity takes into account the efficiency of the installations.

$$A = Q \cdot H_S \cdot C \cdot t \qquad (1)$$

The annual raw energy potential A_R can be obtained by replacing Q by the long term mean runoff MQ. Taking into consideration the measurement units and an efficiency of o.85 given, we get

$$A_R(\mathrm{GWh}) = MQ(\mathrm{m^3/s}) \cdot H_S(\mathrm{m}) \cdot C_1$$

$$C_1 = 0.073$$

Basic data of an Austrian catchment in the prealpine region are presented in figure 6.1. A longitudinal section exhibits the bed level along the main channel and the size of the catchment. The runoff data of five measuring stations are given; three of which are located directly at the main river - the Schwarza.

A simple method for determining the quantity of runoff for non-observed sections consists in establishing a relation between the mean flow and size of catchment. As may be seen from the accompanying figure 6.2 a linear correlation exists so that a runoff resulting from the size of catchment can be concluded. That means that we can assume a standard runoff unit h_A. This statement clearly outlines the scope of application of this simple method. In cases of diverse geological and meteorological conditions varying runoff coefficients will be the result and according to that non-linear relations between catchment and quantity of mean flow must be assumed. In terms of methods that means that the linear regression is to be extended to non-linear regression methods. If, for a certain river area only a few or no long-term runoff data exist, extensions of the so far described methods are necessary. In figure 6.3 a relation between the size of the catchment and the quantity of mean flow has been set up for another Austrian catchment in the pre-alpine area, i.e. the river Ybbs, by adding runoff observations of river areas in the vicinity. The working equation which is determined by a regression equation is also stated. The mathematical and statistical aspects of regression analysis are described in detail in the literature f.e. in YEVJEVICH (1971), SACHS (1968). Procedures for testing a general hypothesis in a linear regression are included as well as the estimation of confidence intervals.

Another possibility to determine the average runoff consists in computing the annual runoff factor from the hydrological and meteorological measuring data. With that according to NORDENSON (1968) the resulting runoff and consequently the runoff quantity is determined from the amount of rainfall and evaporation. For many countries, i.e. river areas, information about the runoff coefficient is available. Another, however, rough estimation of the mean flow, can also be obtained by empirical approaches as e.g. in WUNDT (1949):

$$MQ = M_q \cdot E$$

M_q Specific runoff (l/skm^2)

E Catchment area (km^2)

The specific runoff M_q for different regions is shown in figure 6.4.

After having determined the energy potential of a course of a river with the described methods we have an important basis for designing the project. One must, however, bear in mind that together with economic considerations also the infrastructure, the geological situation and ecological restrictions have to be taken into account.

6.3 FLOOD FREQUENCY ANALYSIS

After having selected an economically interesting river section, extremes in the runoff events (floods and low water) have to be considered for the layout of the plant. Also the time distribution of the runoff has to be taken into account. The weir, i.e. the intake, is to be designed such that with extreme runoffs a drawdown of the water volume can be effected without any damage of the installations. As the flood frequency statistics are based upon probabilistic assumptions this requirement for a flood flow drawdown without any damage can only be expressed in probabilistic quantities.

6.3.1 Analysis of Extreme Events

With the assumption that for a project site a series of runoffs over a long period of time is available, a simple calculation of flood flow probabilities is possible. The maximum annual values are being taken from the observation series as has been done for the gauge of Gloggnitz in the catchment of the river Schwarza (see table 6.1.).

YEAR	HQ (m^3/s)	YEAR	HQ (m^3/s)	YEAR	HQ (m^3/s)
50	40	60	120	70	145
51	120	61	260	71	74
52	100	62	50	72	135
53	140	63	48	73	74
54	105	64	38	74	120
55	95	65	125		
56	82	66	250		
57	145	67	115		
58	105	68	60		
59	200	69	61		

Table 6.1 Annual extremes of the runoff series 1950 - 74, at the gauge of Gloggnitz

The following calculations are based on the assumption, that the readings are stationary and that the flood events are independent of each other. The first condition implies that the parameters such as mean value, variance etc., computed for any portion of the total observation period, are not significantly different of each other. There are various reasons for instationarities like trends within flood series patterns and most of the time the are subject to human activities. Some examples may serve as an illustration:

A large-sized clearing in the catchment reduces the retention potential and leads as a consequence to the increase of flood flow peaks. The same is true for the decreasing retention effect with the growing urbanisation within the catchment. Flood flow decreasing effects result from installing big reservoirs set up f.e. for irrigation purposes. Moreover, such reservoirs reduce the runoff quantity through takeoff, evaporation and seepage. Additionally a reduced variation of runoff may be observed.

In order to detect non-stationarities within records both time series analysis and simple non-parametric testing methods are advisable. A detailed presentation of these methods can be found in YEVJEVICH (1972/2). Some aspects are being discussed by EGGERS (1970) and NACHTNEBEL (1975).

When calculating flood flow probability a connection between the runoff component (flood peak) and the return period is shown. This time period stated in numbers of years indicates that in the mean of all n years an event occurs of at least HQ_n. The probability of annual occurence W_A corresponds to the reciprocal value of n

$$W_A = \frac{1}{n}$$

Based on extensive investigations six distribution functions which turned out to be especially suitable are being listed in the directions of KWK-DVWW (1976):

Pearson-3 - distribution
Log-Pearson - distribution
Log-Normal - distribution
Gumbel distribution
Gamma - distribution
Log-Gamma - distribution

As an illustration the calculation of floods with a given return period is being shown with the Gumbel - and Pearson-3 - distribution.

Gumbel distribution

This method among others is described in detail in the WMO-Report (1970) and in KOBERG et al. (1975) so that mathematical explanations are being left out here. An event X with a return period n of 50, 70 and 100 years is determined with the help of (2) as follows:

$$X_n = X + K \cdot S_x$$

$$K = \frac{Y_n - Y_L}{S_L} \qquad (2)$$

$$Y_n = -\ln \ln \frac{n}{n-1}$$

With the Gumbel distribution the mean value X and S_X can directly be determined from the observations, whereas Y_n are to be defined with the return period of the event that is to be extrapoled. Y_L and S_L are the corresponding parameters of the reduced variate and are to be taken from table 6.2. and 6.3. with consideration of the length of observations.

	0	1	2	3	4	5	6	7	8	9
10	0.4952	0.4996	0.5035	0.5070	0.5100	0.5128	0.5157	0.5181	0.5202	0.5220
20	.5236	.5252	.5268	.5283	.5296	.5309	.5320	.5332	.5343	.5353
30	.5362	.5371	.5380	.5388	.5396	.5402	.5410	.5418	.5424	.5430
40	.5436	.5442	.5448	.5453	.5458	.5463	.5468	.5473	.5477	.5481
50	.5485	.5489	.5493	.5497	.5501	.5504	.5508	.5511	.5515	.5518
60	.5521	.5524	.5527	.5530	.5533	.5535	.5538	.5540	.5543	.5545
70	.5548	.5550	.5552	.5555	.5557	.5559	.5561	.5563	.5565	.5567
80	.5569	.5570	.5572	.5574	.5576	.5578	.5580	.5581	.5583	.5585
90	.5586	.5587	.5589	.5591	.5592	.5593	.5595	.5596	.5598	.5599
100	.5600									

Tab. 6.2. Mean value Y_L of the reduced variate

	0	1	2	3	4	5	6	7	8	9
10	0.9496	0.9676	0.9833	0.9971	1.0095	1.0206	1.0316	1.0411	1.0493	1.0565
20	1.0628	1.0696	1.0754	1.0811	1.0864	1.0915	1.0961	1.1004	1.1047	1.1086
30	1.1124	1.1159	1.1193	1.1226	1.1255	1.1285	1.1313	1.1339	1.1363	1.1388
40	1.1413	1.1436	1.1458	1.1480	1.1499	1.1519	1.1538	1.1557	1.1574	1.1590
50	1.1607	1.1623	1.1638	1.1658	1.1667	1.1681	1.1696	1.1708	1.1721	1.1734
60	1.1747	1.1759	1.1770	1.1782	1.1793	1.1803	1.1814	1.1824	1.1834	1.1844
70	1.1854	1.1863	1.1873	1.1881	1.1890	1.1898	1.1906	1.1915	1.1923	1.1930
80	1.1938	1.1945	1.1953	1.1959	1.1967	1.1973	1.1980	1.1987	1.1994	1.2001
90	1.2007	1.2013	1.2020	1.2026	1.2032	1.2038	1.2044	1.2049	1.2055	1.2060
100	1.2065									

Tab. 6.3. Standard deviation S_L of the reduced variate

As an example the annual series 1950 - 1974 is evaluated with L = 25

$\bar{X} = \bar{Q} = 112.3$ m³/s	mean value of observations
$S_x = S_a = 58.0$ m³/s	standard deviation of observations
$\bar{Y}_L = 0.5309$	(Tab 6.2.)
$S_L = 1.0915$	(Tab. 6.3.)

The result is summarized in table 6.4 and also shown as a graph on fig. 6.5.

n	50	70	100	return per years
Y_n	3.9019	4.2413	4.6001	
K	3.0884	3.3993	3.7281	
HQ_n	291.4	309.5	328.2	m³/s

Tab. 6.4. Flood flow runoffs according to Gumbel / gauge of Gloggnitz

As the results were obtained from a random sample they are subject to errors. That means that the event with the return period of n years will be within the range of fluctuations

$$X_n + \Delta X \text{ and } X_n - \Delta X$$

The component ΔX depends upon the risk of the statement to be made and how the points are scattered around the regression line (see figure 6.5). The greater the requirement for certainty the bigger is ΔX. Likewise the range of fluctuations increases with the deviations of observations from the compensating straight line.

$$\Delta X = t(\alpha) \cdot S_E = t(\alpha) \cdot \beta_n \cdot \frac{S_x}{\sqrt{L}}$$

$$\beta_n = (1 + 1{,}14 \cdot K + 1{,}1 \cdot K^2)^{\frac{1}{2}} \qquad (3)$$

t(α) reflects the standard normal distribution and some values are given in tab. 6.5 together with the corresponding confidence interval.

	68	8o	9o	95	%
t (α)	1.ooo	1.282	1.645	1.96o	
$\beta_{n=100}$	4.532	4.532	4.532	4.532	
ΔX	52.6	67.3	86.2	1o3.1	m³/s

Tab. 6.5 Range of confidence interval (HQ_{100})

Only when knowing X_n and the range of fluctuations X the flood flow analysis is valid for safe statements. Subsequently the HQ_{100} lies with 90 % probability between 242 and 414 m³/s.

Pearson - 3 - distribution

This distribution includes, in addition to the mean value and the variance, the skewness as a third characteristic parameter describing the function. In those cases where a right-side upper limitation of the distribution arises which may be expected with negative skewness, or if there is a negative left-side limitation, this may be the case with small skewness values a transformation of the distribution is performed so that the lower limit is zero. First for initial data logarithms must be used and then the mean value, the variance and the skewness must be determined as in (4).

$$Z_i = \ln X_i$$

$$\bar{Z} = \Sigma \frac{Z_i}{L}$$

$$S_Z = \left(\Sigma \frac{(Z_i - \bar{Z})^2}{L - 1} \right)^{\frac{1}{2}} \qquad (4)$$

$$g_Z = \frac{\Sigma (Z_i - \bar{Z})^3}{(L-1) \cdot (L-2) \cdot S_Z^3}$$

$$z_n = \bar{z} + K(g_z, n) \cdot S_z$$

$$x_n = e^{z_n}$$

g	Return period in years												
	2	2,5	3	5	10	20	25	40	50	100	200	500	1000
0	0,000	0,253	0,440	0,842	1,282	1,645	1,751	1,960	2,054	2,326	2,576	2,878	3,090
0,1	− 0,017	0,238	0,417	0,836	1,292	1,673	1,785	2,007	2,107	2,400	2,670	3,004	3,233
0,2	− 0,033	0,222	0,403	0,830	1,301	1,700	1,818	2,053	2,159	2,473	2,763	3,118	3,377
0,3	− 0,050	0,205	0,388	0,824	1,309	1,726	1,849	2,098	2,211	2,544	2,856	3,244	3,521
0,4	− 0,066	0,189	0,373	0,816	1,317	1,750	1,830	2,142	2,261	2,615	2,940	3,366	3,666
0,5	− 0,083	0,173	0,358	0,808	1,323	1,774	1,910	2,185	2,311	2,686	3,041	3,488	3,811
0,6	− 0,099	0,156	0,342	0,800	1,328	1,797	1,939	2,227	2,359	2,755	3,132	3,608	3,956
0,7	− 0,116	0,139	0,327	0,790	1,333	1,819	1,967	2,268	2,407	2,824	3,223	3,730	4,100
0,8	− 0,132	0,122	0,310	0,780	1,336	1,639	1,993	2,308	2,453	2,891	3,312	3,850	4,244
0,9	− 0,148	0,105	0,294	0,769	1,339	1,859	2,018	2,346	2,498	2,957	3,401	3,969	4,388
1,0	− 0,164	0,088	0,277	0,758	1,340	1,877	2,043	2,384	2,542	3,022	3,489	4,088	4,531
1,1	− 0,180	0,070	0,270	0,745	1,341	1,894	2,066	2,420	2,585	3,087	3,575	4,206	4,673
1,2	− 0,195	0,053	0,242	0,732	1,340	1,910	2,087	2,455	2,626	3,149	3,661	4,323	4,815
1,3	− 0,210	0,036	0,225	0,719	1,339	1,925	2,108	2,489	2,666	3,122	3,745	4,438	4,955
1,4	− 0,225	0,018	0,207	0,705	1,337	1,938	2,128	2,521	2,706	3,271	3,828	4,553	5,095
1,5	− 0,240	0,001	0,189	0,690	1,333	1,951	2,146	2,552	2,743	3,330	3,910	4,667	5,234
1,6	− 0,254	− 0,016	0,171	0,675	1,329	1,962	2,163	2,582	2,780	3,388	3,990	4,779	5,371
1,7	− 0,268	− 0,033	0,153	0,660	1,324	1,972	2,179	2,611	2,815	3,444	4,069	4,890	5,507
1,8	− 0,282	− 0,050	0,135	0,643	1,318	1,981	2,193	2,638	2,848	3,499	4,147	5,000	5,642
1,9	− 0,294	− 0,067	0,117	0,627	1,310	1,989	2,207	2,664	2,881	3,553	4,223	5,108	5,775
2,0	− 0,307	− 0,084	0,099	0,609	1,302	1,996	2,219	2,689	2,912	3,605	4,298	5,215	5,908
2,1	− 0,319	− 0,100	0,081	0,592	1,293	2,001	2,230	2,172	2,942	3,656	4,372	5,320	6,039
2,2	− 0,330	− 0,116	0,063	0,574	1,284	2,006	2,240	2,735	2,970	3,705	4,444	5,424	6,168
2,3	− 0,341	− 0,131	0,045	0,555	1,273	2,009	2,248	2,755	2,997	3,753	4,515	5,527	6,296
2,4	− 0,351	− 0,147	0,027	0,537	1,262	2,011	2,256	2,775	3,023	3,800	4,584	5,678	6,423
2,5	− 0,360	− 0,161	0,010	0,518	1,250	2,012	2,262	2,793	3,048	3,845	4,652	5,728	6,548
2,6	− 0,369	− 0,175	− 0,007	0,499	1,238	2,013	2,257	2,811	3,071	3,889	4,718	5,827	6,672
2,7	− 0,377	− 0,189	− 0,024	0,480	1,224	2,012	2,272	2,827	3,093	3,832	4,783	5,923	6,794
2,8	− 0,384	− 0,203	− 0,041	0,460	1,210	2,010	2,275	2,841	3,114	3,973	4,847	6,019	6,915
2,9	− 0,390	− 0,215	− 0,057	0,440	1,195	2,007	2,277	2,855	3,134	4,013	4,909	6,113	7,034
3,0	− 0,396	− 0,227	− 0,073	0,420	1,180	2,003	2,279	2,867	3,152	4,051	4,970	6,205	7,152

Tab. 6.6. K-values for the Pearson-3-Distribution

If, however, the skewness is smaller than zero the calculation must be repeated directly with the observations. The first and the last formula is omitted (4). If g_x perhaps is negative again or if this is true for d

$$d = \bar{x} \left(1 - \frac{2 \cdot S_x}{\bar{x} \cdot g_x}\right)$$

so we have to put

$$g_x = \frac{2 \cdot S_x}{\bar{x}}$$

K should at any rate be taken from table 6.6. For the purpose of comparison the results according to Gumbel and those according to Pearson are put down opposite each other.

For the same annual series of the Gloggnitz gauge there is the following:

$$\overline{z} = 4.596$$
$$s_z = 0.516$$
$$g_z = -0.1246$$
$$\overline{x} = 112.3$$
$$g_x = 1.145$$

	50	70	100	return per. years
Gumbel	291.4	309.5	328.2	m³/s
Pearson-3	263.4	278.7	293.0	m³/s

Tab. 6.7. Comparison of the flood flow peak for the gauge of Gloggnitz

In conclusion we may state that, in most cases, the Pearson-3-method compared to the Gumbel method would yield somewhat smaller values. In general, forecasts should not exceed the triple length of the observation period because any further extrapolation is subject to a high degree of uncertainty.

Is there only a short series of observations available ($L < 15$ years) the estimation can be improved by the evaluation of partial series. Instead of using the maximum annual values only, all the values are included now into the computation which are above a given limit and whose total number corresponds to the triple number of years of record. With this method special attention must be given to the necessity of independence among all the data. Within the framework of hydrological evaluations for a hydropower plant in Nepal, presented as a case study in paper No 16, the evaluations which are quoted here led to very practicable, corresponding results although the availability of data had been extremely scarce.

6.3.2. Regional Analysis

If there are no high flow readings from the project site at hand at all, information can be taken from neighbouring catchments. This, however, needs certain mathematical proceedings. In DE COURSEY (1973) we find a detailed analysis for the measuring stations of the state Oklanoma, USA. A subset of catchment characteristics has been extracted from tab. 6.8 by means of a discrimination analysis.

- E catchment area
- I_s slope of the main channel
- l length of the main channel
- E_L percentage of lakes
- H_M average altidue of the main channel above sea level

E_W percentage of forested area

h_n average annual precipitation

i_2 intensity of precipitation for 24^h with n=2

i_{100} intensity of precipitation for 24^h with n=100

h_v average annual evaporation (Class A-Pan)

h_i mean infiltration rate

O orientation for the catchment

Tab. 6.8. Parameter for regional flood flow analysis

The four most effective catchment components are pointed out. In order to take the correlation figure of floods of varying return periods into account, the matrix form is advisable (5).

$$X = A\,Y + B$$

$$\begin{pmatrix} X_2 \\ X_5 \\ X_{10} \\ X_{25} \end{pmatrix} = \begin{pmatrix} a_{11} \cdots\cdots\cdots a_{14} \\ a_{21} \cdots\cdots\cdots a_{24} \\ a_{31} \cdots\cdots\cdots a_{34} \\ a_{41} \cdots\cdots\cdots a_{44} \end{pmatrix} \cdot \begin{pmatrix} E \\ i_2 \\ i_{100} \\ h_v \end{pmatrix} + \begin{pmatrix} b_1 \\ b_2 \\ b_3 \\ b_4 \end{pmatrix} \qquad (5)$$

When evaluating the runoff measuring stations of Oklahoma three different A- and B matrices were sufficient for an adequate description of all observations. BUCK (1979) gives an example of a regional high flow analysis by means of reduced runoff values for the area of the river Neckar in Germany (FRG). For small catchments with less than 350 km², these are especially suited for the construction of small hydropower plants, there are hardly ever records available. An extensive investigation of HALASI KUN (1968, 1972) confirms the assumption that the rare high flows, so f.e. the HQ_{100}, are determined in 90 - 95% of the cases by the geological conditions of the catchment and the annual rainfall coefficient. The formulas

$$HQ_{100} = C_2 \cdot E^{C_3} \cdot E = C_2 \cdot E^{C_4}$$

are sufficient for a rough estimate. The special runoffs for Central Europe, especially for the CSSR, were computed for varying geological formations and varying annual precipitation coefficients. The accuracy of this method is stated by the author with ±4 - 7%.

Finally different empirical approaches, the ones of HOFBAUER, ISZKOWSKI, KREPS, HOFMANN and COUTAGNE, and of RÖSSERT (1969) presented in formal comparison are still in use. For some river areas applicable methods can be obtained on the basis of those methods, when they are applied to areas with different climates, however, great precaution is recommended. The suitable application of the rational method, an empirical approach as well, which establishes a relationship between high flow peak HQ

$$HQ = C_5 \cdot i\,(T_c) \cdot E$$

and catchment area E, intensity of precipitation i within the time period T_c and the parameter C_5 was confirmed for smaller catchments in Central-America by BASSO (1973).

6.3.3 Precipitation-Runoff-Schemes

If the observations available are subject to great uncertainties and if the multisite development of a larger-sized river system is considered, an intensive short termed observation of precipitation and runoff within the catchment is advisable. Doubtlessly, the additional costs for improved measuring facilities will be balanced by more reliable design criteria.
A simple black-box model such as the Unit Hydrograph permits the mathematical transformation of the rainfall excess into the flood hydrograph. With simultaneous observation of the distributed precipitation within the catchment and the flood hydrographs at several gauging stations along the river network the Unit Hydrograph curves can be determined by the resolution of a linear equation system. For the separation of the base flow as well as for the splitting of the precipitation into the excess and the losses due to infiltration phenomena confirmed schemes are readily available. The hydrograph can be derived from observations which should be recorded over several years. By including various rainfall events and through variation of the catchment parameters as well as the concentration time, rate of infiltration and storage values of the catchment a variety of flood hydrographs can be generated.

A comparison of peak values which have been obtained through statistical evaluations as well as by means of Unit Hydrograph is shown for a prealpine area in Upper-Austria on figure 6.7. The statistical evaluations are based upon a 16 years series, while for the evaluation by way of a precipitation-runoff-model the readings from a period of four years were used.

6.4 LOW FLOW ANALYSIS

A second important design criteria, f.e. for the estimation of the compensation water, is the available runoff during low flow periods. It is characterized by several factors such as the amount of runoff, the probability of deficit in supply, time of appearance, length of deficit, of a certain runoff limit and limit of deficit. A hydrograph with the characteristics is shown in figure 6.8.

6.4.1 Analysis of Low Flows

The reasons for low flows are natural influences (little precipitation, high degree of evaporation or frost) and human activities (reservoirs, take-offs). The rarity of occurrence of low flows should be dealt with in the same way as high flow probabilities.

Frequently used distributions are the

Weibull distribution
Log Normal distribution
Kaczmarek distribution

A comparison of the different methods with additional consideration of other distributions has been made by KRAUSNEKER (1975).

Following the same observation period as for the high flood events the computing as to Weibull and Kaczmarek is shown below.

YEAR	NQ (m^3/s)	YEAR	NQ (m^3/s)	YEAR	NQ (m^3/s)
1950	3.5	1960	2.40	1970	3.76
51	2.2	61	2.10	71	2.00
52	2.55	62	3.40	72	1.40
53	2.20	63	2.14	73	1.87
54	1.90	64	1.78	74	1.37
55	1.65	65	3.54		
56	2.93	66	2.93		
57	2.40	67	2.95		
58	2.40	68	1.63		
59	2.40	69	2.55		

Table 6.9 Annual minima for the gauge at Gloggnitz

Weibull distribution

The three-parameter distribution with the parameters θ, α and ε (minimum low flow) is frequently used with good adaption for low flow analysis.

$$F(x) = e^{-\left(\frac{x - \varepsilon}{\theta - \varepsilon}\right)^{\alpha}}$$

$$x = \varepsilon + (\theta - \varepsilon)\, e^{\alpha \cdot y_n}$$

$$t_1 = \frac{\bar{x} - x_1}{s_x} \qquad (6)$$

$$\varepsilon = x_1 - \frac{t_1 \cdot s_x}{L^{\alpha} - 1}$$

$$\theta = \frac{\bar{x} - \varepsilon}{\Gamma(1 + \alpha)}$$

Concluded from observations

$$L = 25 \quad \text{(years)}$$
$$x = 2.40 \quad (m^3/s)$$
$$s_x = 0.67 \quad (m^3/s)$$
$$x_1 = 1.37 \quad (m^3/s)$$

Taken from fig. 6.9 with the knowledge of t_1 and L derives exponent α. The complete Γ-function is listed in many mathematical reference books and is shown in excerpts in tab. 6.10.

α	Γ(α)	α	Γ(α)	α	Γ(α)	α	Γ(α)
	0,		0,		0,		0,
1,01	9943	1,26	9044	1,51	8866	1,76	9214
1,02	9888	1,27	9025	1,52	8870	1,77	9238
1,03	9835	1,28	9007	1,53	8876	1,78	9262
1,04	9784	1,29	8990	1,54	8882	1,79	9288
1,05	9735	1,30	8975	1,55	8889	1,80	9314
1,06	9687	1,31	3960	1,56	8896	1,81	9341
1,07	9642	1,32	8946	1,57	8905	1,82	9368
1,08	8597	1,33	8934	1,58	8914	1,83	9397
1,09	9556	1,34	8922	1,59	8924	1,84	9426
1,10	9514	1,35	8912	1,60	8935	1,85	9456
1,11	9474	1,36	8902	1,61	8947	1,86	9487
1,12	9436	1,37	8893	1,62	8959	1,87	9518
1,13	9399	1,38	8885	1,63	8972	1,88	9551
1,14	9364	1,39	8879	1,64	8986	1,89	9584
1,15	8330	1,40	8873	1,65	9001	1,90	9618
1,16	9298	1,41	8868	1,66	9017	1,91	9652
1,17	9267	1,42	8864	1,67	9033	1,92	9688
1,18	9237	1,43	8860	1,68	9050	1,93	9724
1,19	9209	1,44	8858	1,69	9068	1,94	9761
1,10	9182	1,45	8857	1,70	905[illegible]	1,95	9799
1,21	9156	1,46	8856	1,71	9106	1,96	9837
1,22	9131	1,47	8856	1,72	9126	1,97	9877
1,23	9108	1,48	8857	1,73	9147	1,98	9017
1,24	9085	1,49	8859	1,74	9188	1,99	9958
1,25	9064	1,50	8862	1,75	9191	2,00	·0000

Tab. 6.10 Gamma function

ε = 1.11 m^3/s (minimal low flow)

θ = 1.45 m^3/s (characteristic low flow)

On the basis of (6) there has to be computed for various return periods n the low flow runoff which is being compared with the results according to Kaczmarek in tab. 6.12.

Distribution according to Kaczmarek

The distribution suggested by Kaczmarek starts with

$$NQ_n = \bar{X} + A \, . \, S_x$$

and with the use of tab 6.11 it can be easily applied. Furthermore, the confidence interval ΔX is included in tab. 6.13.

$$\Delta X = t\,(\alpha)\,.\,B\,.\,\frac{S_x}{\sqrt{L}}$$

p	0.0	0.5	1.0	2.0	5.0	10.0	20.0	50.0	80.0	90.0	99.0
y	0.000	0.071	0.100	0.142	0.226	0.325	0.472	0.833	1.268	1.517	2.146
A	-1.91	-1.76	-1.70	-1.61	-1.42	-1.21	-0.89	-0.11	0.82	1.36	2.72
B	1.355	1.272	1.240	1.195	1.111	1.027	0.939	0.967	1.379	1.702	2.618

Tab. 6.11 Constant A and B for the Kaczmarek distribution

The minimum limit which is equivalent to the preceding statements corresponds to p = 0.0 and according to that amounts to

$$N\ Qmin = 1.117\ m^3/s$$

n	20	50	70	100	return per. year
Weibull	1.21	1.16	1.15	1.14	m^3/s
Kaczm.	1.45	1.31	1.29	1.26	m^3/s

Tab. 6.12 Low flow runoffs for the gauge of Gloggnitz

α	68	80	90	95	
$t(\alpha)$	1.00	1.282	1.645	1.960	
B	1.160	1.379	1.702	2.014	
Δx	0.155	0.236	0.375	0.529	m^3/s

Tab. 6.13 Confidence intervals according to Kaczmarek

6.4.2 Regional Analysis

The transposition of local observations to a runoff region has been carried out for different river areas in Europe, f.e. by GLOS et al. (1972) for GDR and by WIDMOSER (1974) for forty small catchments in Switzerland. The results of the latter are based on relations between the parameters A and B and the catchment components such as

- E catchment area
- $\Delta 1$ separation of the gauge from the point of gravity
- H_F medium height of altitude of the catchment area
- 1 main channel length

$$NQ_n = A - B\ .\ Y_n$$

$$A = 0.04157\ .\ \frac{E^{1.378}}{\Delta 1^{0.71}\ .\ H_F^{\ 0.321}}$$

$$B = 0.12481\ .\ \frac{1^{0.3007}\ .\ E^{0.7521}}{H_F^{\ 0.5901}}$$

In most cases the basic variables E, $\Delta 1$ etc. may be easily determined. The exponents must be adapted to other regions and can be obtained by means of optimization. This method is based upon the investigation of catchment areas ranging between o.6 and 6oo km^2. Under the condition of a new estimation of the exponents a transposition to other regions may be recomended. The empirical formulas developed for selected river basins by HOFBAUER, SALCHER and ISZKOWSKY are rather limited in application and in transposition too.

Low flow periods

Until now the annual minima were selected from a continuous runoff series and were evaluated. For hydrological and economical utilization, together with the runoffs, also the length of low flow periods is of importance. Therefore, for design purposes often a low flow of a given duration and return period is chosen.

For example,in the U.S. the 10 years flows of a seven days period serve as a basis for designing. With the evaluation,instead of the annual low flow the mean value of the lowest 7-day-period is used. SINGH et al. (1974) investigated topographical, climatic and human activities influencing the low flow discharge of different catchments in Illinois, U.S.

If we want now to determine the low flow runoff for various periods t_p and their probabilities of occurrence, this will formally correspond to the statement

$$NQ_n = F\ (t_p,\ n)$$

which signifies a two-dimensional probability distribution. About this, a problem of great practical significance, there exist only very few studies which have taken up the subject. An empirical evaluation for an Upper-Austrian catchment area already mentioned can be found on figure 6.10. With the help of a diagram for a stated probability of occurence or return period and with a defined length of days the resulting low flow volume can be estimated.

6.5 RUN-OFF SERIES

Together with the distribution functions of high flow and low flow events which serve as a basis for the design of a plant, attention must be given to the time distribution of the runoff when operating small hydropower plants (KWKW). Furthermore the frequency distribution of the daily runoffs, presented if possible with the duration curve (figure 6.11.) furnishes fundamental principles for the installed discharge.

6.5.1 Duration curves for gauging stations

The presentation of a duration line can be easily carried out and is given here only for the sake of completion. From the series of daily runoffs,step by step,for different discharges,the number of days on which a certain runoff is exceeded are counted.

The graphical display of the discharge versus the number of exceedings represents the duration curve, the upper and lower limits are given by the respective runoff maximum and the low flow.

In table 6.14 the duration figures of four years of records may serve as an example (see alsofigure 6.11)

Q	1971	1972	1973	1974	1971-74
200	0	0	0	0	0
150	0	0	0	0	0
100	0	1	0	1	0.5
80	0	3	0	1	1.0
60	0	3	1	2	1.5
40	0	6	6	7	4.75
20	4	35	28	25	23
10	60	124	76	103	90.75
7.5	126	188	121	201	159
5.0	155	289	255	291	247.5
2.5	365	366	365	365	365.2

Tab. 6.14 Duration figures for the gauge of Gloggnitz

From this short evaluation one can clearly recognize the dependence per year. Therefore if possible the duration curve is to be related to a series of readings of several years. The fluctuations of the runoff duration line of a gauge at the river Main have been plotted for 50 years of readings in figure 6.12.

An interesting analytical presentation of the runoff duration curve can be done with consideration of the daily runoffs according to a Log-Normal distribution. For calculating this function three parameters are necessary which can be obtained from the observations.

6.5.2 Regional Analysis

In the case of ungauged catchment-areas information can be furnished through transposition from comparable catchments. This procedure has been performed in regard to a Central European area in the CSSR by NEMEC (1972).

A great number of observed catchments were investigated in order to determine the time distribution of the annual runoff over periods of months and days. The results show that as to monthly runoff values five typical regions can be detected. As to daily runoffs seven duration curves are enough to represent sufficiently all the areas. When setting up these groups the size of catchments and meteorological conditions were taken into account. Instead of runoff values the figures of the relations between the limit of exceedances and the annual mean are given.

If for a non-observed area the duration curve is to be plotted, it is enough to relate to one of the groups and to know the annual mean of the runoff which f.e. can be calculated with regional or empirical procedures. Another possibility can be persued with a short-term observation of the runoff event in order to compare these scarce measuring values with other gauge stations through regression and correlation.

In case of positive results the transposition of a reshaped duration curve is furthermore again possible.

6.6 ANALYSIS OF DESIGN CRITERIA

The basic hydrological values so far discussed allow a preliminary design. Due to the fact that these values are based upon statistical rules - and this holds for all hydrological variables - the design criteria will exhibit a certain deg degree of probability. Each non- performance is connected with financial losses therefore great attention and care must be given to the design of a plant.In th the following steps some guidelines are briefly described using again the catchment of river Schwarza.

6.6.1 Evaluation of project sites.

Based on a longitudinal profile of the river which exhibits the mean discharge and the slope of the river bed a preliminary evaluation of project sites is possible. If no other restrictions are to be taken into account, the section I presented in figure 6.1 is to be preferred for an economic utilization of hydropower. The accompanying raw energy potential reflects an important item for the annual energy output. However, one has to consider that the plants installed utilize the periodically varying yield of water no more than up to the rated discharge.
Furthermore, during low flow periods the dependence of the efficiency upon the discharge is to be considered. According to that the available annual energy output will be proportionally smaller.

6.6.2 Rated discharge

The determination of the rated discharge is elaborated on the basis of the runoff duration curve and the demand of energy. In former times priority was given to an underdesign of small hydropower plants to supply a local grid with firm energy instead of maximazing the generated output.

At present the possiblity exists to cover the demand with a parallel operation connected to a grid or with supplementary energy generation such as thermal plants. Therefore the dimensioning is being enlarged to the 180 - 200 daily runoff or to the mean flow.

Moreover, improved turbine technology allows a greater variation of the discharge without loss of efficiency. Together with the duration curve of this river which shows very distinctly a flat middle section the increase of the rated discharge to the 100 day flow is to be considered. A decision can be made on basis of the chosen efficiency curve of the turbine.

6.6.3 Type of Plants

The river bed and the form of valley in the chosen river section which shows a relative flat channel allow to accept raw heads of 5 - 8m for the plants. The rated discharge of approximately 8 - 1o m^3/s should be preferably utilized by at least two turbines, whereas only one should be completely adjustable.

The question whether a diversion-type plant or a run-of-river plant is to be preferred can only be answered dependent on the cost structure, the risk and the ecological requirements.

Ecological considerations reject the installation of diversion-type plants as with that long river sections would loose its characteristic appearance owing to drastically reduced water volume and flora and fauna of the surroundings would be negatively influenced.

The structure of costs and other risks are in close relationship with each other as risks can only be reduced with a rise in costs at the same time. Without dealing here with the distribution of costs when setting up small-scale hydropower plants, which will be done in other papers,we can say that an essential rise in costs results from precautionary measures against floods.

6.6.4 Precautions against Floods

Together with an example we want to demonstrate the choice of a design flood for the operation- and for the construction period.

For a small hydropower plant during a period of L= 2o years the risk of damage or destruction through exceedance of the design flood should be smaller 10 %. The determination of the respective design flood can be done arithmetically or be taken from table 6.15.

For the not too far off gauge of Gloggnitz there exists a flood statistics so that with the statement of a return period n, HQ_n is immediately given. The return period n results from

$$W_A^{20} = 1 - (1 - \frac{1}{n})^{20} = 0.10$$

$$n = 190$$

If the design is done in view of the 19o years flood the damage probabilty is 10 % for the given time period. An increase of the risk to 20 % permits a design to HQ_{9o}. The corresponding values for the Gloggnitz gauge have been taken from figure 6.5.

$$HQ_{90} = 323\ m^3/s$$

$$HQ_{190} = 363\ m^3/s$$

R	L 2	10	50	100
0.01	198	996	4975	9953
0.10	20	95	475	950
0.50	3.4	15	73	145
0.75	2	7	36	73

Tab. 6.15 Recurrence interval for risk R and period L

We have to proceed likewise if the safety of the project site during the construction time T_B should be estimated.

Together with the risk which presumes an exact knowledge of the distribution, the uncertainty with the estimation of the distribution is to be taken into account, which is expressed in the calculated interval of fluctuations.

$$\Delta X = 66\ m^3/s \quad \text{for } 80\% \text{ and } n = 90$$

$$\Delta X = 95\ m^3/s \quad \text{for } 90\% \text{ and } n = 190$$

Summarizing one may state that an inhomogeneous runoff behaviour, reflected in the HQ_{10}/NQ_{10} or HQ_n/MQ-relation, is connected with higher costs for precautionary measures to prevent flood damages so that a diversion is recommendale in finacial terms.

Similar considerations are true for the great uncertainties in flood estimations which can be due to short periods of observation ore incorrect observations.

6.6.5 Low Flows

Low flow amounts and periods are important for the supply of remaining water ways as well as the installation of balancing thermal plants to ensure a "firm" supply.

An exact statement of the minimum volume in form of NQ_n is not possible as other factors such as the quality and the temperature of the water are to be taken into account. The possibility of the utilization of small reservoirs within the catchment for small-scale hydropower plants should also be mentioned here. This is of special importance with plants on the local grid as an adaption to the load even through the smallest reservoir (f.ex. 6h-reservoirs to cover daily fluctuations) is economically interesting. The design of reservoirs can be done with the help of a long-term discharge hydrograph and the rating curve of load.

With these presentations the most important basic design data are briefly described. The methods, even if scattered here and there, are described in detail in the literature. An important chapter, that is, about sediment transport, had to be omitted because of lack of time and space.

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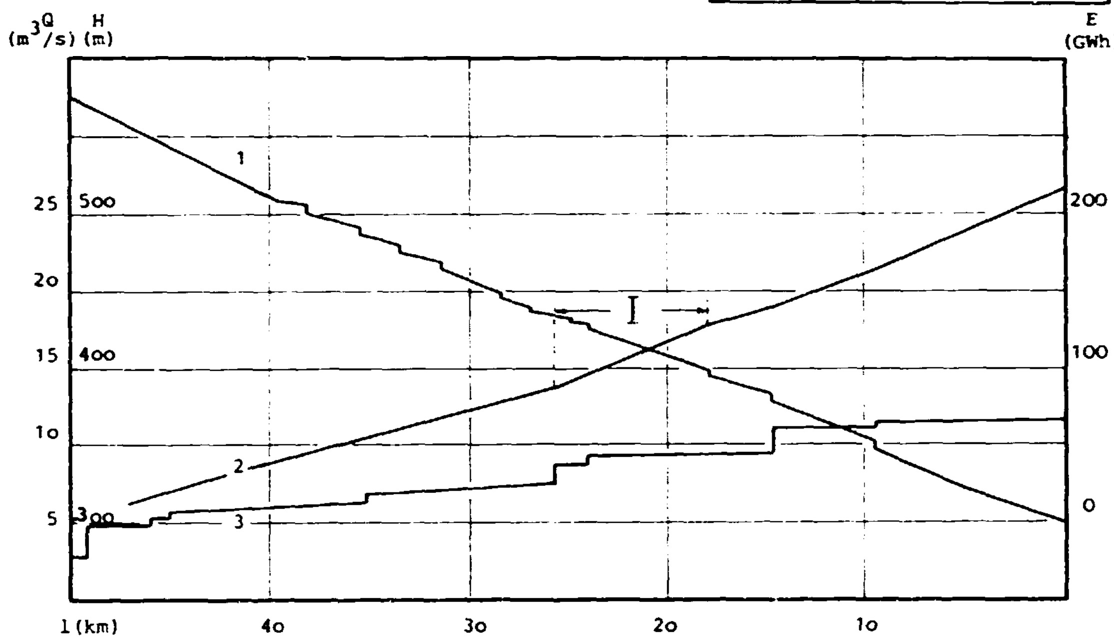
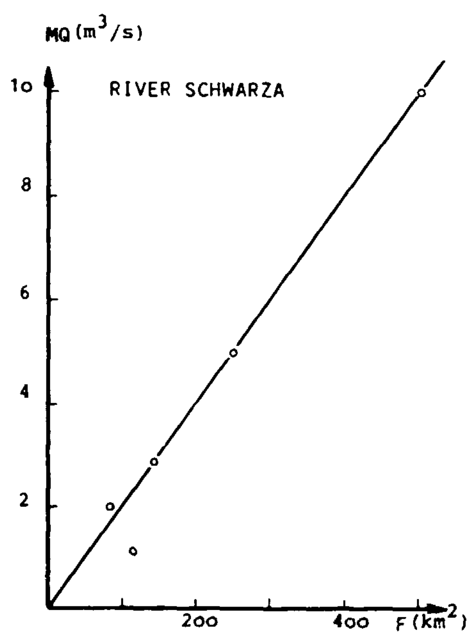


1 ELEVATION OF RIVER BED
FLUSSOHLE

2 RAW ENERGY POTENTIAL
ROHENERGIEPOTENTIAL

3 MEAN ANNUAL DISCHARGE
MITTELWASSER

FIG 6 - 1 LONGITUDINAL SECTION OF RIVER SCHWARZA
LÄNGENSCHNITT DER SCHWARZA



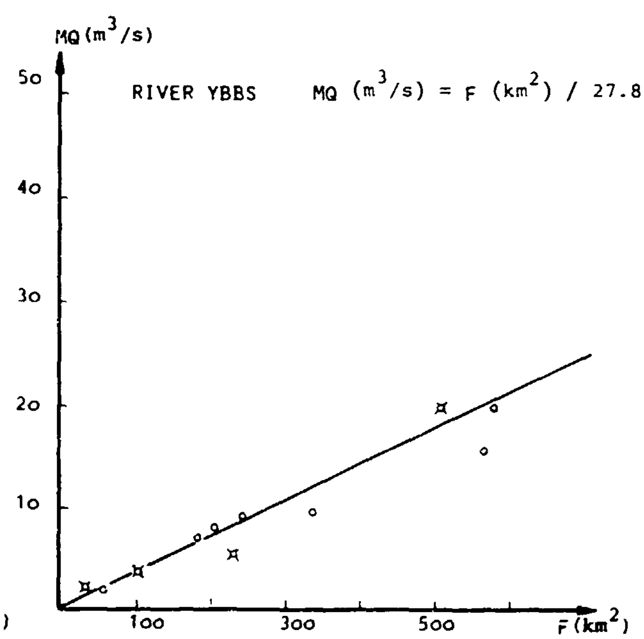


FIG 6 - 2 MEAN DISCHARGE VS. CATCHMENT AREA FIG 6 - 3
MITTELWASSER U. EINZUGSGEB. FLÄCHE

6 HYDROLOGICAL ANALYSIS
HYDROLOGISCHE AUSWERTUNGEN

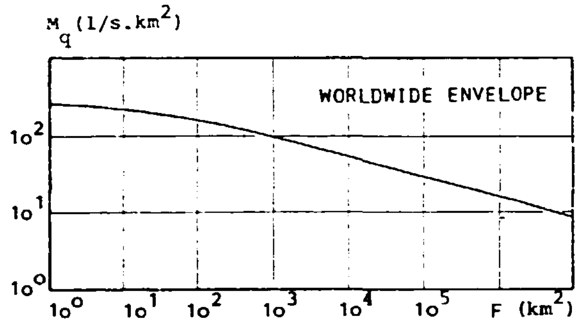


FIG 6 - 4 SPECIFIC RUNOFF
SPEZ. ABFLUSS

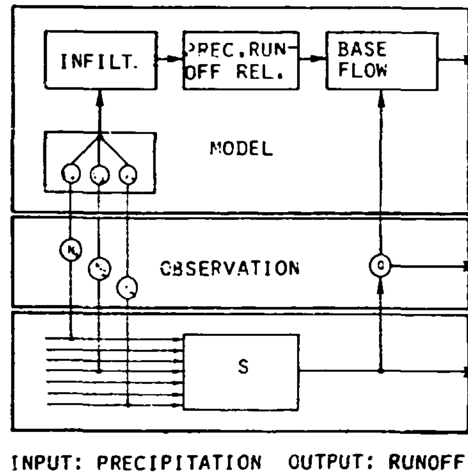


FIG 6 - 6 PRECIPITATION-RUNOFF PROCESS
NIEDERSCHLAGS-ABFLUSSPROZESS

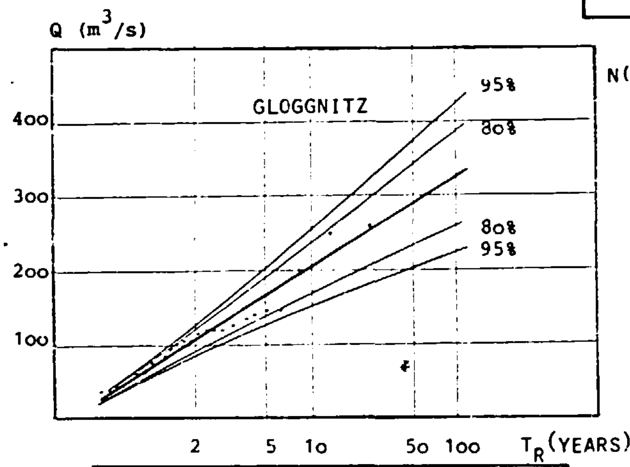
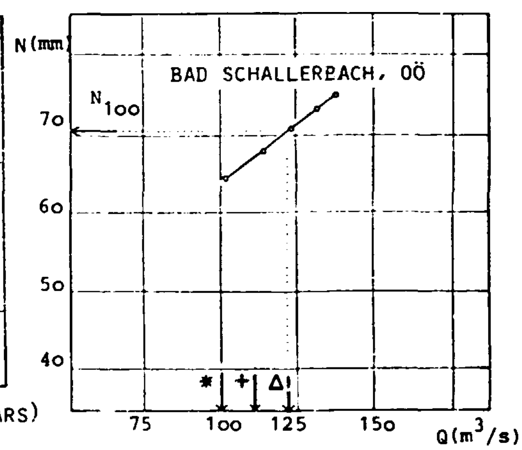


FIG 6 - 5 FLOOD FREQUENCY ANALYSIS
HOCHWASSERSTATISTIK



HQ_{100}: GUMBEL ✱ 100 (m³/s)
PEARSON + 109 (m³/s)
UNIT HYDRO GRAPH Δ 120 (m³/s)

FIG 6 - 7 FLOOD PEAK COMPUTATIONS
HOCHWASSERSCHEITEL BERECHNUNG

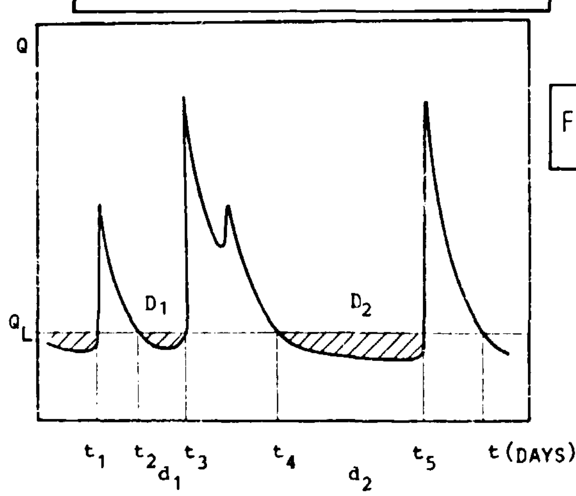


FIG 6 - 8 LOW FLOW ANALYSIS
NIEDERWASSERANALYSE

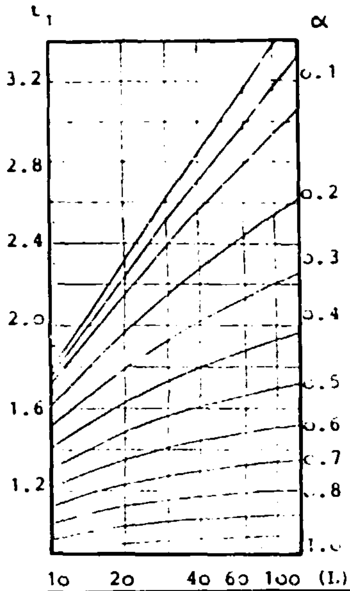


FIG 6 - 9 ESTIMATION OF α
SCHÄTZUNG VON α

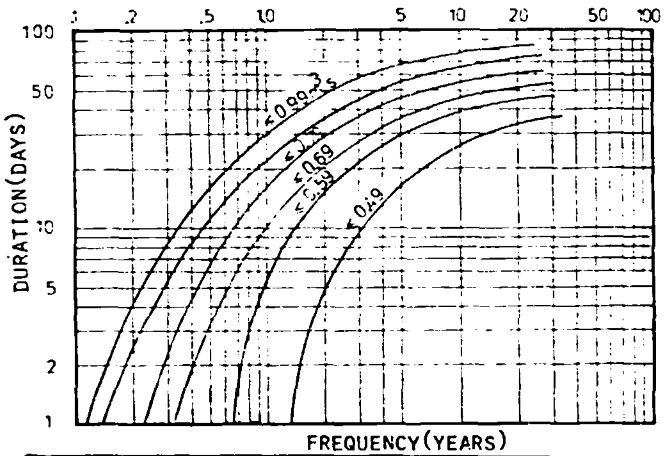


FIG 6 - 10 DURATION AND FREQUENCY OF LOW FLOWS
DAUER UND HÄUFIGKEIT VON NIEDERWASSERN

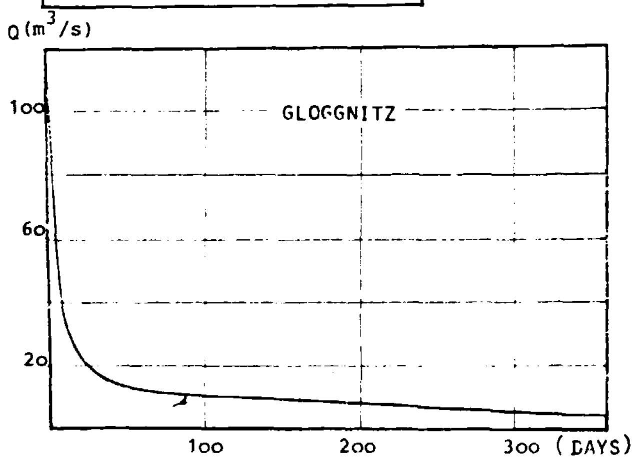
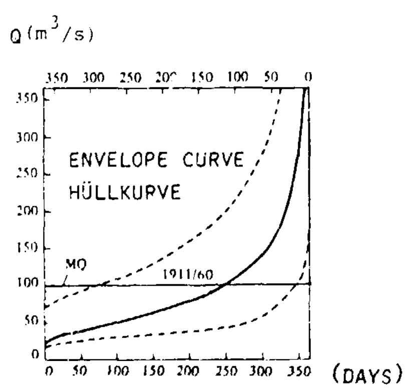


FIG 6 - 11 DURATION CURVE
DAUERLINIE



DURATION CURVES OF RIVER MAIN (1911-1960)
DAUERLINIEN DES MAIN (1911-1960)

GAUGE SCHWEINFURT
PEGEL SCHWEINFURT

FIG 6 - 12 RANGE OF DURATION CURVES
BEREICH DER DAUERLINIEN

7 WEIR INSTALLATIONS AND WEIR GATES IN LOW-HEAD POWER SCHEMES

SIMMLER H.

ABSTRACT:

This Paper discusses weirs suited for mini hydro stations. Dam builders differentiate between fixed-crest structures, which are simple in design and, under certain conditions, can even be used for diversion-type power stations, and gate-controlled structures or barrages, which offer a great variety of design and construction possibilities. In view of their use in hydro power, the discussion should include location problems resulting from bed load transport and aspects of fluid mechanics. A thorough analysis is made of the gate systems used in the construction of mini hydro stations, and of their function and operation in order to enable appraisal of different gate types and their applications. Finally, a weir type is described which has been increasingly used over the last few years, i.e. the inflatable dam. Compared with the conventional gate structures, this is low in cost and, according to the experience gathered so far, represents a safe, automatically controlled gate system.

7.1 GENERAL

Low dams, or weirs, in connexion with the development of hydro power serve to impound and divert turbine supply water, or to gain head in the case of combined weir-powerhouse structures. The structural design of weirs is primarily governed by the required water level, the control of flood discharge and, in some cases, the control of bed load transport. These parameter bear on the fluid mechanics situation and on the design of the dam. Optimisation of this installation in the hydraulic, structural and economic respects is the aim of all project studies. Safety will be a prime consideration, which will have to include the trouble-free utilisation of power water, the safe discharge of flood flow and a safe foundation of the dam.

The international literature differentiates between fixed-crest weirs and movable weirs, or barrages. The fixed-crest weir is safe in operation, simple in design and generally maintenance-free, as it has no movable gates. It needs less operation and is low in initial cost.

Its disadvantages, however, result from the flow-dependence of the water level. This may be of no consequence in narrow virgin gorge sections of backwater areas. However, a fluctuating headwater level may present problems in the case of a diversion. The structures have to be adapted to this range of fluctuation, and for the limitation of the supply water drawn in, a side weir will in general be required to return excess flow into the old stream bed. Many weirs of this type exist in practice, as they are simple and low in construction cost. But in every individual case, provisions will have to be made for the discharge of possible bed load by adequate means, a problem that is not easy to solve.

In dam power stations, variable headwater levels influence turbine output, and the height of the structure must be adapted to the maximum headwater levels, which involves additional cost. Therefore, fixed-crest weirs will very often be combined with a gate-controlled weir opening or with a bottom outlet.

By contrast, a barrage is equipped with a gate on the solid dam structure, which not only prevents the water surface from rising beyond top water level but also enables a certain measure of water level control. Savings in gate cost can be made if the water level is allowed to rise beyond the top water level during the discharge of large flood flows over the weir. Diversion-type power schemes will best be shut down under such circumstances.

7.1.1 Fixed-Crest Weir

There are a great number of different structural designs for fixed-crest weirs. These must be adapted to the local conditions and differ substantially with respect to hydraulic effect, stability and cost of maintenance. Figure 7-1 shows the simplest design consisting of a sheet pile cut-off stabilised by rock fill extending across the stream in order to improve the flow over the weir and to ensure a certain measure of protection against downstream erosion. The same weir type can also be combined with gabions (STEPHENSON, 1980; SCHOKLITSCH, 1950) (fig. 7-2).

The height to which the headwater level can be raised by such weirs is limited, as the difficulties of energy dissipation grow along with increasing height, which in turn jeopardises the stability of the rock fill or gabions. Flushing of the backwater area to remove deposited sand or gravel is naturally not possible. Therefore, it may in some cases be necessary to clear the intake area from time to time. Such solutions are simple and low in cost. However, especially where maintenance is inadequate, it will be impossible to prevent bed load from entering the headrace and being deposited there, so as to cause trouble in operation. Consequently, such simple weir types offer limited applications in practice and their use should be considered only for very unfavourable sites and for the construction of small and simple hydro stations.

From the fluid mechanics point of view, every weir should be located so as to ensure that the water is drawn in free from bedload approximately in the last one-third of a river bend (fig. 7-3) or so as to bring about a flow bend by relocating the stream bed or by providing groynes in the headwater (fig. 7-4). Investigations performed at the Institut für Konstruktiven Wasserbau of the Graz University of Technology have proved control of bedload transport to be feasible even in fixed-crest weirs (fig.7-5) provided that the location of the total structure is favourable from the fluid mechanics point of view and that provision is made for sufficient flushing. Observations made on hydraulic models as well as subsequent experience during operation have shown that during flood flow bedload is carried past the inlet sill and over the - higher - top of the fixed-crest weir or is deposited in the area behind the groyne.

Simple weir types will naturally not satisfy the requirements of dam power stations as these need higher headwater levels. Only concrete barrages can here be considered. A sufficiently large bottom outlet or a weir opening equipped with a movable gate must be provided to ensure the discharge of bedload to the tailwater.

Greater stability is afforded by the classical crib dam, which is constructed relatively quickly and at low cost by skilful carpenters in regions rich in timber (fig. 7-6). A barrier is constructed of round or squared timber and filled with stones. Unless bedrock is exposed, the whole structure is placed on wooden piles. The crest is covered with wooden boards. In any case care should be taken to ensure adequate imperviousness at the upstream face to avoid underseepage and base failure in the foundation.

These designs already allow higher water levels. However, greater difficulties are presented by the problem of energy dissipation in the tailwater. Even rock is not always stable enough to withstand the persistant action of a dropping nappe without suffering deep erosion that may even extend to the dam foundation itself. Subsequent remedial measures are costly and may not be effective enough. It has proved useful to make provisions for adequate scour prevention from the very beginning. A protective board slitted at both ends can be installed immediately next to the crib. This is usually sufficient where bedload is carried over the weir. In any case, adequate tailwater depth should be provided in accordance with the hydraulic requirements for producing a hydraulic jump. Care is, however, indicated where a timber structure is used because, unless continuously submerged, its service life would be limited.

Greatest safety and stability is doubtlessly afforded by dams consisting of a massive concrete structure (fig. 7-1) with a crest and downstream face that should be designed to correspond approximately to a pressure-free profile (CORPS OF ENGINEERS). Subatmospheric pressures up to about 0.2 bars are, however, permissible in rare operating cases, during major floods. Adoption of a pressure-free profile enables the calculation of discharge over the weir using a high discharge coefficient. The free nappe weir (fig. 7-8), which is lower in cost and simpler in design, is likewise used in practice. As its structural length is particularly short, potential underseepage and the resulting risk of base failure should be investigated thoroughly where a solid foundation allows the omission of a stilling basin. In addition, the crest, which is usually straight in plan, is subject to negative pressures when overflown. Therefore, stone facings need adequate anchoring.

The most suitable means of energy dissipation is the classical concrete stilling basin (A.J. PETERKA, 1964), which should at least be equipped with an end sill to shorten and stabilise the hydraulic jump. Baffles in the stilling

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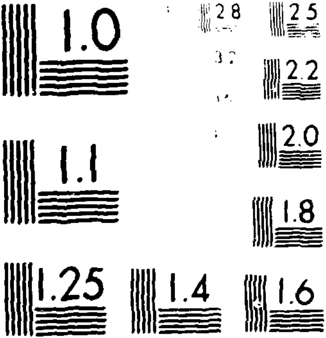

MICROCOPY RESOLUTION TEST CHART

basin, although having a favourable hydraulic effect, are themselves very vulnerable to erosion and cavitation. Prior to adopting such baffles, their future performance should be studied carefully with allowance for the expected frequency of their coming into action. Where bedload transport over the weir attains great magnitudes linings of resistant material (steel, wood, rails) on zones subject to major attack will be useful.

Besides the plane stilling basin, the trough-type stilling basin, although somewhat expensive due to its shape, is used successfully. The advantage of this stilling basin type mainly results from the fact that the water jet makes a flatter angle with the tailwater. The stepped stilling basin, with steps in the bottom towards the tailwater, has proved effective where hydraulic reasons call for a large water depth in the stilling basin.

7.2 BARRAGE

The barrage, or movable weir, (F. PRESS, 1959; F. HARTUNG, 1970) represents a combination between a solid dam structure of concrete and a movable gate, which allows a large measure of headwater control. In mini hydro power, a single gate will be sufficient in most cases. Division into several weir openings appears to be of advantage only where this enables savings in gate cost or where required by safety considerations, as e.g. safety from overtopping when one gate jams. It should, however, be borne in mind that provision of more than one gate drive implies higher cost. Division of the weir into several openings might perhaps be justified where wooden gate leaves can be used as these require smaller spans and can be manufactured at low cost. The gates would in this case be driven by hand or by a motor through shafts. According to the principles of modern construction techniques, however, steel gates are used for plants of moderate size. These differ by their functions and static actions:

Gates hinged to the dam	: flap
Revolving gates	: compression-type radial gate (arms under pressure)
	tension-type radial gate (arms under tension)
Gates carried between piers:	slide gate,
	fixed wheel gate

The flap (fig. 7-9) permits an almost unlimited length of dam without intermediate pier. The height of the flap should, however, range between about 1 m and 3.50 m to allow economical design. Flaps with heights exceeding 3.50 m involve a markedly rising drive cost as the moments increase by the triple power of the hydrostatic head acting on the gate. Also, the hydraulics of the jet delivered to the stilling basin becomes less favourable (fig. 7-9). In cases of doubt, it will be preferable to construct a high solid dam and to choose a low-height flap, whose geometry is governed by the available width of the weir and the magnitude of the maximum flood to be discharged, the allowable surcharge head during floods as well as the passage of floating debris and the compensation of a surge caused by the shutting of the turbine gates. Experience gained in model tests for weirs equipped with flaps has taught that unfavourable and shooting flow towards a weir with a low crest may involve problems due to extremely unsymmetrical loading. If in addition the tailwater level in the stilling basin is very high, it may be impossible to accomplish a stable gate position. It has been found out that a weir structure of sufficient height and improved conditions of flow towards the structure restore the necessary stability.

The manner in which the radial gate affects weir design and operation is different. As the discharge cross section is opened by rotating the gate upwards, different hydraulic flow characteristics prevail (fig. 7-10). Precise headwater control is much more difficult than in the case of a flap. On the other hand, however, lifting of the gate produces a bottom current, which allows flushing of the area immediately in front of the weir to remove mud, sand or bed load. The bed load glides over the weir without touching the gate. In the case of a flap, bed load is discharged not only over its back but also over the bottom seal. Therefore, the radial gate is suited not only as weir gate but mainly for the bottom outlet.

Normally the radial gate is of the compression type, which presents no problems in mini hydro power. The tension-type radial gate with pivot bearings in the headwater zone, in turn, involves a more favourable loading of the piers by compression forces and, moreover, allows a steeper downstream face owing to the position of the tension-type radial gate on the weir sill (fig. 7-11). This in turn allows savings - although minor - in structural length.

The gate types carried between piers include the slide gate and the fixed wheel gate. They consist of a simple panel for which even wooden structures of planks are common where smaller sizes are concerned. Both gate types require grooves in the abutments as gate guides. Slide gates, having friction coefficients of between 0.3 and 0.5, are limited as to their feasible dimensions. With increasing span and height, the driving power requirements soon become prohibitive so that fixed wheel gates, whose friction forces range around 0.1, must be used for major water loads. However, as the manufacture of the latter is fairly complex, this type of solution should be avoided where possible.

Slide and fixed wheel gates are better suited for cross sections of greater height than width. However, for complete opening, the lifting mechanisms have to be located so as to allow raising of the whole gate leaf clear of the discharge cross section, which in turn calls for a high enough location of these mechanisms.

Consequently, selection of the suitable weir gate type should make allowance for the following aspects:
Geometry of the cross section to be closed by the gate, headwater control possibilities, discharge of bed load or bed load removal by flushing, discharge

of floating debris, dimensions and vulnerability of impervious elements, drive mechanisms (fig. 7-12).

The design of barrages is governed not only by the local situation and the operating conditions, but primarily also by the required headwater level and the type of gate. We can differentiate between low and high barrages. In low barrages, the impounding is mainly effected by the movable gate. In such cases, provision of a sill of adequate design above the horizontal upstream floor (fig. 7-13) may often prove of advantage as this allows savings in gate height. High barrages will in most cases consist of a structure of economical design, with any shape being admissible as long as it ensures sufficient stability.

A thorough analysis is required to determine potential underseepage and to avoid the risk of base failure. In fact, engineers often tend to economise on the impervious elements in view of the high cost of a weir. However, the higher the weir, the greater the danger, resulting from seepage under the structure. This is particularly severe in the case of very short weirs, as e.g. the fixed weir (fig. 7-8), especially where a concrete stilling basin is not provided.

Fig. 7-17 shows the basic design of a high weir. The design and dimensioning of the stilling basin is based on the normal hydraulic principles, with the relation

$$t_2/t_1 = 0.5\ (\sqrt{1+8\ Fr_1^2} - 1)$$

holding for the corresponding water depths. A great variety of different formulas, all depending on the quantities t_1, t_2, Fr_1, are used to determine the length. The optimal design has been found when the energy-consuming surface roller resulting from a change in flow from streaming to shooting clings to the downstream face of the weir. It is in fact difficult to arrive at an optimal shape without preceding model tests. Stilling basin designs that have proved successful are those with a straight or trough-shaped bottom and, in the case of very large corresponding water depths in the stilling basin, a stepped floor. The water flowing out of the stilling basin should largely assimilate to the natural flow over an as short as possible reach. The velocity distribution at the cross section and the waving of the water surface should correspond to the original conditions in the stream.

The eroded hole in the river bed should be flat and require no exceedingly deep foundation of the structures. The usual rules of hydraulic design will normally meet the requirements where the discharge cross section through the weir is of greater width than height. Problems will result from exposed layers of sand and silt in the tailwater. But dredging of an artificial erosion hole and possibly its adequate protection will always constitute effective remedial measures.

In low weirs, a change from streaming to shooting flow with a hydraulic jump will rarely be accomplished. Flow will be wavy and require, besides a stilling basin, adequate protection of the banks downstream of the weir over a greater length than in the case of high weirs.

The whole problem of energy dissipation should be appraised in connexion with the frequency of loading through floods and possibly also with bed load transport through the weir. In mini hydro power, structural measures enhancing safety, such as e.g. deeper foundations or larger stilling basin designs, may affect the economy of a project. However, if economy is a prime consideration, supplementary measures in the tailwater area are imperative. In some cases a scour protection as shown on figure 7-6 will be provided. Where major bed load transport is expected, as may occur e.g. in flushing ducts, stone linings, as well as linings of planks and even of old rails have performed very well. By contrast, coats of epoxy resins or slabs of some plastic material glued to the primary concrete have not always yielded satisfactory results.

A weir design of relatively recent origin is the inflatable dam (fig. 7-14; W.E.Z., 1972; J. MAURER 1972), which can today be constructed to heights of between 2 and 3 m. The feasible length of such weirs is almost unlimited. In the CSSR, there are weirs of this type attaining lengths of 62 m. The weir consists of a flexible conduit of textile-reinforced plastic or rubber which is fastened to a concrete bottom slab. The dam profile can be trapezoidal or rectangular. In both cases, connexion of the hose to the banks is possible. The interior of the hose is connected, through the bottom slab, with a filling line, through which water can be supplied to the interior of the hose. The hose is then under a slight excess pressure, which makes it stand upright, thus impounding the headwater. To produce this excess pressure, the hose must be fed either from a sufficiently distant headwater section or from an independent water supply. When the headwater level rises, the relation between internal and external pressures is controlled automatically in such a way that the hose height decreases, allowing the flood to flow over the hose until the hose lies flat on the concrete slab allowing the flow to pass over it unhampered.

The relationships between weir discharge, upstream edge of the hose and overflow height can on principle be seen from the diagram on figure 7-15. Inflatable weirs realised so far have proved practically trouble-free in operation. Some of them are equipped with a small pumping set allowing the weir to be controlled independently of the water level or by other inflows.
Even in winter - under Central European conditions - no particular difficulties have been experienced as in the interior of the hose the ice coat along the skin only forms in the air-exposed tailwater zone. However, the ice coat is thin enough to break as soon as the hose moves a little.

It should be added that in most of the existing inflatable dams, the bed load problem is not a very serious one. Since, however, plastics or rubber show a very high resistance to erosion through bed load, such a weir has been constructed at a high-altitude site in Austria. A second one is under construction. In Switzerland, two inflatable dams exist on the Rhone river, which have performed satisfactorily under extreme loadings from bed load.

The inflatable dams constructed so far have been confined to heights up to 2 m. Larger inflatable weirs are conceivable, but the potential risk of oscillations would have to be studied first. To our knowledge, such investigations have not yet been made on a hydraulic model. A test programme of this type is in the preparatory stage at the Institut für Konstruktiven Wasserbau at the Graz University of Technology. The inflatable dam is no doubt an innovation affording great economy, it has performed satisfactorily and competes primarily with the flap. It is above all in mini hydro power that this weir type will prove competitive in the future and will be realised on a larger scale. A question yet to be answered is that of the service life of the hose where this is not continuously overflown and directly exposed to insolation.

Where the structural combination of a weir with a power station is planned, the following considerations should govern the design:
It will be useful to equip the weir with a gate. If flushing of bed load or deposited suspension load across the weir is impossible, a flushing duct should be provided on the side of the weir. This flushing duct should be located next to the powerhouse, in the first place to keep the power station free of bed or suspension loads (fig. 7-16) and to enable trouble-free operation.

To accomplish optimal flow of water towards the power station (uniform inflow) the alignment of the upstream side wall should be chosen with particular care to ensure that the entire zone of inflow towards the power station presents uniform flow conditions. The flushing duct adjacent to the powerhouse should allow flushing of the area immediately in front of the station. To this end, not only its cross section but also its invert should be carefully designed. The cross section to be closed by the gate must not be too small in order to permit a free flushing flow without backwater effect. The upstream dividing pier should enhance the flushing effect without, however, interfering with the flow towards the powerhouse.

In a great number of hydraulic model tests, it has proved to be of advantage to provide a pier crest sloping to the upstream. Due to this, it has been possible in most cases to make the flushing effect extend to the area of the cutwater of the pier so that the whole inlet area was cleared of sediment load. In addition, the sloping pier crest ensured undisturbed inflow towards the power station.

Where sufficient flushing capacity is provided, the weir can be designed for flood discharge only. Figure 7-17 shows an example constructed in Austria, which, however, does not correspond entirely to the system developed above. Since in this case the headwater is impounded to relatively great height and several weirs exist further upstream, bed load transport is expected not to occur at all during the first years of operation and, later on, to be limited to a very small amount so that safe flushing has not been a design consideration of as much importance as may have been required in other cases.

7.3 GATES

Gate systems (WICKERT - SCHMAUSSER, 1971) have so far been considered only to the extent required for the development of the weir concept as a whole. But there are also fundamental differences with respect to gate function, which will be discussed in greater detail in the following paragraphs.

The gate types indicated above for use in mini hydro power - slide gate, fixed-wheel gate, flap and radial gate - can be controlled manually or are equipped with an automatic control. The latter consists of connecting the gate, via cable control, to a float accommodated in a shaft located in the abutment area. The shaft communicates with the headwater or a control line in such a way that the float moves simultaneously with changes in the headwater level and thus lifts or lowers the gate. Such systems have been very successful in practice, but they require some structural effort in that a separate block is necessary for the accommodation of the float shaft.

A simpler solution is an oil-hydraulic gate control with servo-motors. Its function is also automatic in dependence on the headwater level. The structural requirements involved are relatively small. Slide and fixed wheel gates are usually driven by hand or motor-controlled hoist mechanisms although an oil hydraulic drive can also be used to advantage for such gate systems.

The flap and the radial gate can be provided with a straight or a curved upstream face. The shape is important particularly where a flap is overflown (figs. 7-18, 7-19), because this determines the pressure distribution and the driving power requirements. Subatmospheric pressures should be avoided. On the other hand, adaptation of the shape to the pressure-free parabolic profile yields the best flow coefficients and allows savings in gate height. A circular shape approximating the parabola or a combination of circular arc and straight line is lower in cost. In low flaps, the straight shape is the simplest solution, but it involves the least favourable flow coefficients and relatively high driving power requirements. The author also recommends shapes that start off from the pivot bearing with a circular arc, then continue straight and end again in a circular arc. The driving power requirements are smaller in this case because the end of the flap is relieved of water pressure.

Especially in straight or flat-curvature flaps, problems arise in the area of the hinge, which result from separation of the nappe and subatmospheric pressures when the gate makes too great a negative angle with the horizontal. Therefore, to reduce nappe separation, negative angles should not exceed 6 to 10°. On the other hand, however, in almost horizontal flaps, the nappe is thrown off relatively far, which calls for a fairly long stilling basin.

It follows from the above that the design of a flap should make allowance for a great number of different factors, in particular the pressure conditions on the flap, the resulting driving power requirements, the flow coefficients (fig. 7-21) and the required structural length of the weir.

It is very important in flap design that provision be made for sufficient ventilation of the space beneath so as to achieve atomospheric pressure. It will be useful to supply air across the two weir abutments.

In the case of low overflow heights, the flap shows a great tendency towards oscillation. The oscillating system composed of nappe, flap and air space under the flap reacts readily to ventilation cross sections. Even most careful design of these cross sections may be unable to avoid oscillations altogether in practice. In several cases, it has been possible to attenuate or even eliminate these oscillations by the subsequent insertion of an orifice plate in the area of the aeration cross sections. In addition, the upper edge of the flap is equipped with splitter elements whose spacing should not be too small. A movable device may prove useful because potential oscillations occurring in operation may subsequently be eliminated by changing their position.

Another factor to be considered is the tailwater level. If this is too high, the flap may start floating and cannot be lowered. Thorough studies should be devoted to this problem because tailwater levels are difficult to predict exactly. Many cases are known where the water level in the stilling basin was sufficiently low immediately behind the flap, but it was the higher water level in a stilling basin area further downstream that made the flap float. Radial gates (fig. 7-22) are much less problematic in this respect where they are not overflown. Nevertheless, if the design of the sealing edge is unfavourable and the angle between the gate segment and the dam too acute, oscillations may result. However, enough experience has been gathered in radial gate design that such phenomena can be avoided. Both in slide and radial gates, it should be seen to it that in the rest position there is sufficient pressure against the sill, and this should be of an order of magnitude equal to the hydrostatic head acting on the gate.

In selecting the most suitable gate, consideration should be given to the fact that the flap, in relation to the other gate types, offers the advantage that it can be lowered without energy from outside and without any special mechanism, which may be useful for reasons of safety. All the other gates require sufficient energy supply or a hydraulic control.

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7 WEIRS, GATES: LOW HEAD
WEHRE, VERSCHLÜSSE, NIEDERDRUCK

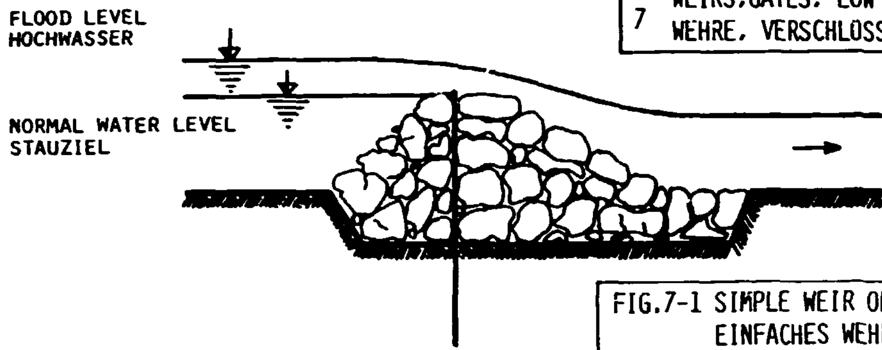


FIG.7-1 SIMPLE WEIR OF ROCK FILL
EINFACHES WEHR MIT STEINWALL

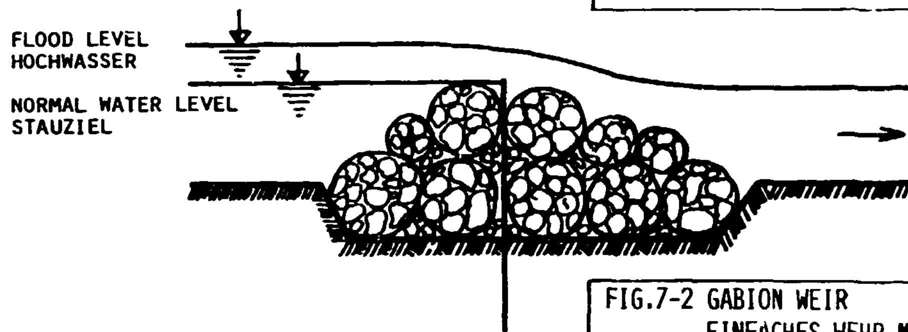


FIG.7-2 GABION WEIR
EINFACHES WEHR MIT SENKWALZEN

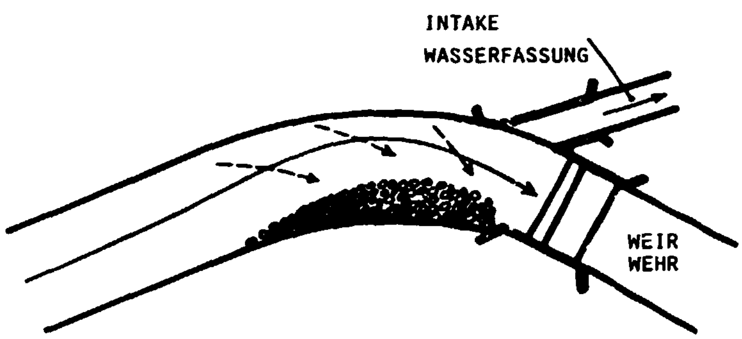


FIG.7-3 LAYOUT OF WEIR AND INTAKE IN A RIVER BEND
SITUIERUNG WEHR UND WASSERFASSUNG IN EINER FLUSSKRÜMMUNG

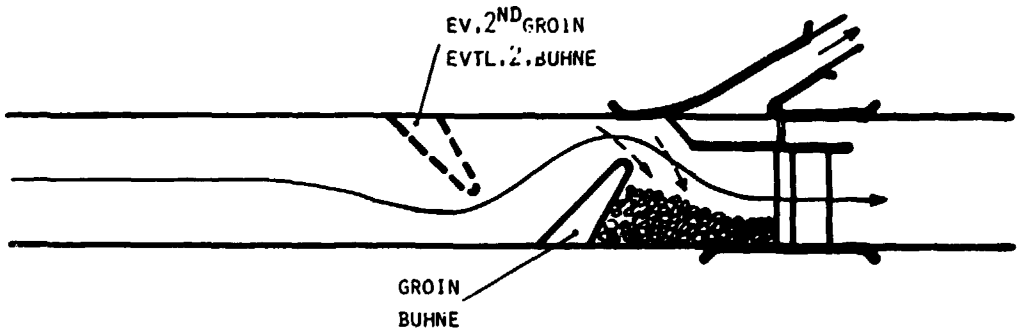


FIG.7-4 LAYOUT OF WEIR AND INTAKE IN A STRAIGHT RIVER
SITUIERUNG WEHR UND WASSERFASSUNG IN EINER GERADEN FLUSS-STRECKE

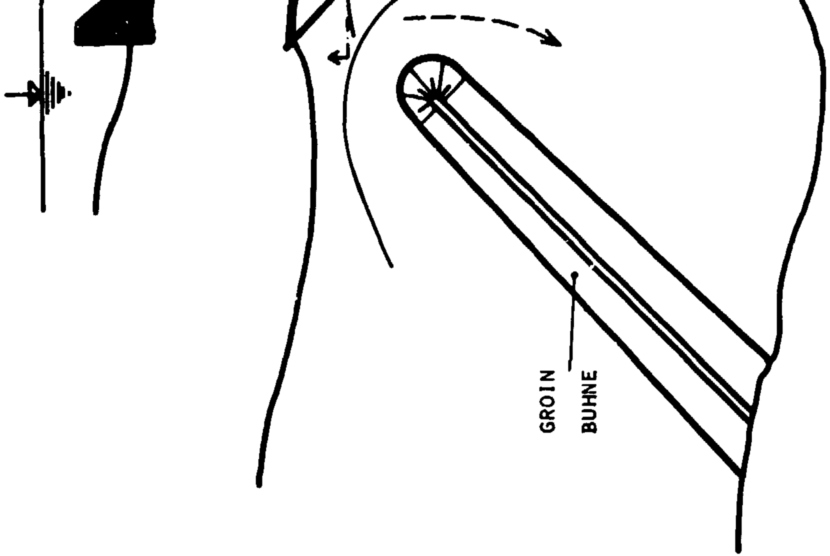
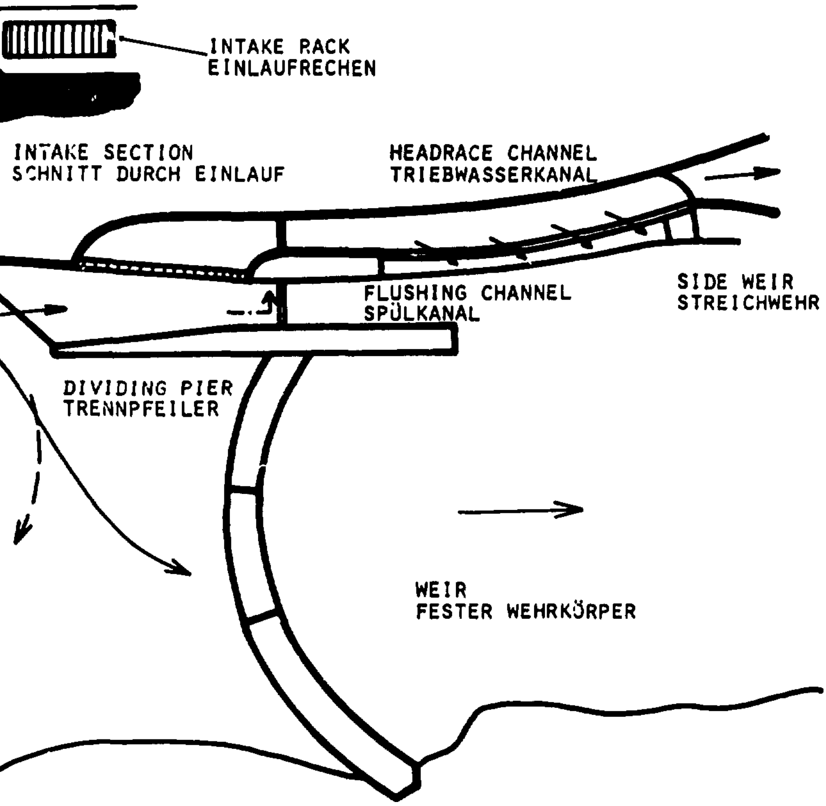


FIG.7-5 WATER INTAKE WITH FIXED WEIR
WASSERFASSUNG MIT FESTEM WEHR (KALSERBACH)



7 WEIRS.GATES: LOW HEAD
WEHRE.VERSCHLÜSSE:NIEDERDRÜCK

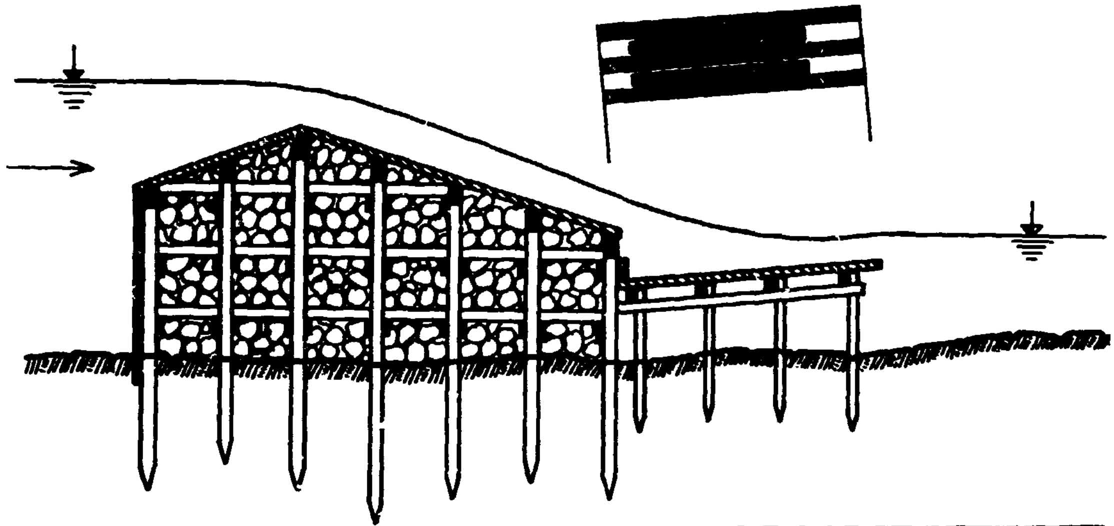


FIG.7-6 FIXED WEIR AND SCOUR PROTECTION
FESTES STEINKASTENWEHR MIT KOLKTAFEL

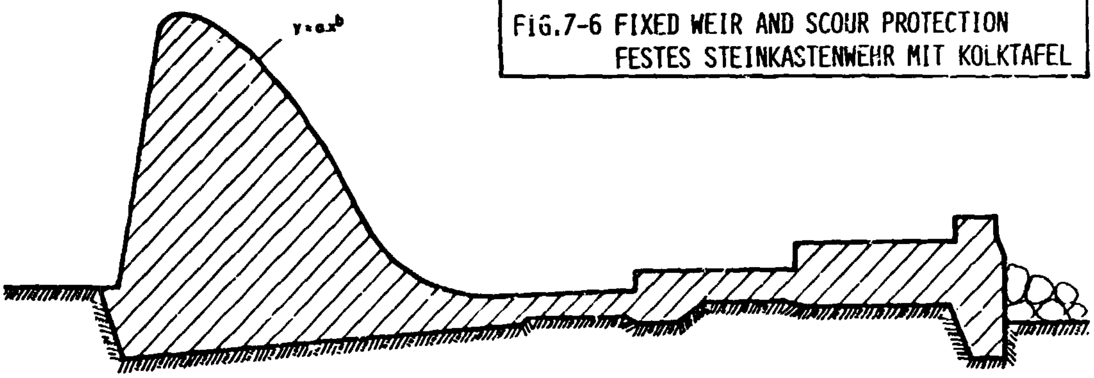


FIG.7-7 FIXED WEIR OF CONCRETE FOR Q = 9,5 M3/SEC,M STEPPED
FESTES WEHR AUS BETON FÜR Q = 9,5 M3/SEC,M

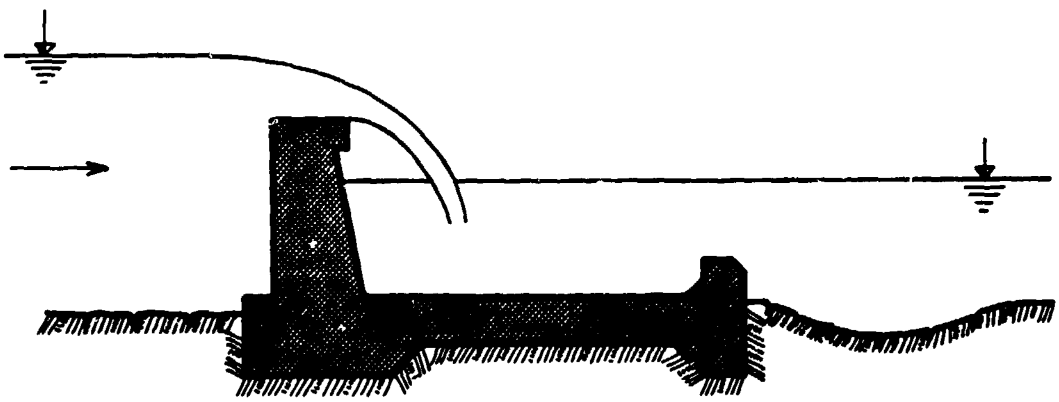


FIG.7-8 FIXED WEIR
STURZWEHR

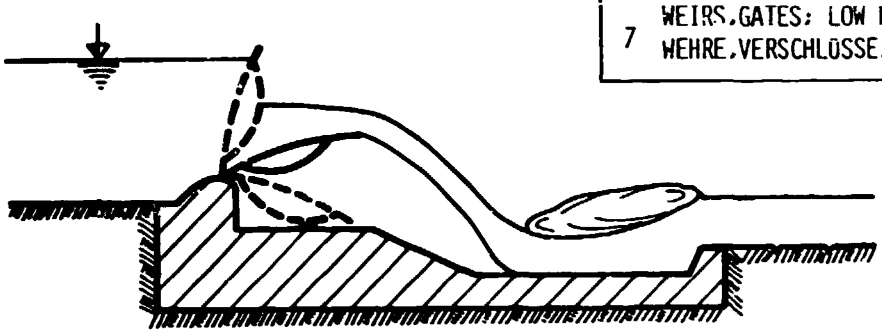


FIG.7-9 MOVABLE WEIR WITH FLAP
BEWEGLICHES WEHR MIT KLAPPE

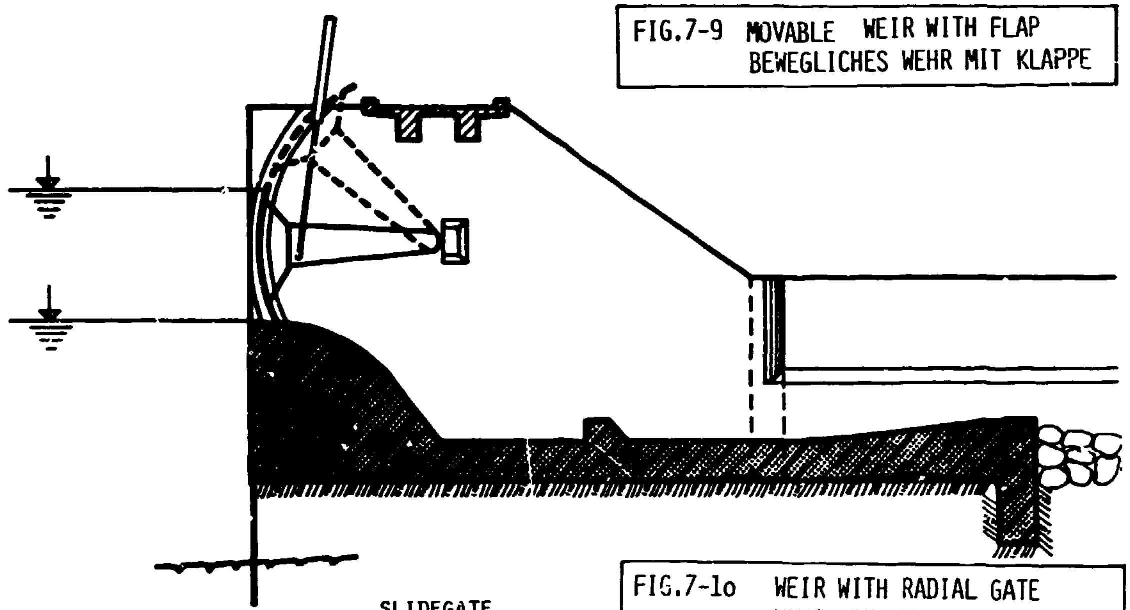


FIG.7-1o WEIR WITH RADIAL GATE
WEHR MIT SEGMENTVERSCHLUSS

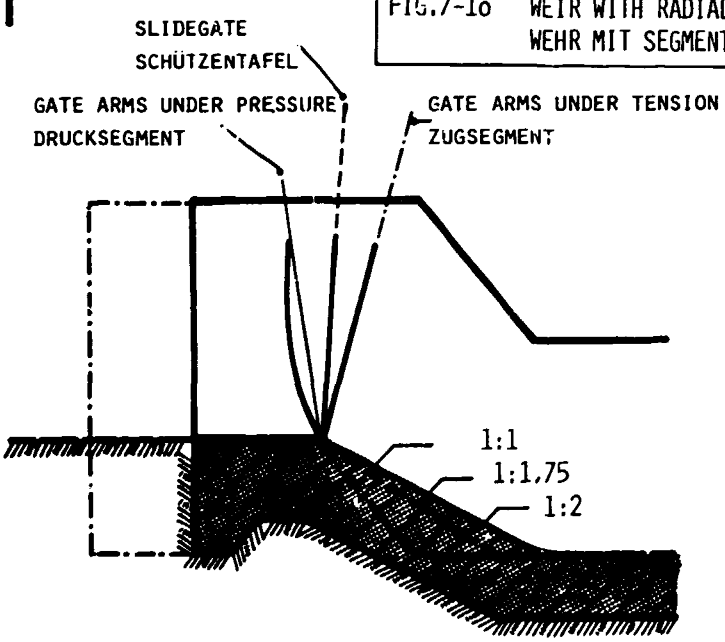


FIG.7-11 INFLUENCE OF GATEPOSITION ON PIER AND WEIR
EINFLUSS DER STELLUNG DES VERSCHLUSSES AUF PFEILER UND WEHRRÜCKEN

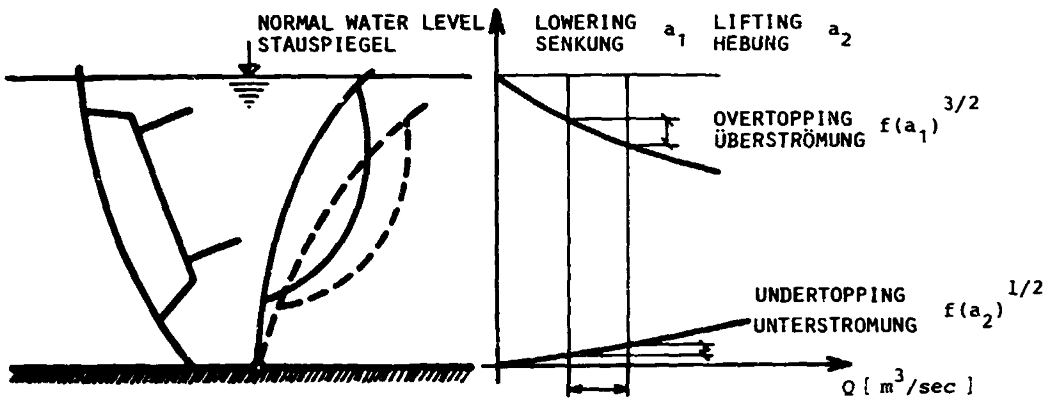
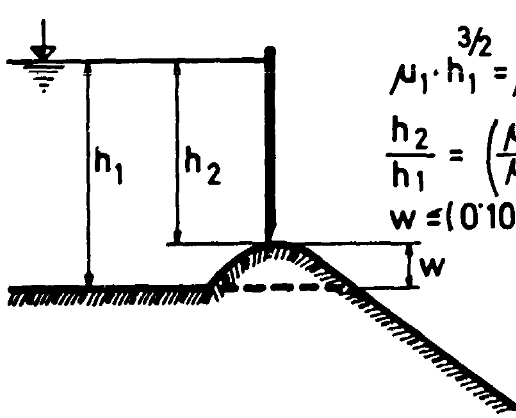


FIG.7-12 DISCHARGE CHARACTERISTICA FLAP - RADIAL GATE
ABFLUSSCHARAKTERISTIK KLAPPE - SEGMENT



$$\mu_1 \cdot h_1^{3/2} = \mu_2 \cdot h_2^{3/2}$$

$$\frac{h_2}{h_1} = \left(\frac{\mu_1}{\mu_2}\right)^{2/3} \sim 0.85 \div 0.90$$

$$w \leq (0.10 \div 0.15)\, h_1$$

FIG. 7-13 ARRANGEMENT OF A HYDRAULIC FAVOURABLE SILL TO REDUCE GATE HEIGHT

ANORDNUNG EINER STRÖMUNGSGÜNSTIG GEFORMTEN SCHWELLE ZUR EINSPARUNG VON VERSCHLUSSHÖHE

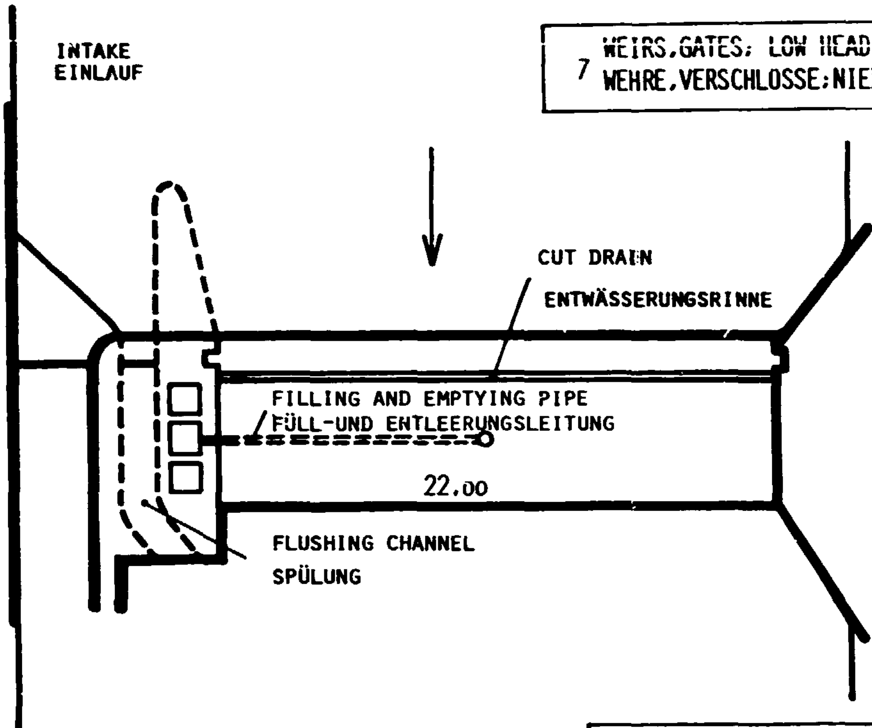
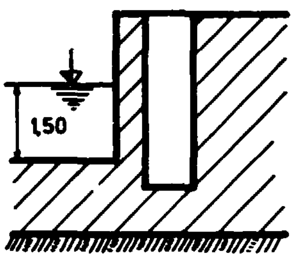
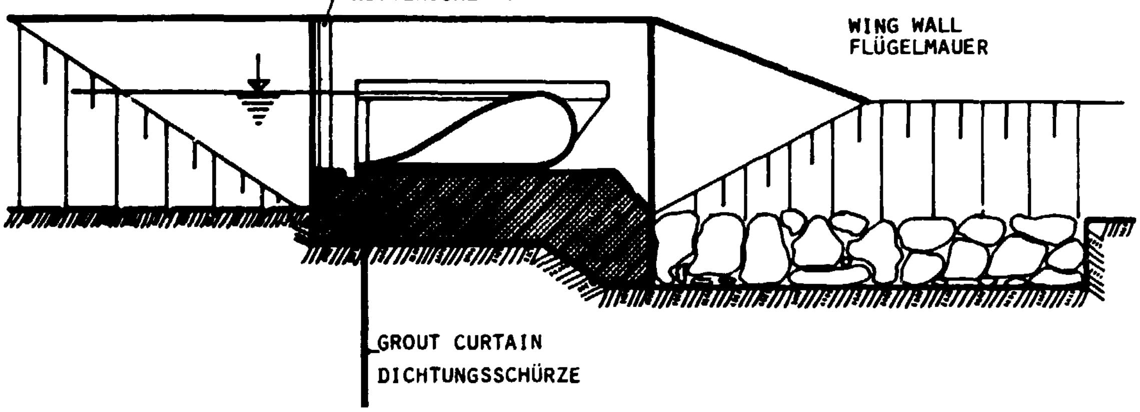
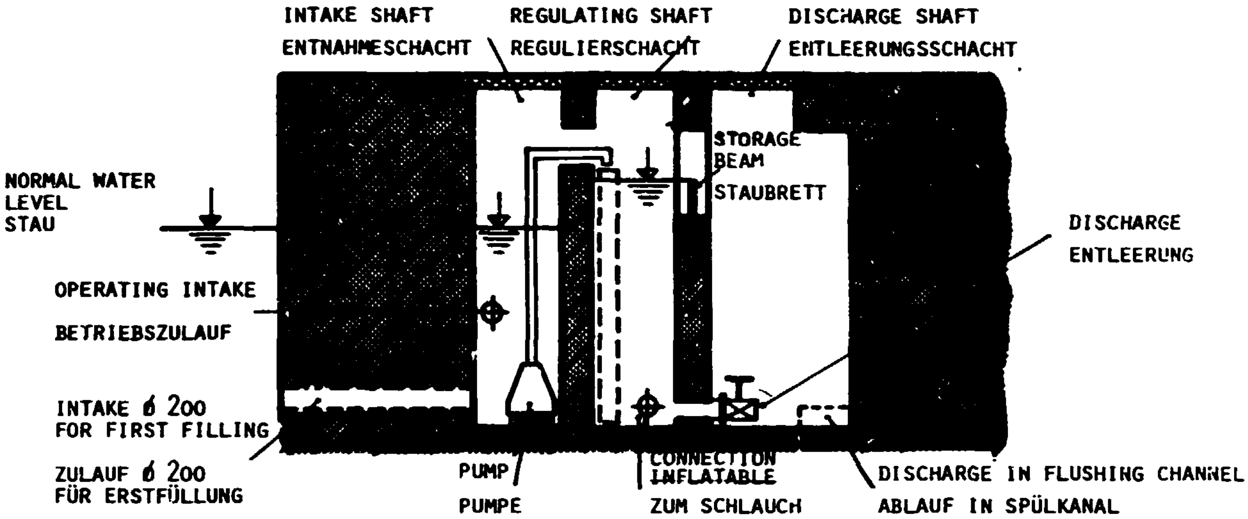


FIG. 7-14 INFLATABLE WEIR
SCHLAUCHWEHR

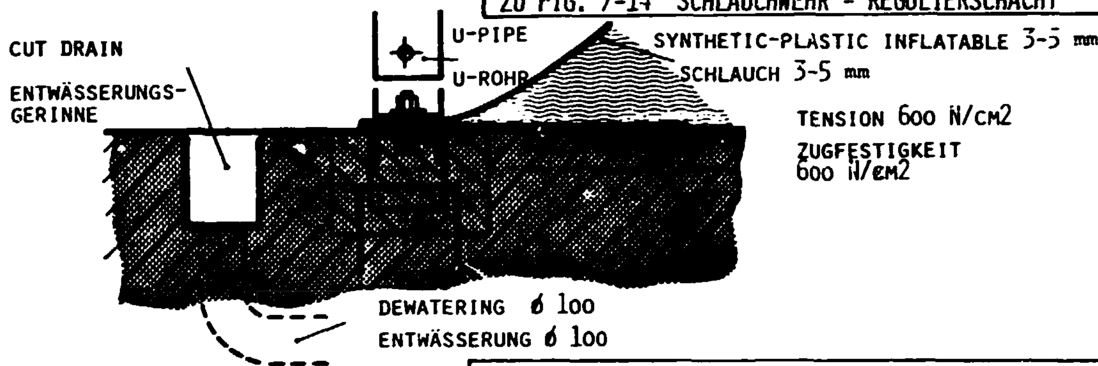




TO FIG.7-14	CROSS SECTION DISCHARGE CAPACITY SCHNITT DURCH EIN SCHLAUCHWEHR-FÖRDERFÄHIGKEIT 3.2 M3/SEC.M
TO FIG.7-14	STEADY NOTES QUANTITY BATCHING OVER INFLATABLE WEIR STÄNDIGE DOTIERWASSERMENGE OBER SCHLAUCH 0.027 M3/SEC.M



TO FIG. 7-14 INFLATABLE WEIR - REGULATING SHAFT
ZU FIG. 7-14 SCHLAUCHWEHR - REGULIERSCHACHT



TO FIG. 7-14 DETAIL INFLATABLE WEIR CONNECTION
ZU FIG. 7-14 DETAIL SCHLAUCHANSCHLUSS

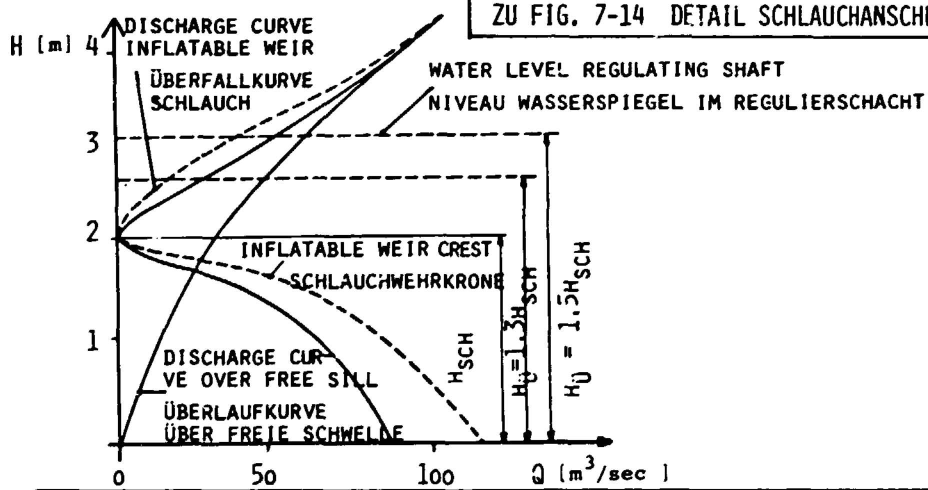


FIG. 7-15 INFLATABLE WEIR; STAGE DISCHARGE-, INFLATABLE WEIR CREST-, RATING CURVES
SCHLAUCHWEHR: ÜBERFALL-, SCHLAUCHWEHRKRONE-, ABFLUSSKURVEN

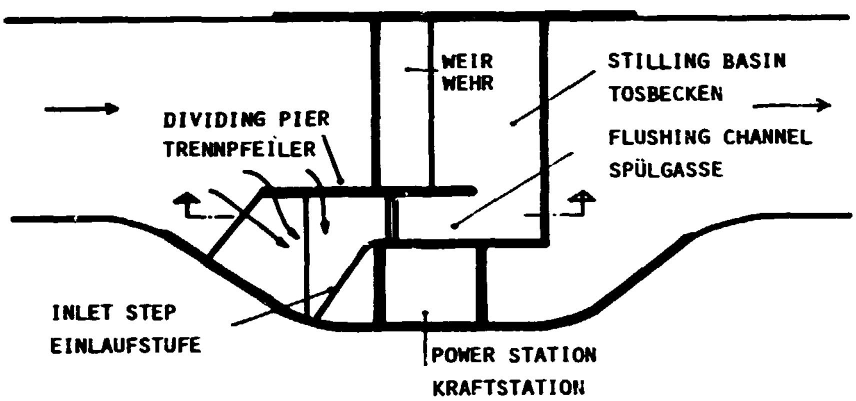
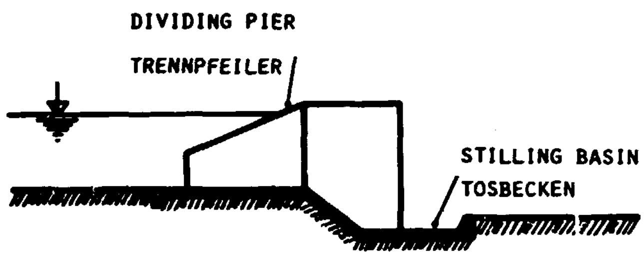


FIG.7-16 PRINCIPLE SITUATION OF WEIR AND POWER STATION
GRUNDSÄTZLICHE ANORDNUNG WEHR MIT KRAFTSTATION



TO FIG. 7-16 FLUSHING CHANNEL SECTION
ZU FIG. 7-16 SCHNITT DURCH SPOLGASSE

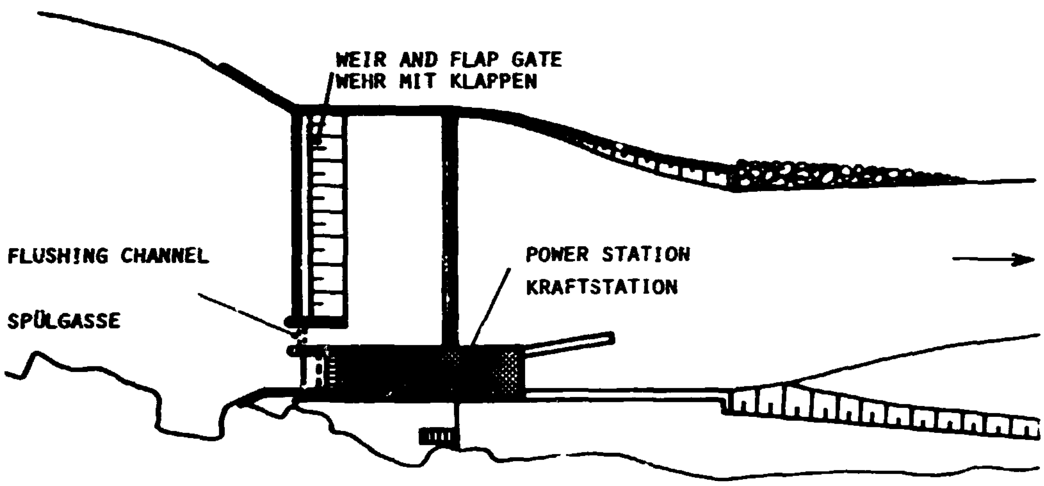
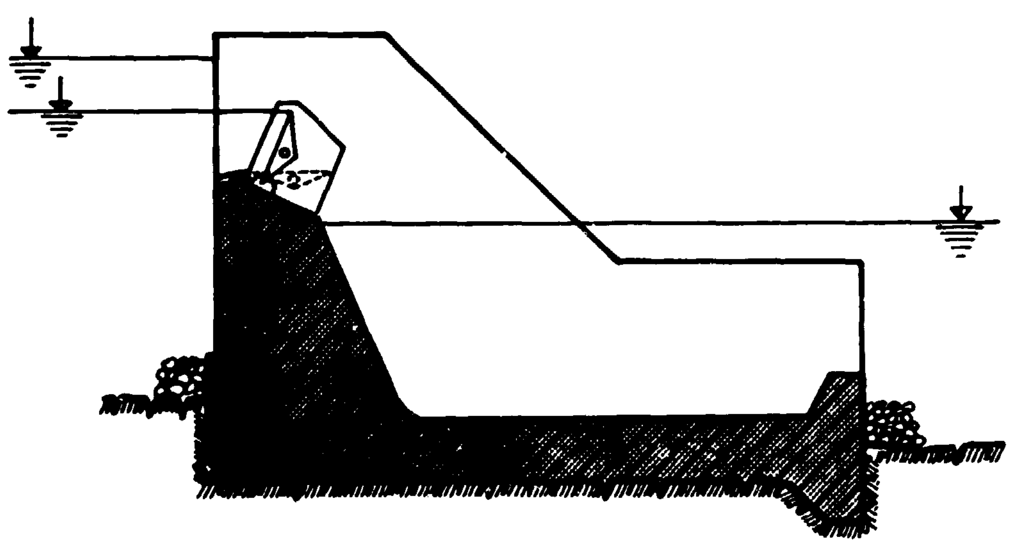


FIG.7-17 WEIR AND POWER STATION
WEHR MIT KRAFTSTATION



TO FIG.7-17 CROSS SECTION WEIR AND STRAIGHT FLAP
ZU FIG.7-17 SCHNITT DURCH DAS WEHR MIT GERADER KLAPPE

WEIRS.GATES; LOW HEAD
7 WEHRE.VERSCHLOSSE; NIEDERDRUCK

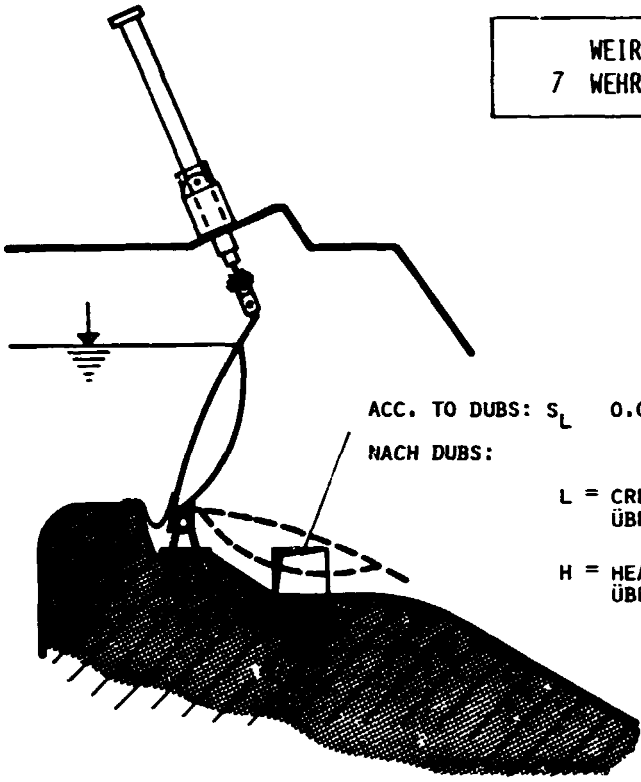


FIG.7-18 FLAP - HYDRAULIC ON PIER
KLAPPE MIT OBENLIEGENDER HYDRAULIK

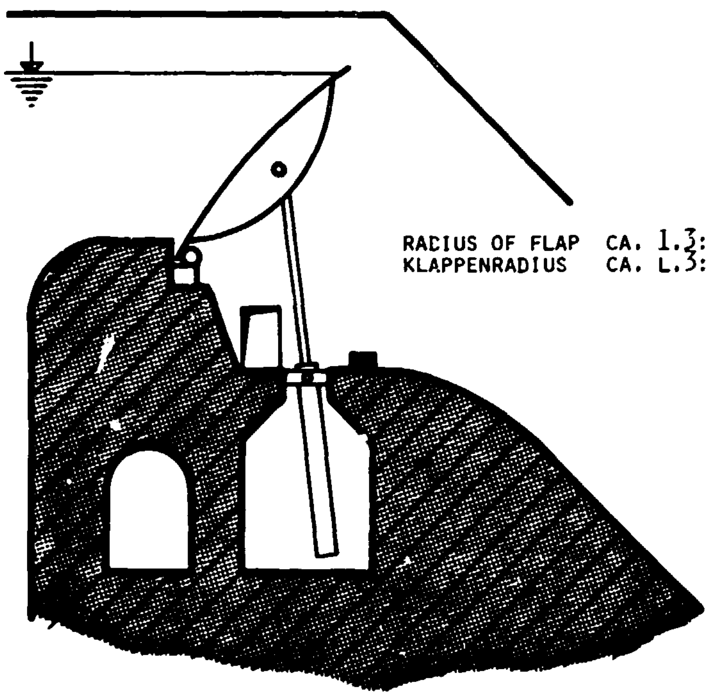


FIG. 7-19 FLAP WITH HYDRAULIC BENEATH
KLAPPE MIT UNTERLIEGENDER HYDRAULIK

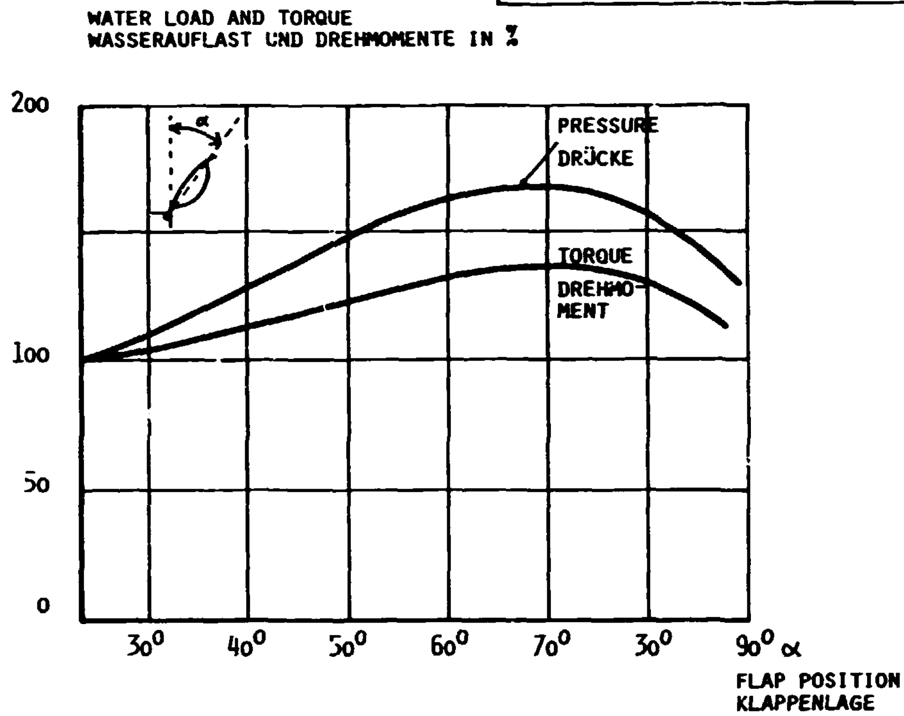


FIG.7-2o FLAP - WATER LOAD AND TORQUE
KLAPPE - WASSERAUFLAST UND DREHMOMENT

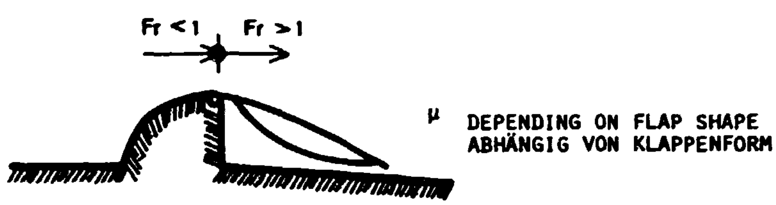


FIG.7-21 HYDRAULIC CRITERIAS OF FLAPS WITH AND WITHOUT WEIR
HYDRAULISCHE KRITERIEN BEI KLAPPEN MIT UND OHNE WEHRHÖCKER

WEIRS, GATES: LOW HEAD
7 WEHRE, VERSCHLOSSE: NIEDERDRUCK

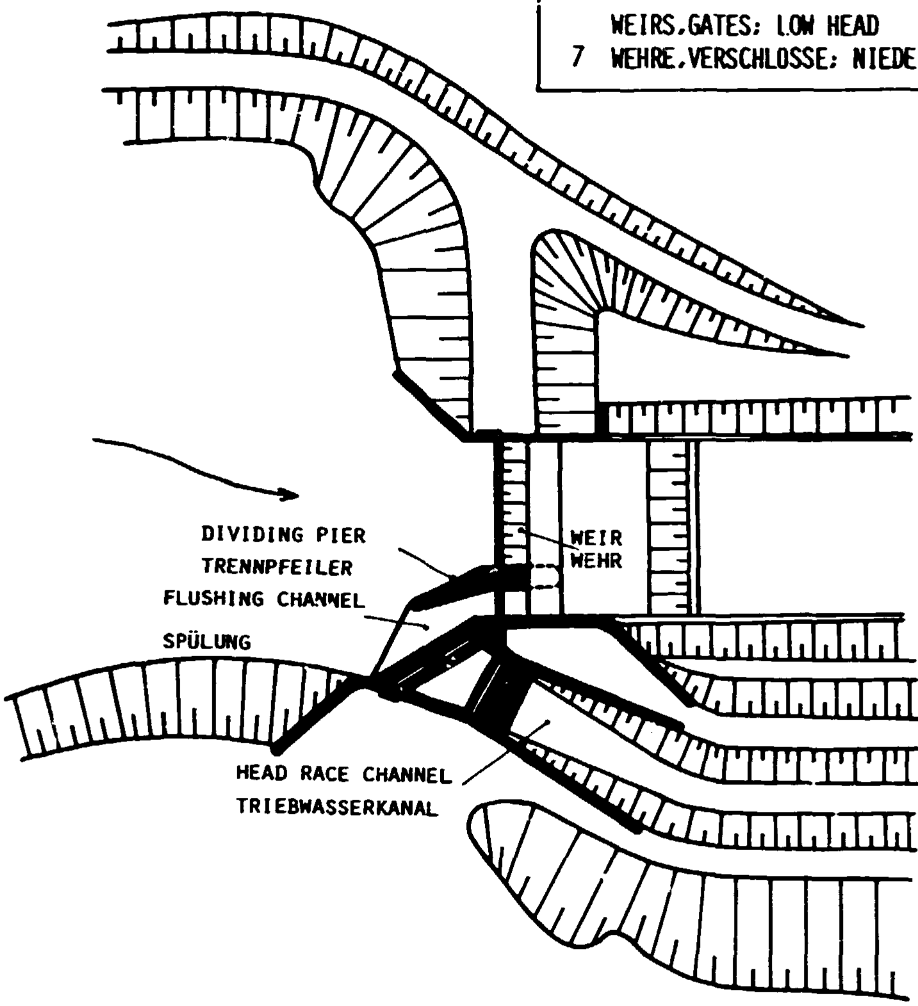
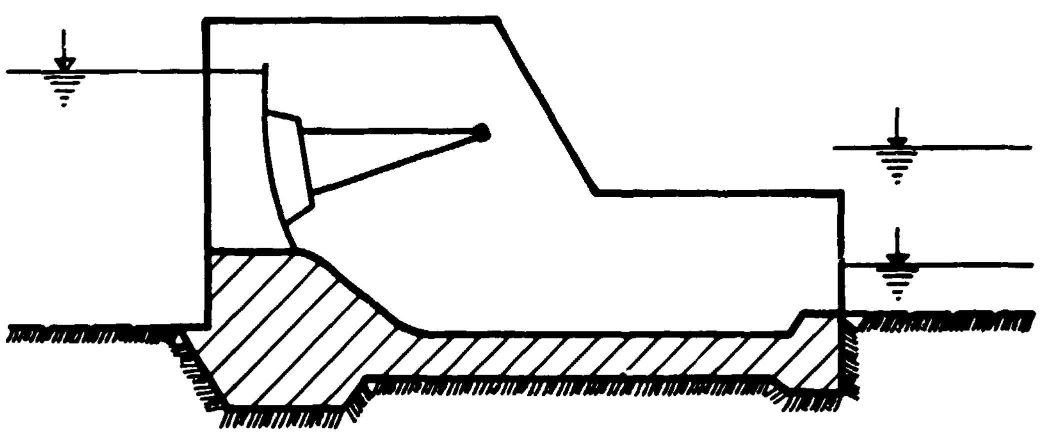
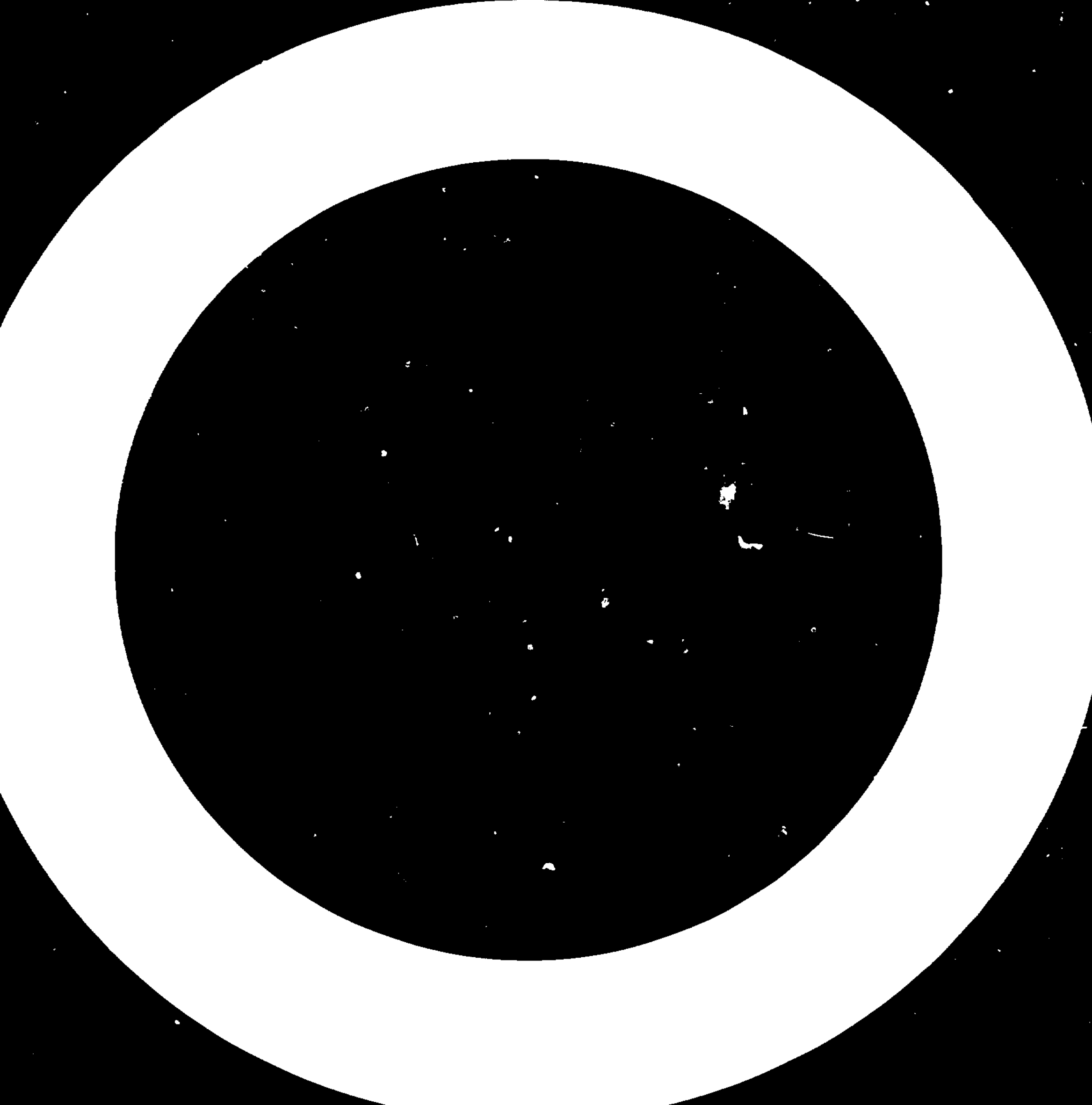


FIG.7-22 WATER INTAKE WITH RADIAL GATE
WASSERFASSUNG MIT SEGMENTWEHR



TO FIG.7-22 CROSS SECTION -WEIR AND RADIAL GATE
ZU FIG.7-22 SCHNITT DURCH WEHRANLAGE MIT SEGMENT



8 POWER INTAKE AND SAND TRAP IN LOW-HEAD POWER DEVELOPMENTS

STEINER R.

ABSTRACT:

This Paper discusses power intakes and sand traps in low-head power developments. Basic considerations regarding the location of power intakes are given in Paper 7, whereas this Report only deals with the fluid mechanics aspect as well as the operation and flushing of power intakes. Particular mention will be made of the so-called cantilever sill developed at the Graz University of Technology, a design allowing intake structures to be kept practically free of bed load. Depending on the suspended-load transport and composition, it may be of advantage even in mini hydro power to provide for sand removal from the turbine supply water. Problems relating to inlets into sand traps and to the design of desanding chambers as well as flushing possibilities are discussed. Intermittent flushing will probably be the method of preference in low-head power as continuous flushing takes 10 percent of the total inflow, which represents a constant energy loss.

8.1 GENERAL

Water intakes with a diversion by-passing the stream are often chosen for mini hydro stations as they permit gains in head to be derived from the alignment in elevation of the headrace or pipe line, with little impounding needed at the intake. A disadvantage of this project idea is the dewatering of the natural stream bed as well as the generation loss should the supply of compensation water be required. Its merits are the low structural height of the water intake and the small amount of interference with the groundwater conditions.

Two principal demands must be made on a water intake. Firstly, the flow diverted from the stream should be free of bed and suspension loads to the largest possible degree, and secondly, removal of bed load should be ensured by the flow remaining in the stream bed or by intermittent flushing. In general, a water intake consists of the following structural components: weir, flushing basin as well as inlet or intake works. Considerations of a fundamental nature regarding the location of water intakes for fixed-crest weirs and barrages have already been dealt with in the Paper on "Weirs". This Paper will discuss the optimal design and function of the power intake from the fluid mechanics point of view as well as flushing facilities for bed load removal. The second chapter will separately treat the problems of desilting.

8.2 BASIC STREAM ENGINEERING CONSIDERATIONS

The most difficult problems in civil engineering hydraulics include the fight against bed load in water intakes because the processes of transport, sedimentation and erosion of bed load are difficult to determine accurately. Basic information of prime importance will, consequently, be obtained from a study of the natural conditions of bed load transport in a stream. This is no easy task as the problem will be different according to whether we are dealing with a latent stream bed, a state of equilibrium or an erosion reach. The complexity of these conditions calls for profiles to be taken over decades to establish the changes in the mean bed level, a requirement that is rarely satisfied in mini hydro developments. Although it is difficult, both in theory and in practice, to determine the flow at which general bed load transport begins, this saves the troubles usually resulting from insufficient information on this fact.

In exceptional cases, mini hydro stations can be affected by sudden bed load impacts artificially produced e.g. by the flushing of major upstream hydro stations.

8.3 WATER INTAKE

Several requirements must be met should a water intake function properly. The operating safety should be appraised from the viewpoints of protection from bed load and the prevention of choking with floating debris and bed load. In addition, some consideration should be given to a flushing possibility to remove deposited bed load as may result from inaccurate operation or floods.

Investigations made in Switzerland (MÜLLER, 1955) have shown almost bed load free inflow to be possible provided that the flow drawn in during a period of bed load transport accounts for not more than half the total arriving flow. Other important aspects are the location of the intake works and the layout of the individual structural components. By making use of the effects of curvature in the stream and by making the diversion branch off at an acute angle, it will be possible to find a perfect solution. Practical experience has shown, however, that it should be made sure that curvature effects are actually present. In some cases it may be necessary to bring them about by artificial means. Still, it will be necessary to keep a constant check on the plant during operation.

In respect of the system, we differentiate between power intake works with bed load diversion towards the water inlet and those with bed load removal in front of the inlet (Fig. 8-1).

In the case of bed load removal in front of the inlet, the separation of the headwater from the bed load carrying tailwater is effected by providing an intermediate floor or slots (Fig. 8-2). Obstruction of these bed load ducts by bed load, wood or other bulky objects is a potential risk especially during floods, but removal is much easier and takes less time.

The intake works would at any rate be provided with a step in the intake floor and with a scum board and a coarse rack as a protection from floating debris. The assumption that an as high as possible intake step, or sill, and a low inflow velocity were effective means of solving the bed load problem has been defeated by tests (BULLE, 1926). The top of the inlet sill should be 1.2 to 2.0 m below the ungated weir crest or the top water level. In many cases it will also be of advantage to provide an inlet step rising from the dividing pier towards the bank.

In order to ensure the permissible inflow velocity it will generally be preferable to provide deep and narrow inlets and to avoid great inlet widths, which are difficult to keep free of bed load. In practice, there will be advantage in risking to set the inlet step too low and to heighten it subsequently in the light of experience gathered during operation.

The inlet front alignment should not differ too much from the general direction of inflow, the branching-off angle losing in importance with rising headwater level. According to our experience, a diversion branching off at right angles is possible where the flow velocities do not exceed 1 m/s. At any rate, the design of the diversion should correspond in an optimal manner to the inflowing water, avoiding dead water zones and return currents because this is where bed load is deposited.

In search for an effective means of fighting the bed load problem, the Institut für Wasserwirtschaft und Konstruktiven Wasserbau of the Graz University of Technology has developed a cantilever sill. This new sill design has yielded excellent results not only in new structures but also in the reconstruction of old structures with unsatisfactory bed load protection (SIMMLER, 1979).

Model tests and operating experience have shown that mobilisation of the curvature effect is not an essential requirement, which enables this intake type to be used even for straight inlet reaches without calling for the provision of an artifical curvature.

To allow a cantilever sill with sufficient space underneath to be provided, the inlet step should be at least 1 m to 1.5 m high. Other structures required are a dividing pier in the headwater and a flushing outlet pointing in the direction of flow. Besides the cantilever sill, the design and location of the dividing pier is important. Dividing piers rising beyond the top water level produce severe turbulences at the cutwater of the pier, which renders development of a uniform flow pattern in the inlet area practically impossible (Fig. 8-4). In order to eliminate these objectionable disturbances, a submergible pier was studied (KROLL, 1977). This led to a substantial reduction in turbulences and produced a uniform velocity distribution in the inlet area (Fig. 8-5). In case the top of the pier is too low in relation to the cantilever sill, unfavourable secondary flows might form behind the dividing pier. The best results have been obtained from a dividing pier decreasing in height towards the upstream (Fig. 8-6).

This design not only keeps the inlet area practically free of bed load, but also ensures a very uniform flow over the cantilever sill. During the flushing process, a bed-load carrying roller forms under the cantilever. The roller can be reinforced by a slight constriction in the area of the flushing gate.

In our experience, flushing should take place already upstream of the intake, in such a manner that the whole inlet zone can be cleared of material. This requires a sufficiently large dimensioning of the flushing duct and of the gate as well as a sufficiently sloping upstream floor, that is to say, 10 percent if possible. To render flushing easier it is useful to concrete the upstream floor between the dividing pier and in the inlet sill.

Tests carried out so far confirm that by provision of a cantilever sill, excellent flushing of the basin between the dividing pier and the intake front is achieved. A condition of this is, however, the lowering of the water level in the basin and a free discharge through the flushing basin.

For reasons of safety, an additional flushing facility may be provided downstream of the inlet, although flushing upstream of the intake should always remain a basic principle.

Gate types chosen for flushing are usually slide gates, rarely fixed wheel gates. These are driven via racks or chains with hand or motor operation, or by means of a hoisting rod with a hydraulic cylinder. To allow maintenance and repair of the flushing gate, it is recommended that in any case an emergency gate be provided. The simplest solution is in the form of planks, for which adequate guides must be provided. The same applies to the inlet of the power intake, to allow a check to be made on the headrace when necessary.

8.4 SAND TRAP

Practically all streams carry bed load as well as suspended matter in amounts varying according to the slope of the respective stream bed. A carefully designed intake structure will enable the waterway to be kept free of bed load, whereas suspended matter will remain in the power water. Further downstream this risks to have harmful effects on all machine parts coming into contact with the power water, and this the more so, the greater the effective head, that is, the velocity of the flowing water. Therefore, depending on the suspended load concentration, it may be useful even in low head power developments to make provisions for the desanding of the power water. Decision in favour of a sand trap to ensure trouble-free operation should be preceded by a careful study of the bed and suspension load conditions.

Sedimentation of undesirable suspension loads is relatively easy. The principle of effective desanding consists in retarding the power water flow and distribute it as uniformly as possible over the whole cross section. The residence time of flow in the sand trap must not be shorter than the sedimentation time of the suspended material. This calls for relatively long structures of favourable hydraulic design. As the flow velocity towards the sand trap must not be too small to avoid premature sedimentation of the suspended material, inflow into the sand trap will be turbulent. Therefore, it is recommended that at any rate several stilling screens be provided at the upstream end of the sand trap. It should be pointed out in this context that uniform flow is very difficult to accomplish where the flow from the weir is very turbulent or where the supply channel to the sand trap is curved. This can be allowed for by providing a sufficient sand trap length and adding a certain extra length.

For the dimensioning of sand traps, observation of certain measurements has proved useful. These can be read off the relevant literature (MOSONYI, 1966).

Thus, basic research and practical experience have yielded definite guide lines for the design of sand traps as e.g. the water depth in the sand trap or the relation of basin width to basin length, which lead to the correct dimensioning of the sand trap.

Flushing of the sand trap can be continuous or intermittent. Continuous flushing involves a permanent flow loss of some 10 percent. Therefore, in mini hydro stations, intermittent flushing will be preferred unless supply of compensation water to the stream bed is required by the authorities. Ir continuous flushing, the sand sinks, through screens embedded in the basin floor, into the flushing duct, and is then returned to the original stream bed (Fig. 8-9).

In intermittent flushing major amounts of sand first settle in the basin and are then removed by a flushing process started either by hand or automatically. Since the flow velocity through the sand trap increases as the basin sills up, timely flushing is important. To facilitate flushing, certain rules should be observed. Besides providing smooth walls, it is useful to design the floor zone as a trapezoidal profile. The floor width should account for about one third of the total basin width and the lateral shoulders should have a slope not less than 1 to 1. For the longitudinal gradient of the basin floor, value exceeding 2.5 percent is desirable if the topography permits it (Fig. 10). In addition, adequate dimensioning of the flushing outlet is required, for which a flushing flow of 1.5 to 2 times the rated discharge has proved useful. In order to accomplish effective flushing, it is necessary that the water surface in the sand trap drops to the free flow level. It is only then that backward erosion can completely flush the sand catching basin free of material, the lateral shoulders encouraging the gradual sliding of settled sand into the flushing duct. In many cases, a radial gate is used as a flushing gate. This can be controlled by very simple, fully automatic means, via pressure cells embedded in the floor of the sand catching basin. Naturally, also hand operated slide gates - more rarely, fixed wheel gates - can be used. The necessity of returning the flushing water into the natural stream bed dictates the impounding height at the water intake.

The desanded power water can be conveyed to the powerhouse either in a free-flow channel or in a pipe line. Where a pipe line is used, it should be seen to it that neither during operation nor during the flushing process air is drawn in. The water surface downstream of the overflow crest of the sand catching basin should be at least 1.5 m above the top of the pipe line. Turbine operation should be stopped during flushing. In the design shown on Fig. 8-12, a partition protects the pipe line from running dry during the flushing process.

Since in fixed-crest weirs, the water level in the inlet area is dependent on the discharge, the sand catching basin must in such a case be provided with an overflow spillway (Fig. 8-11) to enable the discharge of surplus flow. For the precision control of the water level in the sand trap to obtain the rated discharge, it is useful to render the crest of this spillway variable by placing planks.

The quantity of desanding is determined not only by the limit particle size but also by the degree of desanding. In the low-head range, settling of grain sizes 0.2 to 0.5 mm in diameter are considered useful, the suspension load concentration representing a determining parameter. Streams in level country normally carry less suspended material, which, however, is likely to attain 10 to 50 times this value when the flow increases. Where the suspension load concentration is very high even with low flows - approximately 5 to 10 kg/m³ - desanding to a limit particle size of 0.1 mm or less may be necessary to protect the gates, the pipe line and in particular the turbine from premature wear. Where the suspension load contains a major proportion of quartz particles, the lower limit screen size deserves particular attention.

The degree of desanding is defined as the percentage relation of the suspension load concentration of the desanded flow to that of the flow prior to desanding. Where the composition of the suspension load transport is known, the degree of desanding is easily calculated by fixing the limit particle diameter.

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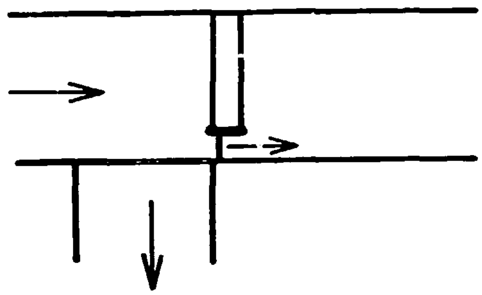
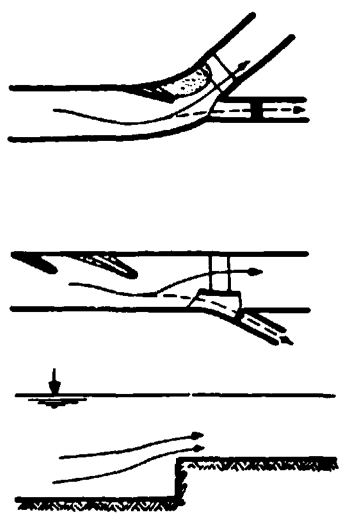
BED LOAD DRAWN OFF IN FRONT OF INTAKE
GESCHIEBEABZUG VOR DER EINLAUFFRONT

CONVENTIONAL TYPE
KONVENTIONELLER TYP



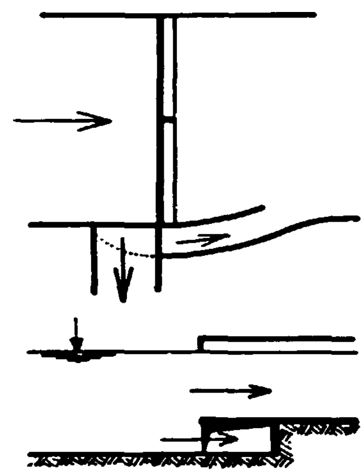
ZURÜCH UNVERSITY OF TECHNOLOGY TYPE
TYP ETH ZÜRICH (MÜLLER)



GRAZ UNIVERSITY OF TECHNOLOGY TYPE WITH CANTILEVER
SILL AT INTAKE STEP
TYP TU GRAZ MIT KRAGSCHWELLE AN DER EINLAUFSTUFE

BED LOAD DIVERSION INTO WATER INTAKE
GESCHIEBEAUSLEITUNG IN RICHTUNG EINLEITUNG

CONVENTIONAL TYPE
KONVENTIONELLER TYP



BED LOAD DRAW OFF BENEATH INTAKE
GESCHIEBEABZUG UNTER DER EINLAUFSOHLE

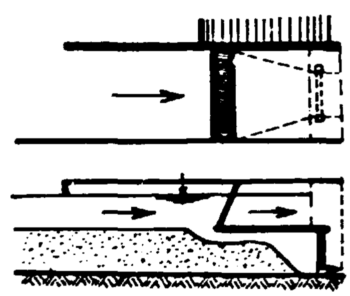


FIG. 8 - 1 TYPES OF WATER INTAKES
SYSTEME VON WASSERFASSUNGEN

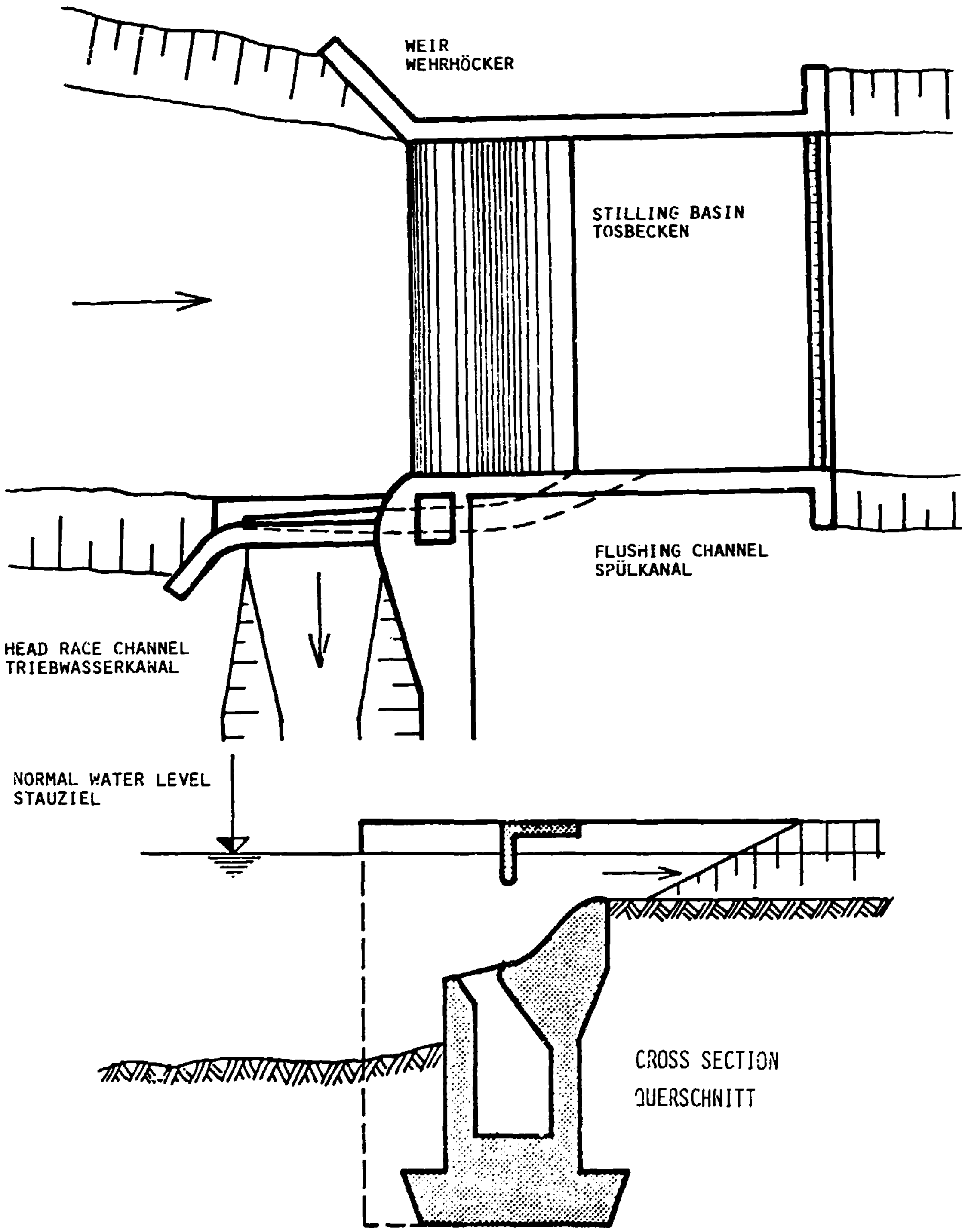
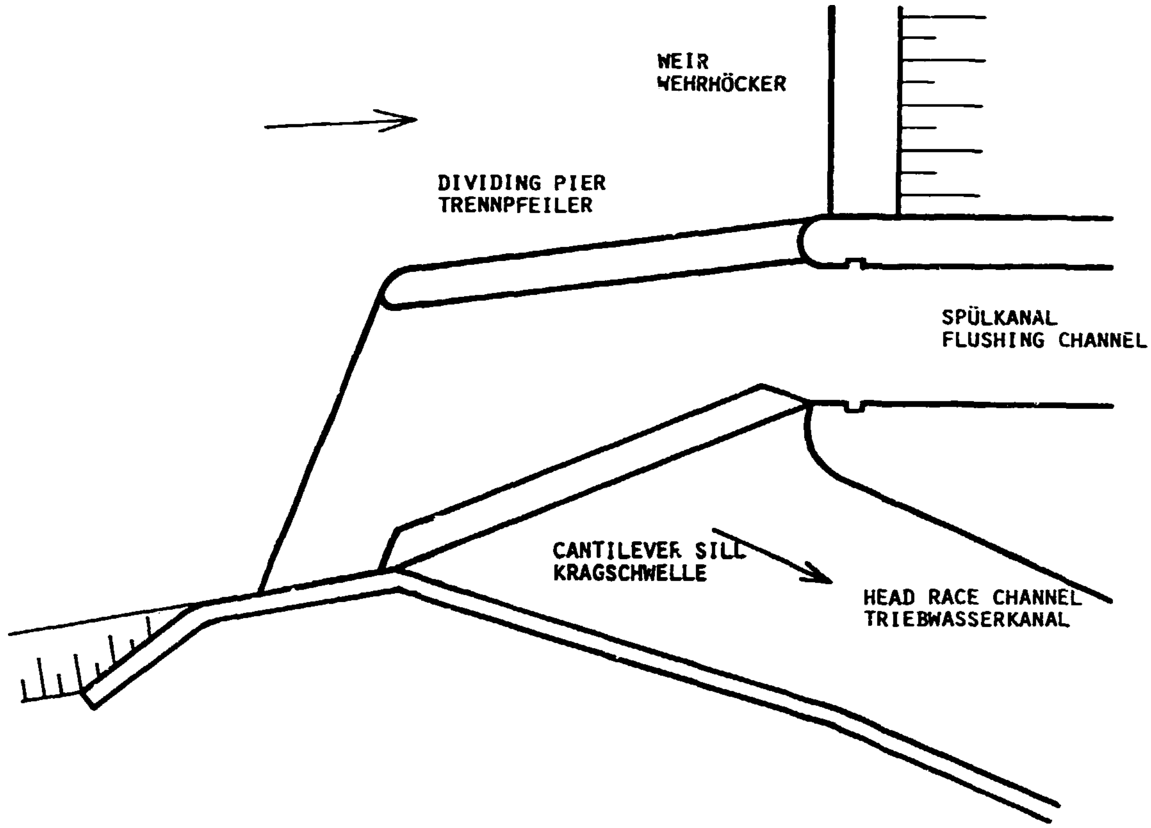


FIG. 8-2 WATER INTAKE WITH BED LOAD DIVERSION INTO WATER INLET
WASSERFASSUNG MIT GESCHIEBEAUSLEITUNG RICHTUNG EINLEITUNG



CROSS SECTION
QUERSCHNITT

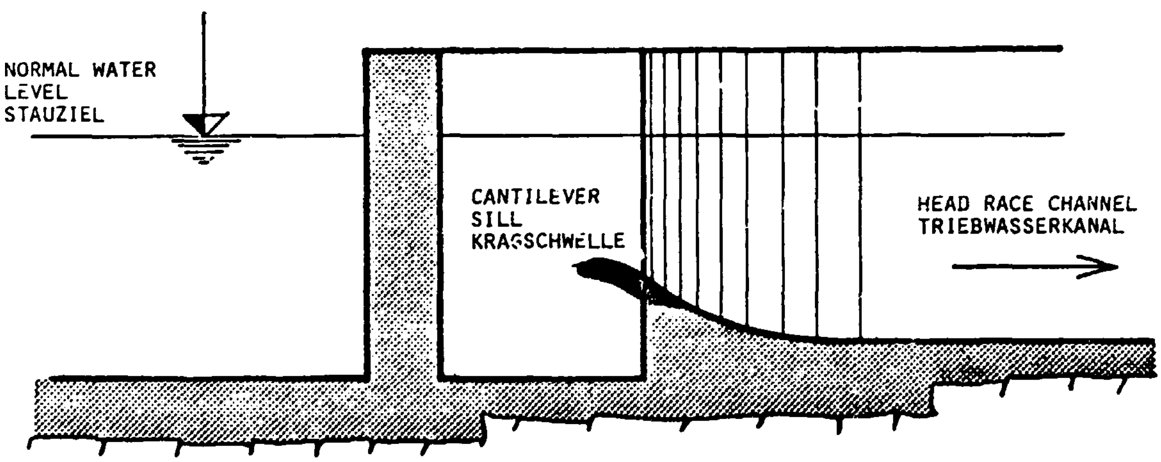
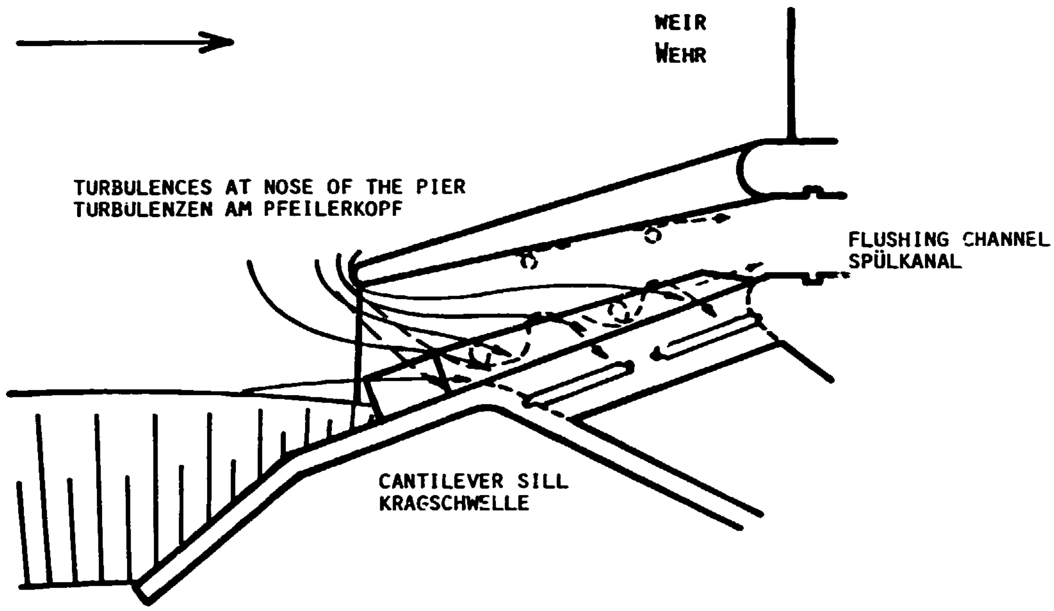


FIG. 8-3 WATER INTAKE WITH CANTILEVER SILL
WASSERFASSUNG MIT KRAGSCHWELLE



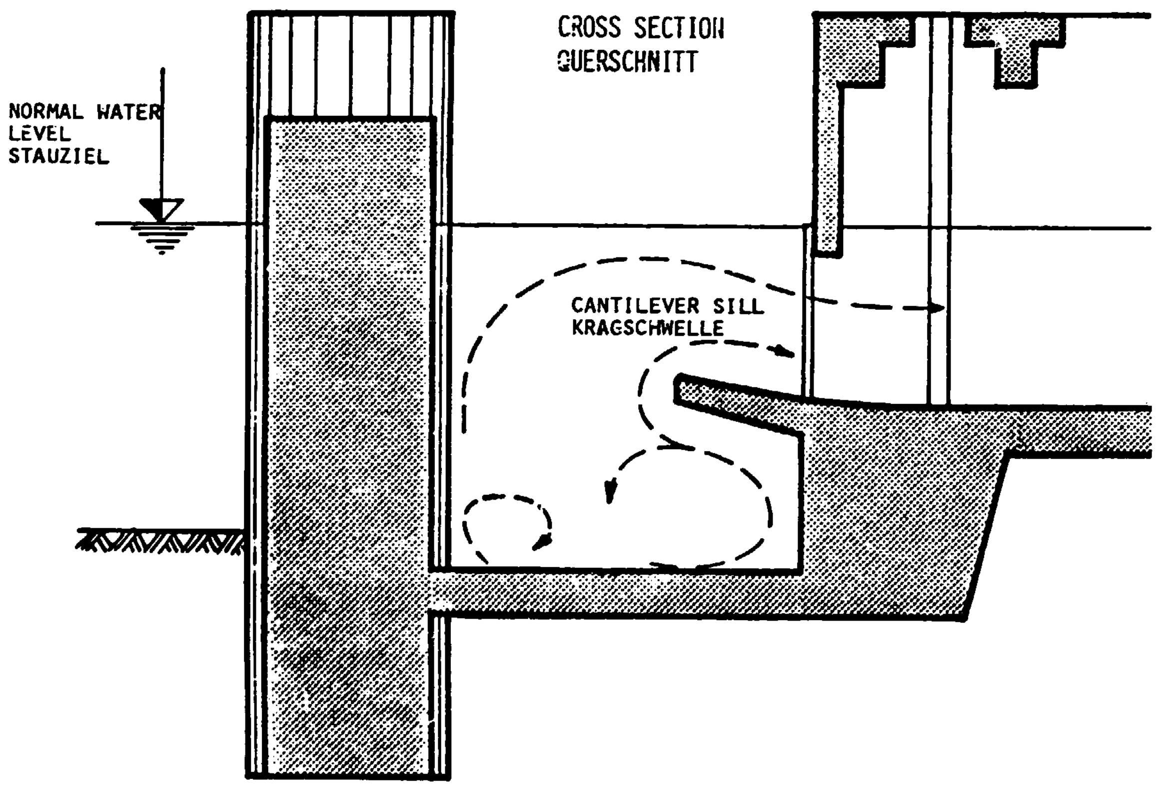


FIG.8-4 INTAKE WITH CANTILEVER SILL, DIVIDING PIER RISING ABOVE NORMAL WATER LEVEL
EINLAUF MIT KRAGSCHWELLE, TRENNPFEILER OBER STAUZIEL

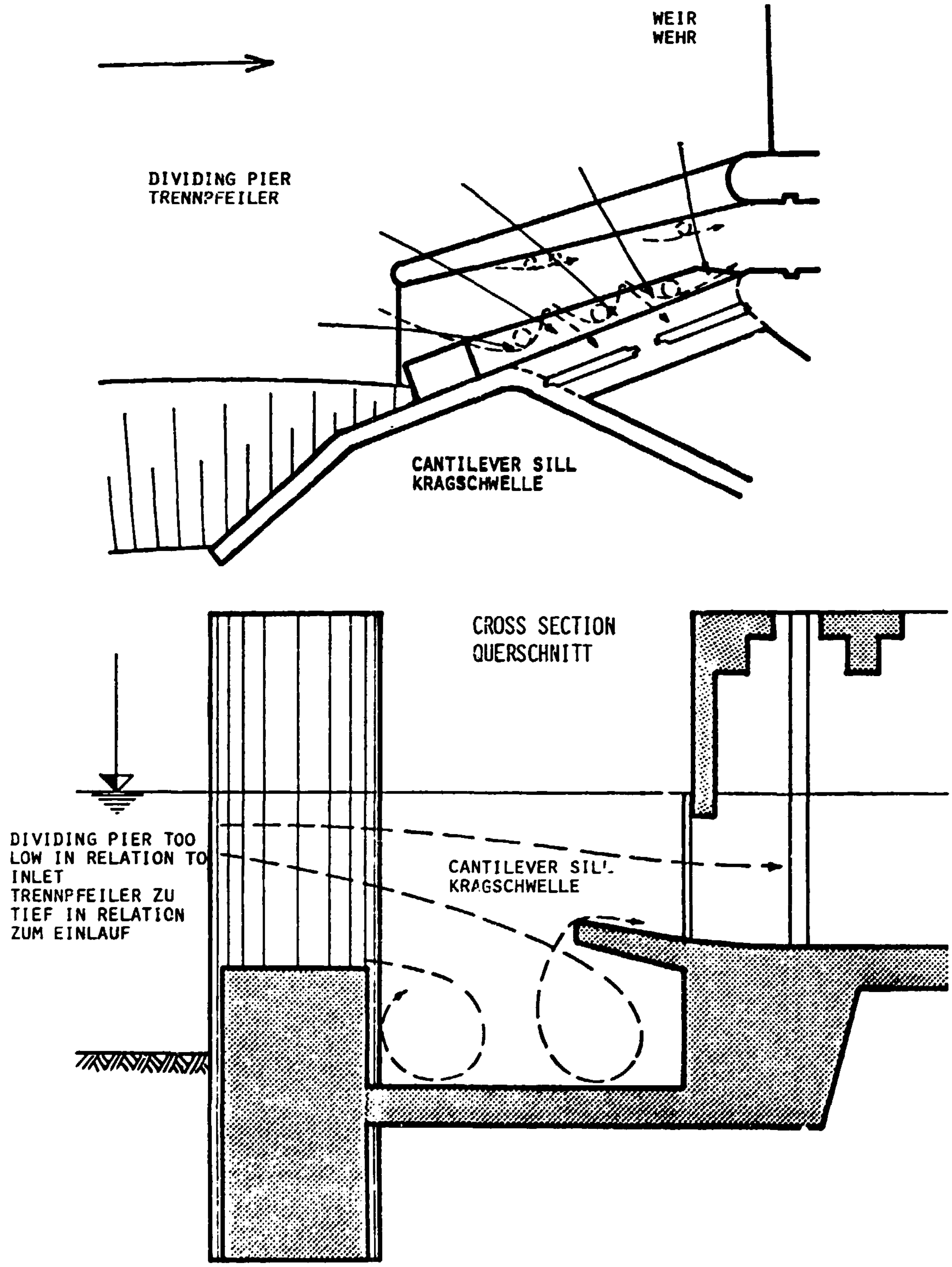


FIG.8-5 INTAKE WITH CANTILEVER SILL WITH DIVIDING PIER OVERFLOWN
EINLAUF MIT KRAGSCHWEILLE, TRENNPFEILER OBERSTROMT

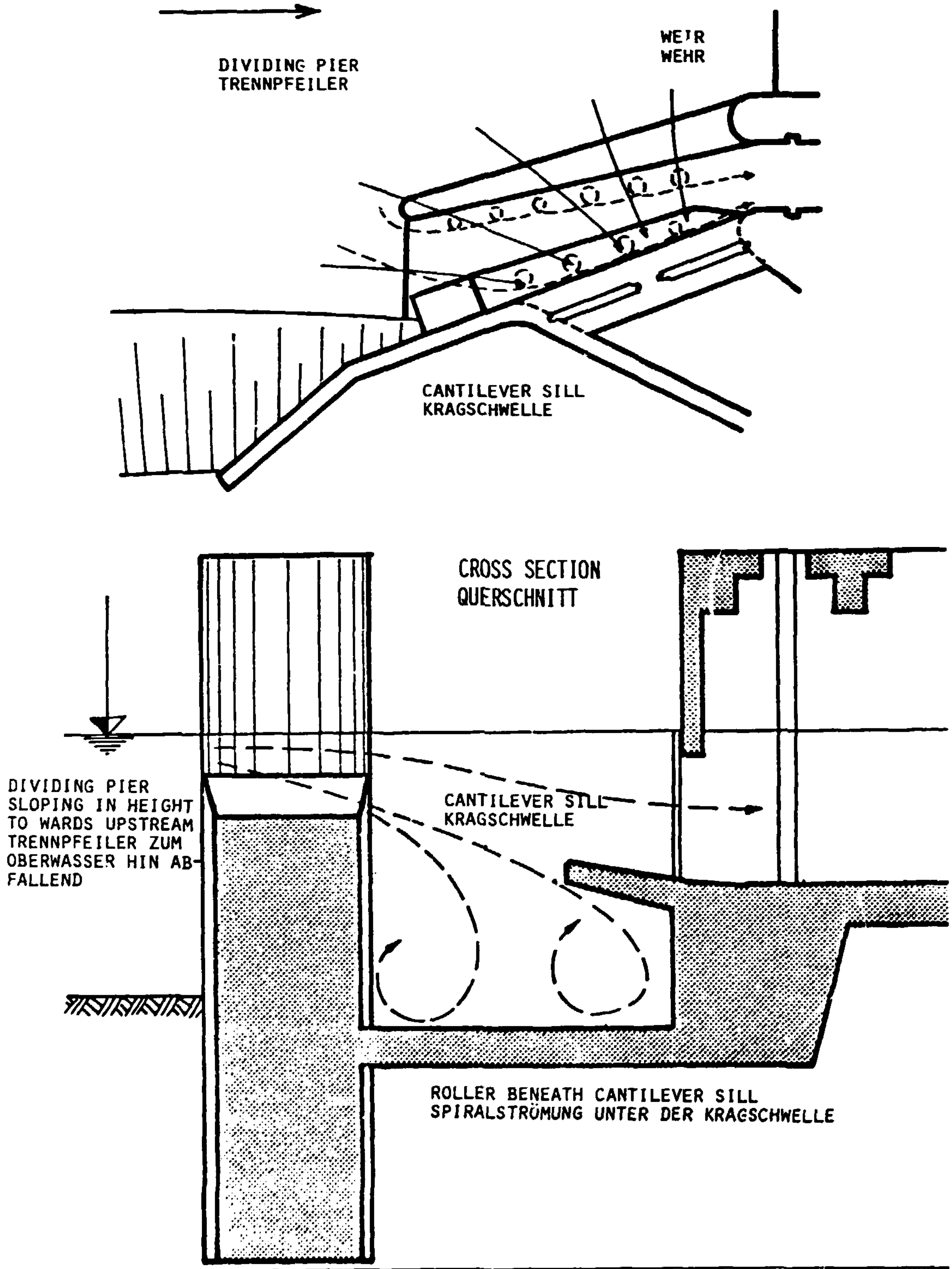


FIG 8 - 6 INTAKE WITH CANTILEVER SILL, DIVIDING PIER SLOPING TOWARDS UPSTREAM
EINLAUF MIT KRAGSCHWELLE, TRENNPFEILER ZUM OBERWASSER HIN ABFALLEND

USED AS FLUSHING GATE FOR WATER INTAKES AND DESILTER BASINS
VERWENDET ALS SPÜLSCHÜTZ BEI WASSERFASSUNG UND ENTSANDER

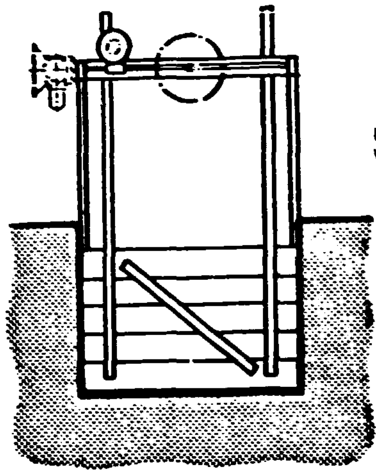


FIG. 8-7 WOODEN GATE
HOLZSCHÜTZ

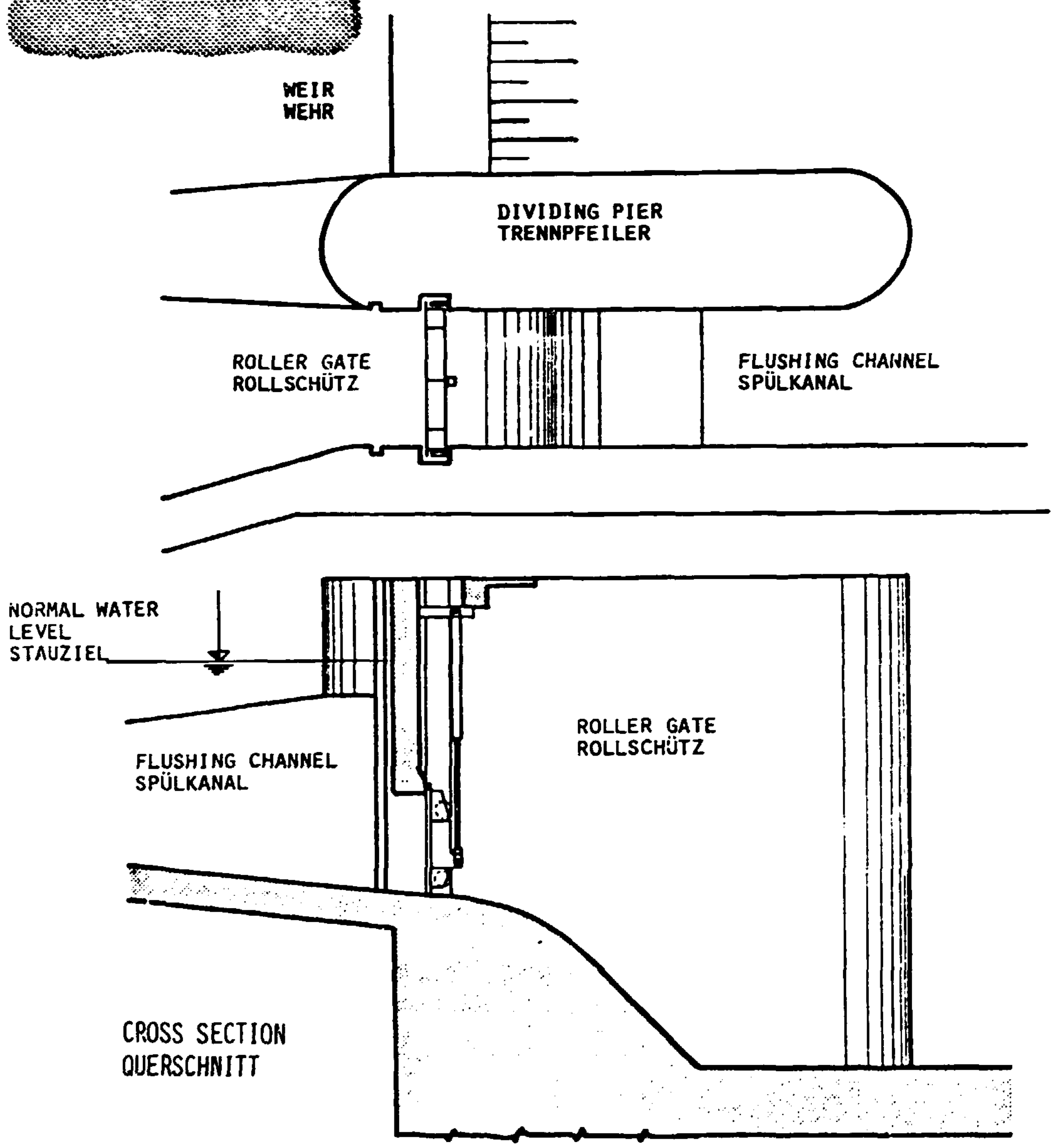


FIG. 8-8 ROLLER GATE
ROLLSCHÜTZ

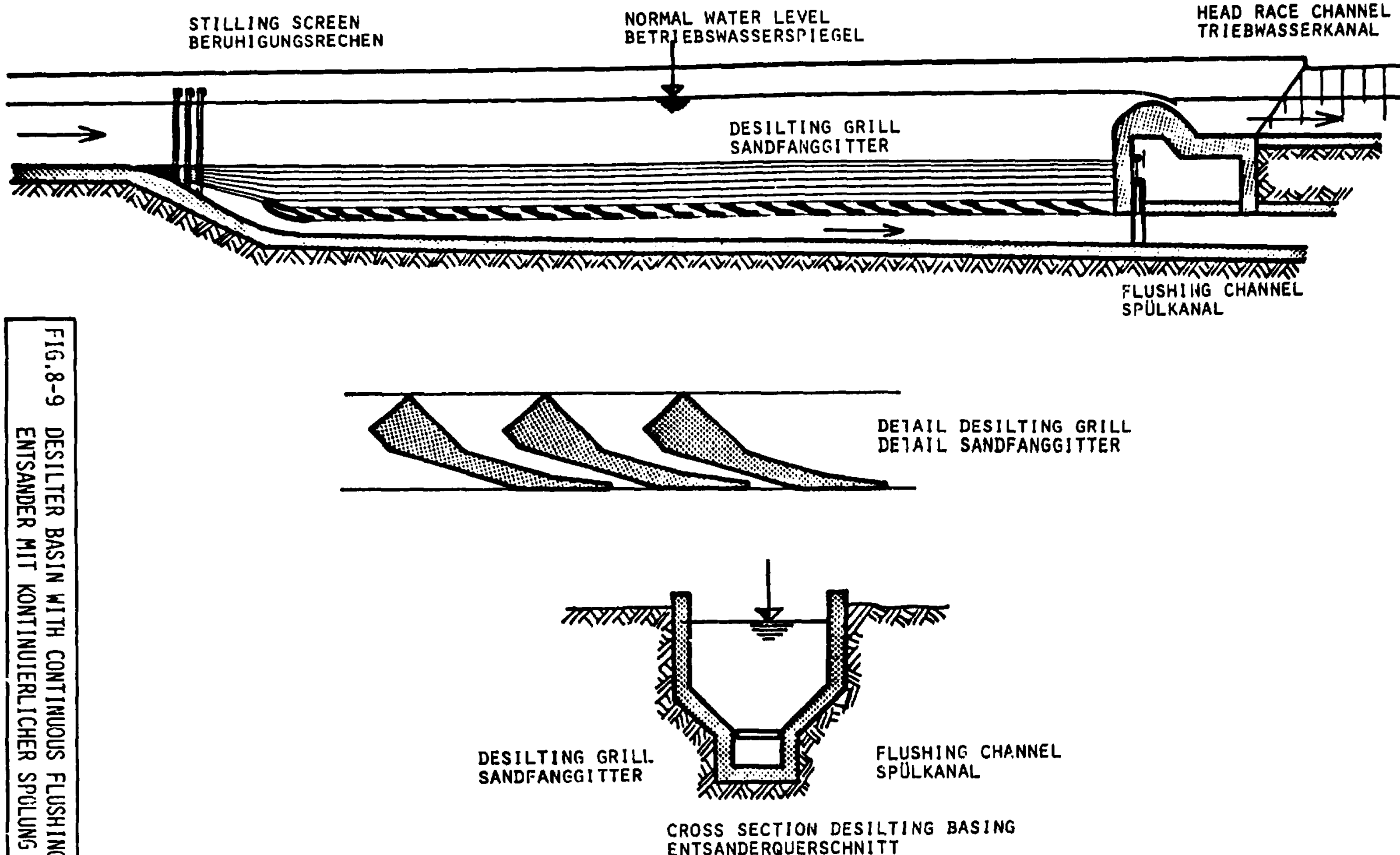


FIG.8-9 DESILTER BASIN WITH CONTINUOUS FLUSHING
ENTSANDER MIT KONTINUIERLICHER SPÜLUNG

8 INTAKE, DESILTER; LOW HEAD
FASSUNG, ENTSANDER; NIEDERDRUCK

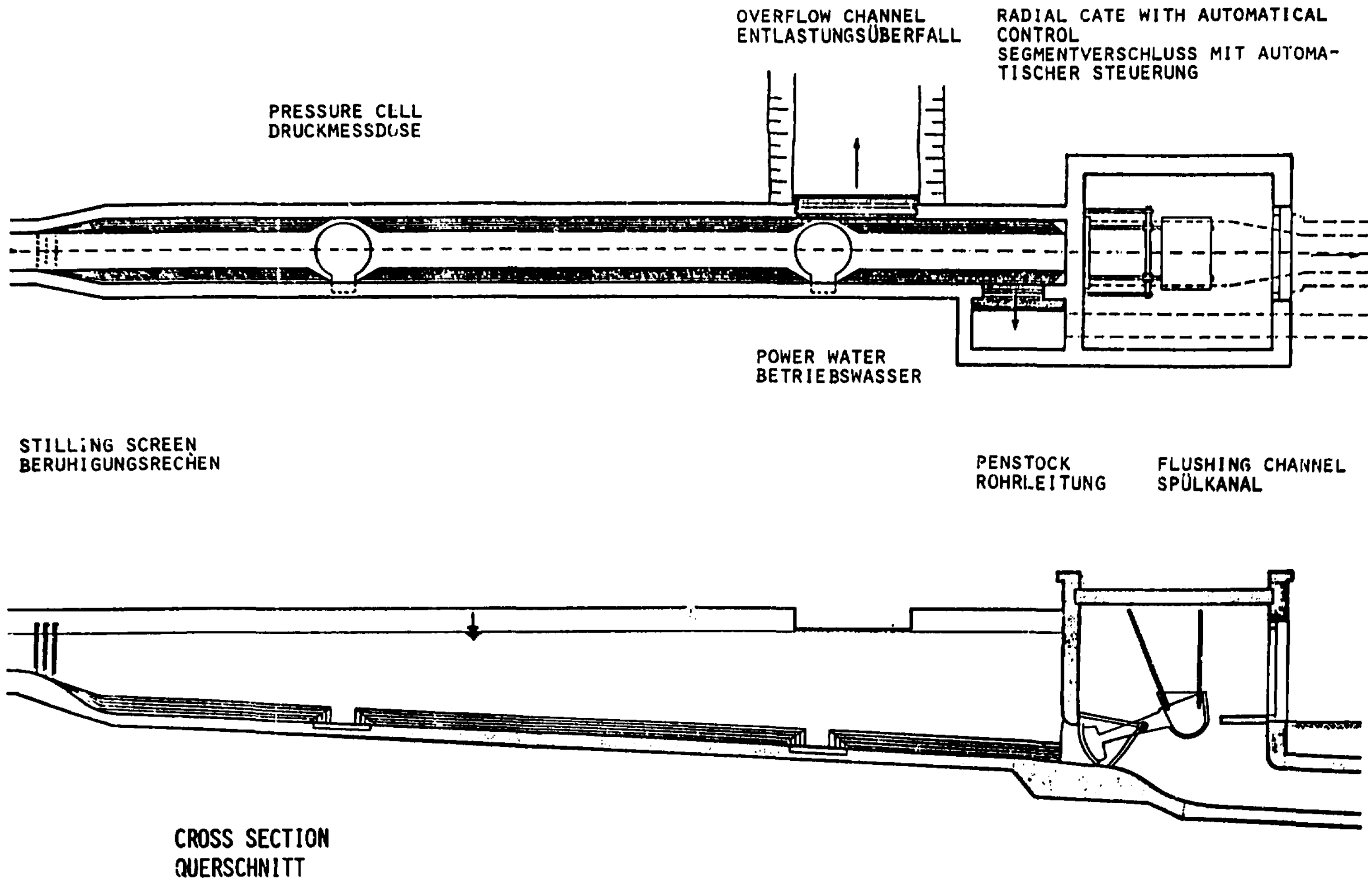


FIG.8 - 1o DESILTER BASIN WITH INTERMITTENT FLUSHING
ENTSANDER MIT INTERMITTIERENDER SPÜLUNG

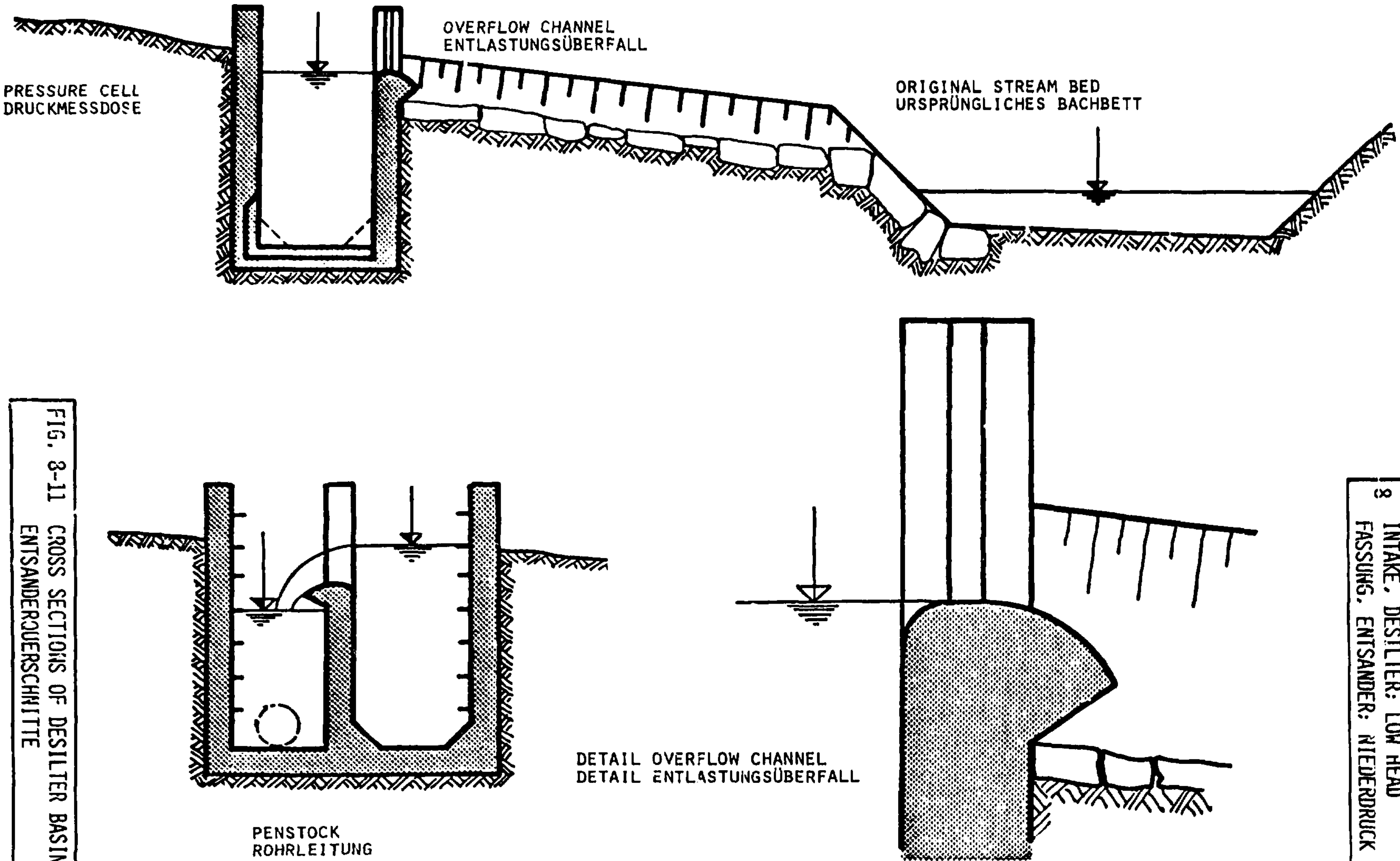
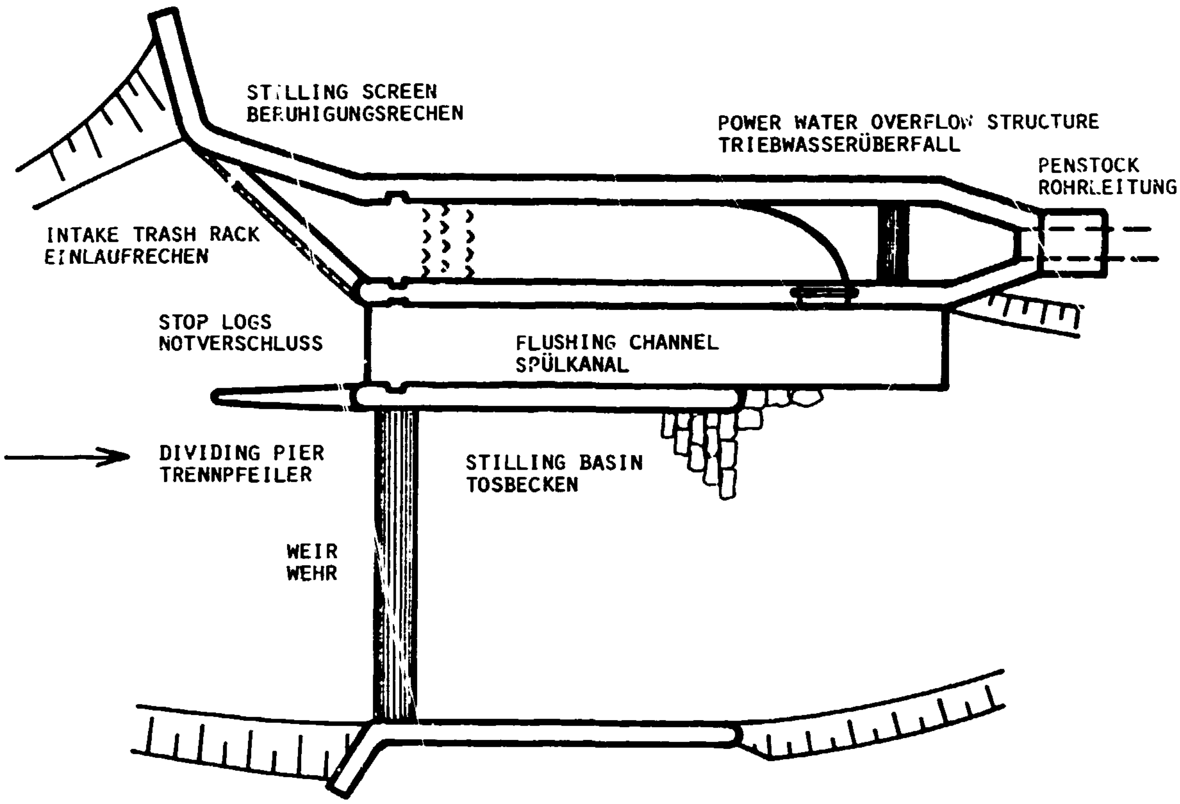


FIG. 8-11 CROSS SECTIONS OF DESILTER BASIN
ENTSANDERQUERSCHNITTE



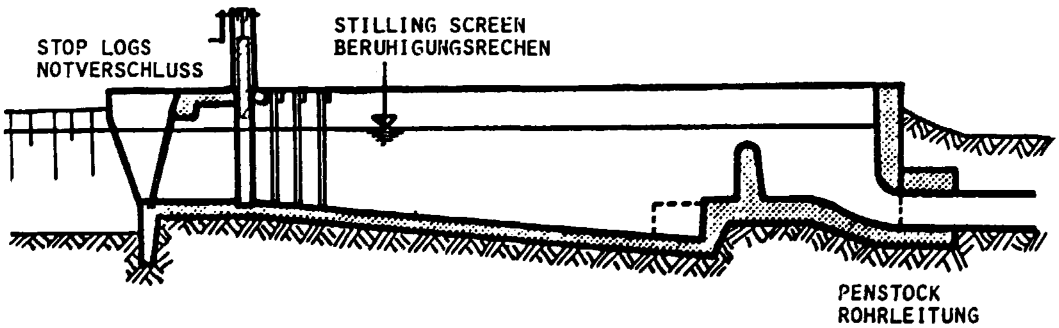
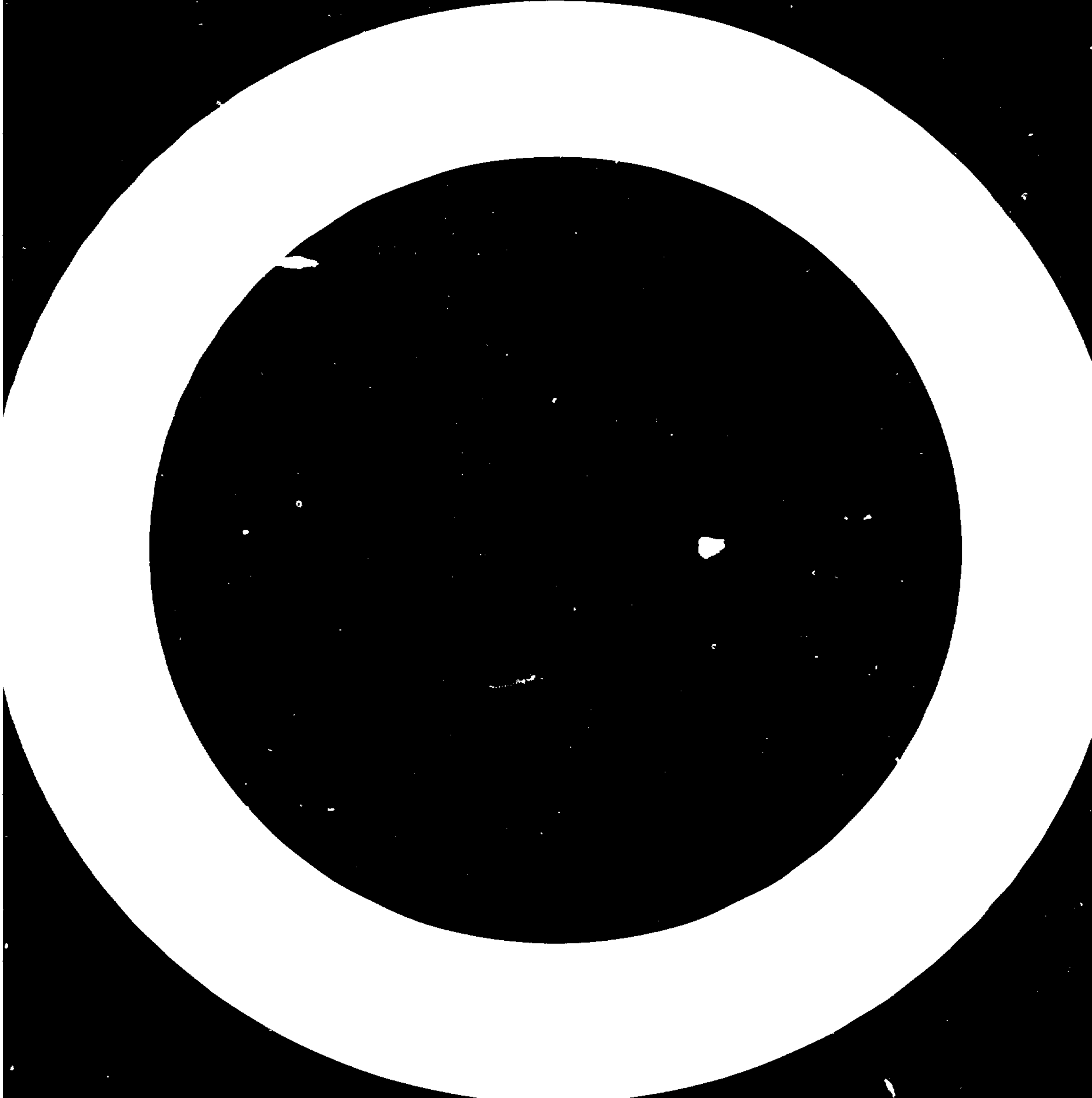


FIG. 8-12 WATER INTAKE WITH DESILTER BASIN
WASSERFASSUNG MIT ENTSANDER



9 WATER CONVEYANCE STRUCTURES IN LOW-HEAD HYDRO-DEVELOPMENTS

RADLER S.

ABSTRACT:

The typical water conveyance facility in low-head power developments is the open water channel. The point at which flow is developed for energy generation divides the waterway into an upstream part and a downstream part. This division is made according to topographical and geological aspects as well as in terms of water management and is reflected by economic considerations in that the upstream conveyance structures are usually concrete-lined whereas the downstream part, or tailrace, is normally not.

The waterways of mini hydro developments are generally subject to the same rules, design principles and construction materials as apply for other hydro plants.Special designs and construction methods may sometimes be dictated by the unusual characteristics of a project site. Departures from conventional designs are possible in view of the small magnitude of the design data involved.

9.1 GENERAL

The waterway in a power development comprises the intake works and the upstream conveyance structures, whose function consists in conveying the flow with minimum loss and turbulence from the intake - as e.g. the reservoir - to the power station and from there to potential downstream utilisation or back into the natural stream bed. To account for the particular importance of power intakes in mini hydro developments, this subject is dealt with in a separate report (8).

In low-head storage stations, that is to say, in power schemes where the power station is located in close proximity to a dam so as to derive its head from the impounded water only, the waterway will be short. Characteristic locations of such developments are rivers abounding in water and with gentle slopes. Hence, their proportion is small in mini hydro developments. Depending on the design (compact, open-layout,submergible), there are three possibilities of construction.

In storage-and-diversion power schemes, however, head can be gained from the water impounded at the intake and from savings in conveyance losses along the diversion, at very variable ratios. By-passing a stream by a diversion is of interest only where provision of an artificial channel allows substantial savings in head as compared with the natural channel. This is the case in a small stream with a fairly straight course and a large natural gradient (2 ‰) or in strongly meandering streams with small gradients. Optimal development of a head is possible in cases where a great slope is combined with a meandering stream course.

9.2 PROJECT STUDIES

Cross section and alignment are fixed on the basis of considerations of hydraulics and construction engineering, which may sometimes be conflicting. The dimensions and the geometry are adopted on the basis of technical and economic aspects. Topography and discharge govern the decision on whether the upstream waterway should be constructed as an open channel, a free-flow tunnel or a pressure tunnel.

9.2.1 Cross Section and Alignment

The hydraulic performance of a cross section is expressed by its "hydraulic quality" (fig. 9 - 1).A cross section represents a hydraulic optimum if its area has a minimum wetted perimeter, and that is - ideally - the half-circle. The equal-leg trapezoid is nearest to this.

The cross section of open channels largely depends on the type of ground, the configuration of the ground and the type of lining required, or desirable from the hydraulic point of view (fig. 9 - 2). In addition, allowance must be made for compulsory points such as intersections with transport routes or streams.

The layout will be fixed on a contour map. If an open channel is adopted, a rough preliminary layout with a straightforward alignment will be developed first, whereas accurate fitting into the topography will then be accomplished on transverse profiles. In the case of free-flow tunnels, which are recommended only under special circumstances, the slope will be fixed in accurate conformity with the friction slope of the rated discharge. Where penstocks are chosen, it should be made sure that even under unusual operating conditions, the hydraulic grade line never falls below the tunnel roof.

Where possible, the waterway should be given a uniform characteristic profile over its whole length. Changes in cross section involve energy loss and cause delays in construction. Where the configuration of the ground calls for changes in cross section, transitions should be long and of suitable shape so as to ensure that the water jet never separates. Fig. 9 - 3 shows several transition designs safe from jet separation. These can be translated to any desired cross section.

9.2.2 Sealing

The choice of the type of sealing for the waterway is mainly governed by the connexions with the water table. The additional lining cost involved on the one hand should be compared with the smaller cross section, the smaller friction losses and the safety of the channel wall from erosion on the other hand. Fig. 9 - 4 shows basic possibilities. Sealing by means of a continuous internal lining (fig. 9 - 4a) presents all the above mentioned merits. Where the channel is founded on alluvial deposits or other types of loose material, and imperviousness can be accomplished by means that are justified from the economic and engineering points of view, protection from the natural water table can be accomplished also by other measures (fig. 9 - 4 b and 9 - 4 c). Such a situation is found e.g. in the case of diversions following relatively shallow impounding basins without impervious elements and where the water table of the accompanying terraces is high in relation to the reservoir water level. The cross sections of such channels should be designed with account being taken of these high ground water levels, which will result in very wide and shallow shapes. Installation of a continuous internal lining in the zone affected by water table fluctuations will in this case present problems not only because dewatering is necessary to allow construction operations, but also with a view to subsequent shutting-off possibilities. It should be noted in this context that the asphaltic concrete lining withstands an uplift pressure not greater than that corresponding to about 2.5 times the lining thickness. Hence, a lining 15 cm in thickness can at best resist an excess head of several decimetres.

The downstream waterway returning the flow to the natural stream bed will in most cases consist of an unlined excavated channel, whose design must allow for the water table of the stream. The slope inclination will depend on the type of ground and the stability of the foundation, and will range from 1 to 0.2 (rock) through approximately 1 to 2 (gravel) to 1 to 4 (loam, drift sand etc.).

9.2.3 Design and Location

The design usually adopted for low-head schemes is the open channel. It is only in exceptional cases that the provision of free-flow tunnels and, more rarely still, pressure tunnels is required.

Free-flow tunnels are cheaper than pressure tunnels. Their longitudinal profile is dictated by the hydraulic requirements. Free-flow tunnels will mainly be adopted for normal and transmountain stream diversions into a reservoir, where very small flow variations and insignificant changes in intake water level are involved, that is to say, especially downstream of stream intakes, e.g. through a "TYROLEAN WEIR". The merits of the free-flow tunnel consist in the smaller loading of the rock mass and the more favourable hydraulic behaviour; but it has the disadvantage that its discharge capacity is determined by its longitudinal slope being chosen for the rated discharge only and that, therefore, it is difficult or impossible to adjust it to the changing requirements of power plant operation. Linings for free-flow tunnels are intended not only to reduce friction losses but also to withstand ground pressures and to prevent leakage. An additional reason for the unwillingness of engineers to adopt free-flow tunnels for low-head power schemes is the fact that construction economy calls for a certain minimum excavation section (6 to 8 m^2), which is uneconomical in view of the cross section actually required. Use of tunnelling machines, which work the ground by means of disc or cone bits, is unprofitable in view of the usually minor excavation lengths involved.

Pressure tunnels usually have small slopes. The lining is primarily intended to withstand the internal pressures during operation or, when empty, to resist joint water pressures. Construction of pressure tunnels presents greater problems. The longitudinal profile of its alignment can largely be adopted at random. Besides unlined structures in very competent rock, simple concrete or reinforced-concrete linings are the usual practice.

9.3 HYDRAULIC DESIGN

9.3.1 Friction Losses

Open channels and free-flow tunnels will yield optimal results only if they are designed for the rated discharge, the cross section, the actual wall roughness and the resulting friction slope. This is accomplished by assuming a uniform steady state for the generally turbulent, streaming flow. Hence, for a given cross section adopted as a basis for the design, with the discharge being fixed and the wall roughness assumed, the friction slope is calculated by choosing different flow velocities (or filling depths).For the purposes of approximate calculations and preliminary dimensioning, use of the GAUKLER-MANNING-STRICKLER formula

$$v = k_s \cdot R^{2/3} \cdot I^{1/2} \qquad \dots\dots\dots\dots (1)$$

will give sastisfactory results. The notations are:

v ... flow velocity in the channel at full load, in terms of m/s

I ... slope of the energy line for uniform movement = = slope of water surface = invert slope

R ... hydraulic radius = F/U, where

F .. water-filled cross section in terms of m^2

U ... wetted perimeter, in terms of m

k_s ... roughness coefficient.

The value k_s varies between 15 for very rough, small channels and 100 for large smooth channels. Guide values for the k_s value:

Natural water courses:	20 to 40
Earth channels:	20 to 60
Rock channels:	15 to 30
Masonry channels:	45 to 80
Concrete channels:	50 to 100
Wooden channels:	65 to 95
Steel plate channels:	65 to 95
Asphaltic concrete:	70 to 80

The large variation of this value suggests that clear definition of the roughness coefficient is difficult. Due to the dimensional impurity of the formula, the absolute value of roughness also varies with the size of the channel and the REYNOLDS number

$$R_e = \frac{v \cdot d}{\nu} = \frac{v \cdot 4R}{\nu} \qquad \dots\dots\dots\dots (2)$$

where:

ν ... kinematic viscosity, in terms of m^2/s.
This is a function of the water temperature and is approx.
$1.8 \cdot 10^{-6}$ for 0° C, $1.3 \cdot 10^{-6}$ for 10° C and $1.0 \cdot 10^{-6}$ m^2/s for 20°C.

Therefore reliable statements regarding the roughness coefficient k_s according to STRICKLER can be made only for similar channels having the same dimensions, whose roughness coefficients have been derived from measurements.

A much more accurate determination of the friction losses is possible using DARCY's formula

$$h_r = I \cdot l = \lambda \cdot \frac{v^2}{2g} \cdot \frac{l}{d} \quad \ldots\ldots\ldots\ldots (3)$$

for pipe lines.
The notations used are:

λ ... friction coefficient dependent on the channel wall and the geometry of the cross section

d ... diameter of the pressure pipe line or, in open channels, four times the hydraulic radius (4R).

Calculation of the λ value is based on the roughness quantity "s", also called "absolute, equivalent or technical roughness". This quantity essentially corresponds to the actual irregularities of the channel walls, whether these are accurately worked steel surfaces (s = 0.01 mm) or the gravel surface of a stream bed (dm = 5 to 15 cm). NIKURADSE's formula is used to determine the friction coefficient via the relative roughness ($\frac{d}{s}$ or $\frac{4R}{s}$):

$$\frac{1}{\sqrt{\lambda}} = 1.14 + 2 \log \frac{d}{s} \quad \ldots\ldots\ldots\ldots (4)$$

The values of the roughness quantity s can be calculated with much greater accuracy and, moreover, incorrect estimation of the s value has a much smaller effect on the friction loss:

Steel penstock (welded):	0.01	to	0.1	mm
Steel penstock (riveted):	0.1	to	2	mm
Plastic pipe - smooth:	0.001	to	0.01	mm
Asphaltic concrete (rolled):	0.2	to	0.5	mm
Concrete (steel formwork):	0.2	to	0.6	mm
Concrete (planed formwork):	0.5			mm
Concrete (rough formwork):	1	to	2	mm
Wooden flume (joints in the direction of flow), planed:	0.2	to	0.5	mm
Wooden flume (rough):	0.5	to	2	mm
Gneiss slabs (flush joints):	5	to	10	mm
Meadow:	10	to	20	mm
Placed stones:	20	to	40	mm
Riprap:	30	to	70	mm
Natural stream course:	average bed load grain size (d_m)			

The coefficient of STRICKLER's formula and DARCY's formula are connected by the relation

$$k_s = \frac{1}{R^{1/6}} \cdot \sqrt{\frac{8g}{\lambda}} \quad \ldots\ldots\ldots\ldots (5)$$

Since the STRICKLER formula is easy to handle, it is useful to estimate the s value and to find, by means of the above relation and on the basis of fig. 9 - 5, the value k_s as a function of the cross sectional geometry. This calculation can be carried out according to the STRICKLER formula.

9.3.2 Surge and Trough Analysis

After determining the longitudinal slope and the water depth at a selected cross section, we will fix the freeboard, that is to say, the vertical distance between the normal operating level and the top of the channel. For short channels, it is the usual practice to adopt a horizontal crest for the longitudinal embankment. In the case of long channels, however, it may be of advantage to make the dam crest parallel the invert to reduce construction material requirements. Here allowance must, however, be made for the fact that for flows < Q_A (rated discharge) a smaller energy gradient predominates so that the water level may rise into the space reserved as freeboard, even to the extent of overtopping the dam. In such a case, energy-dissipating measures should be taken at the inlet, i.e.either operation under a lower headwater level or flow reduction by means of a gate.

Apart from the above considerations, exceptional operating conditions, in particular start-up and shut-down of the turbines, should be taken into account for the dimensioning of the headrace. Causes of surge and trough are shown on fig. 9 - 6.

As regards turbine engagement (start-up), a time pattern should be determined by the electrical and mechanical equipment manufacturers .The start-up time of any type of hydro machinery is usually very short and almost exclusively dependent on the capacity of the water conveyance structures.

This is clearly reflected by the approximate formula for the theoretical regulating time

$$t = \frac{l \cdot v}{g \cdot h} \qquad \ldots\ldots\ldots\ldots (6)$$

where

t ... time, in terms of sec
l ... length, in terms of m
v ... velocity, in terms of m/sec
g ... acceleration (9.81 m/sec^2)
h ... net head, in terms of m

The longer the waterway and the smaller the head, the clumsier the plant. This is also why diversion-type power schemes with unfavourable head-length ratios call for the intercalation of a surge tank (at the downstream end of a tunnel) or a compensating basin (at the downstream end of a channel) in order to reduce the regulating time. This will balance sudden lack, or instantaneous surplus, of water in the event of transition from the state of rest to the state of operation, or vice versa.

Therefore, the size of the compensating basin should be determined as a function of the opening time of the turbine. Basicly, turbine start-up generates a trough which travels upstream at a speed of approximately

$$v_w = \sqrt{g \cdot t} \qquad \ldots\ldots\ldots\ldots (7)$$

where:

t ... water depth, in terms of m
g ... acceleration.

This wave velocity also characterises the transition from streaming flow to shooting flow. The full supply flow is not mobilised until the trough has arrived at the intake and the discharge capacity so produced becomes effective due to the difference between the compensating-basin and water-intake levels.

The magnitude of the trough forming in a channel with a trapezoidal cross section can be determined accurately enough, by iteration, from the nomogram shown on fig. 9 - 7.

The travel time the trough takes up to the weir and the initial depth of the trough can be used for deriving the start-up time of the turbine (engagement time) or, if a very short start-up time is necessary for reasons of power supply, for calculating the required size of the compensating basin.

The reverse process takes place when the plant working under full load must suddenly be shut down due to a defect on the generator, turbine or transmission line. The water hammer so generated propagates at a speed of approx. 1000 m/sec through the upstream waterway (penstock, pressure shaft or turbine inlet) to the compensating basin, where it produces a surge of height z_o due to power water continuing to flow into the headrace. The height of the surge is again easily determined for a trapezoidal cross section, on the basis of fig. 9 - 8. The surge so formed now propagates upstream, at the above-mentioned wave velocity minus inflow velocity into the channel, impounding behind it the power water continuing to arrive, by filling the freeboard space. The inflow of power water into the waterway will not be reduced until the surge has arrived at the intake and, due to the equalisation of heads, there is no longer any cause for water to flow in. Consequently, for a first approximation, we must calculate the travel time of the surge up to the intake works, and use this to calculate the volume of power water that has meanwhile entered the channel. The final surge height is obtained by iteration from the freeboard volume that has been filled with water with a horizontal surface. It is for this final surge height that the freeboard of a headrace should be designed unless the compensating basin is equipped with a spillway.

9.4 CONSTRUCTION MATERIALS

As in all the other design considerations, the construction materials to be adopted for waterways in mini hydro schemes will be the same as those used in large-scale hydro development. However, particularly for the open channel, which is the typical water conveyance structure in the mini hydro scheme, the smallness of the cross section may allow special construction methods that cannot be applied in large developments.

9.4.1 Concrete and Asphaltic Concrete

The conventional design of a stabilised headrace used to be the flat trapezoid (maximum slope, 1 to 1.5). Its rough grade, whether in cut or embankment sections, was stabilised and provided with a concrete lining 12 to 25 cm in thickness. Concrete was placed by sections. As a rule, the invert was completed first for subsequent use as a runway for transport vehicles. Then the slopes were produced in vertical strips 3 to 5 m in length. To allow for the setting process, the pouring sections were not completed in a continuous row. In a first operation, every second section was concreted and after setting, the intermediate sections of equal size were closed. As a rule, construction joints did not receive any subsequent treatment. Where expansion joints were left, these had subsequently to be closed by mastic asphalt or another joint sealing mastic.

The steep trapezoid (fig. 9 - 2) should be mentioned in connexion with concrete retaining walls in embankment sections or with revetment walls or other lining or stabilisation measures in cut sections made in rock.

The concrete lining has lately been replaced almost totally by the asphaltic concrete construction method. It is again the flat trapezoidal profile with slopes not exceeding 1 to 1.5 to which an exact rough grade is produced. For stabilisation, the slopes are in many cases sprayed with bitumen or treated with other chemical means of stabilisation. The one or two layer asphaltic concrete lining is then applied continuously. Rolling-in to compact the mix results in a very smooth surface, which compares favourably with concrete linings from the hydraulic point of view.

9.4.2 Stone Slabs

Use of concrete or asphaltic concrete may be impossible at very exposed locations. Then one will have to resort to local materials. Especially good results have been obtained from gneiss, in particular gneissic slate or marl slate slabs with thicknesses of between 3 and 8 cm. After completion of the work (excavation and placement of fill) and adequate compaction, the slabs are laid on a levelling course of sand or imbedded in cement mortar and pointed flush with the surface by means of cement mortar or another joint sealing product (mastic asphalt, plastic sealing compound etc.) (fig. 9 - 9).

9.4.3 Timber

The timber linings formerly used for power water or transport channels in mill construction and brine lines are again today preferred for small power stations with short headraces. A suitable shape is the flat rectangular cross section. The timber, serving either as a loadbearing element or only as a simple lining, is joined by means of slits and feather-joint tongues and laid in the direction of flow. In this type of lining, problems result primarily from the short service life (5 to 12 years, depending on the type of wood) and from potential drying for a major period.

9.4.4 Other Materials

Mention should here be made of the possibility of accomplishing the sealing effect by means of simple plastic foils. Fig. 9 - 10 shows a plastic foil which is laid in sufficiently large widths (of at least several metres) on the rough grade - a circular or parabolic cross section will suit best - and covered with gravel at least o.3 m thick. This method can be applied only to channels with very low flow velocities (not exceeding o.7 m/s), where erosion is largely avoided.

9.5 CONCLUSIONS

The alignment as well as the choice of cross section and material are essential factors in the design of waterways for diversion-type low-head power schemes. Therefore, during the project studies, the flow of information should always proceed from the general layout, through the selection of material, the properties of construction materials and the hydraulic analyses, to economic studies in order to ensure optimal solutions. Basic detailed optimisations, e.g. for a given diversion, should be based on a division into upstream and downstream sections. Usually this decision clearly follows from the site topography. The problem will be complex where it is possible to vary the water level at the intake, which results in several alternative waterway alignments. A decision will in this case have to be based on economic studies, comparing the present power tariff on the profits side with the capital structure in connexion with the present rate of interest on the expenditure side. The fact that both the power tariffs and the rates of interest are subject to constant variation demonstrates the necessity of allowing for this range of fluctuation in the economic studies.

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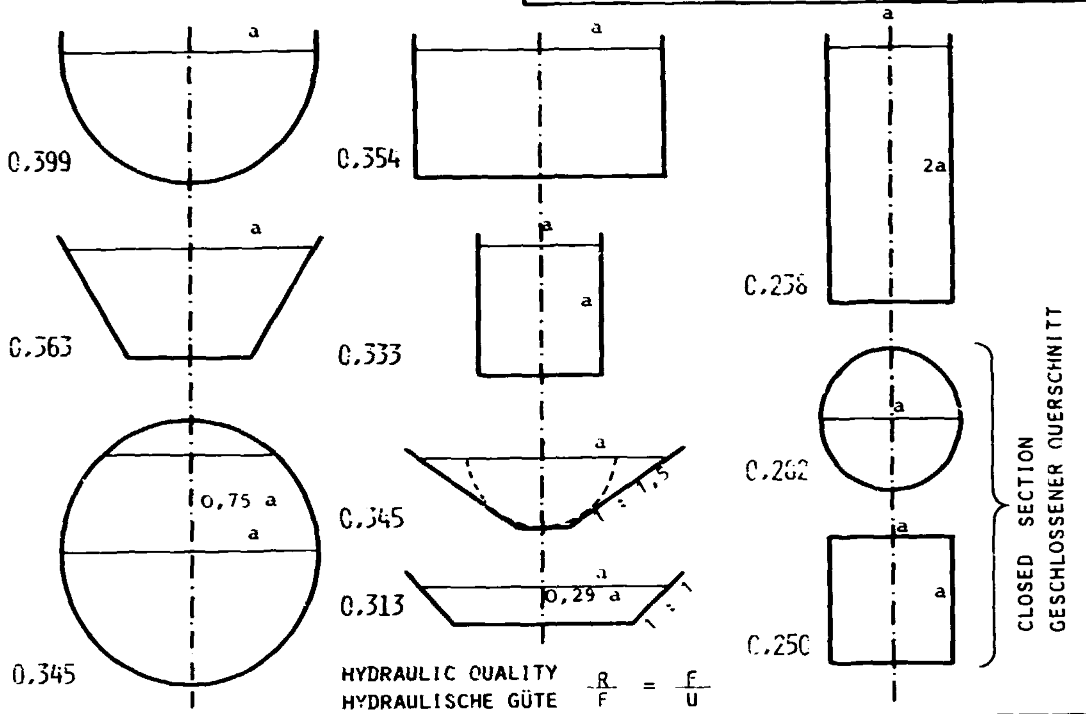


FIG 9 - 1 HYDRAULIC QUALITY
HYDRAULISCHE GÜTE

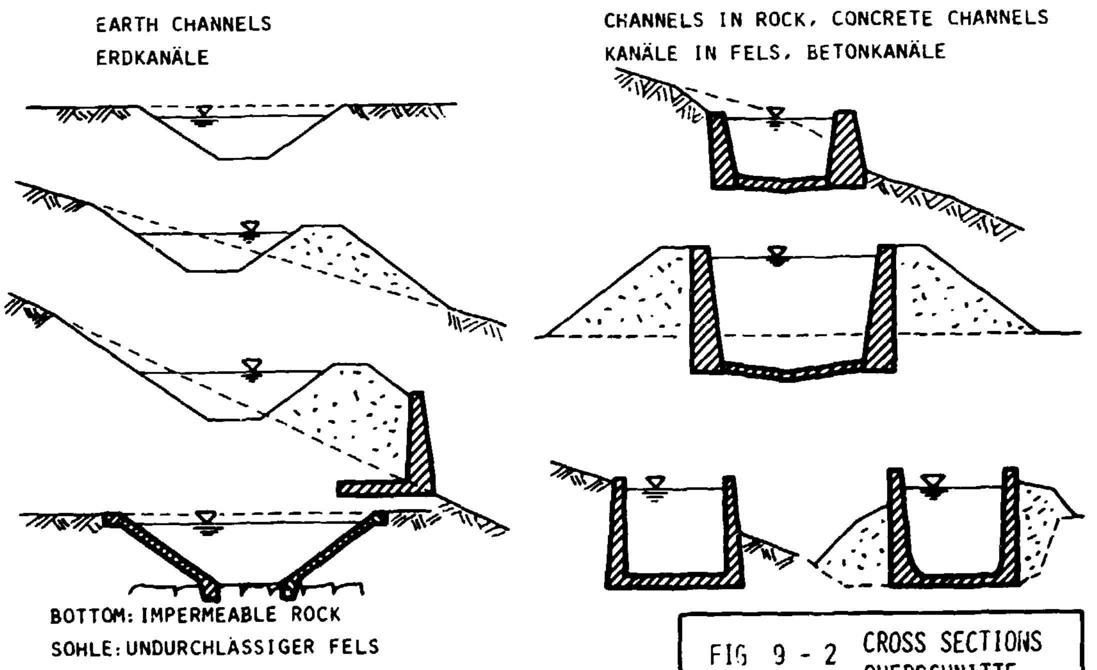


FIG 9 - 2 CROSS SECTIONS
QUERSCHNITTE

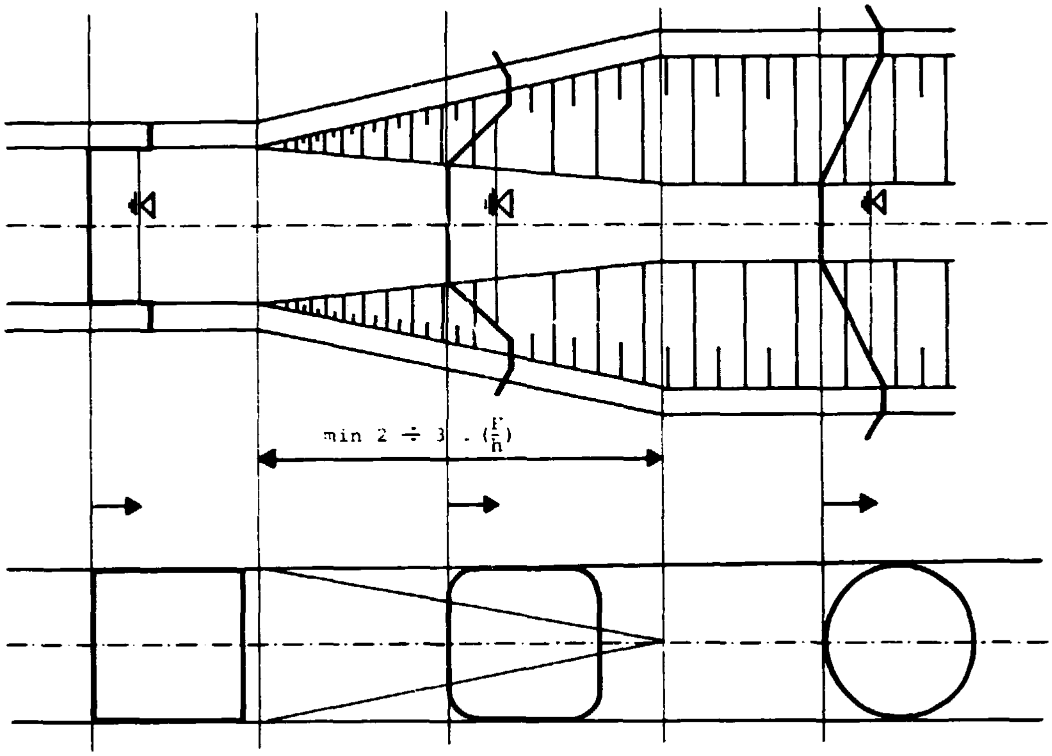


FIG 9 - 3 TRANSITION SECTIONS
ÜBERGANGSSTRECKEN

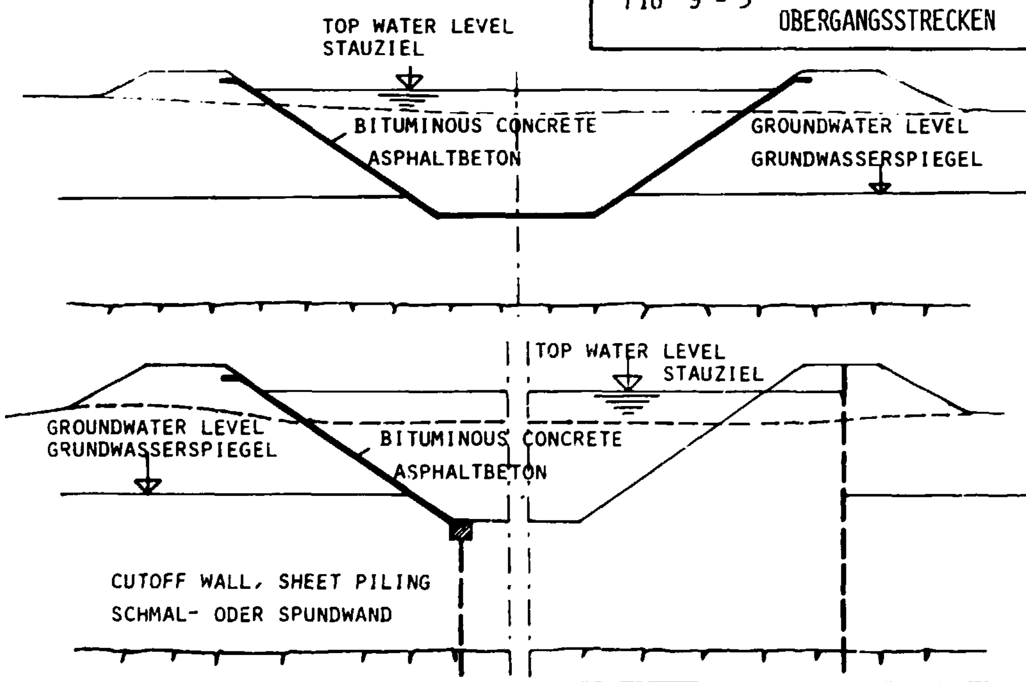
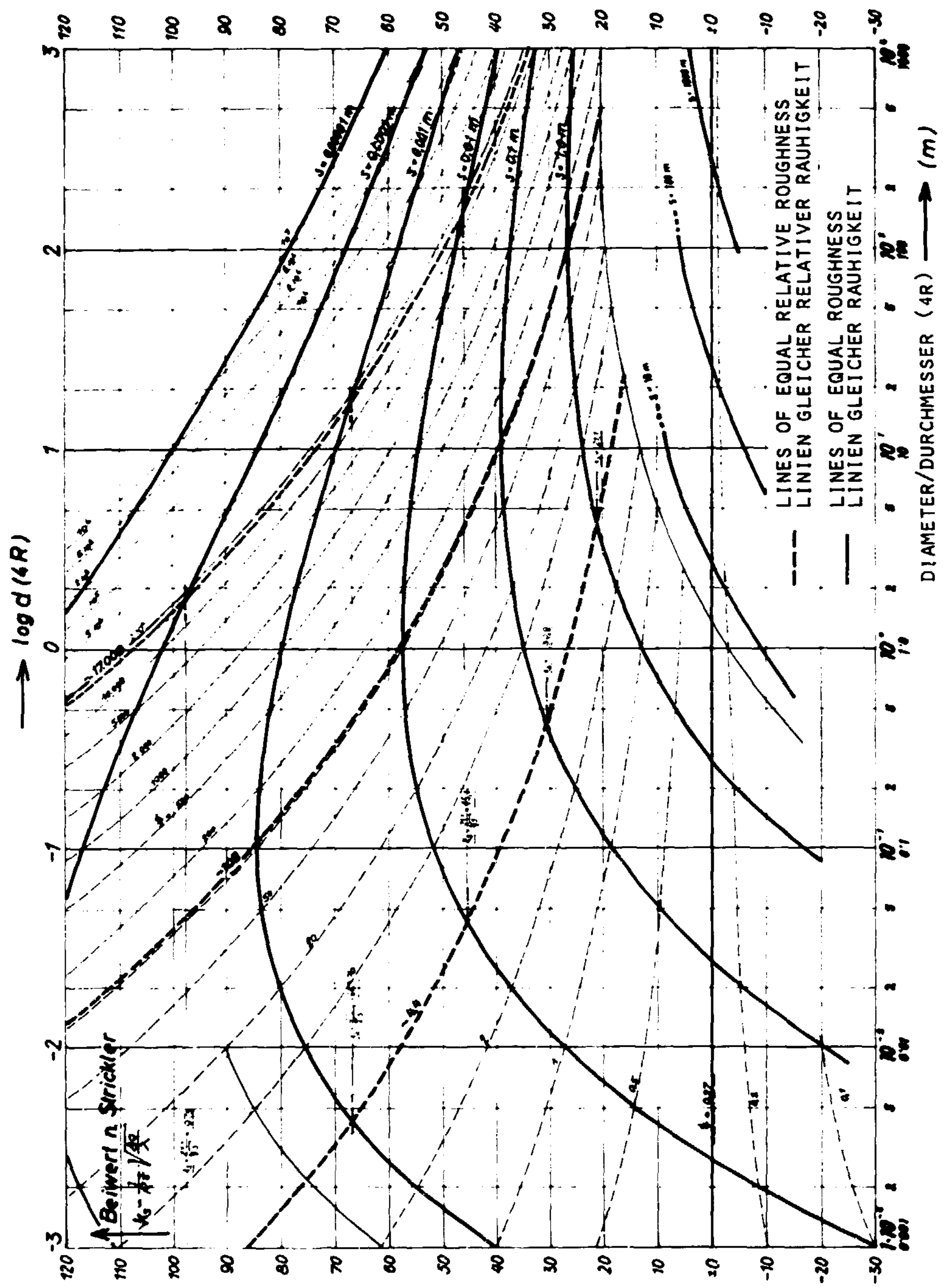


FIG 9 - 4 IMPERVIOUS ELEMENTS
ABDICHTUNGSMÖGLICHKEITEN



RADLER, 1964

FIG 9 - 5 RELATION BETWEEN K_S, S AND R
ZUSAMMENHANG ZWISCHEN K_S, S UND R

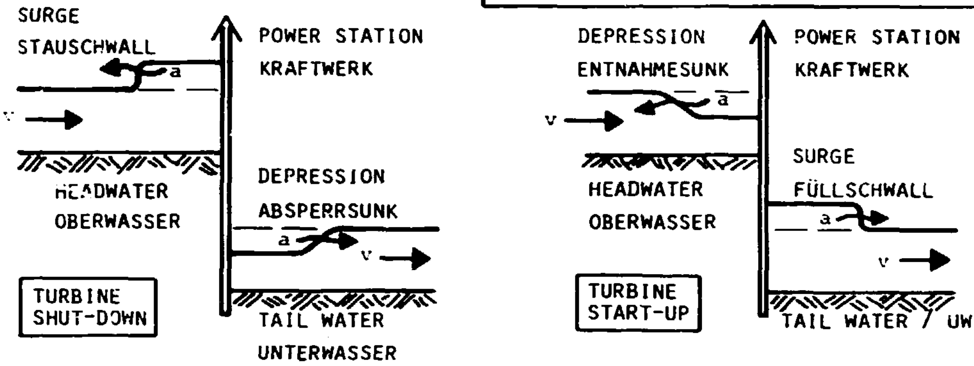
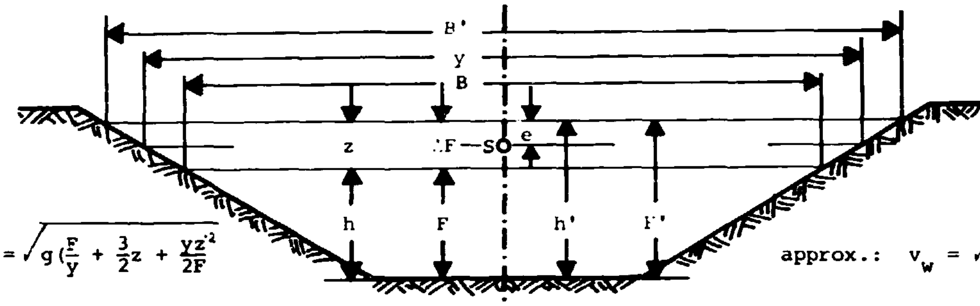


FIG 9 - 6 CAUSES OF SWELL AND SUNK
URSACHEN DER SCHWALL- UND SUNKBILDUNG



$$v_w = \sqrt{g\left(\frac{F}{y} + \frac{3}{2}z + \frac{yz^2}{2F}\right)}$$

approx.: $v_w = \sqrt{g.t}$

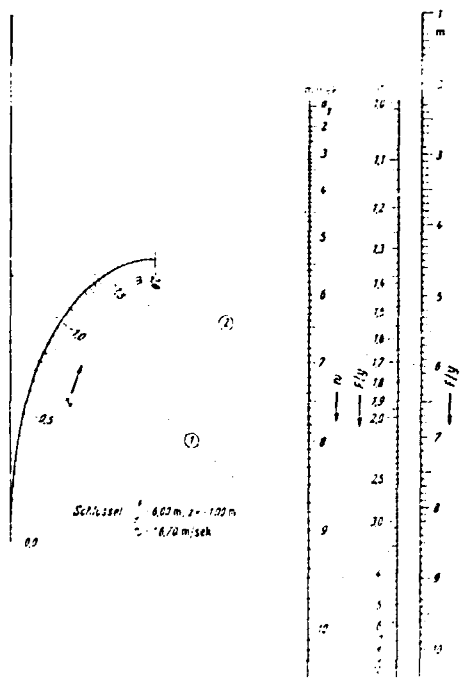


FIG 9 - 7 NOMOGRAPH OF DEPRESSION
NOMOGRAMM SUNK

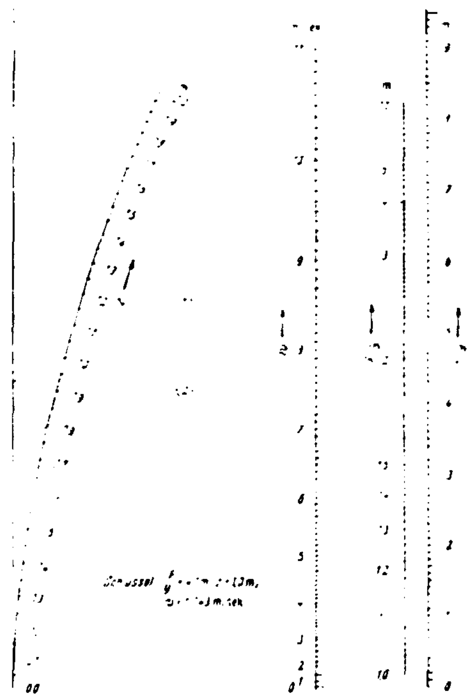


FIG 9 - 8 NOMOGRAPH OF SURGE
NOMOGRAMM SCHWALL

FRANK, 1957

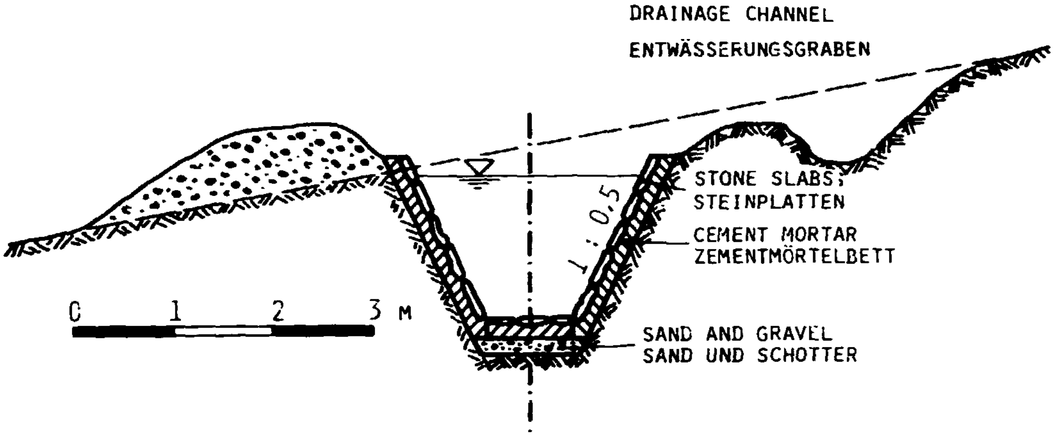


FIG 9 - 9 TYPICAL SECTION
REGELQUERSCHNITT

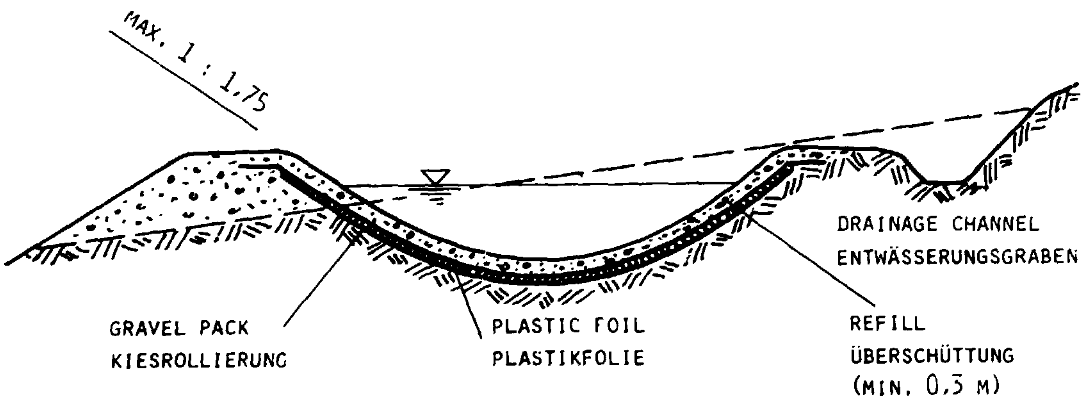


FIG 9 - 10 PLASTIC FOIL SEALING
ABDICHTUNG MITTELS PLASTIKFOLIE

10 WATERWAY AND SPECIFIC PROBLEMS OF HIGH-HEAD SCHEMES

KELLER S.

ABSTRACT:

High-head hydro schemes develop relatively small flows under relatively high heads. Such installations will mainly be located at medium and high altitudes, that is to say, in sparsely populated mountainous regions. The preferred power generating machine is the impulse turbine with small jets. This involves special problems relating to desilting and prevention of water contamination as well as to the protection of hydraulic machinery from sand and excessive wear. Possible solutions are presented. Fields of economical application of pressure pipe are defined and suitable types of pipe and raw materials are suggested. Ductile cast-iron pipe lines are discussed in detail.

10.1 DEFINITION OF HIGH-HEAD SCHEMES AND DESCRIPTION OF WATERWAYS

If "h" denotes the useful head and "Q" the rated discharge, mini hydro schemes can be defined as high-head plants according to the rule-of-thumb

$$\frac{h}{\sqrt[3]{Q}} \geq 100$$

where h .... net head in terms of m
Q .... rated discharge in terms of m^3/s

Figure 10-1 shows this region delimited by a dash-dotted line. The waterways of such hydro schemes are usually characterised by great lengths and relatively small discharge diameters. In most cases, these will be pipe lines of plastic, asbestos cement or iron functioning under high internal pressure. In exceptional cases, wood or concrete piping may be used.

For temporary use or in installations of minor size, such pressure pipe lines will be laid above-ground, whereas in final plants calling for maximum safety in operation, pipes should be laid in trenches and be backfilled. In exceptional cases, power water conveyance structures may be laid in (accessible) ducts (collector ducts), in tunnels or in drill holes.

Whereas in low-head schemes, the electrical and mechanical equipment is the central plant component accounting for the greater proportion of the total cost, this place is taken by the waterway in high-head developments. Careful design and construction of the penstock will be a prime consideration as this has an important bearing on the economy of the whole project. This subject will be dealt with in greater detail in the following chapters.

10.2 CHOICE OF THE RAW MATERIAL FOR THE WATER CONVEYANCE STRUCTURES

In most moderate-sized high-head schemes, the penstock will take the central place in the design considerations. It connects the intake works with the powerhouse. Choice of the suitable raw material is essential. This will affect not only the total construction cost, but also safety in operation and service life, and hence the utilisation period of the whole power scheme.

10.2.1 Effects on Design and Dimensioning

The following effects constitute essential factors for, and should govern the choice of the design and dimensioning of pressure pipe lines:

2.1.1 Accessibility of the site, costs of transport route construction and transport
2.1.2 Rated discharge "Q"
2.1.3 Hydrostatic pressure line, station head or gross head "h"
2.1.4 Hydrodynamic pressure line with the limits "h_{min}" and "h_{max}"
2.1.5 Ratio h:Q
2.1.6 Horizontal length of penstock "l"
2.1.7 Ratio l:h. The smaller the ratio, the better. Values of 1....2 are ideal and allow negligence of effect 2.1.4. With values l:h > 10 and major rated discharges, a surge tank should be intercalated if possible. Provision of air receivers as was the former practice is not recommended for reasons of safety.
2.1.8 Geometry of the longitudinal section through the pressure pipe alignment - convex or concave
2.1.9 Slope inclination and maximum slope of penstock alignment
2.1.10 Mean and maximum cross falls on steep slopes, as compulsory factors
2.1.11 Natural and artificial obstacles on rough ground and/or in built-up areas, existing structures etc.
2.1.12 Stability of foundation along the adopted alignment (firm ground, slowly creeping slope, risk of releasing failure, risk of reducing the natural angle of repose in steeply sloping ground by inevitable construction measures, disturbance of the neutral equilibrium on steep slopes etc.)
2.1.13 Potentially aggressive chemical action of the foundation material in the case of buried pipes
2.1.14 Potentially aggressive chemical action of the power water including inevitable abrasion risks, and
2.1.15 Resistance to severe abrasion from sand-bearing power water.

Fig. 10-1 is a diagram showing economical ranges of application of various pipe materials under normal geological and hydrological conditions as e.g. prevail at medium and high altitudes in the European mountains.

10.2.2 Plastic Pipe

Where minimum discharges $Q \leq 0.03\ m^3/s$ with internal hydrostatic pressures $h < 100$ m are involved, flexible plastic pipes of soft polyethylene can be used. Pipe-laying even on rough ground involves no problems. Being UV resistant, plastic pipe may even be laid above-ground. Thus, one of the 15 effects listed above risks to constitute a reason for ruling out this type of pipe, which has also yielded excellent results in the construction of drinking water lines. Due to the substantial pipe wall thicknesses, even the abrasion risk (effect 2.1.15) is negligible.

10.2.3 Plastic and Asbestos Cement Pipes

Where discharges $Q < 0.2\ m^3/s$ and heads $h < 160$ m are concerned, considerations of economy will dictate the use of either rigid polyethylene or PVC pipes or asbestos cement pipe. All these pipes have simple sleeve joints, which,however,offer no resistance to pull. This renders their application limited in respect of effects 2.1.9 to 2.1.12 listed above - if these are unfavourable. Should their use be ruled out by these reasons, spun-type high-pressure iron tubing as described below should be adopted instead.

In addition, where use of the corrodible asbestos cement pipes is considered, the effects listed under items 2.1.13 to 2.1.15 should be allowed for. Severe abrasion (according to 2.1.15) may be a reason for precluding even PVC pipe from consideration.

10.2.4 Spun-Type Ductile Iron Pressure Tubing

For discharges $Q \approx 0.20$ to $\sim 2.0\ m^3/s$ and heads up to $h = 250$, 320 and 400 m, high-strength spun-type iron pressure tubine made of a Fe-C-Si-Mn alloy can be used as an alternative to asbestos cement, which, although low in cost, has many disadvantages. Ductile iron pressure tubing is widely used in Central Europe, not only in drinking water supply systems but increasingly also in mini hydro developments, mainly because it offers long-term safety in operation. The fundamental difference from conventional steel pipe resides in the fact that no welding is necessary during pipe laying.

Two types of pipe are offered all over the world:

(a) Ductile iron pressure tubes with screwed fittings and Tyton joints, which are not resistant to pull; standard diameters range from 80 mm to more than 1000 mm; pipe lengths, about 5.00 m.

(b) Ductile iron pressure tubes with VRS joints, which are pull resistant to the full nominal pressure; standard diameters range from 80 mm to 500 mm at present; pipe lengths, about 5.00 m. The two joint types are shown on figs. 10-4 to 10-5.

Whereas the applications of tubing with screwed fittings are limited in respect of the effects of items 2.1.9, 2.1.10 and 2.1.12 above, tubing with VRS bell- and -spigot joints can also be laid in slopes subject to sliding risks and in unstable soils.

Ductile pressure tubing, although much less vulnerable to the attack of corrosion than steel would be, should be provided with adequate insulation in severely aggressive environments (effects 2.1.13 and 2.1.14).

VRS tubing is at present allowed for internal pressure stages not greater than 25, 32 and 40 bars, but continued development will soon enable tubing of higher permissible internal pressures to be offered on the market.

Supplemental details regarding the properties and applications of ductile iron tubing will be given in chapter 4 below.

10.2.5 Welded Steel Pipe

Welded steel pipe is the last of the pipe groups shown on the application diagram (fig. 10-1). As this is commonly applied in medium-sized and large power developments as well and is amply discussed in the relevant literature, welded steel pipe will not be treated in greater detail here. The transition between ductile pressure tubing and steel is not quite clear. Both types will perform satisfactorily within a relatively wide range of applications, so that a comparative study should be carried out in each individual case. Comparison will always clearly demonstrate that the purchase price of steel pipe is substantially lower than that of ductile iron tubing, whereas the relationship of cost items is reverse for pipe laying. According to the experience gathered so far, the use of ductile pressure tubing with VRS joints will be preferred in unfavourable and difficult terrain in sparsely populated mountainous regions.

10.3 SIMPLE PROTECTION FROM OVERLOADING, CONTAMINATION AND SAND ACCRETION

10.3.1 The Problem

A characteristic feature of high-head mini power schemes is normally a very small rated discharge "Q" emerging from the jet (or jets) of an impulse turbine. This results in jet diameters less than 0.03 m. At partial load the ring-shaped water jet between nozzle and needle shrinks to a thickness of a few millimetres. In order to minimize wear and troubles, it is essential even in mini developments that the water supplied by the turbine be purified in an optimal degree and that provision be made for the safe prevention of sand or mud accretion in the pipe line in unusual inflow situations (e.g. heavy rainfall in the catchment with sudden arrival of major bed load concentrations).

The intake and desilting facilities also used in medium-sized and large power developments have been discussed in Reports 8 and 9. As a supplement to these, a hand operated desilter with an overload protection of extremely simple design and an automatic round-type desilter for small inflows will be discussed in chapters 10.3.2 and 10.3.3 below.

10.3.2 Simple Intake Structure with Overload Protection (See fig. 10-2)

For minor power stations with relatively small inflows, a prefabricated tank of (galvanised) plate will suffice. If the necessary routine servicing (e.g. on a daily or weekly basis) is omitted or if increased bed load transport begins, discharge will be inhibited at the movable scum board (e.g. of wood) installed in the tank, that is to say, inflow is interrupted by water-borne particles deposited under the scum board. The power station breaks down, but inadmissible sedimentation in the penstock and turbine wear are prevented. A similar effect results from clogging of the fine rack in front of the entrance to the penstock.

Fig. 10-2 should be considered as a scaleless typical drawing. In realistic dimensions, the settling tank would have to be extended towards the weir, i.e. beyond the right-hand edge of the page.

10.3.3 Round-type Desilter with Intermittent Flushing (See fig.10-3)

To satisfy the demands of more sophisticated power schemes with small inflows, a surplus-flow controlled automatic flushing facility was developed in the years 1955 to 1963.

Its main functions are as follows: Coming from the weir, the water flows tangentially into the round-type desilter and thus maintains a slowly circulating movement. The sand settles in the direction towards the axis of the desilter. A flushing outlet, which is closed by a vertical pipe, is provided at the lowest point of the basin. When the inflow from the weir is greater than the flow drawn into the penstock, surplus water flows into a suspended tank. A simple mechanism, oscillation-free and reliable in operation, starts the flushing process and is stopped again when the tank, which constitutes a counterweight, is empty. The slightly lowered water level of the round-type desilter is raised again and the next flushing process starts. The time intervals between the individual flushing processes are dependent on the magnitude of inflow into the desilter. During these processes, full turbine supply flow is maintained. The flushing processes are not stopped until the inflow has decreased to or below the flow drawn in.

A helical pure-water duct winding round the vertical pipe prevents clogging in the area of the flushing outlet and above. Backflow during flushing leads to self-cleaning of the fine rack provided the space behind the rack is adequately dimensioned for this purpose.

Conditions of trouble-free function: Turbine discharge must not be much greater than that magnitude of discharge in the water course (stream) which experience has shown to carry the maximum amount to bed load. An inflow-dependent turbine gate opening control that may be required should allow for the water level fluctuations produced by the flushing processes.

Dimensioning of the automatic desilter control can be based on experience gathered during the operation of existing installations. This refers to the hydrological data and the characteristics of the stream to be developed.

Fig. 10-3 is a scaleless drawing demonstrating functions. Realistic dimensions would essentially change the proportions shown in the drawing.

10.4 SUPPLEMENTARY DATA ON DUCTILE IRON PRESSURE TUBING

10.4.1 Strength Data

Tensile strength = 400 N/mm² Extensibility = 10 % E = 1.7 . 10^5 N/mm²
Tensile yield point = 300 N/mm² Compressive strength = 900 N/mm²

A compressive strength of 900 N/mm² is by far the highest value so far reached by a pipe material, and is about twice as high as that of pure steel. This gives a great resistance to internal and external loadings. See fig. 10-9.

10.4.2 VRS Bell-and-Spigot Joints

The advantages of this type of joint are as follows:

(a) The section of the rubber gasket and the high pushing-in pressure applied ensure absolute water tightness.

(b) A joggle renders superfluous any other sort of locking device against push and pull.

(c) The connexion system allows angling and is not rigid, in spite of the joggle, which is a particular merit when pipes are laid in mountainous terrain and unstable soil. Safety criteria are the welded bead and the joint design.

10.4.3 Laying Technology for the VRS System

In practice, pipelaying progress rates of up to 700 m per shift have been attained. This is feasible as pipe-to-pipe connexion requires a single pushing force (e.g. lever, lifting block, shovel). It is also possible to join the push-resistant pipe material near the edge of the trench to lengths of up to 25 m before lowering it in, or to joint pipes at any location and e.g. pull the pipe line up or down a mountain slope by means of hoisting gears or earthwork equipment. Application of such a method is demonstrated by the example shown on figs. 10-11 and 10-12, where a 200 mm penstock was installed ready for operation over the whole length of a 400-mm bore of 130-m length with a gradient of 78 percent in not more than 5 working hours.

10 WATERWAY FOR HIGH HEAD SCHEMES UNDER SPECIFIC ASPECTS
TRIEBWASSERWEG UND SPEZIFISCHE PROBLEME VON HOCHDRUCKANLAGEN

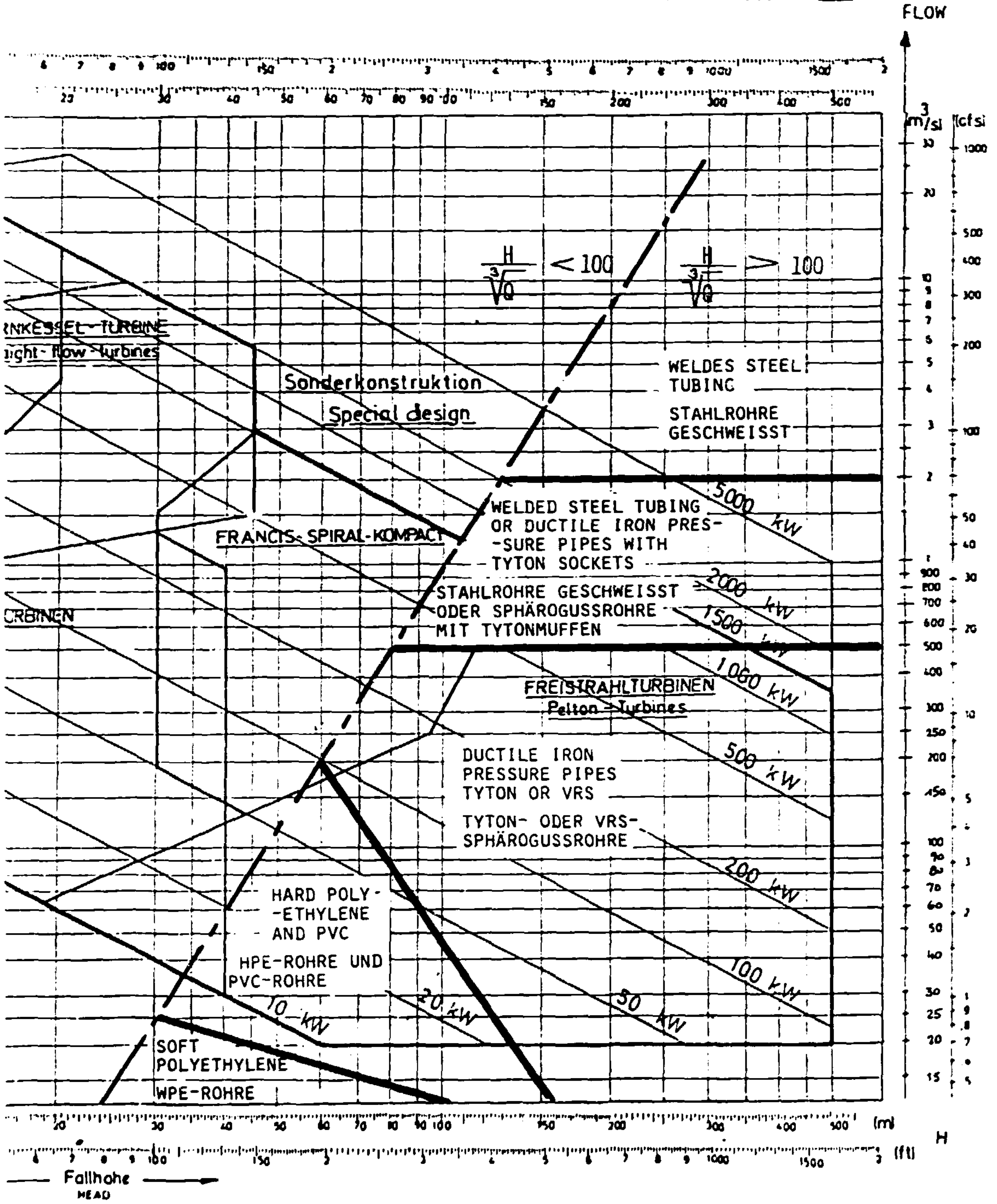


FIG 10-1 PIPE MATERIAL AND LIMITS OF ECONOMICAL APPLICATION
ROHRMATERIAL UND WIRTSCHAFTLICHE ANWENDUNGSBEREICHE

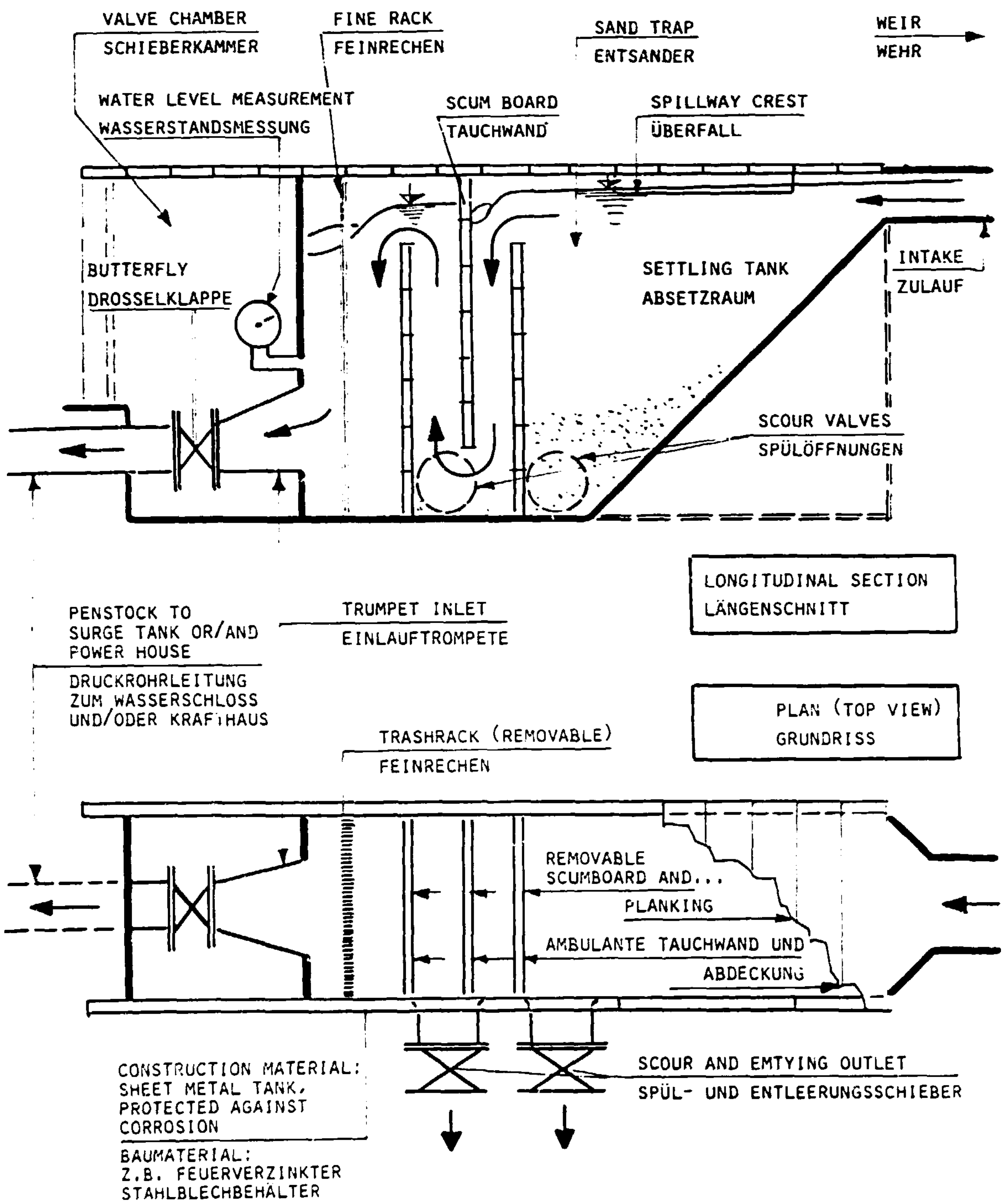


FIG 10-2 SIMPLE INTAKE STRUCTURE
EINFACHES EINLAUFBAUWERK MIT ÜBERLASTUNGSSCHUTZ

10 WATERWAY FOR HIGH HEAD SCHEMES UNDER SPECIFIC ASPECTS
TRIEBWASSERWEG UND SPEZIFISCHE PROBLEME VON HOCHDRUCKANLAGEN

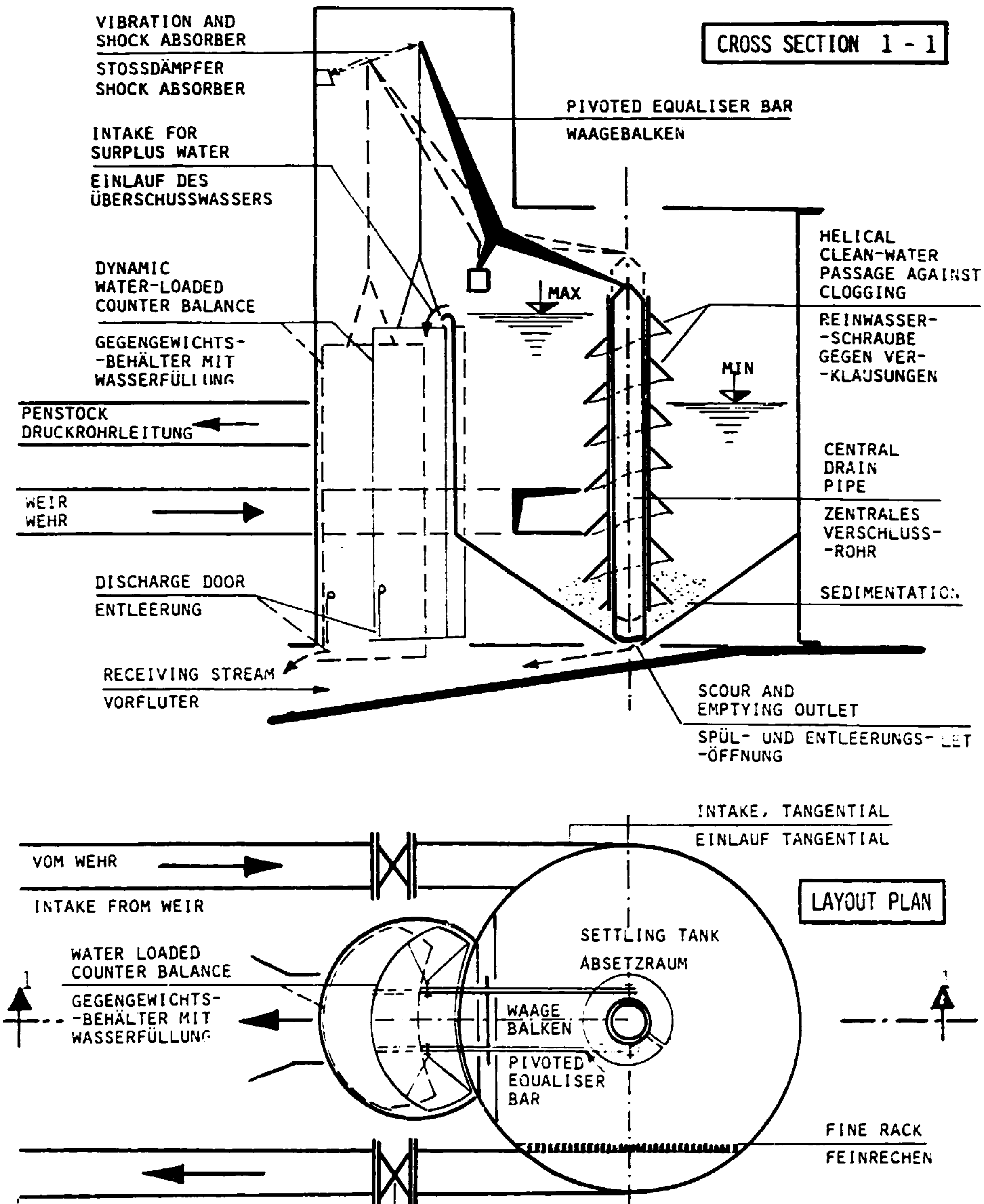


FIG 10-3 ROTARY-TYPE SAND TRAP
RUNDENTSANDER

10	WATERWAY FOR HIGH HEAD SCHEMES UNDER SPECIFIC ASPECTS TRIEBWASSERWEG UND SPEZIFISCHE PROBLEME VON HOCHDRUCKANLAGEN

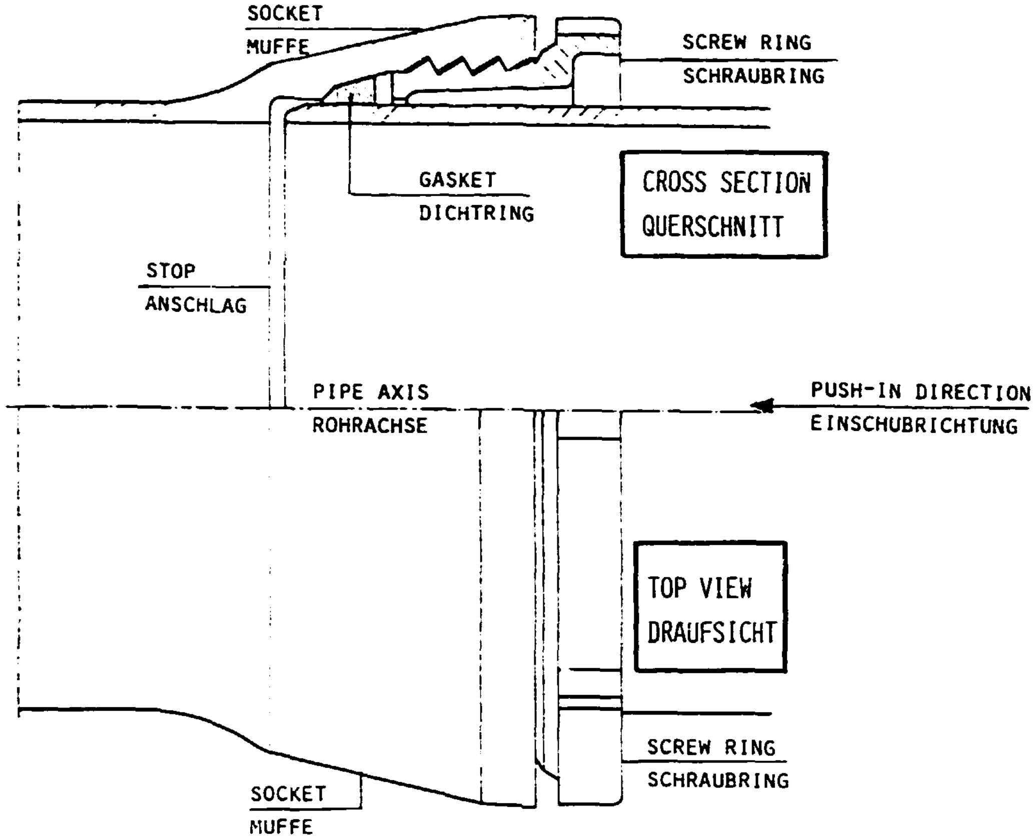


FIG 10-4 DUCTILE IRON PRESSURE PIPES - SCREW SOCKET CONNEXION
SPHÄROGUSSROHRE MIT SCHRAUB MUFFENVERBINDUNG

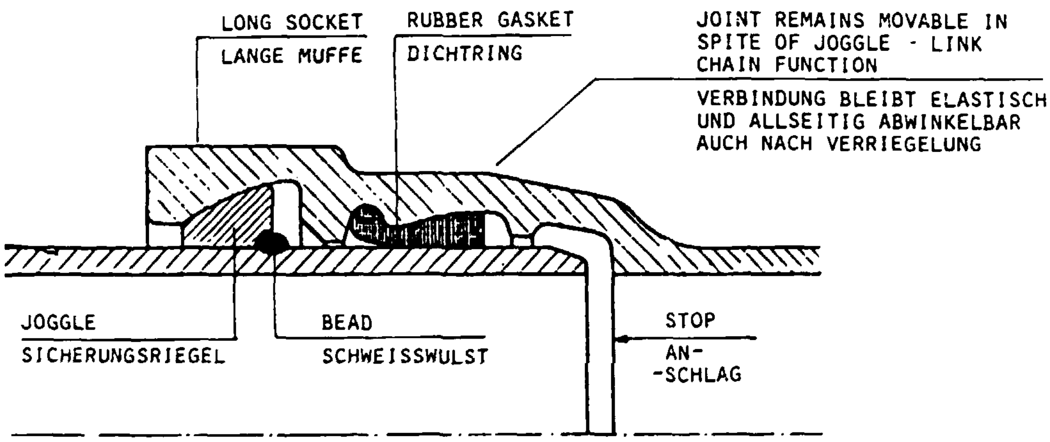


FIG 10-5 DUCTILE IRON PRESSURE PIPES - VRS CONNEXION
SPHÄROGUSSROHRE MIT VRS-VERBINDUNG

10	WATERWAY FOR HIGH HEAD SCHEMES UNDER SPECIFIC ASPECTS TRIEBWASSERWEG UND SPEZIFISCHE PROBLEME VON HOCHDRUCKANLAGEN

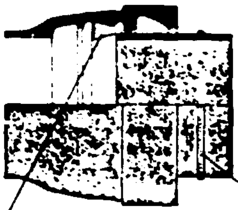
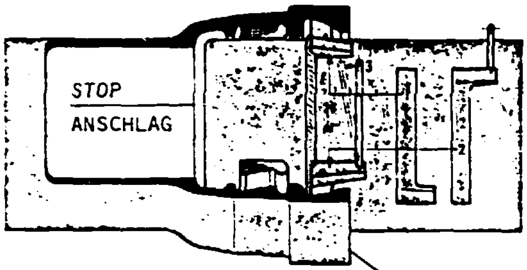


FIG 10-6 DUCTILE IRON PRESSURE PIPES - VRS CONNEXION
SPHÄROGUSS-VRS-VERBINDUNG ROHREINSCHUB



1 ... PUT IN SAFETY BOLT TO THE RIGHT
RIEGELEINSCHUB RECHTS

2 ... PUT IN SAFETY BOLT TO THE LEFT
RIEGELEINSCHUB LINKS

3 ... PRESS IN SAFETY WIRE
SICHERUNGSDRAHT EINDRÜCKEN

FIG 10-7 VRS CONNEXION
VRS VERBINDUNG

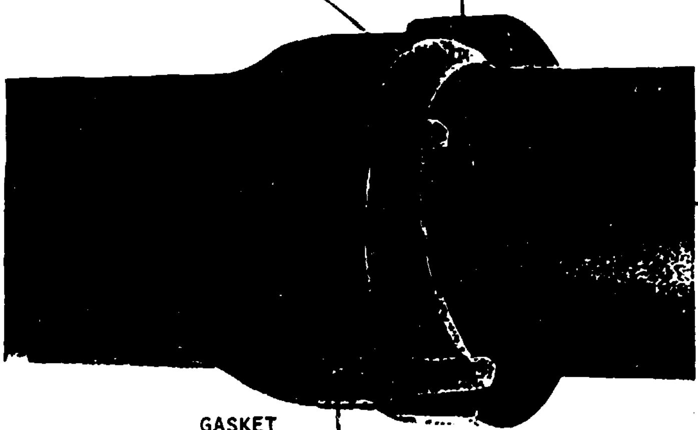


FIG 10-8 VRS CONNEXION
VRS VERBINDUNG

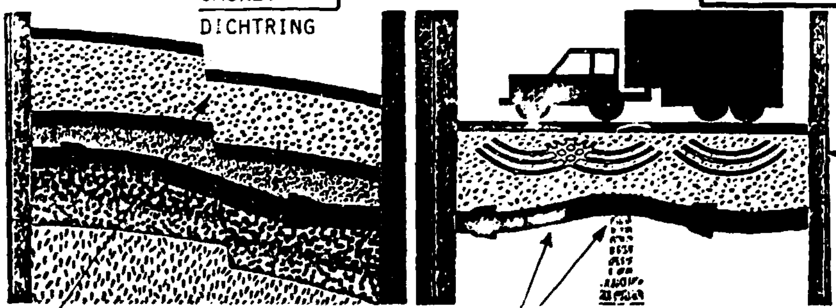


FIG 10-9 EXAMPLE FOR APPLICATION
ANWENDUNGSBEISPIEL

10	WATERWAY FOR HIGH HEAD SCHEMES UNDER SPECIFIC ASPECTS TRIEBWASSERWEG UND SPEZIFISCHE PROBLEME VON HOCHDRUCKANLAGEN

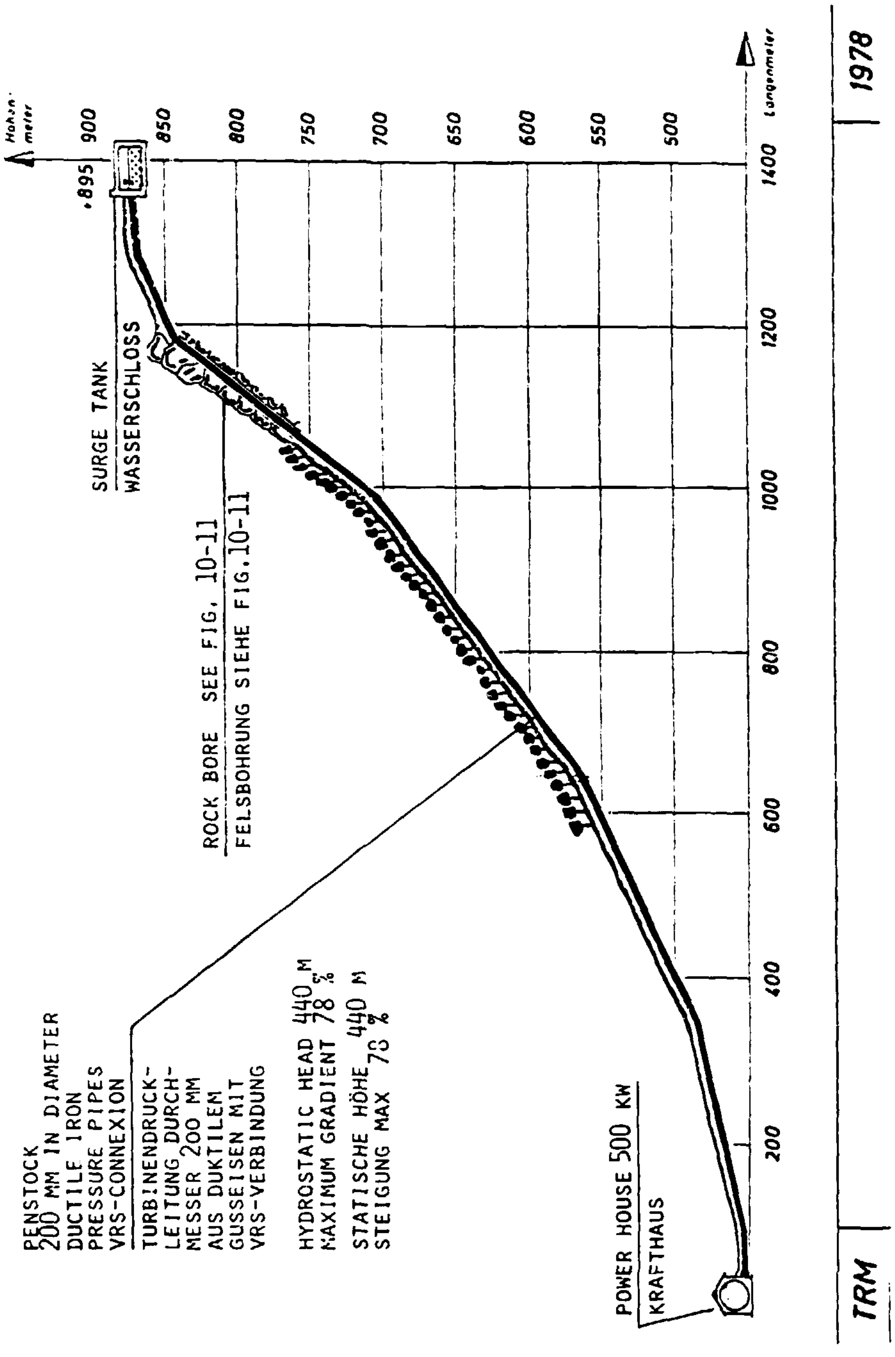


FIG 10-10	DUCTILE IRON PRESSURE TUBES - PRACTICAL EXAMPLE DRUCKROHRLEITUNG AUS SPHÄROGUSSROHREN - ANWENDUNGSBEISPIEL

10 WATERWAY FOR HIGH HEAD SCHEMES UNDER SPECIFIC ASPECTS
TRIEBWASSERWEG UND SPEZIFISCHE PROBLEME VON HOCHDRUCKANLAGEN

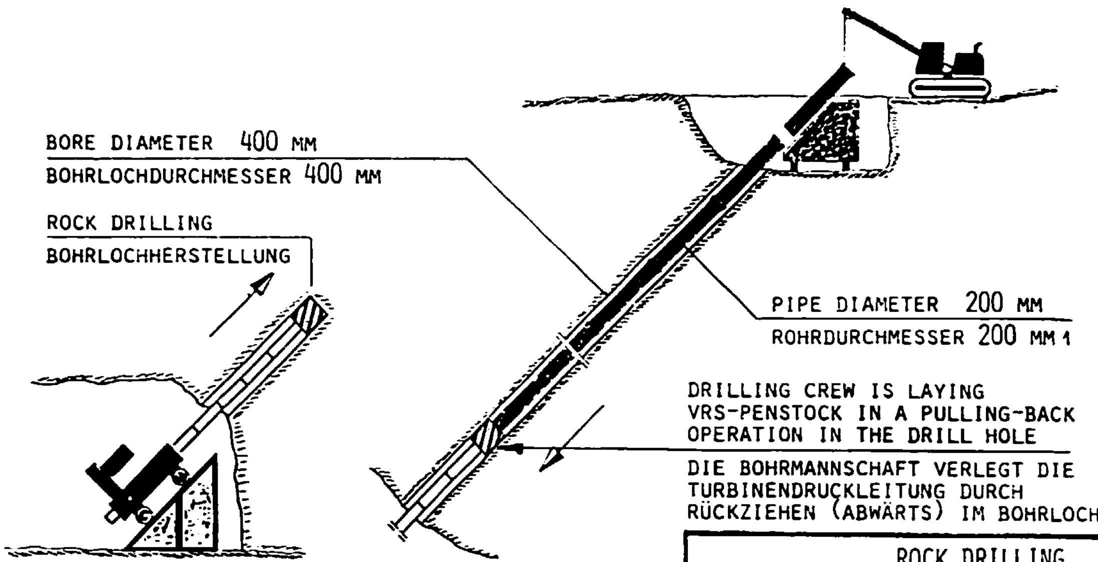


FIG 10-11 ROCK DRILLING FOR PENSTOCK
FELSBOHRUNG FÜR TURBINENDRUCK- -LEITUNG

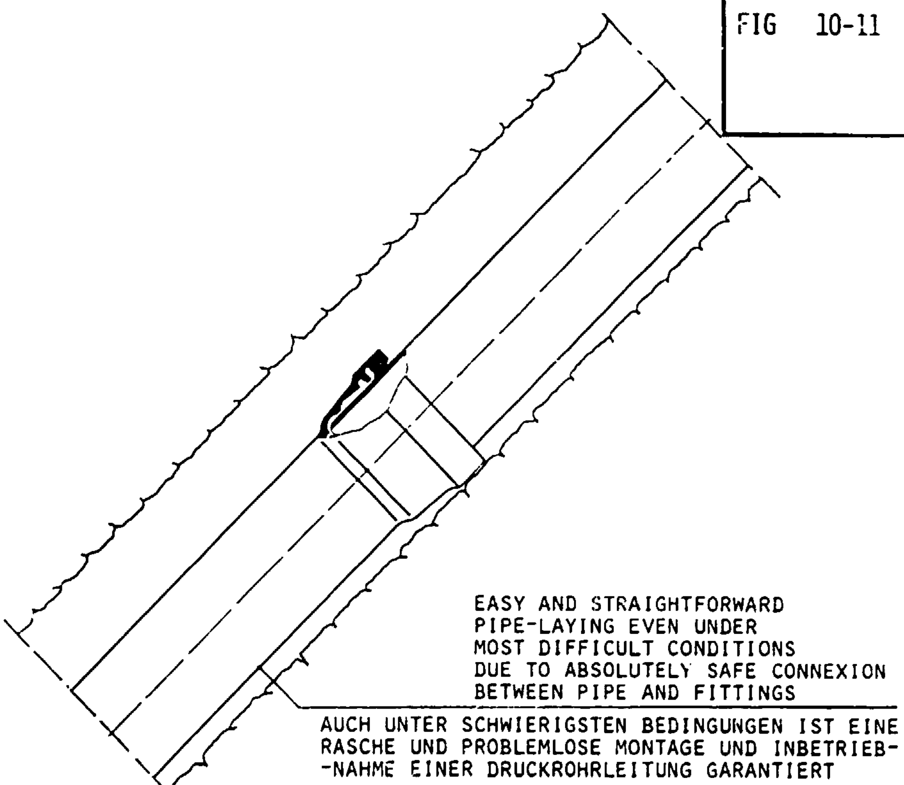
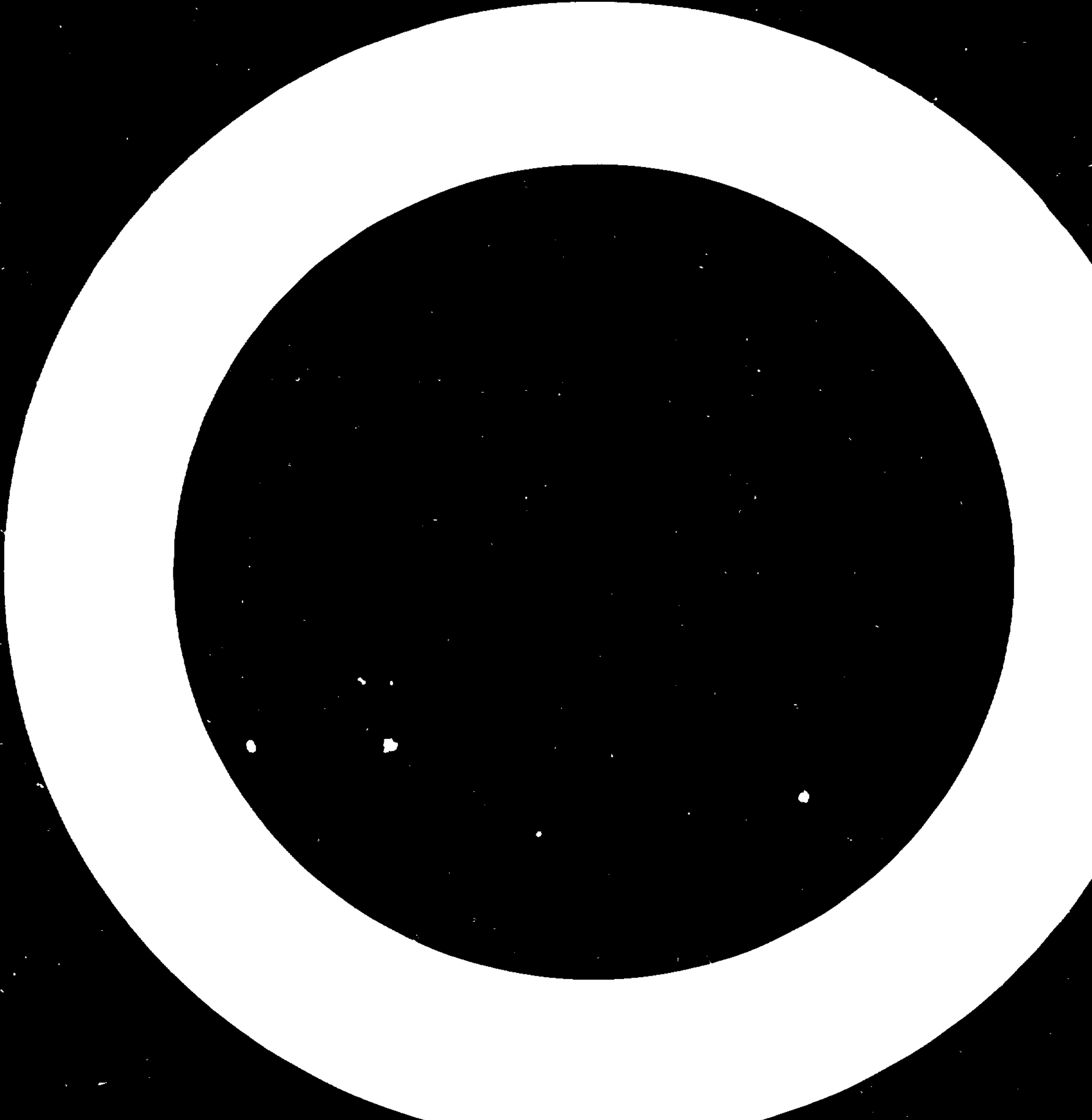


FIG 10-12 PIPE-LAYING IN THE DRILL HOLE
ROHRVERLEGUNG IM BOHRLOCH



11 GENERAL DESCRIPTION AND APPLICATIONS OF SMALL HYDRO TURBINES

TSCHERNUTTER P.

ABSTRACT:

The market offers extensive standardised product lines from different manufacturers. The main design principles will be presented and approximate dimensions will be indicated in the following paragraphs.

11.1 FUNDAMENTAL APPLICATIONS

On a general basis, purchase of a turbine should be preceded by decisions regarding the desired number of operating hours per year, the expected service life, as well as the type of stream to be developed (high rate of bed load transport, aggressive water etc.). These parameters have an important bearing on the choice of materials and, hence, on the costs of all parts subject to major loadings. So far as the materials are concerned, severely loaded parts with long expected service lives, such as runners, nozzles, etc., are made e.g. of chrome - nickel - molybdenum - steel alloys instead of a chrome steel, or cast steel instead of a bronze alloy. It should, however, be added that the cost of such high-strength material is substantially higher than that of conventional materials.

The choice of the turbine type is governed not only by the available head and the rated discharge, but also by the partial-load behaviour. Where a choice must be made between two eligible turbine types, it will often be the one with the better speed control that will be preferred.
Below is a figure showing the basic curves of the different turbine efficiencies. The magnitudes indicated are mean values. The actual values may be slightly greater or smaller.

11.1.1 Low-Head Range

In the low-head range, a decision must primarily be taken between

Kaplan or bulb turbines,
normal Francis or high-speed Francis turbines,
cross-flow turbines with a three-part runner gradation.

High-speed Francis turbines and partly also normal-speed turbines have unfavourable partial-load efficiences for major development factors (greater than 1.5 times the mean flow) and major flow variations (mean low-water flow to mean flow). In some cases operation of the plant may be impossible at all (e.g. development factor 1.5 x mean flow; ratio of mean low-water flow to mean flow greater than approximately 1:5). In most cases, however, provision of 2 turbine units for moderate-sized power plants will be precluded from consideration for financial reasons.

Cross-flow turbines exhibit a slightly more favourable distribution of the partial-load efficiency within a larger range of flow variation.

11.1.2 High-Head Range

In the high-head range, the following turbine types are normally applied:

Pelton turbines (with one or two nozzles),
low-speed Francis or normal-speed Francis turbines,
cross-flow turbines with a two-part runner gradation.

These turbine types are much more sensitive to high development factors and major flow variations than low-head turbines are. With a development factor of approx. 1.5 times the mean flow and a flow variation of mean low-water flow to mean flow = 1:5, it will probably be only Pelton turbines and perhaps cross-flow turbines (a single unit in each case) that will yield a satisfactory efficiency for low-water flows.

For Francis turbines, the relation of mean low-water flow to mean flow, under otherwise the same conditions, ranges around 1:3. If this requirement is not met or at least approached, it will be necessary to provide 2 turbine units or - if possible - use a different turbine type, e.g. where the applications of Francis and Pelton turbines overlap, a decision will be taken in favour of Pelton.

11.2 OUTLINE OF THE STANDARD PRODUCT LINES OF SEVERAL TURBINE MANUFACTURERS

The standard product lines offered at present by the various turbine manufacturers are substantial. Therefore, it will not be possible in this context to discuss them as a whole. It will be attempted to outline certain design principles of the individual turbine types in order to afford some basis for decision. Detailed information will best be obtained from the individual manufacturers.

11.2.1 Kaplan and Bulb Turbine

On a general basis, Kaplan and bulb turbines allow double control (by means of the wheel vanes and the guide vanes). The market also offers single-control types, in which the wheel vanes are usually mounted rigidly on the hub. Depending on head and capacity, a wide range of different designs have been developed and manufactured. Figs. 11-1 to 11-7 present the main design types. Besides the turbine ratings, main measurements and dimensioning aids are indicated.

11.2.2 Cross Flow Turbine

The cross flow turbine is a transition type between the impulse (Pelton) turbine and the reaction turbine (Francis, Kaplan). The transitional designs are characterised by the fact that the pressure in the gap between guide vanes and runner is not yet above atmospheric, although the water completely fills the openings between the wheel vanes. This turbine is known in literature also as Mitchell-Ossberger or Banki turbine. It has the advantage of allowing good speed control (three-part runner graduation) at partial load in the low-head range, for great stream flow variations. However, their peak efficiencies are substantially below those of other turbine types. Where major heads are developed, three-part gradation of the runner is not possible for mechanical reasons, which partly impairs the partial-load behaviour. Fig. 11-8 shows the fundamental design of a cross flow turbine with a three-part runner gradation.

11.2.3 Francis Turbine

Francis turbines are manufactured for a very large range of heads and discharges. However, the design principles employed vary considerably. For minor capacities or heads, Reiffenstein and straight-flow turbines are often used. Open-flume turbines are normally chosen for major flows and moderate heads. Francis vertical-shaft mini sets are applied for major flows and moderate heads. Fundamentals of design as applied by different manufacturers are shown in Figs. 11-9 to 11-15.

11.2.4 Pelton Turbine

There is only a very small product line for standard wheels. This is mainly due to the fact that the effect of head and discharge on the design of the wheel and other components is much greater than in other turbine types. For greater speed-control requirements, adoption of a two-nozzle type with separate nozzle controls is recommended. Figs. 11-16 and 11-17 show possible design principles and indicate a standardised diameter series for wheels.

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11 GENERAL VIEW AND APPLICATION FOR SMALL HYDRO TURBINES
ÜBERSICHT UND EINSATZBEREICH VON KLEINEN WASSERKRAFTTURBINEN

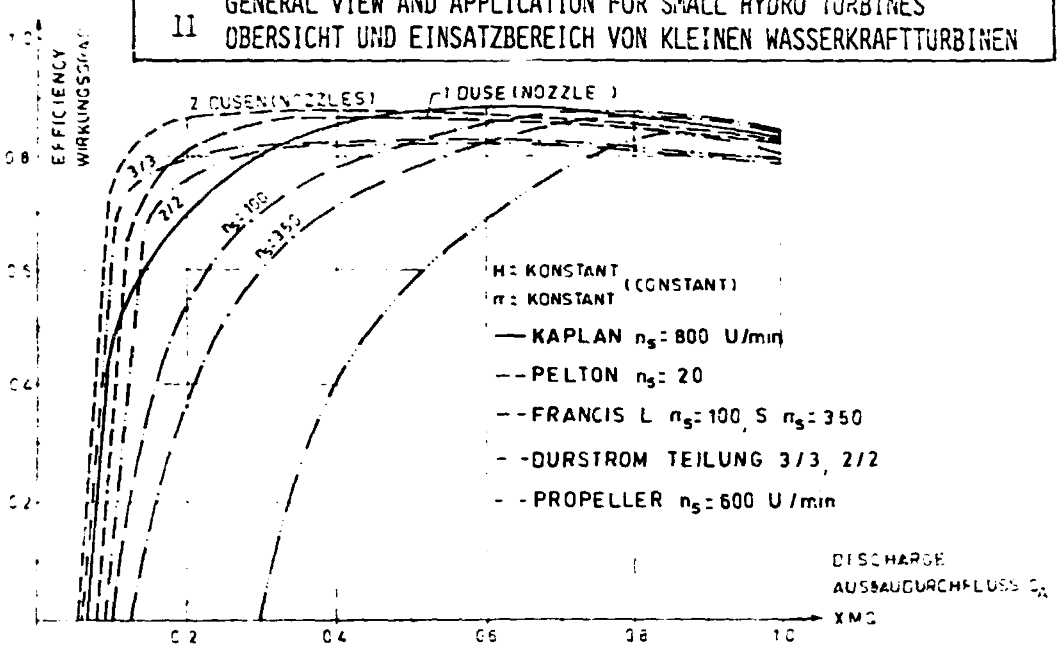


FIG 11-1 EFFICIENCIES OF DIFFERENT TURBINES
WIRKUNGSGRADE VERSCHIEDENER TURBINEN

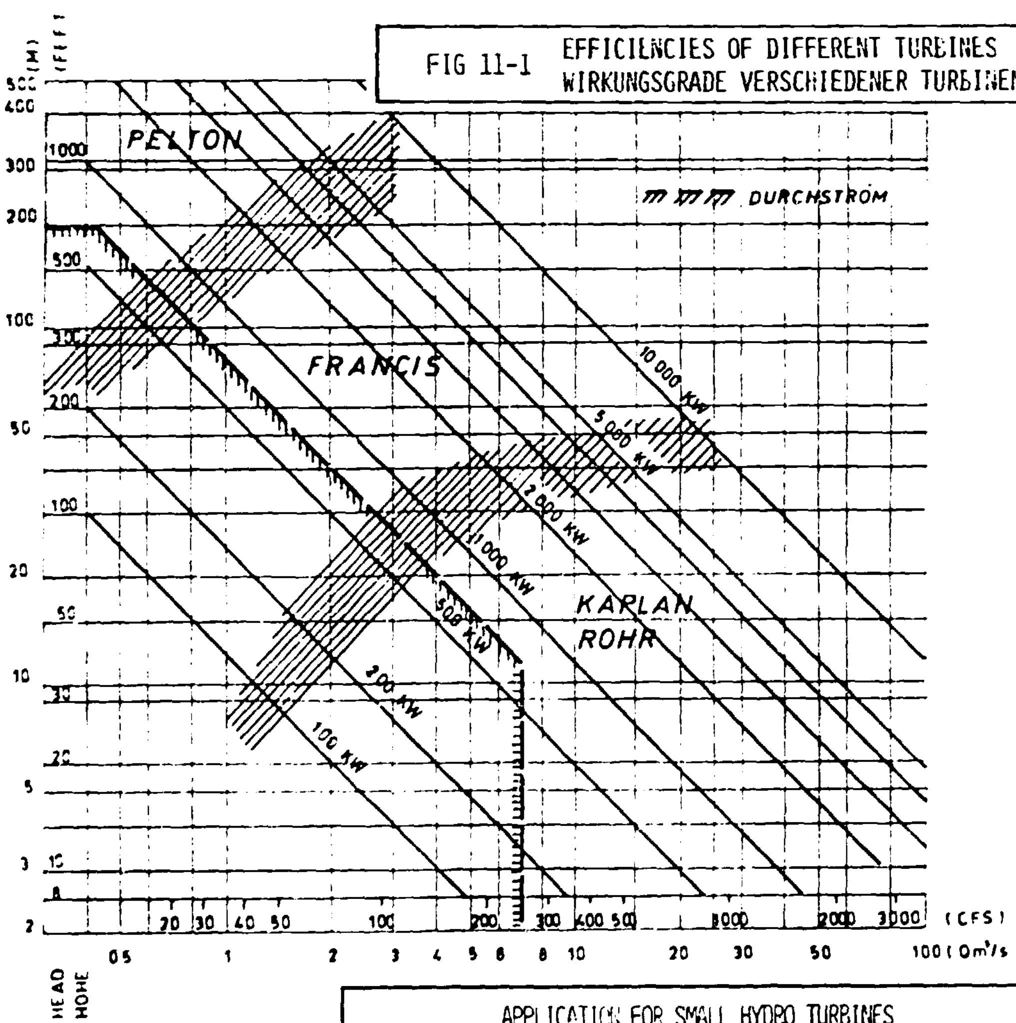
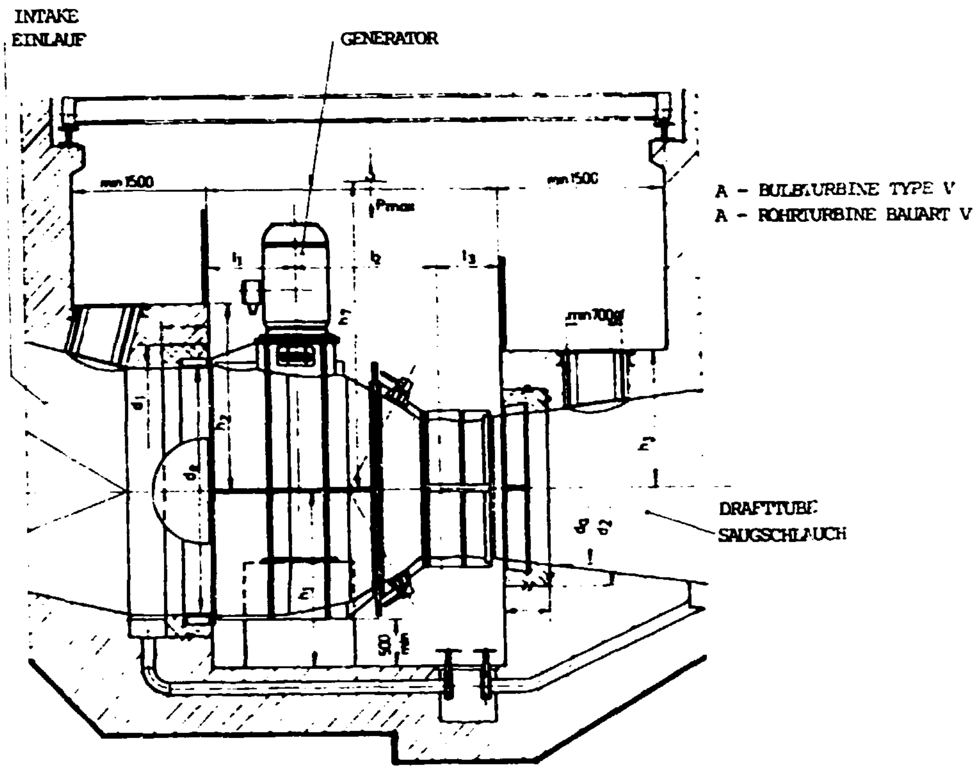


FIG 11-2 APPLICATION FOR SMALL HYDRO TURBINES
EINSATZBEREICH VON KLEINEN WASSERKRAFTTURBINEN

11	GENERAL VIEW AND APPLICATION FOR SMALL HYDRO TURBINES OBERSICHT UND EINSATZBEREICH VON KLEINEN WASSERKRAFTTURBINEN



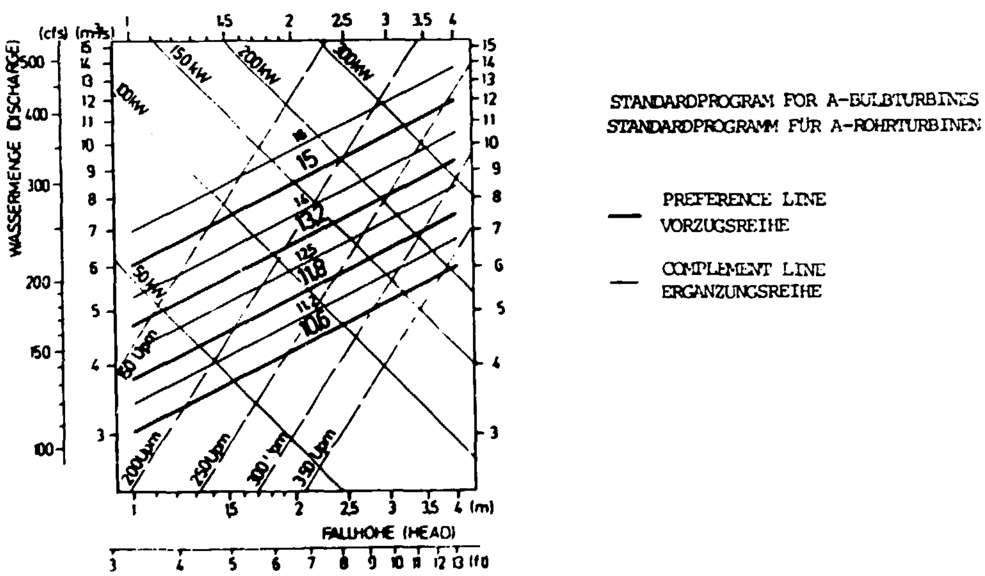
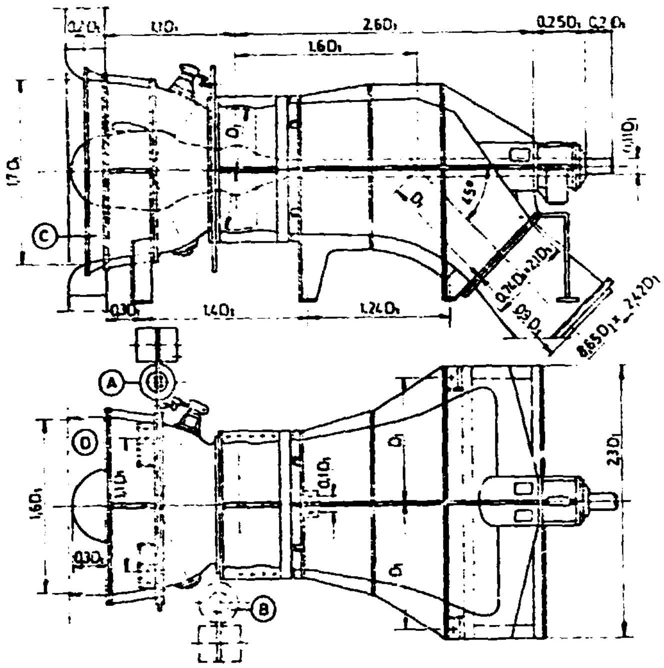


FIG 11-3	STANDARD PROGRAM FOR A-BULBTURBINES FROM PRODUCER KÖSSLER STANDARDPROGRAMM FÜR A-ROHRTURBINEN DER FIRMA KÖSSLER

11	GENERAL VIEW AND APPLICATION FOR SMALL HYDRO TURBINES ÜBERSICHT UND EINSATZBEREICH VON KLEINEN WASSERKRAFTTURBINEN

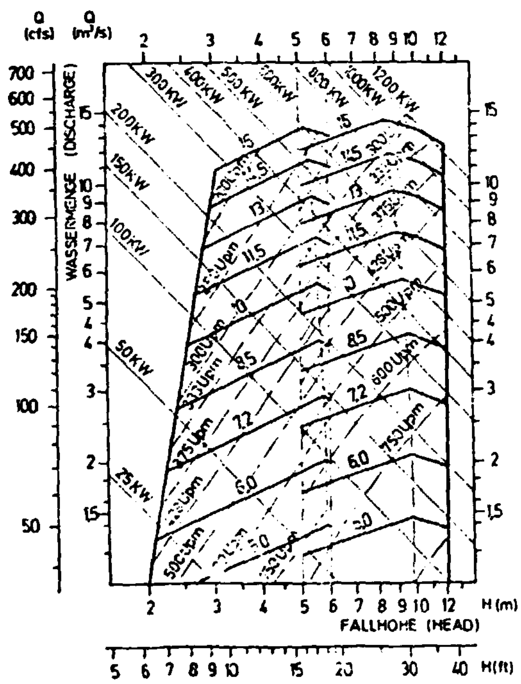


S - BULBTURBINE FROM PRODUCER KÖSSLER

S - ROHRTURBINE DER FIRMA KÖSSLER

BASIC DIMENSIONS

GRUNDABMESSUNGEN



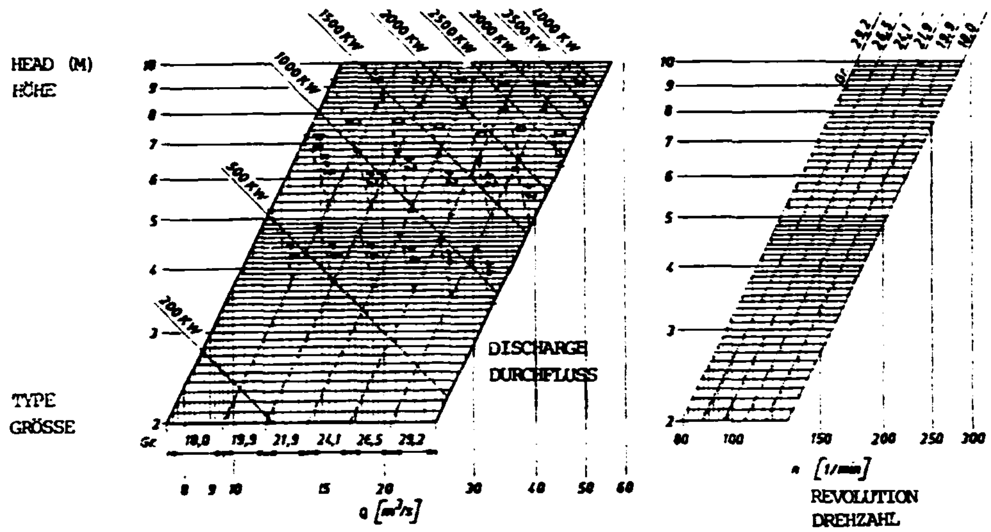
LAY OUT DIAGRAM FOR S - BULB-TURBINES

AUSLEGUNGSDIAGRAMM FÜR S - ROHR-TURBINEN

FIG 11- 4	STANDARD PROGRAM FOR S-BULBTURBINES FROM PRODUCER KÖSSLER STANDARDPROGRAMM FÜR S-ROHRTURBINEN DER FIRMA KÖSSLER

11 GENERAL VIEW AND APPLICATION FOR SMALL HYDRO TURBINES
OBERSICHT UND EINSATZBEREICH VON KLEINEN WASSERKRAFTTURBINEN

RUNNER WITH 4 BLADES
LAUFRAD MIT 4 SCHAUFELN



RUNNER WITH 5 BLADES
LAUFRAD MIT 5 SCHAUFELN

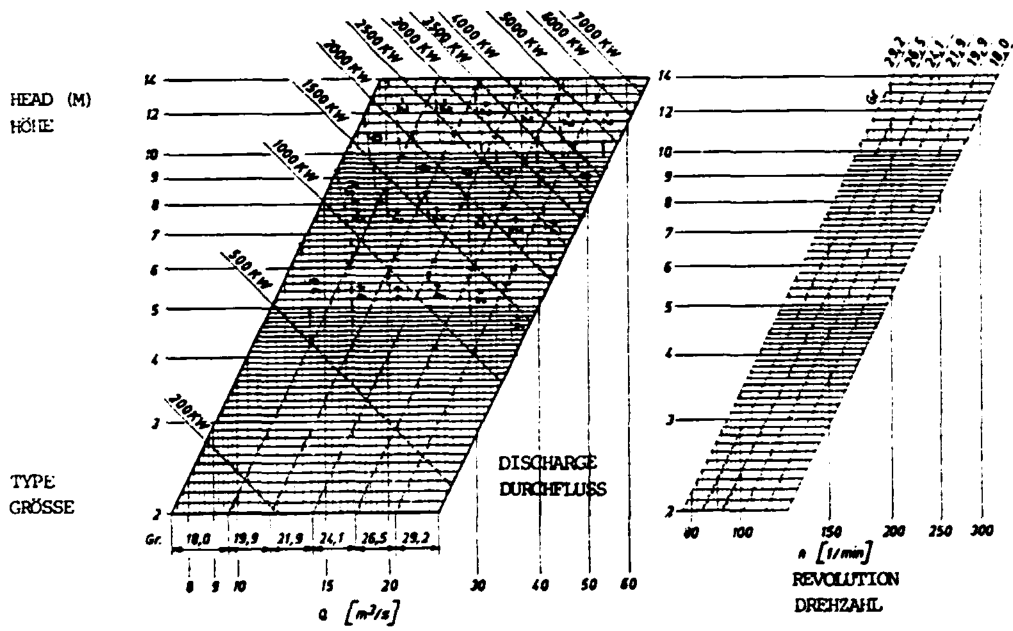
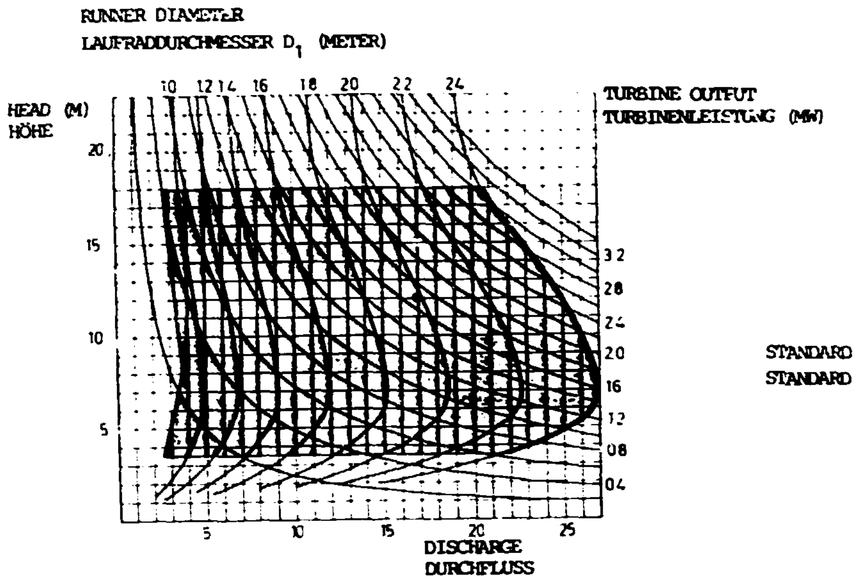


FIG 11-5 STANDARD PROGRAM FOR BULBTURBINES FROM PRODUCER VOITH
STANDARDPROGRAMM FÜR ROHRTURBINEN DER FIRMA VOITH

11 GENERAL VIEW AND APPLICATION FOR SMALL HYDRO TURBINES
ÜBERSICHT UND EINSATZBEREICH VON KLEINEN WASSERKRAFTTURBINEN



STANDARD PROGRAM
STANDARD PROGRAMM

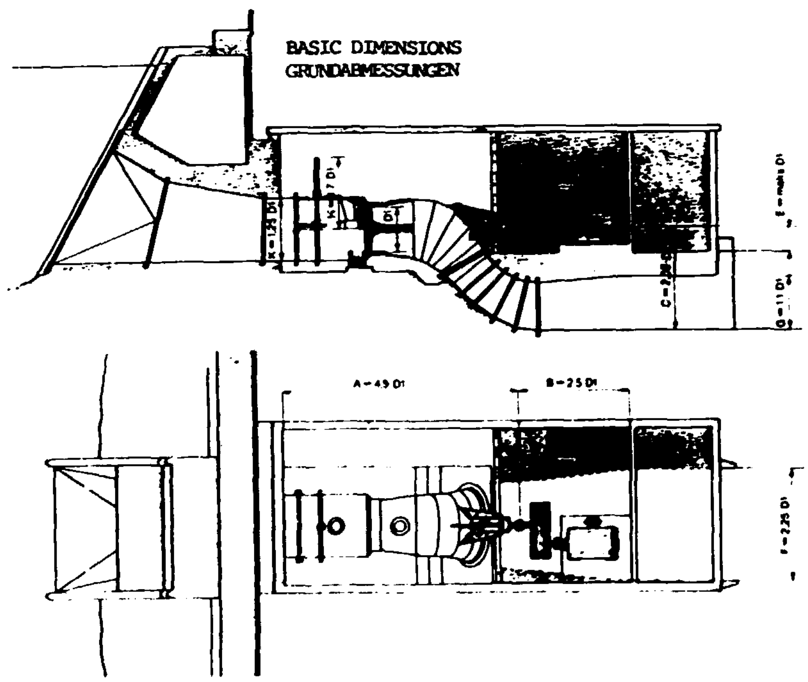


FIG 11-6 STANDARD PROGRAM (PRODUCER SANDEN)
STANDARDPROGRAMM (FIRMA SANDEN)

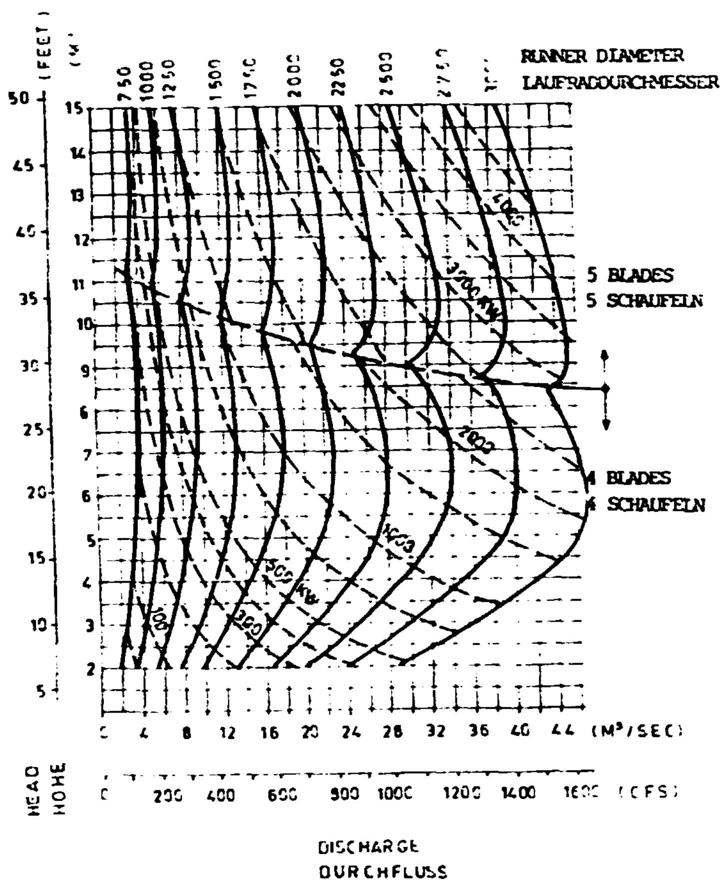
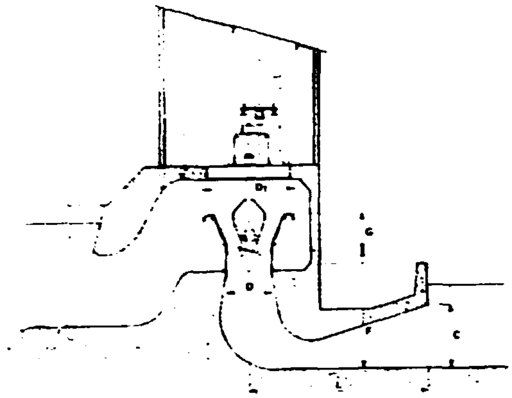


FIG 11-7 ALLIS CHALMERS TUBE TURBINES
ALLIS CHALMERS ROHRTURBINEN

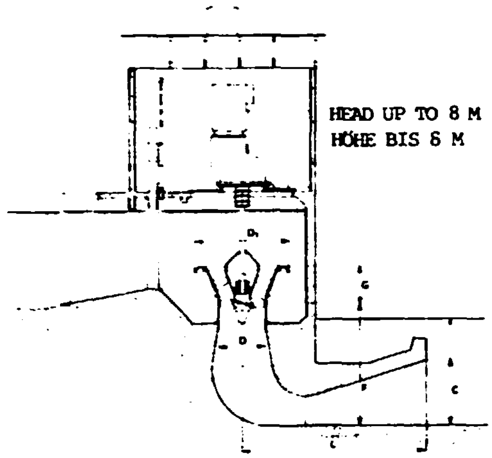
11	GENERAL VIEW AND APPLICATION FOR SMALL HYDRO TURBINES OBERSICHT UND EINSATZBEREICH VON KLEINEN WASSERKRAFTTURBINEN

SIPHON ARRANGEMENT
SIPHON ANORDNUNG



$B = 2{,}67 \times D;\ C = 1{,}33 \times D;\ D_1 = 2 \times D$
$F = 2{,}5 \times D;\ G = 0{,}84 \times D;\ L = 4{,}44 \times D$

OPEN FLUME ARRANGEMENT
SCHACHTANORDNUNG



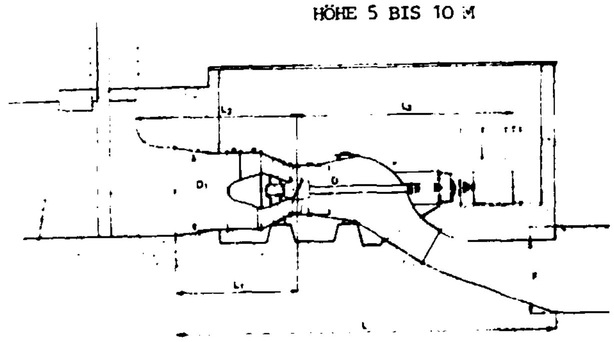
$B = 2{,}67 \times D;\ C = 1{,}33 \times D;\ D_1 = 2 \times D$
$F = 2{,}5 \times D;\ G = 0{,}84 \times D;\ L = 4{,}44 \times D$

B = DRAFTTUBE WIDTH / SAUGSCHLAUCHBREITE   D = RUNNER DIAMETER / LAUFRADDURCHMESSER

D : 500, 700, 900, 1150, 1400, 1700, 2000, 2250, 2500, 2750, 3000 MM

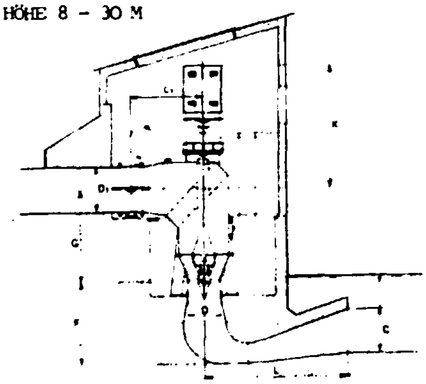
HORIZONTAL ARRANGEMENT
HORIZONTALE ANORDNUNG

HEAD 5 TO 10 M
HÖHE 5 BIS 10 M



$B = 2{,}11 \times D;\ C = 2{,}62 \times D;\ D_1 = 1{,}56 \times D$
$F = 1{,}52 \times D;\ L = 8{,}53 \times D;\ L_1 = 2{,}63 \times D;\ L_2 = 3{,}46 \times D$

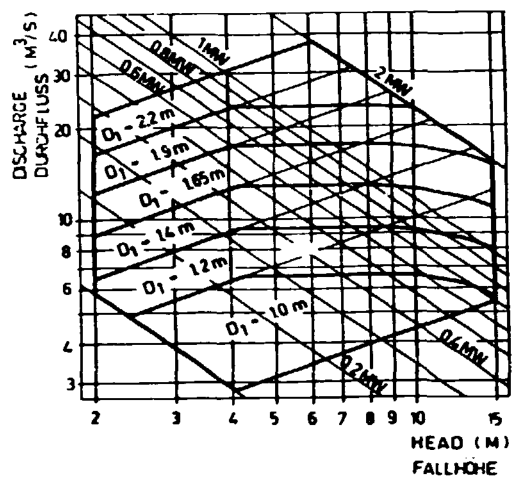
VERTICAL ARRANGEMENT
VERTIKALE ANORDNUNG
HEAD 8 - 30 M
HÖHE 8 - 30 M



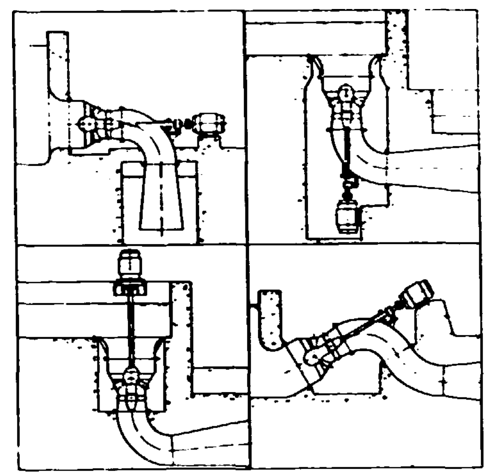
$B = 2{,}67 \times D;\ C = 1{,}33 \times D;$
$D_1 = 1{,}33 - 1{,}5 \times D;\ F = 2{,}5 \times D;$
$G = 2{,}78 \times D;\ K = 4{,}7 - 1{,}9 \times D;$
$L = 4{,}44 \times D;\ L_1 = 2{,}33 - 2{,}5 \times D$

FIG 11-8	BOFORS - NOHAB TURBINE PROGRAM BOFORS - NOHAB TURBINENPROGRAMM

11	GENERAL VIEW AND APPLICATION FOR SMALL HYDRO TURBINES ÜBERSICHT UND EINSATZBEREICH VON KLEINEN WASSERKRAFTTURBINEN



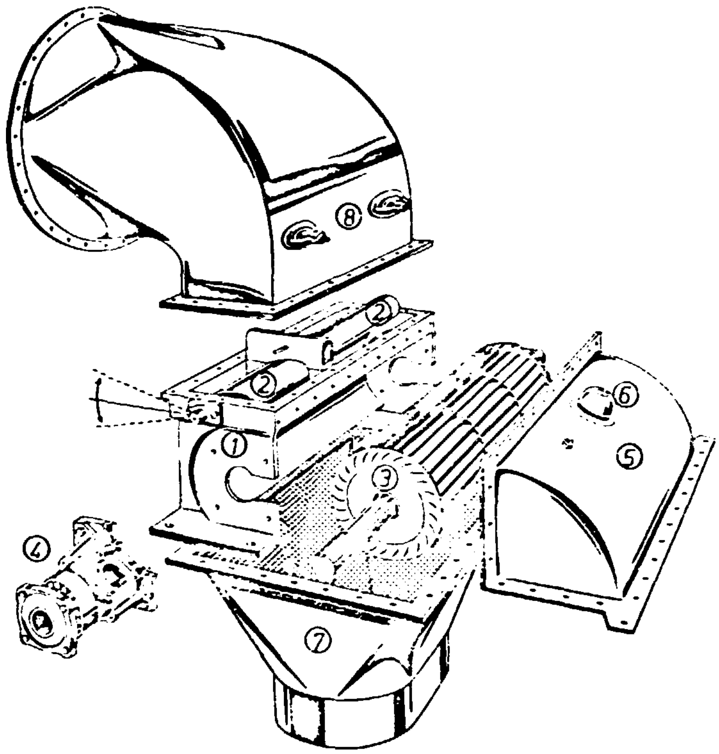
LAY OUT DIAGRAM
AUSLEGUNGSDIAGRAMM



ARRANGEMENTS
ANORDNUNGSMÖGLICHKEITEN

FIG 11-9	STANDARDIZED BULBTURBINES (PRODUCER BELL) STANDARDISIERTE ACHSIALTURBINEN (FIRMA BELL)

11	GENERAL VIEW AND APPLICATION FOR SMALL HYDRO TURBINES ÜBERSICHT UND EINSATZBEREICH VON KLEINEN WASSERKRAFTTURBINEN



PRINCIPLE SYSTEM OF AN OSSBERGER TURBINE

PRINZIPIELLES SYSTEM EINER OSSBERGER TURBINE

1 CASING
GEHÄUSE

2 GUIDE VANES
LEITAPPARAT

3 RUNNER
LAUFRAD

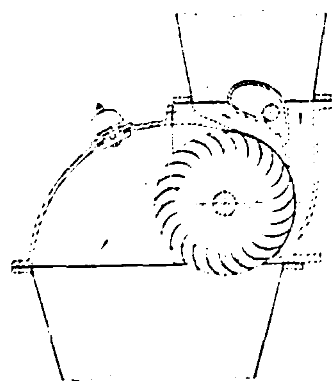
4 MAIN BEARING
HAUPTLAGER

5 CORNER CASING
ECKKASTEN

6 AIR INLET VALVE
BELÜFTUNGSVENTIL

7 DRAFTTUBE
SAUGROHR

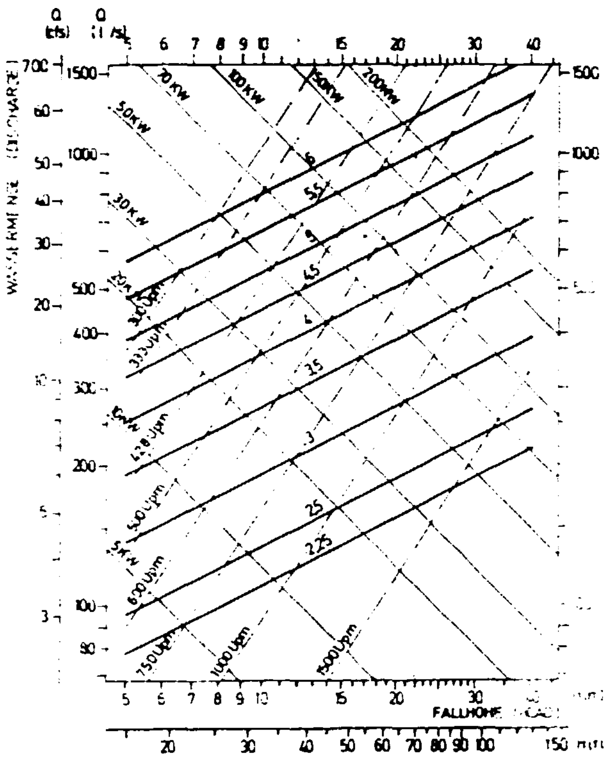
8 REDUCER
ÜBERGANGSSTÜCK



GENERAL CROSS SECTION
QUERSCHNITT

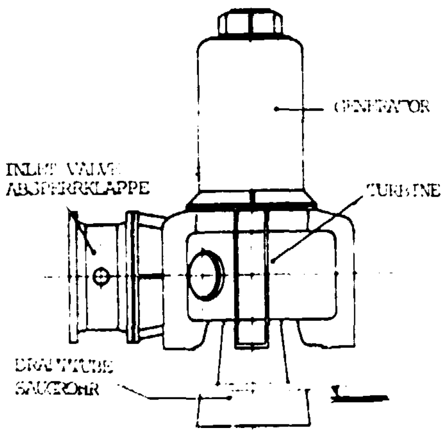
FIG 11-1o	OSSBERGER - CROSS - FLOW - TURBINE OSSBERGER DURCHSTRÖMTURBINE

11 GENERAL VIEW AND APPLICATION FOR SMALL HYDRO TURBINES
ÜBERSICHT UND EINSATZBEREICH VON KLEINEN WASSERKRAFTTURBINEN



LAY OUT DIAGRAM FOR KÖSSLER REIFFENSTEIN TURBINES

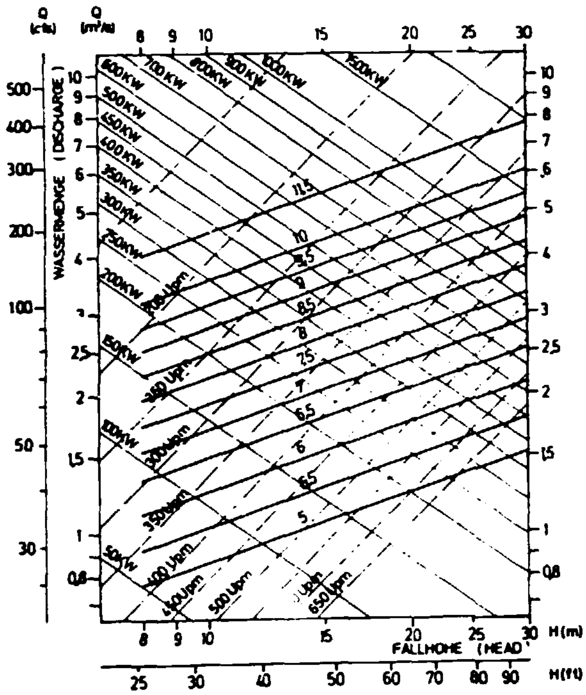
AUSLEGUNGSDIAGRAMM FÜR KÖSSLER REIFFENSTEINTURBINEN



GENERAL SYSTEM OF REIFFENSTEIN TURBINES
GENERELLES SYSTEM DER REIFFENSTEINTURBINEN

FIG 11-11 REIFENSTEIN-TURBINE (PRODUCER KÖSSLER, RUNNER TYPE 200)
REIFFENSTEINTRUBINE (FIRMA KÖSSLER, LAUFRADTYPE 200)

11	GENERAL VIEW AND APPLICATION FOR SMALL HYDRO TURBINES ÜBERSICHT UND EINSATZBEREICH VON KLEINEN WASSERKRAFTTURBINEN

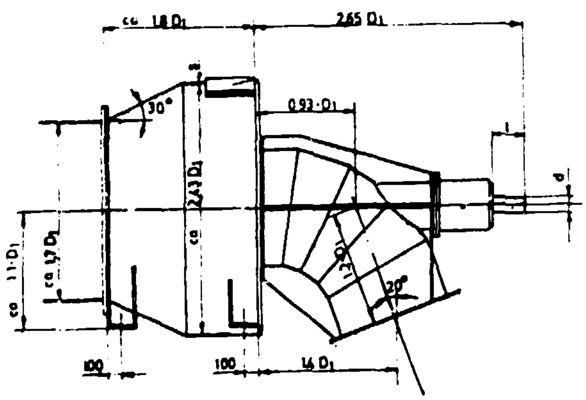


LAY OUT DIAGRAM FOR KÖSSLER STRAIGHT-FLOW-TURBINES

AUSLEGUNGSDIAGRAMM FÜR KÖSSLER STIRNKESSELTURBINEN

RUNNER TYPE 250 (n_s)

LAUFRADTYPE 250 (n_s)



BASIC DIMENSIONS

REGELABMESSUNGEN

DIAMETER SERIES D_1 (dm): 5; 5,5; 6; 6,5; 7; 7,5; 8; 8,5; 9; 9,5; 10; 11,5; 13

DURCHMESSERREIHE

FIG 11-12	STRAIGHT-FLOW-TURBINES FROM PRODUCER KÖSSLER STIRNKESSELTURBINEN DER FIRMA KÖSSLER

11 GENERAL VIEW AND APPLICATION FOR SMALL HYDRO TURBINES
OBERSICHT UND EINSATZBEREICH VON KLEINEN WASSERKRAFTTURBINEN

FALLHOHE (m) HEAD

TYPE
LEISTUNG (kVA) 400/231V,50Hz TURBINE OUTPUT
WASSERMENGE (l/s) DISCHARGE

30	35	40	45	50	55	60	65	70	75	80	85	90	95	100
2,25/6 50 190	2.5/4 50 165	2,5/3 50 145												
2,5/6 63 235	2,75/4 63 205	2,5/4 63 180	2,5/3 63 160											
2,25/8 80 295	2,5/5 80 255	2,75/4 80 225	2,75/3 80 200	2,75/3 80 180										
2,5/8 100 365	2,75/6 100 315	2,75/5 100 280	2,75/4 100 245	3,0/3 100 220	2,75/3 100 205									
	2,5/8 125 390	2,75/6 125 345	3,0/4 125 305	3,0/4 125 270	3,0/3 125 250	3,0/3 125 230								
	2,75/8 160 490	3,0/6 160 430	2,75/5 160 385	3,25/4 160 345	3,0/4 160 315	3,25/3 160 285	3,0/3 160 265							
3,5/8 200 710		2,75/8 200 535	3,0/6 200 475	3,0/5 200 430	3,25/4 200 390	3,25/4 200 360	3,25/3 200 330	3,25/3 200 305						
4,0/8 250 880	4,0/6 250 750	3,0/8 250 660	2,75/8 250 590	3,25/6 250 530	3,0/5 250 485	3,5/4 250 440	3,25/4 250 405	3,5/3 250 380	3,25/3 250 355	3,25/3 250 335				
	4,0/8 320 960	4,0/6 320 840	3,0/8 320 755	3,0/8 320 670	3,25/6 320 610	3,25/5 320 560	3,25/5 320 515	3,5/4 320 480	3,25/4 320 445	3,5/3 320 425	3,5/3 320 400			
	4,5/8 400 1215	4,0/8 400 1060	4,5/6 400 940	3,25/8 400 850	3,25/8 400 770	3,5/6 400 700	3,5/5 400 645	3,25/5 400 605	3,5/4 400 560	3,5/4 400 535	4,0/3 400 500	3,5/3 400 475	3,5/3 400 445	
		4,5/8 500 1300	4,0/8 500 1150	4,5/6 500 1025	4,5/5 500 940	3,5/8 500 855	3,5/6 500 790	3,5/5 500 740	3,5/5 500 685	4,0/4 500 650	4,0/4 500 605	4,0/3 500 570	4,0/3 500 545	4,0/3 500 515

← 1000Upm | 1500Upm →
rpm rpm

FIG 11-13 FRANCIS MINI SET VERTICAL (PRODUCER KÖSSLER)
FRANCIS KOMPAKT-SPIRALTURBINEN (FIRMA KÖSSLER)

11 GENERAL VIEW AND APPLICATION FOR SMALL HYDRO TURBINES
ÜBERSICHT UND EINSATZBEREICH VON KLEINEN WASSERKRAFTTURBINEN

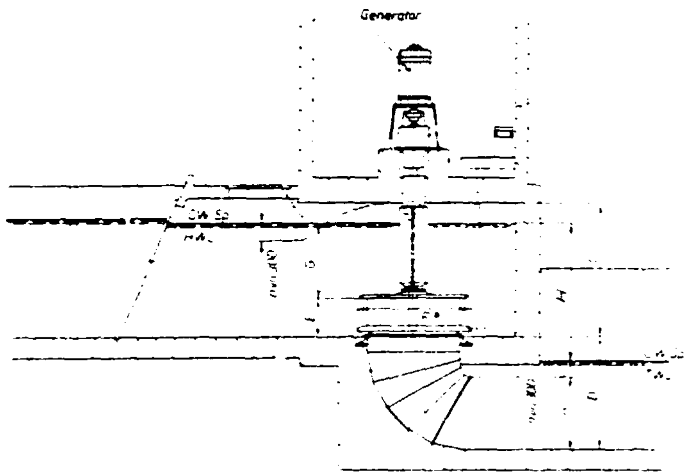


FIG 11-14 CONSTRUKTION EXAMPLE FOR SMALL HEAD FRANCIS TURBINES (PRODUCER VOITH)
EINBAUBEISPIEL FÜR FRANCISTURBINEN MIT KLEINER FALLHÖHE (FIRMA VOITH)

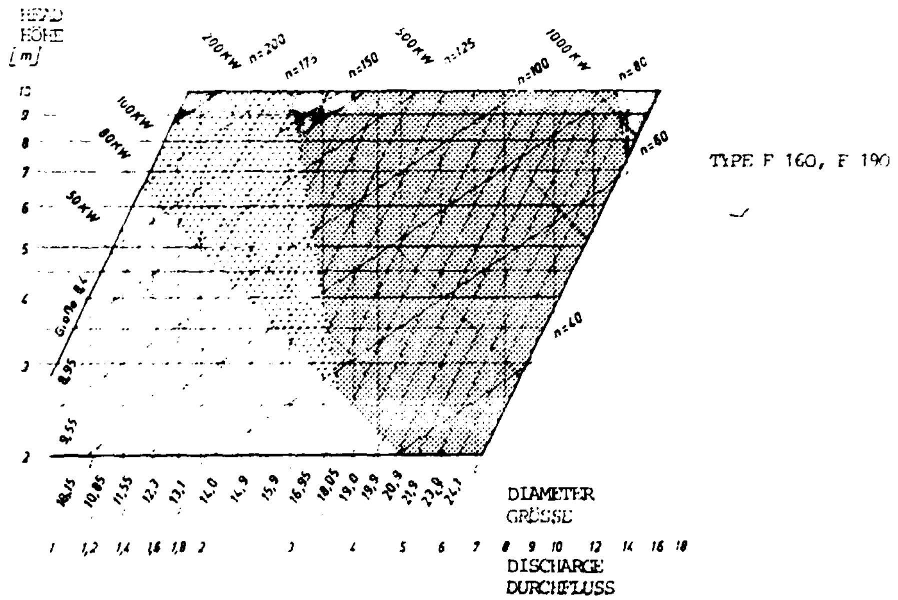
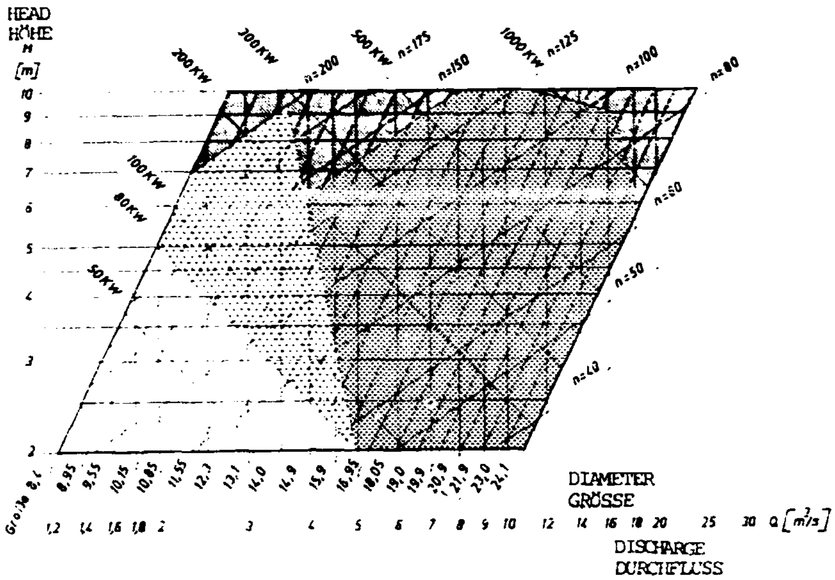


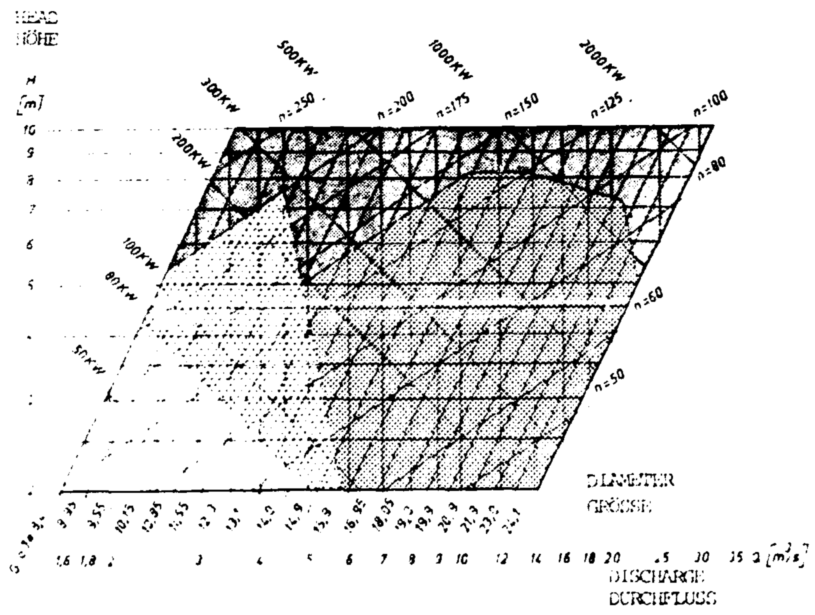
FIG 11-15 LAY OUT DIAGRAM FOR VOITH-FRANCIS TURBINES
AUSLEGUNGSDIAGRAMM FÜR VOITH-FRANCISTURBINEN

11 GENERAL VIEW AND APPLICATION FOR SMALL HYDRO TURBINES
ÜBERSICHT UND EINSATZBEREICH VON KLEINEN WASSERKRAFTTURBINEN



TYPE F 225

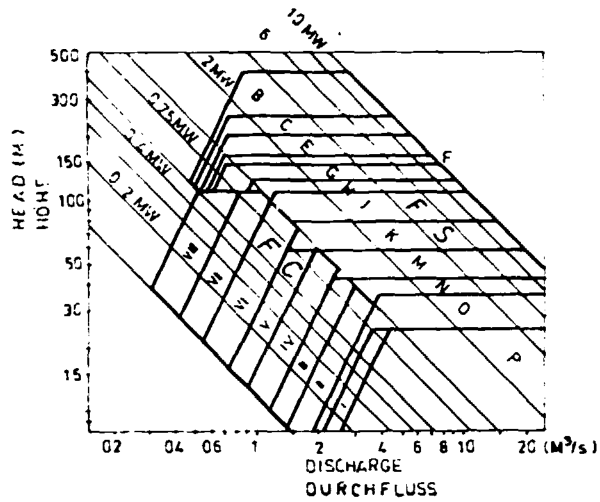
DIFFERENCE EQUIPMENTS
VERSCHIEDENE AUSSTATTUNGEN



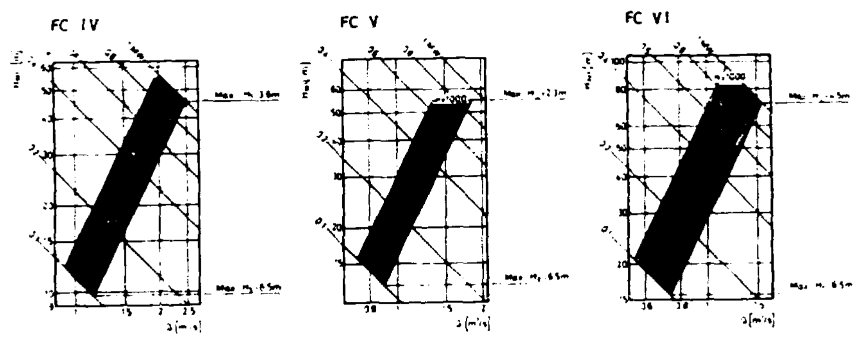
TYPE F 260, F 295
F 330

FIG 11-16 LAY OUT DIAGRAM FOR VOITH-FRANCIS TURBINES
AUSLEGUNGSDIAGRAMM FÜR VOITH-FRANCISTURBINEN

11	GENERAL VIEW AND APPLICATION FOR SMALL HYDRO TURBINES OBERSICHT UND EINSATZBEREICH VON KLEINEN WASSERKRAFTTURBINEN

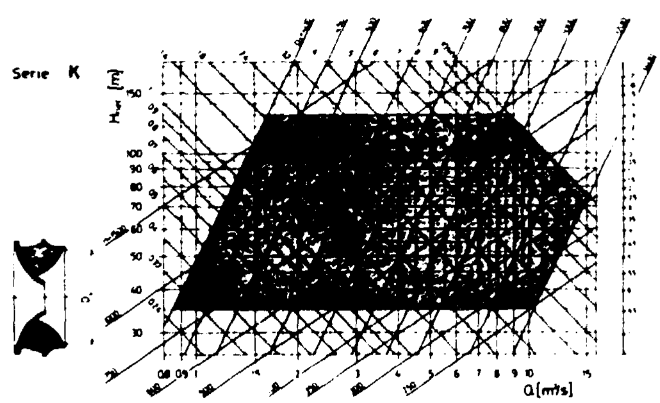


STANDARD PROGRAM FOR FRANCIS TURBINES
STANDARDPROGRAMM FÜR FRANCISTURBINEN



LAY OUT EXAMPLES FOR TURBINES TYPE FC

AUSLEGUNGSBEISPIELE FÜR TURBINEN DER TYPE FC



LAY OUT EXAMPLE FOR TURBINES TYPE FS

AUSLEGUNGSBEISPIEL FÜR TURBINEN DER TYPE FS

FIG 11- 17	FRANCIS-TURBINES FROM PRODUCER SANDEN FRANCISTURBINEN DER FIRMA SANDEN

11 GENERAL VIEW AND APPLICATION FOR SMALL HYDRO TURBINES
ÜBERSICHT UND EINSATZBEREICH VON KLEINEN WASSERKRAFTTURBINEN

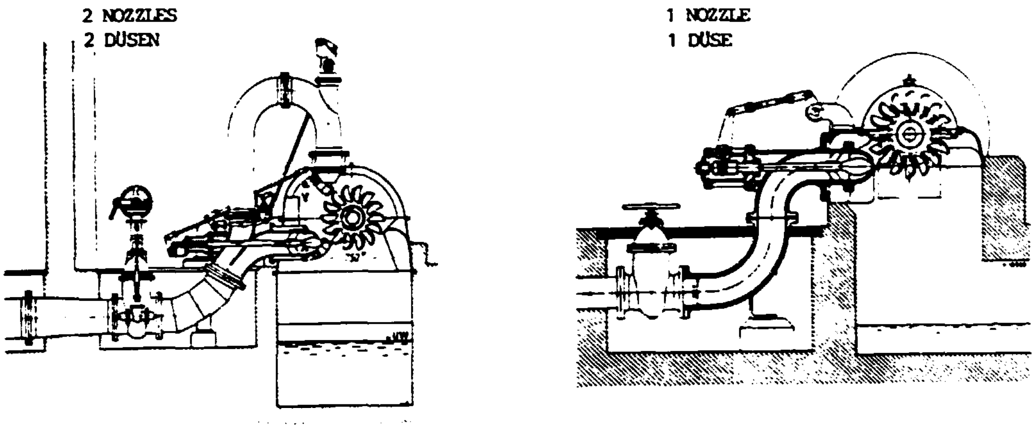


FIG 11- 18 PRINCIPLE CONSTRUKTIONS FOR PELTON TURBINES (VOITH)
KONSTRUKTIONSPRINZIPIEN FÜR PELTONTURBINEN (VOITH)

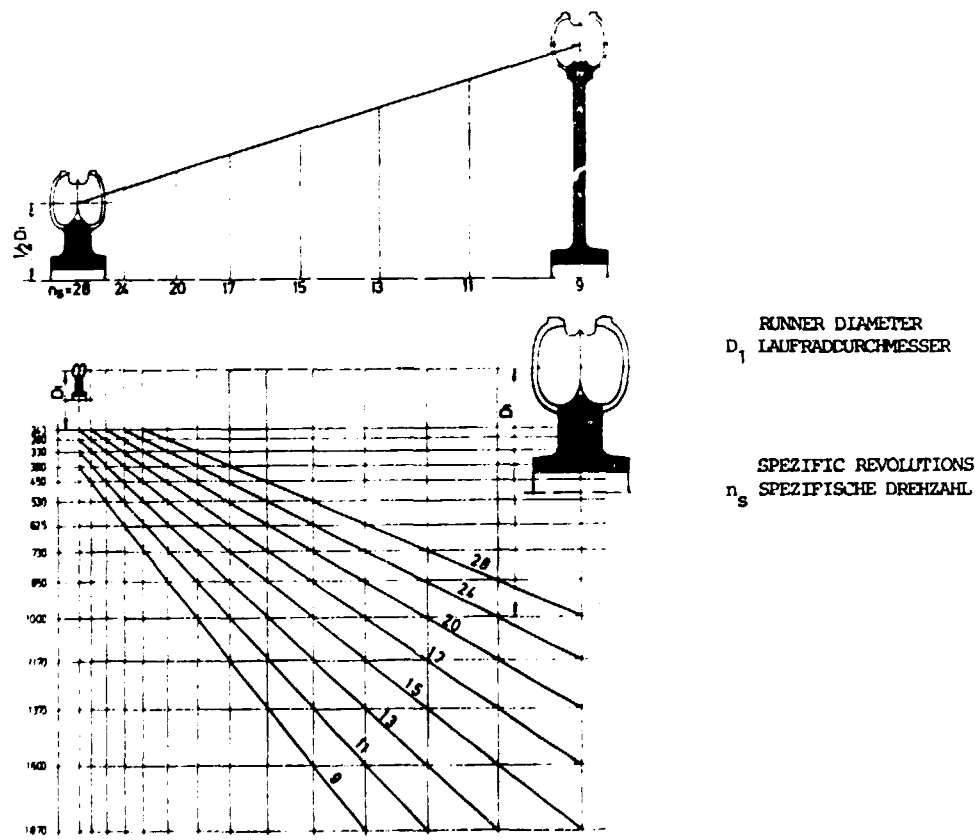
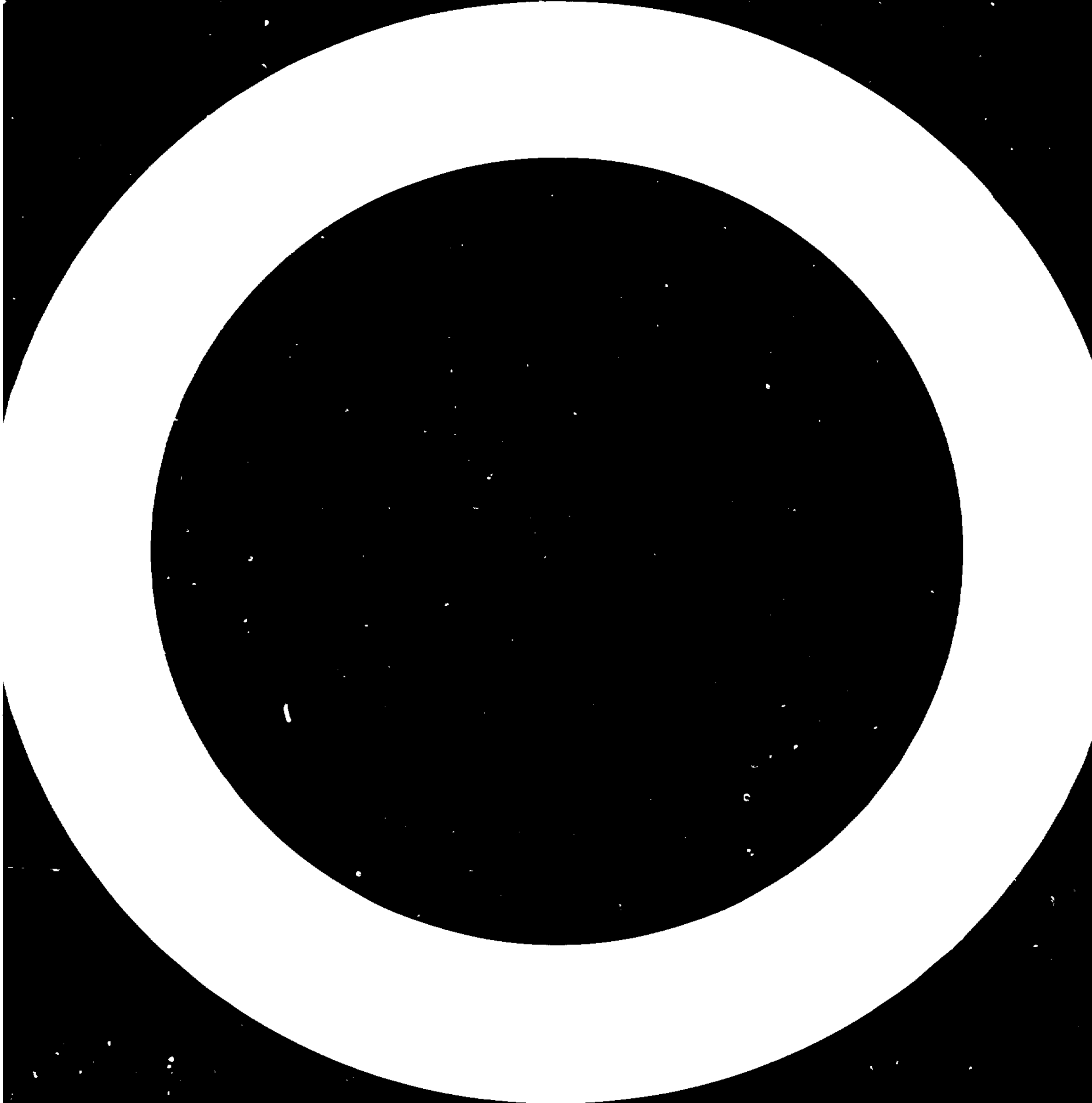


FIG 11-19 STANDARDLINE FOR KÖSSLER IMPULSE RUNNERS
NORMENREIHE FÜR KÖSSLER FREISTRAHLLAUFRÄDER



12 ELECTRICAL EQUIPMENT IN MINI HYDRO STATIONS

HOFBAUER E.

ABSTRACT:

The operation of mini hydro stations requires electrical switching, metering and control equipment. In addition, provision must be made for installations that ensure a reliable and economical operation. In the following chapters, the author will describe these installations as well as all the other plant components necessary for the use of electricity in a private producer's industrial plant as well as for the supply of electricity to the public system.

12.1 TECHNOLOGICAL STANDARD OF THE ELECTRICAL EQUIPMENT

In mini hydro power, economy is normally a prime consideration. Nevertheless, or perhaps for this very reason, a certain technological standard should be guaranteed.

Where a mini power station supplies electricity to a superior system of an electricity undertaking, account must necessarily be taken of the operating conditions of this superior network.

These introductory remarks have been made to emphasise the chief problem in the planning, design and operation of mini hydro stations, namely reliability.

In the following, questions of detail will be discussed. Figure 12-1 is a schematic diagram of such an installation. This is based on the assumption that there is a low-voltage generator whose output is fed to a station bus and that this can be connected, through appropriate equipment, to the bus bar of the electricity undertaking. The high-voltage side of this is connected to the public supply system.

Although the type described here is the most usual, there is also a possibility of providing a generator transformer immediately after the generator so that the station bus is supplied on its high-voltage side right away.

Even in this case, which is fairly rare, all the installations described in the following should be provided analogously.

12.2 GENERATOR

The generator power rating should be adapted to that of the turbine with allowance being made for existing product lines. Excessive variations from the power reached for the highest number of utilisation hours p.a. reduce efficiency. The annual number of utilisation hours is derived from the electrical energy generated in a year, divided by the installed generator power.

Figure 12-2 is a diagram showing a typical efficiency curve as a function of power. The generator should be of the synchronous type. This also enables isolated operation of a separate network independently of the public supply system. It is only in cases where operation of an isolated network is practically precluded from consideration that the much cheaper asynchronous generator should be chosen.

An important criterion for the selection of the generator is the design of its bearings. These should be designed for a period of service of two-hundred-thousand hours.

12.3 GENERATOR CONTROL AND PROTECTIVE RELAYING EQUIPMENT

The following are indispensable components of the control and protective relaying equipment:

- voltage regulation
- speed control
  This is probably one of the most important elements in power plant operation.
- turbine overload protection
- generator overload protection
- reverse power protection
  This protection is particularly important because in the case of faults in the control equipment or in low-flow periods, there is a potential risk of the generator passing over to motor operation, being driven by the public system. The lack of a reverse power protection is one of the most frequent causes of failure in badly designed mini hydro stations.
- earth leakage protection
- overcurrent protection
- voltage increase protection
- underfrequency protection
  This is essential to orderly parallel operation with a public network in order to avoid potential disturbance from irregularities in the operation of the mini hydro station.

Apart from these protections, in mini power stations with capacities exceeding 1000 KW,

- temperature detectors in the windings

and a

- bearing temperature monitor

can be provided.

12.4 METERING AND INSTRUMENTATION

The operation of the power station and the private producer's plant requires instruments indicating the following quantities:

- voltage
- current of all phases
- power
- power factor cos phi
- turbine speed
- generator frequency
- power printer if necessary for the requirements of the private producer's plant
- two-rate meter for the electrical energy generated, both for high-load hours and low-load hours
- hour meter
- frequency of superior network

12.4.2 Metering and Instrumentation for the Supply to the Superior Network

Two sets of instruments of the same type should be provided for the supplies to and from the superior network. These should consist of the following instruments:

- power meter with maximum indicator and automatic resetting after a controllable time, usually five minutes
- two-rate meter for active energy during high-load and low-load hours
- two-rate meter for reactive energy during high-load and low-load hours
- power factor meter for cos phi if necessary

12.5 POWER STATION BUS

The generator output is fed, through a power switch, to the station bus.

Connected to this bus is also the station-service system supplying energy to the station auxiliaries such as lighting, pumps, gate controls etc.

Prior to power station start-up, these auxiliaries must be supplied from a battery or an emergency plant - as e.g. the lighting system and the governors - or hand operated as e.g. the gate leaf.

This station bus should be connected either directly to the system of the private producer's plant unless this is fed from the public system, or the power-station and public-supply busses should be synchronised prior to connecting the two systems.

12.6 SYNCHRONISING RELAY

Depending on the size of the private producer's plant, a simple or an automatic synchronising relay should be furnished. In fully automatic installations requiring no operating staff, provisions can be made for automatic start-up of the mini hydro station following an interruption of operation. These start-up processes should be prevented from automatic restarting after failure of two attempts to start, and should in such a case be checked by the service crew.

12.7 PRIVATE PRODUCER'S SYSTEM

The private producer's system is energised either from the public supply bus or from the station bus, depending on whether or not sufficient energy is available from the station auxiliaries.

Connected to the private producer's system are all consumers that are not part of the power station operation proper, i.e., in an industrial plant, all production machinery and other plant for meeting lighting, power and heat requirements.

For the intra-plant energy balance, it is usually necessary to furnish metering devices on all the individual outgoing lines that are equipped with switching and cut-out mechanisms. Provision of single rate meters for the active energy is usually sufficient.

12.8 CONNEXION OF THE PRIVATE PRODUCER'S SYSTEM TO THE PUBLIC SUPPLY

For the supply both from and to the public system, provision should be made for connecting the two systems.

Normally the systems are connected on the high-voltage side. For this purpose, a transformer station is furnished in the private producer's plant and this supplies, through an appropriate switchgear, a low-voltage bus, which we will call "public-supply" for the example under discussion.

In selecting the electrical data, careful allowance should be made for the characteristics of the public supply system.

In this case, it is the short-circuit power of this system which can only be determined in close cooperation with the operators of the public system.

The rating, in particular that of the switching devices, should be based on that maximum short-circuit power which can occur over the service life of these plant components, i.e. ordinarily a period of twenty years.

12.9 VOLTAGE LEVEL OF METERING FOR ELECTRICITY SUPPLIES

The question to be answered first is whether metering for electricity supplies should be on the high-voltage or on the low-voltage side. This depends on the average load percentage and the number of utilisation hours of the plant.

In the case of three-shift operation, demands not greater than 500 KW will already require the provision of high-voltage metering.

In plants with low average load percentages, that is to say, in the case of non-continuous supply to the station, and for demands not exceeding about 600 KW for supplies both to and from the plant system, low-voltage metering can be provided.

12.10 SUPPLY STATION

The supply point of which the instruments and metering devices of the supply undertaking as well as the power switch are accommodated will have to be accessible to the staffs of both the supply undertaking and the private producer's plant. The minimum metering requirements to be furnished consist in one set of instruments each as specified under 12.4.2, i.e.

power meter and
two-rate meters both for active energy and reactive energy.

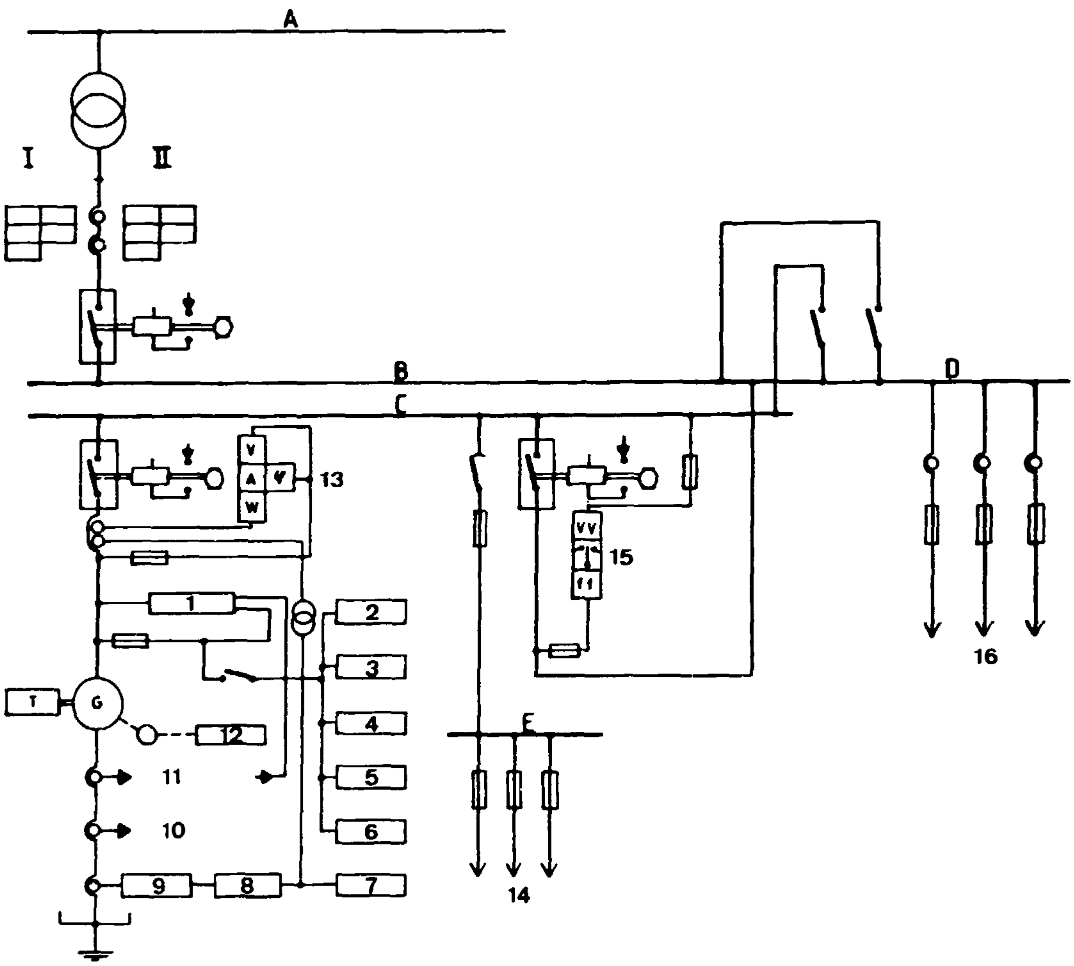
It will be useful to provide a maximum demand monitor to prevent demand peaks.

12.11 SUMMARY

The electrical equipment of private industrial generating plants and mini power stations should be of simple and clear design.

In the case of faults or irregularities during operation it is necessary that these facilities be automatically islolated from the system or that the turbine be shut down, especially where these are fully automatic with no operating staff being present.

The plant components likely to have effects on the public supply system should be selected jointly with the operators of the supply system already at the design stage.



I ... Purchase Metering / Bezug - Messung
II ... Delivery Metering / Lieferung - Messung
A ... Public Supply System / EVU-Netz
B ... Public Supply Bus / EVU-Schiene
C ... Powerhouse Bus / Kraftwerkschiene
D ... Supply System / Betriebsnetz
E ... Auxiliary Bus / Eigenbedarf-Schiene
G ... Generator / Generator
T ... Turbine / Turbine
1 ... Voltage Control / Spannungsregelung
2 ... Reverse Power Protection / Rückleistung
3 ... Turbine Overlood Protect. / Turbine Überlast
4 ... Voltage Increase / Spannungssteigerung
5 ... Frequency Deviation Protect. / Frequenzdifferenz
6 ... Overvoltage Protection / Überspannung
7 ... Earth Leakage Protection / Erdschluß
8 ... Overcurrent Protection / Überstrom
9 ... Generator Overlood Protect. / Generator Übersicht
10 ... Excitation Support / Stützerregung
11 ... Static Control / Reglerstatik
12 ... Speed Control / Drehzahlregelung
13 ... Generator Metering Devices / Generator Meßanzeigen
14 ... Powerhouse Auxiliaries / Kraftwerk Eigenbedarf
15 ... Synchronising Equipment / Synchronisiereinrichtun
16 ... Consumers / Verbraucher

FIG 12 - 1 SCHEMATIC DIAGRAM OF A MINOR POWER PLANT
SCHALTSCHEMA EINES KLEINKRAFTWERKES

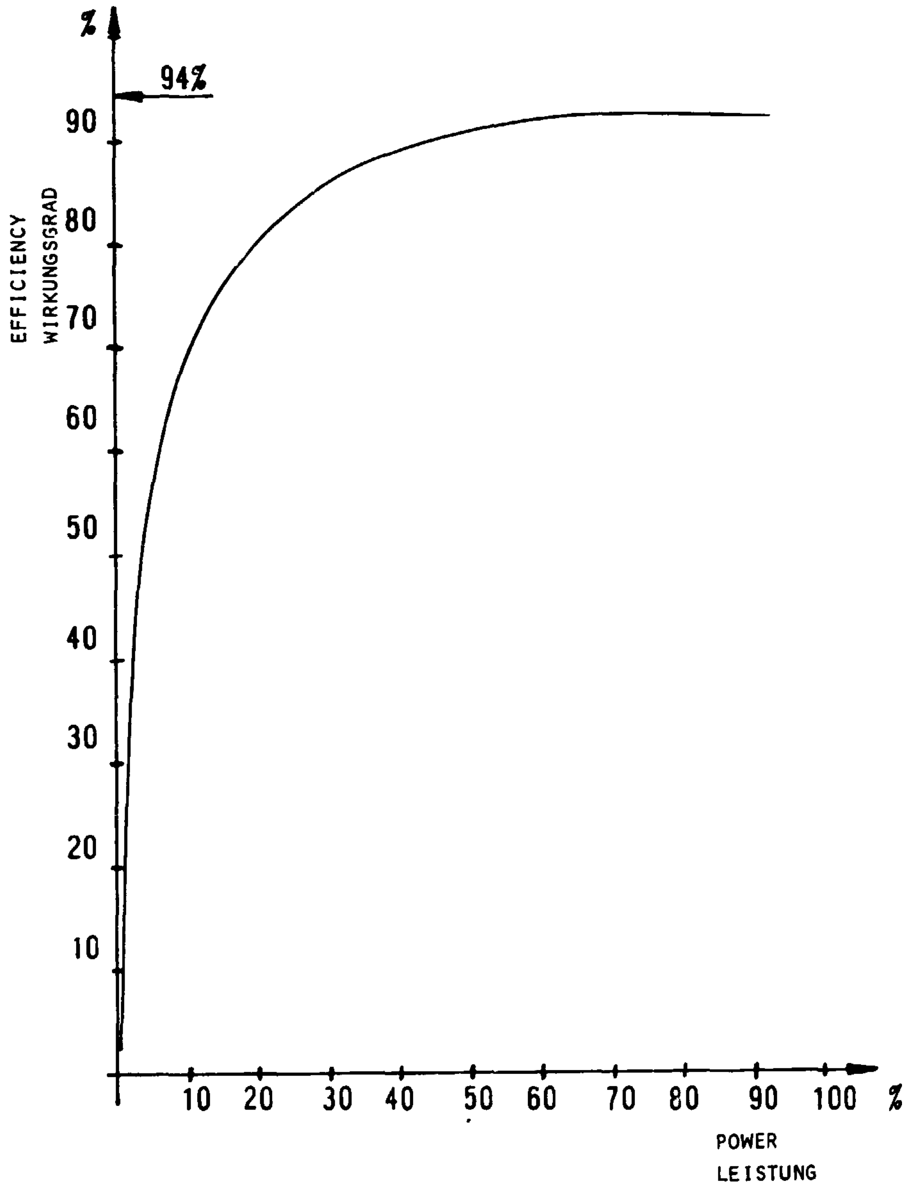
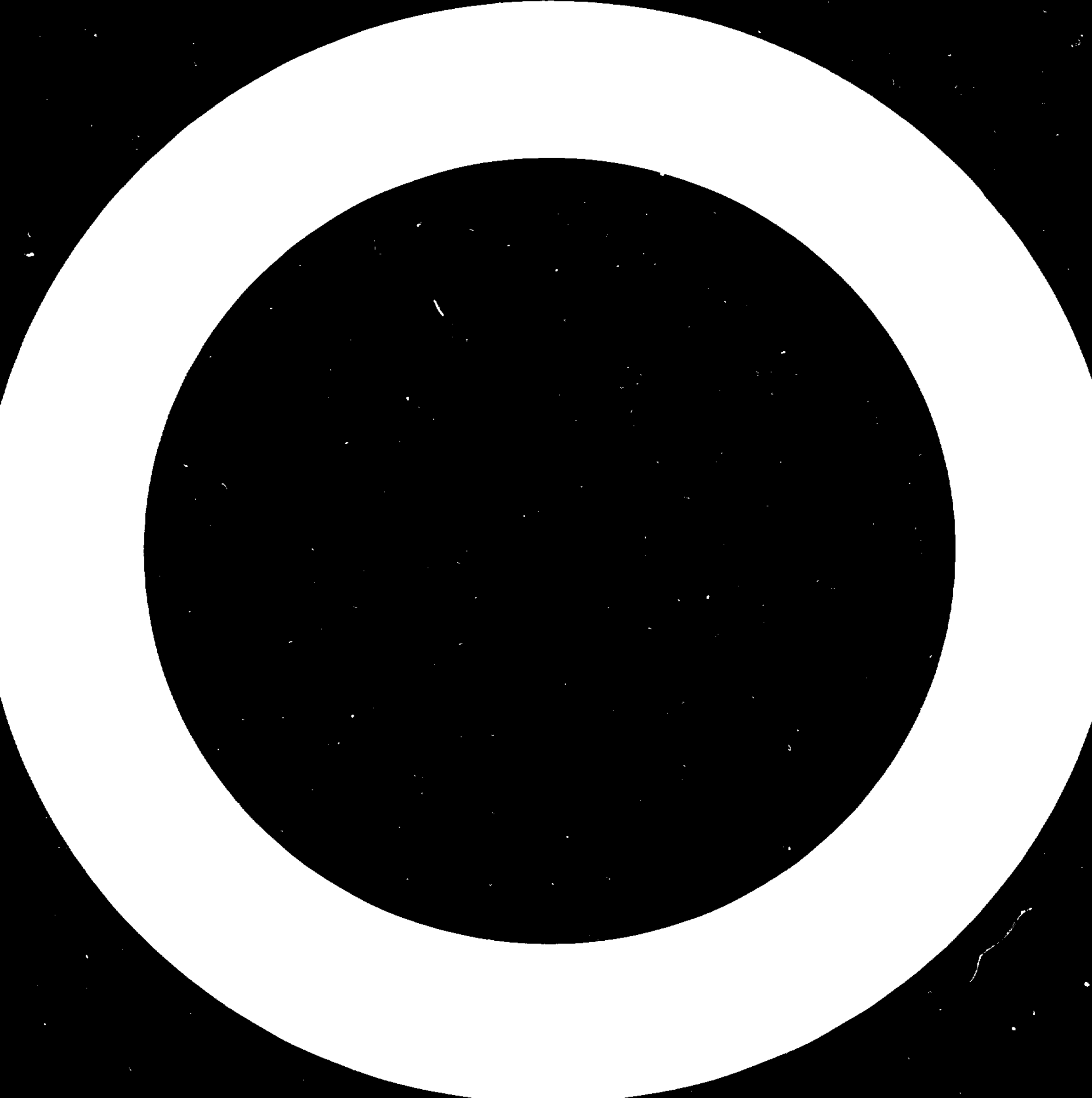


FIG 12 - 2 EFFICIENCY OF A SYNCHRONOUS ALTERNATOR
WIRKUNGSGRADKENNLINIE EINES SYNCHRONGENERATORS



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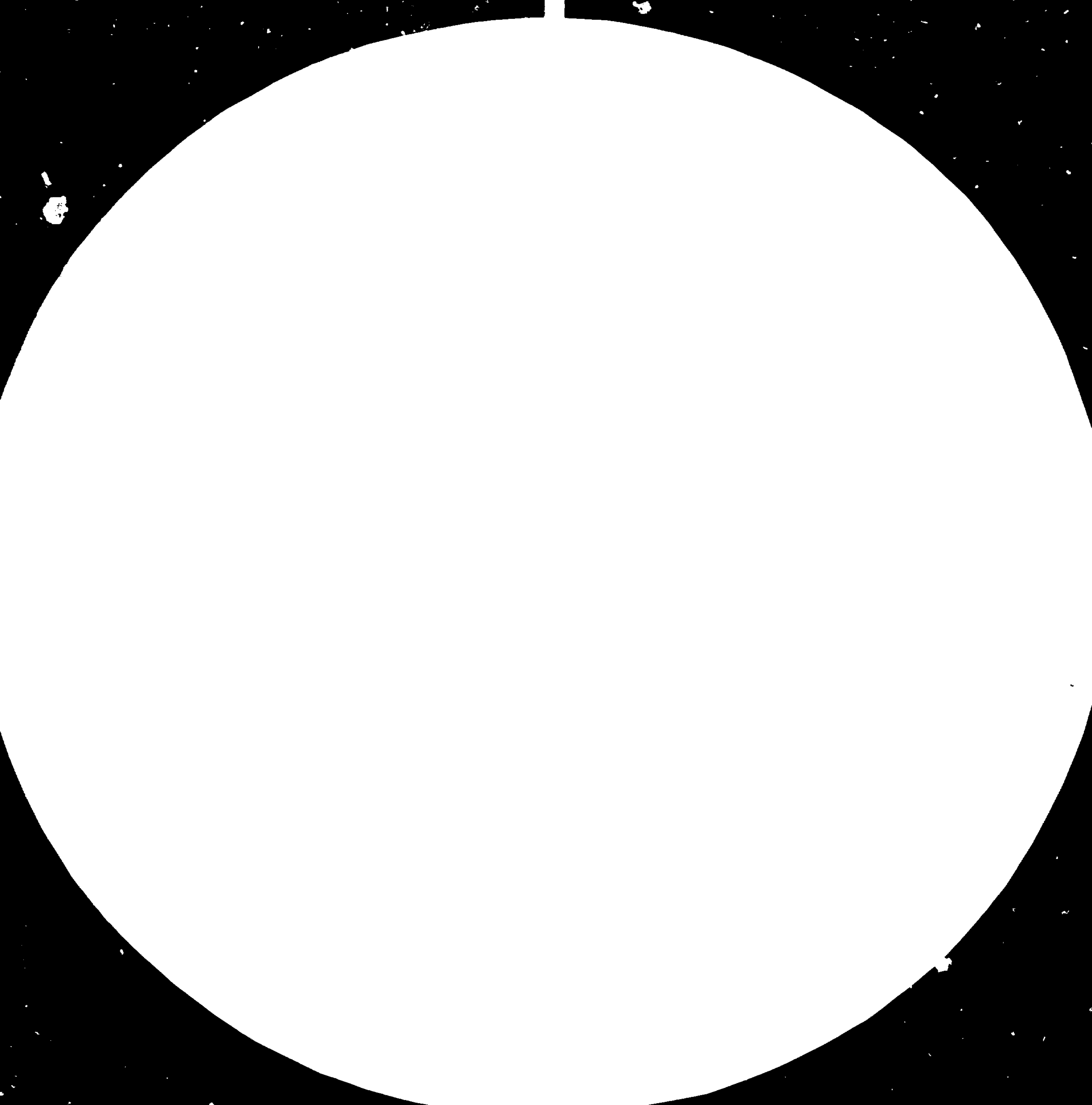

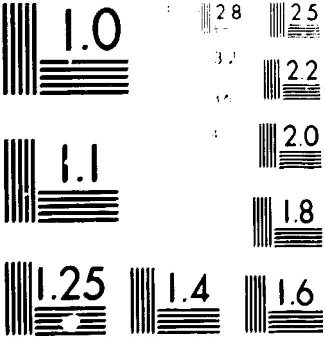

MICROCOPY RESOLUTION TEST CHART

13 STUDY OF THE MACROECONOMIC IMPORTANCE AND COST STRUCTURE OF MINI HYDRO SCHEMES

STIGLER H.

ABSTRACT:

This Report presents a thorough study of Austria's macroeconomic situation of her balance of payments and current accounts as well as her balance of trade. Problems dealt with in detail include Austria's dependence on power imports, the trend of the exchange rate of the Austrian schilling in relation to power clearing currencies, the necessity of economic growth to ensure full employment, and the increased power consumption involved, as well as national indebtedness with all the implications of these factors and their development over the last few years. A brief survey is given of Austria's precarious power situation in view of her limited fossil reserves and her dependence on imports, which is believed to increase in the future as her resources are running out. For the appraisal of the economy of mini power installations, an analysis of the cost structures of the main plant components will be presented in mathematical form. Evaluation of the electrical energy produced will be based on typical hydrographs from the different types of drainage basin.

13.1 INTRODUCTORY REMARKS

Ever since the energy crisis of 1973-74 and the new one of 1980, mini hydro power has been in the focus of power discussions besides other possibilities of utilising non-conventional sources of energy. A characteristic feature of hydro power is its constant self-renewal and regeneration, which render it inexhaustible. On the other hand, however, its potential is limited. The importance of mini hydro for Austria's power economy can be appraised only when considering it against the background of the macroeconomic situation. Therefore, this Report will discuss in some detail the relevant spheres of economy, study the possibilities of overcoming the problems involved and present the conclusions to be drawn.

In order to be able to appraise mini hydro power from the viewpoints of the country's economy as a whole and of the economy of individual plant management, we will have to study the initial cost of such an installation, and this will be done by means of cost-structure analyses for individual plant components. The value of the electrical energy generated is strongly dependent on the annual pattern of power production. In this respect we can differentiate roughly four types of stream basins, which are characterised by different hydrographs underlying evaluation in terms of power economy.

13.2 AUSTRIA'S POWER SITUATION

Austria's energy production showed the following pattern in 1979 (see table 13-1):

Primary energy	Proportion (%)	of which imported percentage
Petroleum and petroleum products	50.4	84.6
Natural gas	17.9	55.7
Coal	15.1	78.4
Hydro	12.5	-10.0
Other	4.1	3.0

Table 13.1: Power production in Austria in 1979 (BRENNSTOFFSTATISTIK)

Austria's energy production in 1979 was 1.079 . 10^{18}J ≈ 1.07^ EJ. Out of this, about two thirds came from abroad. Whereas in 1953, Austria had been completely independent as to her power supplies, by 1980, she imported already two thirds of her power requirements. In 1990, Austria will have to import three quarters. This proportion will then continue to increase as is demonstrated in a simple manner by the situation with respect to the reserves of fossil sources of energy (see table 13.2)

in terms of 10^{18}J = EJ	certain and probable	possible and hypothetical
Lignite	3.4	6.4
Petroleum	0.97	1.45 - 1.7
Natural gas	0.51	2.22 - 2.33
Total	4.88	10.07 - 10.43

Table 13.2: Fossil energy resources in Austria, in terms of 10^{18}J (=EJ) (BAUER 1980)

13.3 MACROECONOMIC SITUATION

13.3.1 Balance of Payments on Current Accounts

Assessment of the macroeconomic situation should primarily be based on a study of the balance of payments and current accounts and its development over time. The following table shows the structure of the Austrian balance of payments and current accounts and its composition in terms of money (thousands of millions of Austrian schillings) for the year 1980:

1 Balance of trade (of which energy - 44.1)	- 87,5
2 Balance of service transactions	+ 40.3
3 Balance of effective transfers	- 0.2
4 Balance of payments on current accounts (1 + 2 + 3)	- 47.4
5 Statistical difference	+ 20.0
6 Balance of visible and invisible items (4 + 5)	- 27.4

Table 13.3: Net positions on the balance of visible and invisible items for the year 1980 in Austria, in thousands of millions of Austrian schillings (WIFO 1981)

The balance of trade, which will later be discussed in greater detail, includes not only imported food, semiluxuries, raw materials, chemical products, processed goods, data processing machinery, vehicles and other finished goods, but in particular fuel and power imports, which represent a debit balance of approx. 44,100 million Austrian schillings, i.e. about half the total liabilities resulting from the unfavourable balance of payments and current accounts (the only favourable item among all the above positions of the balance is that of processed goods).

The balance of service transactions (primarily composed of the profits from the tourist and travelling industries) is a traditional credit item and contributes a great deal towards improving the unfavourable results of the balance of payments on current accounts.

The effective transfers are composed of donations, retirement pensions and similar transactions and have developed from slightly favourable to slightly unfavourable balances.

The sum total of the three above positions is the balance of payments on current accounts, which has shown an alarmingly unfavourable trend over the last few years. Since differences in the evaluation of imports and exports as well as financial factors of influence are not reflected by the balance of payments on current accounts, this does not yet constitute a representative picture of the foreign relations of Austria's economy. It is only by introducing the so-called Statistical Difference that we obtain a corrected picture of the economic relations with foreign countries, which is expressed by the balance of visible and invisible items.

The largely parallel development of expenditures on power imports and of the balance of visible and invisible items is clearly recognised as shown plotted on fig. 13-1 (WIFO, 1981).

There is a marked increase in the net expenditure on power from 1974. The power expenditure development that has followed over the last few years speaks a clear enough language. The parallel development of the balance of visible and invisible items and of power imports reflects this relationship. Anticipated purchases made in 1977 in view of the impending introduction of the raised VAT rates on luxury articles led to a pronounced increase in the net liabilities of the balance of payments on current accounts, which however, was largely compensated by the recessional development of the year 1978, as also suggested by fig. 13-1.

13.3.2 Structural Comparison of the Trade Balances of 1970, 1975 and 1980

The relative developments of the individual positions of the respective trade balances are shown on the table on the next page.

The deficit shown on the 1980 trade balance mainly derives from the following positions (as percentages of the total trade balance deficit):

Power (-50.6 %), Vehicles (-19.0 %), Finished Goods (-16.3 %), Foodstuffs (-10.6 %), Chemical Products (-10.3 %). Only a very small percentage of this is balanced by the surpluses achieved on Processed Goods (+21.6 %). Imports increased by as much as 17 % (i.e. 315,000 million Austrian schillings) from 1979 to 1980, whereas exports rose by not more than 9 % (i.e. 227,000 million Austrian schillings) during the same period. This resulted in a 41 % increase in the trade balance deficit within a year (the balance of payments and current accounts rising by as much as about 80 % in the same period).

	1975	1980	1980: Percentage proportion of the the trade balance deficit
Foodstuffs	155.8	251.6	- 10.6
Semiluxuries	125.0	125.0	- 0.0
Raw materials	211.0	109.0	- 1.2
Oils	160.3	151.3	- 1.3
Chemical Products	94.1	189.2	- 10.3
Processed Goods	189.5	228.4	+ 21.6
DP Machinery	387.5	954.2	- 2.6
Vehicles	182.0	300.0	- 19.0
Other Finished Goods	260.5	1186.2	- 16.3
Other Goods	400.0	1300.0	- 0.1
Fuels, Energy	312.2	788.7	- 50.6
Total	180.5	498.2	-100.0

Table 13.3: Relative developments of the individual items on the trade balance for the years 1975 and 1980 as against 1970 (= 100); + denoting credit balance, - denoting debit balance (WIFO, 1981).

By 1980, specially the net power imports had risen to 7.9 times their value of 1970, accounting for more than half the total trade balance deficit. The net expenditures of the "Vehicles" item had trebled during the same period.

Although the power imports remained almost unchanged in terms of quantity from 1979 to 1980, the prices per unit of power rose by almost one-half. Whereas in 1970, the proportion of power imports accounted for not more than 8 % of the total imports and for about 32 % of the trade balance deficit, the corresponding proportions rose to 15 % and 50.6 %, respectively, by 1980.

13.3.3 Effects of Changes in Rates of Exchange with Clearing Currences, Power-GNP Elasticity and Import Dependence

It should be stressed in this context that there is a close relationship between the expenditure on power imports and the development of exchange rates, in particular that of the US dollar, as a large proportion of our power imports is invoiced in this currency. From 1970 the parity of the US dollar dropped continuously in relation to the Austrian schilling, which had a favourable effect for the hard-currency countries, to which Austria belongs, with respect to the foreign currency to be spent on power imports as compared with the price increase of the primary energies on the world market (which are for the greater part invoiced in US dollars). Soaring prices were paid by means of ever cheaper US dollars. The development of the rate of exchange between the US dollar and the Austrian schilling since 1970 is shown plotted on Figure 13-2.

Since 1979, the exchange rate of the US dollar has no longer dropped, but has begun to rise again. This means that the advantage derived from the relationship (rising world market prices of power) x (decreasing rate of clearing currency) is dwindling and that the higher world market prices are making themselves increasingly felt in our country.

It should be pointed out furthermore that the growing amounts of imported power are of paramount importance for our country's economy. Investigations carried out by the Österreichisches Institut für Wirtschaftsforschung (Austrian Institute of Economic Research) have shown that maintenance of full employment requires a growth in the gross national product of about 2.5 to 3.5 percent p.a. However, a growth in gross national product implies, in the long run, an increase in power consumption. This relationship can be expressed by the power-GNP-elasticity index following from the formula

$$\text{power elasticity} = \frac{\frac{\Delta E}{E}}{\frac{\Delta GNP}{GNP}}$$

where ΔE = growth in power consumption
E = total power consumption
ΔGNP = growth in GNP

Computations made by the above Institute have yielded an average index of 0.98 for the years from 1960 to 1975, which corresponds to a growth in power consumption of approx. 1 percent for a growth in GNP of 1 percent. This value dropped to 0.31 for the period 1973 to 1980. The elasticity index expected for the eighties is about 0.75, that is to say, a growth in GNP of 1 percent will involve an additional power consumption of about 0.75 percent.

There is an additional reason allowing the conclusion that the amount of imported power will continue to increase, and will do so at a greater rate. The small domestic reserves of energy (mentioned earlier in this Report) will necessarily call for constantly increasing power imports in the years to come. The Austrian Institute of Economic Research has calculated a power import proportion of about two thirds for the year 1980 and precasts a proportion of about three quarters for the year 1990.

The causality chain: full employment - growth GNP, GNP growth - increased power consumption, increased power consumption as well as decreasing domestic reserves - increasing power imports, increased power imports and recovered clearing currency as well as rising world market prices - increased expenditures on power imports - this clearly demonstrates what difficulties we will be faced with in respect of our balance of trade and our balance of payments on current accounts.

13.3.4 Austria's Indebtedness

Austria's total liabilities amounted to 1,000,000 million Austrian schillings in 1980. This corresponds to 114 percent of the gross domestic product.

The national debt, that is to say, the nation's total liabilities resulting from the sum total of all the budget deficits hitherto incurred was, in 1980, 261,000 million Austrian schillings, which corresponds to 97 percent of last year's national revenue, i.e. 26,500 Austrian schillings per inhabitant. The following table compares these figures with those of foreign countries:

Switzerland	20,200	AS per inhabitant
Federal Republic of German	20,800	-"-
Austria	26,500	-"-
Great Britain	41,000	-"-
U.S.A.	44,000	-"-
Sweden	47,000	-"-

Table 13.5: National debt per inhabitant in 1980 (FINANZSCHULDENBERICHT)

Thus, Austria is situated in the medium range, but it should be pointed out that in some of the above countries the gross national product per inhabitant is much higher than in Austria. Expressing national debt as a percentage of the gross domestic product gives the following international situation:

Federal Republic of Germany	14.0 %
Austria	23.6 %
Sweden	33.3 %
U.S.A	37.3 %
Great Britain	52.0 %

Table 13.6: National debt as percentage of the gross domestic product (FINANZSCHULDENBERICHT)

The development of national indebtedness and the share of foreign debts from the year 1960 is shown plotted on fig. 13-3. This clearly illustrates the marked increase in indebtedness since the mid-seventies, which has now already exceeded the foreign currency reserves.

13.3.5 Possible Remedial Measures

13.3.5.1 Devaluation of the Austrian Schilling

This measure would reduce the purchasing power auf the Austrian schilling abroad, which in turn would lead to an increase in import prices. This would raise the price level of imported goods on the domestic market, which would then lead to a decline in purchases. On the other hand, however, the cost of a unit of Austrian currency abroad would be reduced, which means that Austrian products would become cheaper abroad, and this ought to encourage Austrian exports. The sum total of these two effects would improve the unfavourable ratio of the Austrian trade balance. Since, however, the performance of production at home requires foreign products for further processing, foreign raw materials, chemical products, machinery, vehicles and, in particular, power, the goods to be exported would become more expensive due to the proportion of foreign supplies contained in them, which would in many cases swallow up the greater part of the favourable effect of lower export prices. The net result remaining in the end would often be nothing but higher import prices. Hence, this method does not appear to hold any promise of success.

13.3.5.2 Further Increase in the Balance of Service Transactions

An answer to the question regarding the feasibility of such a measure - which would largely correspond to an encouragement of tourist trade - would probably have to be a political one. If the answer is in the affirmative, intensification of tourist trade would result. The implications would be an increased load on the Austrian landscape, construction of holiday villages, increased employment in the sector of services such as skiing instructors, chambermaids, waiters etc. Already today, voices are raised against a continued and increased extension of tourist trade and against the conception of Austria as a holiday paradise. Thus, this also appears not to be the right course towards a solution of the problem.

13.3.5.3 Increased Industrialisation

The Austrian economy was characterised by "disindustrialisation" during the seventies. Thus, industrial investments in 1970 accounted for 20 percent of the total investments. This decreased to 12.7 and 13 percent by the years 1979 and 1980, respectively. The number of persons employed in the industry dropped from 672,000 in 1972 to 612,000 in 1978 and 630,000 in 1980.

Reanimated industrialisation results in more goods being produced at home so that imports decrease and exports increase. Here again, however, we must consider the fact that performance of production at home requires supplies from abroad, such as power, semifinished products, raw materials, machinery etc., which have an unfavourable effect on the balance of trade. Any reanimation of industrialisation ought to concentrate on classes of products which are for the most part manufactured at home and contain only small amounts of foreign supplies. This results in a list of goods that are power-extensive and contain only small proportions of raw materials or other foreign supplies. Therefore, efforts of encouragement or new installation should be directed at product classes whose manufacture involves intelligence (intelligent products), know-how, expert knowledge etc., whereas products requiring great amounts of power and raw materials (primary industries) should be repressed or should not be newly installed, as their output indirectly comes to a worsening of the balance of trade.

Mini power stations can contribute in two ways towards improving the situation described in the preceding paragraphs. On the one hand, they involve the utilisation of a regeneration and inexhaustible domestic source of energy, which would reduce the expenditure on power imports. On the other hand, Austria possesses experience gathered over many decades and an excellent international reputation in the construction of such installations, and this manifests itself in the export of such plants to countries all over the world. The export of mini power installations represents an export of top-quality industrial products, which could aid in improving our balance of payments and current accounts. Increased development of large-scale and mini-hydro power at home as well as the increased worldwide demand for Austrian hydro installations demonstrate that we have adopted the right course, as the other countries of the world have likewise recognised the necessity of an efficient utilisation of hydro power as a domestic source of energy in view of its importance for foreign trade relations.

13.4 ANALYSIS OF THE INITIAL COSTS OF SELECTED MINI HYDRO PLANT COMPONENTS (PRIEWASSER, 1979)

Economic assessment of a mini hydro scheme should preferably be based on the initial costs as modern automatic installations require only a small measure of operation and maintenance. As the specific costs (i.e. costs per unit of power) of a mini hydro scheme are not directly dependent on power, as would largely be true of a thermal plant, but are a function of the product of head and discharge, a direct dependence on power cannot be established. It should also be pointed out that the construction cost, being strongly dependent on the geological and hydrological conditions of the selected site, naturally defies mathematical limitation. Relationships permitting mathematical representation have, however, been found for certain other plant components, but not for all of them as certain facts may render expression by mathematical means impossible. The costs indicated in the following are based on 1978 prices. It should, however, be mentioned that innovations lately introduced have slightly changed some of the cost items. At any rate, offers should be invited when purchase is considered.

13.4.1 Specific Costs of Turbines Including Gates and Gearing

As the capacity of a hydro power scheme (in kilowatts) results from the product of head (in metres) and discharge (in cubic metres per second) multiplied by a constant factor (allowing for the efficiency of approx.8), the specific costs (in Austrian schillings per kilowatt) are also dependent on the respective head-discharge ratio. Fig. 13-4 shows the results of a mathematical determination of specific costs above the plane formed by head versus discharge. Plotting of this diagram was based on the values from more than 100 hydro installations. It clearly follows from this diagram that among hydro schemes with the same total capacity, those with the higher heads and the smaller discharges exhibit the more favourable cost pattern.

Naturally it is necessary to state probabilities of occurrence for cost figures as these are capable of reflecting cost as found in reality only with a certain probability. The following table shows the cost data underlying Fig. 13-4 as well as the interval limits for a certain probability of occurrence of the value to be found:

Turbine type	Cost function k=........AS/KW	Interval limits as percentages for a probability of occurrence of		
		95 %	67 %	50 %
Kaplan and bulb turbines	$78200.Q^{-0.35}.H^{-1.01}$	+ 33 - 26	+ 14 - 13	+ 10 - 9
Francis turbines	$158100.Q^{-0.46}.H^{-1.01}$	+115 - 54	+ 42 - 29	+ 27 - 21
Pelton turbines	$147300.Q^{-0.43}.H^{-0.83}$	+ 37 - 27	+ 15 - 14	+ 11 - 10

Table 13.7: Cost functions of turbines including gates and gearings for a certain probability of occurrence (Q in m³/s, H in m)

The above relationships will be explained with the help of a further diagram. For this purpose, we have taken a section along a 200-KW line on fig. 13-4. The result is shown on fig. 13-5. This clearly illustrates the above mentioned relationship between increasing head and decreasing specific cost for a constant total plant capacity. It is also seen from this figure that the transitions from one turbine type to the next are all marked by a major cost reduction, which can be explained by the fact that each following turbine type shows better adjustment to the respective head-discharge ratio. Fig. 13-5 indicates in addition the region of probability of occurrence of the specific cost forecasts for Pelton turbines, which suggests that the forecast value will fall within the hatched zone with a probability of 2 to 3.

13.4.2. Transformer Costs

Transformer costs are relatively easy to bring into a relationship as they are for the greater part dependent only on transformer ouput. The fundamental relationship is given by the following formula (based on 1978 prices):

$$\text{Transformer costs } k = \left(160 + \frac{41\,000}{P(kW)}\right) \text{ AS/kW;}$$

P = active transformer output

13.4.3 Steel Hydraulics Structures

In respect of steel hydraulics structures, it has been possible to establish cost functions for only part of the plant components concerned. These are listed in the table below:

Plant component	Cost function in AS/kW	Remarks
Rack	$700 \cdot Q^{0.11} \cdot H^{-1.0}$	Q > 2 m³/s variations are possible
Rack cleaner	(145500 + 10400.Q)/P	Q > 2 m³/s
Gate	50000 - 500000	no clear dependence
Shutters	23300 + 27500 . A	A = shutter area in m²; cost ± 25 %

Table 13.8: Specific costs of selected steel hydraulic structures

13.4.4 Generator Cost

The cost of a generator mainly depends on two factors: Firstly, the cost decreases continuously with increasing generator capacity, secondly it is favourably influenced by the speed, that is to day, the higher the motor speed (the smaller the number of pairs of poles), the lower the specific cost. Costs are shown on fig. 13-6.

13.4.5 Cost of Switchgear

We must differentiate between low-voltage switchgear (380 V) and medium-voltage switchgear (20 KV). Cost figures include ancillary parts such as power switches, protective equipment, synchronising equipment, short-circuit and differential protections, overcurrent and overvoltage protections, incidentals and the associated labour costs; no allowance has been made for the necessary transformers.

Specific cost curves can be seen from fig. 13-7.

13.5 PROPORTION OF STRUCTURAL ENGINEERING IN THE TOTAL COST

Various authors in Austria and abroad have sought to find a mathematical formulation for the share of structural items in the total cost. As mentioned earlier, however, this is extremely difficult in view of the substantial variation of both hydrological and geological factors. Below is a table listing empirical values for mini hydro schemes furnished by experts. For the reasons stated above, however, these only hold for Austrian conditions and should be considered as nothing but approximate reference values.

New construction of a high-head plant	75 %
New construction of a medium-head plant	65 %
Extension of an existing plant, rehabilitation	50 %

Table 13.9: Empirical values of percentage share of structural in total cost

13.6 HYDROGRAPHS AND DURATION CURVES OF STREAMS IN TYPICAL DRAINAGE BASINS IN AUSTRIA

The hydrographs and duration curves of streams have an important bearing on the value of the power generated. The smaller the difference between the amounts of energy generated in the winter and summer half-years, respectively, and the earlier or later in the course of the year the energy can be generated, the higher the value of generation.

For our purposes, Austria's geography allows differentiation of four types of drainage basin:

- High-altitude streams with glaciers in their catchments
- High-altitude streams without glaciers in their catchments
- Streams in the Alpine foreland, at the border of the Alps and at medium altitudes
- Streams in the Waldviertel and Weinviertel regions (Bohemian Massif to the north of the Danube).

The high-altitude streams with glaciers in their catchments exhibit pronounced high-water levels particularly in June and July as intense insolation causes increased melting of the snow and ice reserves. In autumn, in winter and well into the spring, their flows are small as precipitation is tied up in the form of snow or ice and thus prevented from running off.

The high-altitude streams without glaciers in their catchments have more balanced hydrographs as compared with the first group although flows also decrease substantially in winter. However, already in the months of March, April and May, flows rise markedly since the snow melts due to the increased temperatures and spring precipitation runs off without delay.

The rivers at the border and in the foothills of the Alps show a great variety of different flow characteristics. Maximum flows may occur very early in the year. The curves shown on fig. 13-8 are envelopes for the respective stream types. During the winter months, flow decreases to not more than 50 percent of the maximum values.

The streams in the Bohemian Massif to the north of the Danube have hydrographs that differ substantially both from large-size hydro and from the other stream types, which have their peak flows in the months of February and March, which is extremely favourable from the point of view of power generation. They show a pronounced secondary peak in autumn, and flow does not drop below 60 or 50 percent of the maximum values throughout the winter.

The hydrographs shown on fig. 13-8 relate the respective monthly mean of flow to the maximum flow, thus representing only relative values, which nevertheless illustrate very well the actual conditions.

13.7 CONCLUDING REMARKS

Against a background of dwindling domestic energy reserves and the growing import dependence involved, the importance of utilising any domestic source of energy cannot be over-estimated. Austria's economic situation is characterised by a rapidly increasing deterioration of the foreign-trade situation as well as by the unfavourable boundary conditions of foreign indebtedness, adverse balances of trade and of payments and current accounts, constantly rising world market prices of imported sources of primary energy of an ever decreasing quality, the necessity of an increasing power consumption to ensure growth of the gross national product and, hence, full employment etc. This calls for using to full advantage any possibility of domestic power production. Power development in mini hydro installations represents a domestic source of energy which is not subject to cost increase resulting from rising primary energy prices. Mini hydro schemes have long service lives; their operation involves no foreign currency expenditure and the investments made benefit the national economy.

On the other hand, however, mini power schemes are often higher in cost than comparable thermal plants. The smaller power units concerned involve higher costs of maintenance and personnel. The seasonal pattern of power production cannot be balanced and the potential is limited.

Decisions in favour or against mini hydro power should be based on a long-term appraisal as afforded by a cost-benefit analysis and should make allowance for all the aspects mentioned in the preceding paragraphs. Provided that the economy is ensured, however, any available possibility should be used.

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13 MACROECONOMIC RELEVANCE AND COST STRUCTURE
GESAMTWIRTSCHAFTLICHE BEDEUTUNG UND KOSTENSTRUKTUR

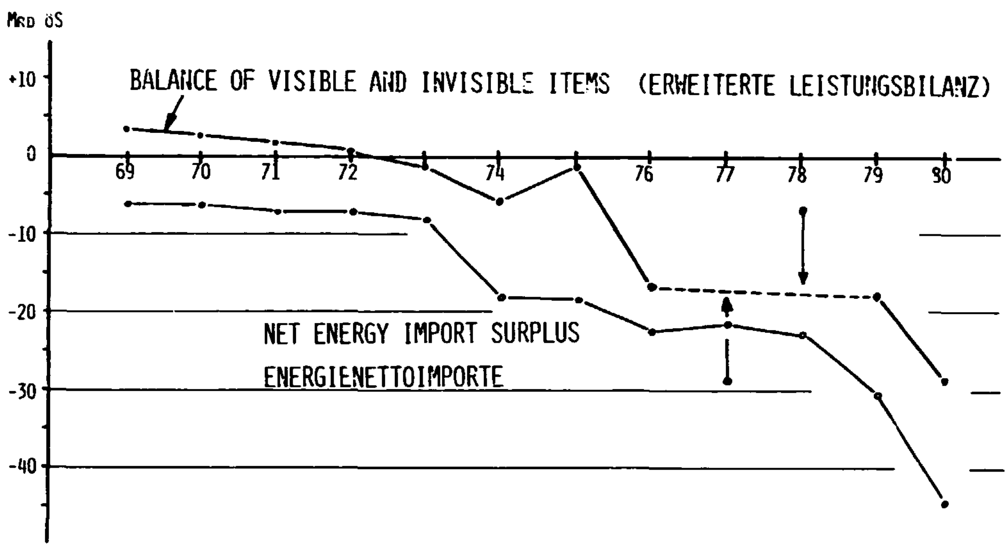

FIG 13 - 1 DEVELOPMENT OF EXPENDITURE FOR ENERGY IMPORT SURPLUS AND BALANCE OF VISIBLE AND INVISIBLE ITEMS SINCE 1970 (AS)

ENTWICKLUNG DER AUSGABEN FÜR NETTOENERGIEIMPORTE UND LEISTUNGSBILANDSALDO AB 1970 IN MRD ÖS (WIFO, 1981)

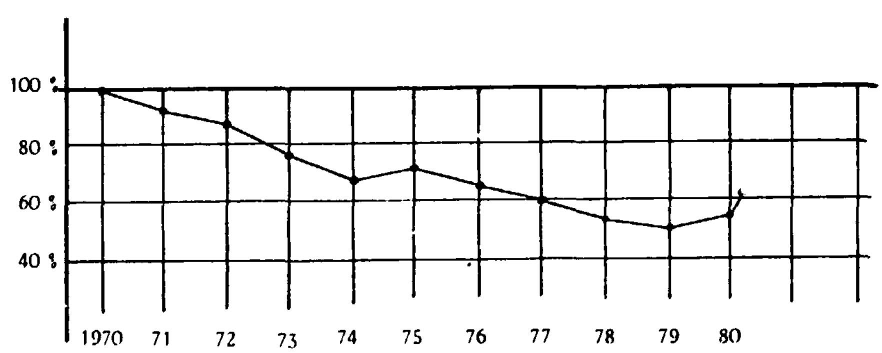

FIG 13 - 2 DEVELOPMENT OF DOLLAR-RATE TO AS AT THE END OF EACH YEAR
KURSENTWICKLUNG DES US-$ ZUM ÖS ZU JAHRESENDE (LÄNDERBANK, 1981)

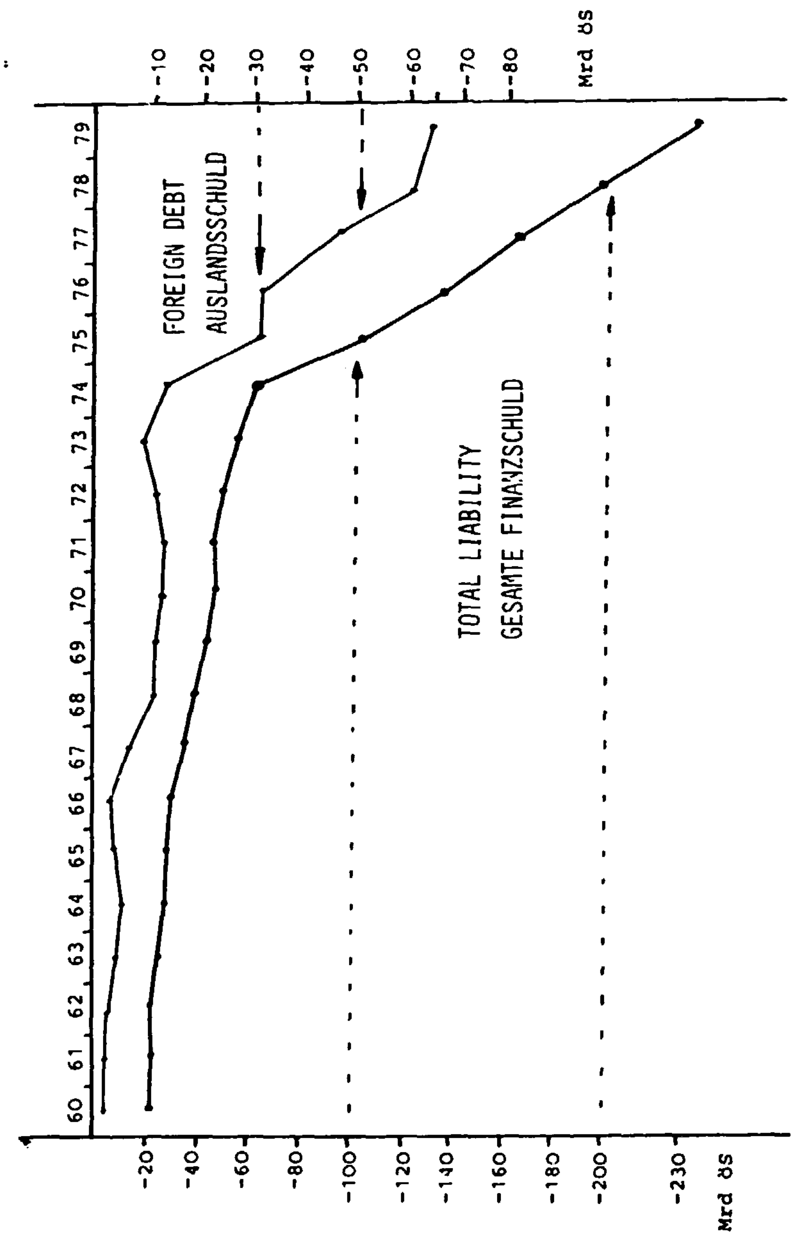

FIG 13 - 3 DEVELOPMENT OF TOTAL DEBT AND FOREIGN LIABILITY SINCE 1960
ENTWICKLUNG VON STAATSSCHULD UND AUSLANDSSCHULD SEIT 1960
(FINANZSCHULDENBERICHT 1981)

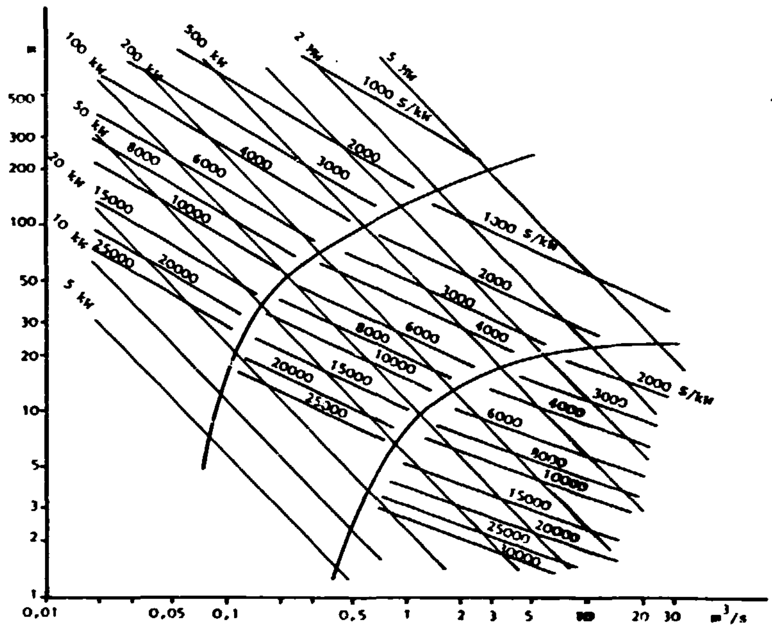

FIG. 13 - 4 SPECIFIC COSTS OF TURBINES INCLUDING FLYWHEEL, COUPLING AND TRANSMISSION
SPEZ. KOSTEN VON TURBINEN INCL. SCHWUNGRÄDERN, KUPPLUNG UND GETRIEBE

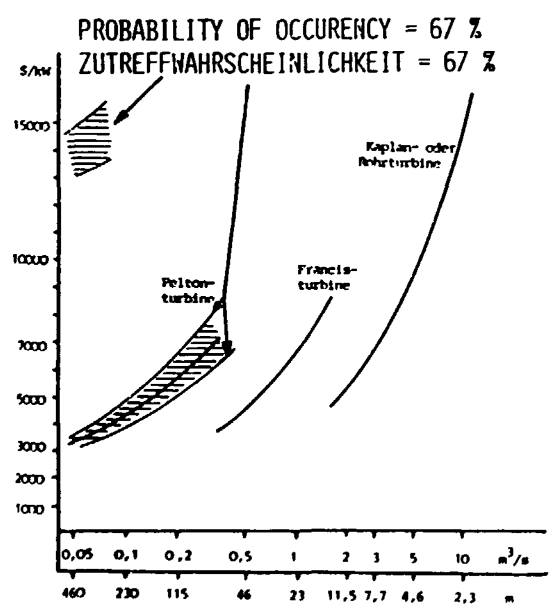

FIG. 13 - 5 CURVES OF SPECIFIC COSTS OF TURBINES WITH CONSTANT POWER OF 200 KW
KURVEN SPEZ. KOSTEN VON TURBINEN MIT KONSTANTER LEISTUNG VON 200 KW

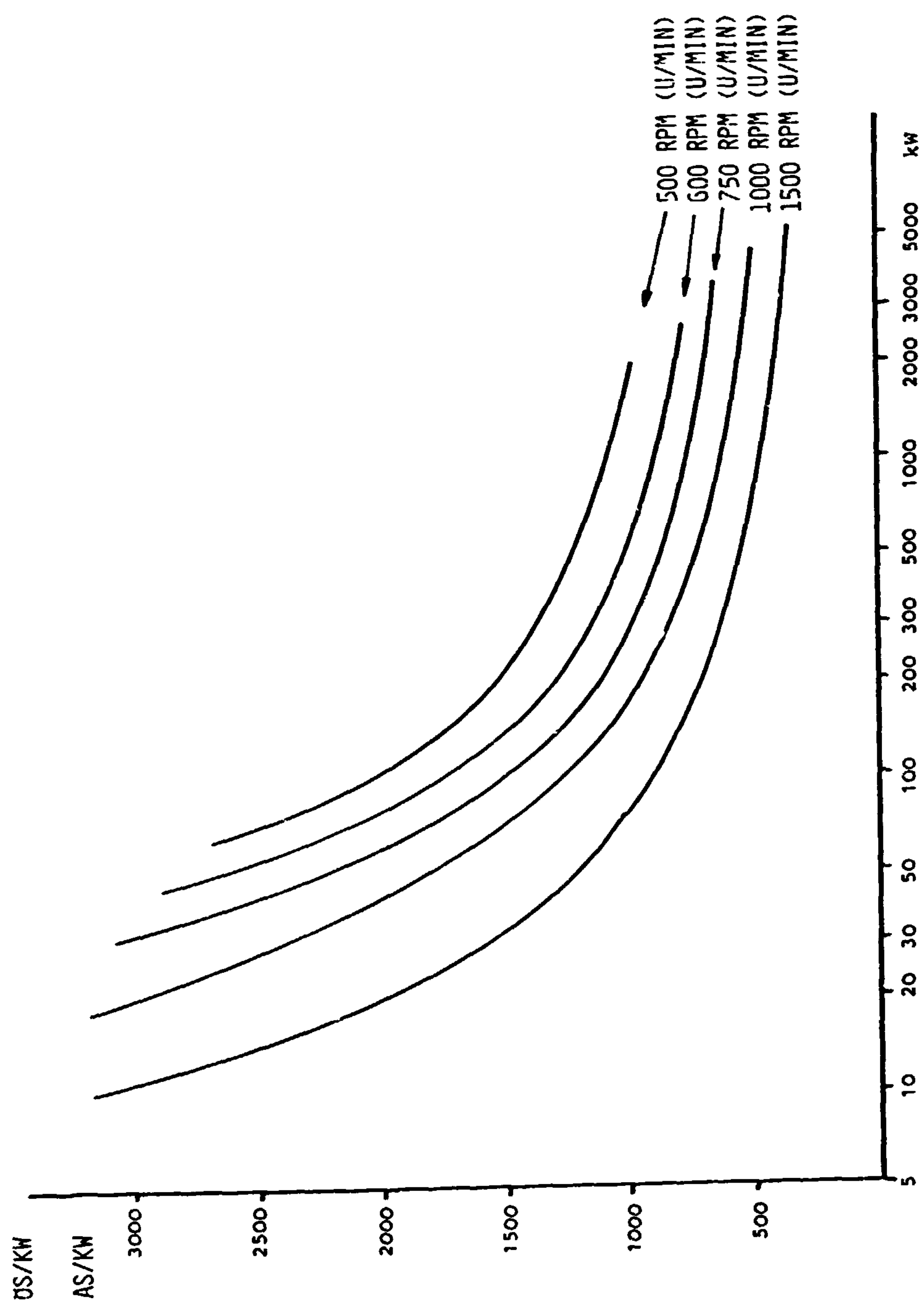

FIG 13 - 6 SPECIFIC COSTS OF GENERATORS DEPENDING ON POWER AND REVOLUTIONS PER MINUTE (RPM) (PRICE LEVEL 1978)

SPEZ. KOSTEN VON GENFRATOREN IN ABHÄNGIGKEIT VON LEISTUNG UND DREHZAHL (PREISBASIS 1978)

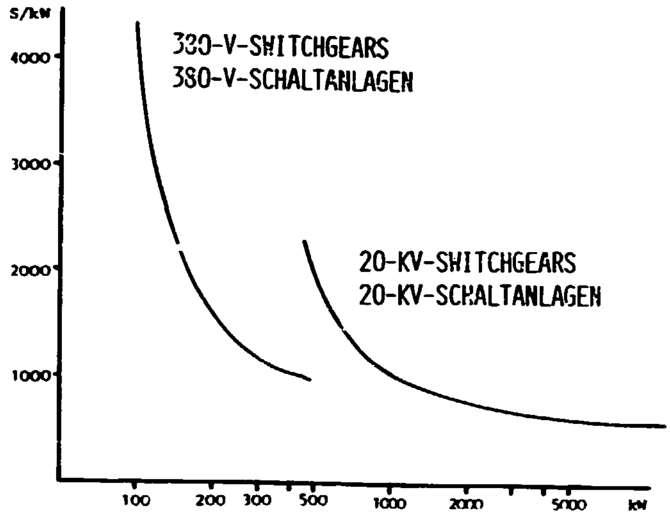

FIG 13 - 7 SPECIFIC COSTS OF SWITCHGEARS (PRICE LEVEL 1978)
SPEZ. KOSTEN VON SCHALTANLAGEN (PREISBASIS 1978)

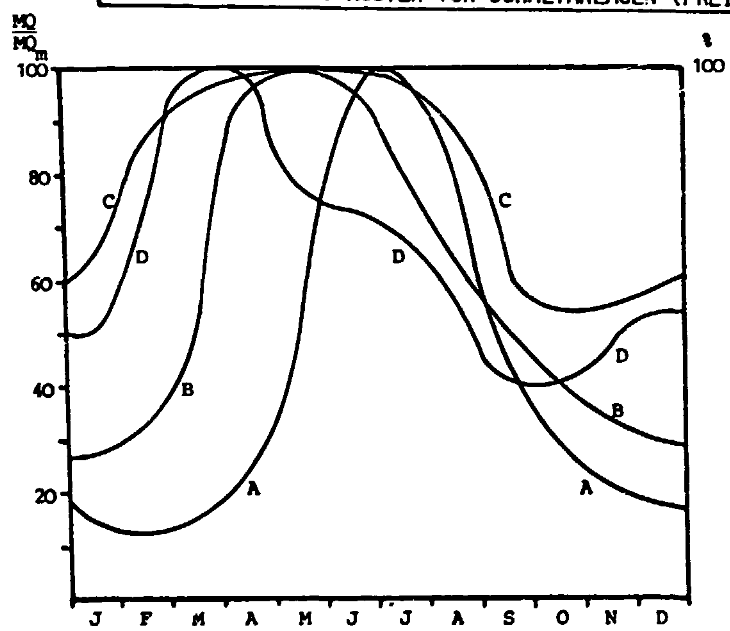

FIG 13 - 8 TYPICAL FLOW-OFF OF AUSTRIAN RIVER BASINS
TYPISCHE GANGLINIEN ÖSTERREICHISCHER FLUSSGEBIETE

A ... ALPINE RIVERS WITH GLACIERS (HOCHGEBIRGE MIT GLETSCHERN)
B ... ALPINE RIVERS WITHOUT GLACIERS (HOCHGEBIRGE OHNE GLETSCHER)
C ... FORELAND OF ALPS (ALPENVORLAND)
D ... WALD- UND MOHLVIERTEL (STIGLER, 1981)

14 ECONOMIC EFFICIENCY OF SMALL-SCALE HYDROPLANTS

NACHTNEBEL H.P.

ABSTRACT:

The economic efficiency of small-scale hydroplants may be assessed from the entrepreneur's viewpoint as well as in terms of political economy.

The following examples will describe several evaluation techniques, also including other possibilities of energy generation. Furthermore, the possibilities of a decision on grounds of economic efficiency, in the face of insufficient data, viz. uncertainty, will be discussed.

In conclusion of this contribution, there follows an interesting projection and realization of multipurpose projects, under consideration of multiple evaluation as well as qualification of costs and benefits.

14.1 INTRODUCTION

In determining the economic efficiency, various economic activities, like investment, have to be evaluated. This implies that the initial situation is known, that the feasable set, i.e. all possible alternatives, is determinable, that one or several targets are given, and it necessitates a scale for measuring the efficiency of the various alternatives.

As, in reality, each of these conditions will be satisfied to a certain degree only, this analysis of economic efficiency will frequently be restricted to special cases.

The target of this analysis is a definite evaluation of the alternatives, in order to choose the most favourable one. As is shown by means of the following techniques, definiteness is well possible here, if and when the viewpoint has accurately been determined. This means that, irrespectively of the technique applied, the same result will ensue.

A changed viewpoint will, in most cases, effect a change in the evaluation. That means that an investment, which is unprofitable for the entrepreneur, may be of interest in terms of political economy. The differences are rooted in the additional consideration given to influencing factors like energy imports, trade balance and secondary effects.

The next sections will discuss simple decisions. With the cash flow accurately given, economic efficiency of an investment will be examined by means of various methods. Following this, uncertainty of data will also be included.

This discussion will also extend to decisions to be made in the case of multiple targets.

14.2 CLASSIC ANALYSIS OF ECONOMIC EFFICIENCY

With each investment two cash flows have to be taken into account. Both, income (or benefits) and costs are characterized by amount and point of time of the cash flow item, which in a simplified way, is demonstrated in figure 14.1. The diagram in this figure exhibits the series of payments during the construction of a small-scale hydroplant, where relatively high investment costs are followed by low operation costs. In the income flow, petty fluctuations, in correspondence to the slightly varying annual energy generation, may be registered.

For the purpose of determining the economic efficiency of an investment, the classic evaluation techniques are most suitable:

- o Present Worth Method
- o Annual Costs Method
- o Benefit-Cost Ratio Method
- o Rate of Return Method

JAMES et. al. (1971) comprises, inter alia, a short description of the listed methods.

Before expanding on the description of the methods, we shall briefly discuss some basic notions by means of a payment, X_t. If a payment, X_t, is brought into relation to a point of time, $T > t$, the worth, X_t, due to interest yield, will rise to a future worth, E.

$$E = X_t \cdot d_{T-t}$$
$$d_{T-t} = (1+i)^{T-t} \qquad (1)$$

If brought into relation to the initial point of time, t=0, the present worth, B, of a payment, X_t, is to be determined by means of deduction of unaccrued interest or discounting.

$$B = X_t \cdot \frac{1}{d_t}$$
$$d_t = (1+i)^t \qquad (2)$$

X_t	payment at a point of time, t
B	present worth
E	future worth
i	market interest rate
$1/d_t$	discounting factor
t	point of time
T	end of the period covered

Both calculation processes, addition and deduction of unaccrued interest are determined by the period of time and the market interest rate. In case of a high interest rate, future payments will only trivially effect the cash value. This will be demonstrated by comparing the present values of payments made at the point of time t = 0, t = 10 and t = 20 years in table 14.1.

i (%)	t = 0	t = 10	t = 20
6	B	0.53 B	0.28 B
8	B	0.46 B	0.21 B

Table 14.1 Discounting in relation to the interest rate

If there are cash flows, each factor of a series must refer to the same point of time, which, in most cases, is placed in the beginning. If payments are the same each year, this will essentially simplify the calculation process. In the following we shall assume that assessment should be made for a project only. This means that we may choose between at least two alternatives: Implementation of the project or no implementation.

Furthermore, we shall presuppose exact knowledge of income and costs during the whole covered period, T, with the market interest rate, i, being definitely given.

14.2.1 Present Worth Method

The present worth is the sum of a series of payments, which fall due during the economic life, T, of a project, in relation to an initial point of time.

$$B_N = \sum_{t=1}^{T} \frac{N_t}{d_t}$$

$$B_K = \sum_{t=1}^{T} \frac{K_t}{d_t} \qquad (3)$$

$$KW = B_N - B_K$$

N_t benefits at a point of time, t
K_t costs at a point of time, t
B_N present value of benefits
B_K present value of costs
KW present worth

Equality of payments, e.g. N_t, simplifies the calculation, as the capital recovery factor, WF,

$$B_N = \frac{N_t}{WF}$$

$$WF = \frac{i \cdot (1+i)^T}{(1+i)^T - 1} \qquad (4)$$

may speedily be calculated.

If the project exhibits a positive present worth, realization is justified.

14.2.2 Annual Costs Method

Payments, whose sequence may frequently vary, are transformed into an equivalent series with constant intervals.

$$K' \Sigma \frac{1}{d_t} = \Sigma \frac{K_t}{d_t}$$
$$N' \Sigma \frac{1}{d_t} = \Sigma \frac{N_t}{d_t} \qquad (5)$$

If the annual benefit, N', exceeds the annual costs, K', realization of the project ist justified. Close relation to the Present Worth Method can be seen from (5).

14.2.3 Benefit-Cost Ratio Method

The quantities B_N, B_K viz. N' and K', which were derived in the above methods, yield the benefit - cost ratio NKF, which is frequently used for economic evaluation of investments.

$$NKF = \frac{B_N}{B_K} = \frac{N'}{K'}$$

As the benefits should exceed costs, the benefit-cost ratio must exceed one in order to justify an investment.

14.2.4 Rate of Return

As the choice of the interest rate entails some uncertainties, the present worth, in the case of the Rate of Return Method, is calculated by means of various interest rates, until the present worth comes down to zero.

If the market interest is lower, realization of the project is recommendable.

The calculation process of this technique is somewhat extensive, uncertainty regarding the interest rate, however, is almost excluded.

14.2.5 Application of the classic methods

The four methods will be demonstrated by means of a simple example. The economic efficiency of a small-scale hydroplant with a capacity of L = 500 KW is to be examined. We suppose an underestimated life time of 25 years and an interest rate of 7 %. Further data are indicated in table 14.2.

	small-scale hydroplant KW 1
capacity (KW)	500
specific costs (S/KW)	28 000
investment costs (mio. S)	14.0
interest rate (%)	7
income (mio. S/a)	1.83
operation and maintenance costs (% of investment)	2.5
operation and maintenance costs (mio. S/a)	0.35
capital recovery factor WF(i=7%, T=25 years)	0.0858
present value of benefit (mio. S)	21.32
present value of costs (mio. S)	18.06
present worth (mio. S)	3.26
annual benefits (mio. S/a)	1.83
annual costs (mio. S/a)	1.55
net annual benefits (mio. S/a)	0.28
benefit-cost ratio	1.18
rate of return (%)	9.8

Table 14.2 Evaluation criteria for KW 1

14.2.6 Evaluation of the Discounting Techniques

If thoroughly implemented, each of the four methods yields the same result, which means that preference is given to the same alternative.

The Methods of Present Worth and Annual Costs are easily accomplished and closely interrelated. The former one yields large numerical values, which, sometimes, lack a certain perspicuity, while the annual benefits and costs are easily comprehensible.

Many governmental authorities would prefer the Benefit-Cost Method, especially for projects of hydraulic engineering. Problems in the assessment of several alternatives will be discussed in detail later on.

The Rate of Return method is a good evaluation instrument in the presence of uncertainties. In order to avoid mistakes, one should stick to accurate performance in cases of complex formulation.

14.3 ECONOMIC EFFICIENCY UNDER UNCERTAINTY

Up to here, the interest rate, investments, as well as income and costs were considered given quantities. This supposition is not applicable to long-term projects. Furthermore, especially on the energy sector, price development is uncertain, which makes an evaluation of the future development inevitable.

Two methods are frequently applied. The first one, the Sensitivity Analysis, examines the effects on the evaluation criterion.

The second method, the Risk Analysis replaces the fixed quantities by probable quantities, which also yields, of course, a result based on probability only.

14.3.1 Sensitivity Analysis

The sensitivity analysis would examine the admissable fluctuation margin of the individual influencing quantities. Here, by turns, each quantity which had been considered a given quantity during the previous section, is replaced by a variable one, in order to examine its effects on the result. Thus the sensitivity degree of the result in relation to each variable quantity, or also "influencing quantity", is being determined.

As regards the previous example, there are uncertainties concerning investments, income and costs, interest rate, and economic life of the plant. In order to evaluate the effects of a variable quantity, the change in the present worth is examined in the previous instance.

Table 14.3 expresses the effects of the modified variable quantities on the present worth in percentages of the modifications.

	Investm. (mio.S)	Benefits (mio. S/a)	Operation Costs (mio.S/a)	Economic Life Time (years)	Interest Rate %	Present Worth (mio.S)	Change in Present Worth
Initial Situation	14	1.83	0.35	25	7	3.26	
+ 10 % Investm.	15.4	1.83	0.35	25	7	1.86	-42 %
- 10 % Investm.	12.6	1.83	0.35	25	7	4.66	+42 %
+ 10 % Benefits	14	2.01	0.35	25	7	5.40	+65 %
- 10 % Benefits	14	1.64	0.35	25	7	1.12	-65 %
+ 10 % Oper.Costs	14	1.83	0.385	25	7	2.85	-12 %
- 10 % Oper.Costs	14	1.83	0.315	25	7	3.66	+12 %
+ 10 % Ec.lifetime	14	1.83	0.35	27.5	7	3.85	+18 %
- 10 % Ec.lifetime	14	1.83	0.35	22.5	7	2.13	-34 %
+ 10 % Interest R.	14	1.83	0.35	25	7.7	2.21	-32 %
- 10 % Interest R.	14	1.83	0.35	25	6.3	4.39	+34 %

Tab. 14.3 Sensitivity Analysis

From this table one may see that a change in the income has the greatest effect on the present worth. Due to the present energy situation one may expect that the energy price will be on the increase during the following decades, which would result in an intensified growth of the present worth. Economic life time and operation costs exert the least important influence, while benefits, investment and the interest rate are more important. At the same time these data serve as a support for decision-making, as they would indicate, for which influencing quantities an accurate evaluation is possible and for which variable quantities, in the course of simple evaluation, satisfactory results would be available. Finally there is the possibility to determine admissible limits for each variable quantity, up to which the project would still be justified in terms of economic efficiency. In special cases, even a pessimistic evaluation of the influencing quantities will still advocate a realization of the project.

14.3.2 Risk Analysis

During the discussion of the sensitivity analysis, the various quantities, e.g., the interest rate or income were variates, while supposing, implicitly that within the variation margin, all values would exhibit the same probability distribution to the variable quantities. These degrees indicate, how much probability should be attributed, for instance to a certain interest rate. The argument that this probability may not be determined objectively, is justified to a certain extent, but one must point out that the realization probability of a fixed given rate or income flow is zero. By means of evaluations, which are based on longterm observation of sequences, e.g. of the interest rate, or which take into consideration projects which have already been realized, the probability distribution of the influencing quantities may be determined in a simple way. An approximative characterization of the variable quantity by means of the most favourable value as well as fluctuation or distribution will do. If a constant exceeding or failing to come up to the estimate is admissible, the application of a standard distribution will be most favourable. If the probability of exceeding or failing to come up to the estimate is irregular, skewed distributions are recommendable. There are examples for two variable quantities in table 14.3.

The result will also exhibit the probability degree for the investment criterion. From the diagram one may find out the degrees of probability, for which realization of the project is justified in terms of economic efficiency.

Calculation of the final distribution function, however, involves a lot of calculation work and may be implemented by means of simulation, bearing in mind, at the same time, several influencing quantities, which were only statistically determined. Here, according to the probability distribution of the variable quantities, a great number of possibilities may be figured out, and the frequency of the results is determined.

Calculation of the interest rate depending on the probability distribution of the individual variable quantities constitute an important means for assessing the project as well as for a comparison of several alternatives.

Distribution as charted in table 14.4 allows a determination of the probability of exceeding a limit, as well as a risk-estimate which is expressed in the confidence interval of distribution.

In conclusion we shall mention some mistakes which may occur in the course of the procedure. The most probable total result ought not to be considered the logical result of the most probable individual data. This assertion holds especially for skewed distributions.

Furthermore one must heed the fact that many variable quantities, in terms of statistics, are not independent from each other. There is, for instance, a relation between the expenses which comprises the interest payments and the interest rate. When determining this relation in a mathematic way, one may also apply correlative statements or conditional probability distributions.

Although, admittedly, this procedure is a rather lengthy one, it allows a far better description of all factual data than the previously treated methods.

Inflationary trends stay unheeded, if benefits and costs are equally subject to increases. In case of irregular developments regarding income and costs, additional corrections will have to be made. This attitude leads us to the dynamic methods. For time reasons, the application of the dynamic method will not be discussed here.

14.4 ECONOMIC EFFICIENCY IN CASE OF SEVERAL ALTERNATIVES

Up to here, decision bases for the evaluation of a project had been worked out. When looking at the matter more closely, however, one shall find several alternatives, some of which would exclude each other. Let us discuss here, as an example, utilization of certain water resources, with, on the one hand, installation of a few big-scale plants carried on quickly, and, on the other hand, gradual installation of small plants with realization timing adapted to the growing demand. Although the number of alternatives exceeds two here, it will, even after having taken into account various combination possibilities, stay a limited one. Plant dimensioning possibilities, however, exhibit a lot of alternatives. Variation of the rated discharge and the fall head at a certain project site, may bring forth an immense number of possibilities. As, at said site, both quantities may constantly be varied, there results a - theoretically - unlimited number of alternatives. Due to the turbine producers' standard programmes, however, reduction to a low number is possible. As both instances require similar treatment, they are comprised and discussed in this chapter. In the same order, the four classic methods will be, for supplementary purposes, briefly discussed here.

14.4.1 Present Worth Method

As payments may be considered equivalent only, if the amounts, in relation to a joint reference point, are the same, one must also fix a joint reference point, e.g. the year 1980, in case there are several alternatives.

These requirements have to be satisfied, even if the alternatives are to be realized at different points of time.

Furthermore, a joint interest rate, as well as a uniform economic life, have to be applied to all alternatives. This economic life shall be a medium period. Alternatives with a longer economic life will, therefore, at the end of the evaluation period, avail of a rest value which has to be included into the analysis on economic efficiency. The present worth of all alternatives has to be figured out and those, exhibiting a positive present worth, will be chosen. In case of projects excluding each other, the one exhibiting the highest present worth will have to be favoured, with its costs being exactly known . If the evaluation criterion is known, the alternative accompanied by the lowest costs is to be favoured.

14.4.2 Annual Costs Method

The same conditions as in the afore mentioned techniques will apply here. In the case of several alternatives excluding each other, the one yielded the best annual net benefit will be the most favourable one in terms of economic efficiency.

14.4.3 Benefit-Cost Method

Having in mind the conditions which, also in future, will apply to the interest rate, evaluation period, and reference point, those projects with their benefit-cost ratio exceeding one, have to be determined. Additional analysis will become necessary for those alternatives, which are mutually exclusive. The group will be arranged in a series according to the cost increases, and, starting from the cheapest alternative, the incremental cost-benefit ratio has to be determined. This procedure is to be determined until the value of the cost-benefit factor sinks below one. The alternative added last, therefore, is to be considered unprofitable, while the last but one is the most favourable one.

14.4.4 Rate of Return Method

After having chosen a joint evaluation period and reference point, the values of the rate of return have to be figured out for all projects. On the basis of a comparison with the minimum value, a preliminary choice will be made. Thereafter, in case of the alternatives excluding each other, the rates of return of the incremental benefit-cost ratio will be the determining factors. As soon as the interest rate of the increments falls below the admissible minimum limit, the most profitable alternative will be established. Depending on the market interest rate, different evaluation results may be expected from this method, as can be seen from figure 14.5.

14.4.5 Example of Application

The above explanation will now be domonstrated in a simplified way by means of an example: A gradual utilization variant will be added to the project discussed in 14.2.5. So, there are the following alternatives now.

In the first case there is a project for the installation of a 500 KW plant. The second alternative envisages the installation of a 350 KW plant, which will generate sufficient energy for some years, and to which, after 10 years, the second small-scale 200 KW hydroplant will be added. The economic life of the plant is fixed at 25 years. For simplification purposes, the present worth of the plants at the end of the period covered by the analysis will not be considered.

	alternative I	alternative II	
capacity (KW)	500	350	200
specific costs	28000	31000	36000
investment costs (mio. S)	14	10.8	7.2
interest rate (%)	7	7	7
income (mio. S)	1.83	1.45	0.95
operation- and maintenance-costs (mio. S/a)	0.35	0.25	0.10
start of operation after n years	0	0	10
economic life (in years)	25	25	25
present value of the benefit (mio. S)	21.3	16.9	4.4
present value of costs (mio. S)	18.1	13.8	4.1
present worth (mio. S)	3.2	3.1	o.3
annual benefits (mio. S)	1.83	1.45	o.38
annual costs (mio. S)	1.55	1.18	o.35
benefit increments (mio. S)	o.o2	——	
costs increments (mio. S)	o.2o	——	
benefit-cost ratio	1.18	1.19	
incremental benefit-cost ratio	< 1	——	
rate of return	9.5	9.7	

Table 14.4 Evaluation example with various alternatives

All evaluation techniques advocate the gradual utilization of water resources. In applying the Cost-Benefit Method, the incremental benefit-cost ratio would constitute a value below zero, which means that this additional investment is no more justified.

The next instance poses the question for the most economic dimensioning of a small-scale hydroplant. Economic criteria of energy, which refer to reliable energy, to summer and winter output as well as various tariff periods, will not be discussed in detail. They would partially show in the income.

Regarding the project site, there are estimates for the investment and operation costs, as well as for income from energy generation, as is charted in figure 14.6. By increasing the net head as well as the rated discharge, one may effect a rise of the annual output. Following this, only the capacity values of a small scale plant will be used for reference purposes. Regarding figure 14.6 one ought to mention that such diagrams would, still, frequently exhibit inconsistencies in the investment function which are due to the transition, for technical reasons, to more expensive construction methods, which become necessary as soon as, for instance, marginal limits of rated discharge or fall head are exceeded.

capa. (KW)	spec.co. (S/KW)	invest. (mio.S)	maint.& op.costs (mio.S)	in-come (mio)	pr.val. of benef. (mio.S)	ben. incr. (mio.S)	pr.val. of co. (mio.S)	costs incr. (mio.S)	NKF	NKF of growth
380	35260	13.4	0.35	2.0	23.3		17.48		1.33	
510	31600	16.1	0.36	2.4	27.96	4.66	20.29	2.81	1.38	1.65
640	27800	17.8	0.38	2.65	30.87	2.91	22.22	1.93	1.39	1.50
760	25520	19.4	0.40	2.85	33.20	2.33	24.05	1.83	1.38	1.27
890	23250	20.7	0.44	2.98	34.71	1.51	25.81	1.76	1.34	0.85

Table 14.5 Determination of the optimum plant factor

As becomes evident from table 14 5 the incremental benefit-cost ratio between 760 KW and 890 KW falls below one. This means that the plant factor for proportioning is to be found within this margin. The demand for an optimum factor may also be satisfied by an equivalence of the marginal benefits and costs.

In the above analyses, benefits and costs were considered given values without having their components discussed in detail.

There follows a brief on the benefit-cost structure as well as on the latter's influence on hydroplant efficiency.

14.5 STRUCTURES OF BENEFITS AND COSTS

This section provides a survey on the structure of benefits and costs of small-scale hydroplants. Here, special attention is attributed to the reference scope, as, depending on the extent of the analysis, different criteria will be applied.

Previous evaluation of economic efficiency was based on micro-economic analysis, which compared energy generation, expressed by the latter's price, to capital utilization and maintenance costs.

However, different evaluation criteria will be decisive, if evaluation of small-scale hydroplants is implemented on a regional or national level. Before proceeding to evaluation on a broadened basis, the structure of benefits and costs will be discussed on a micro-economic basis.

14.5.1 Micro-economic benefit structures

In the case of a micro-economic analysis of small-scale hydroplants, energy generation constitutes the benefit of small-scale hydroplants. A monetary assessment of energy depends on several aspects, with the form of energy utilization and temporal coincidence of consumption and generation being the most important ones.

Satisfaction of the private demand by means of the energy generated, deserves highest assessment. In this case delivery from elsewhere is substituted, and the energy generated may, approximatively, be adapted to the energy purchase price Deviations may occur due to a potential restriction on the sector of supply-reliability.

Assessment for the same energy generated would, however, be lower, if the total amount is fed into the grid, that is, if producer and consumer are not identical. In this case, the tariff system with its tariff periods, which are subject to the seasons, as well as the tariff hours, which cover the demand fluctuations during the day, will be applied.

In Austria, tariff arrangements are up to the electricity-generating enterprises, which on their part, would obtain their guidelines from the tariff system of the Österreichische Verbundgesellschaft (Austrian National Grid Company). Assessment of electric energy from small-scale hydroplants by the Niederösterreichische Elektrizitätswerke AG (NEWAG,1980) (Electricity Supply Company of Lower Austria), will serve as an example here.

The number of feeding hours during high-tariff periods and hours, i.e. during week days between 6^{00} and 22^{00} hours, including, from April to September, Saturdays between 6^{00} and 13^{00} hours, will be decisive. Feeding hours result from the quotient of the active output fed during high-tariff periods and hours, and the established nominal capacity of the plant's generator.

high-tariff feeding hours	percentage of the grid working price
1200	50 %
2000	65 %
2500	80 %
3000	90 %
3000	100 %

Table 14.6 Assessment of the hydroplant feeding.

From this table one may see that special consideration is given to continual feeding. Nevertheless, preference is given to low plant factors which make constant feeding possible.

Due to the rising energy consumption, however, one should aim at a complete economic utilization of water-power (OBERLEITNER, 1981), which is feasible by means of increasing the plant factor.

For the assessment of the energy-generation of small-scale hydroplants, therefore, both aspects, efficient utilization of the water power as well as feeding reliability will be considered. Besides, the total contribution of small-scale hydroplants with consideration to the load duration curve of a grid shall be included into the assessment.

Observation of the total energy is advantageous, as fluctuations in discharge and, consequently, in energy generation of the individual plants may be compensated.

In considering very large regions one may realize a certain positive supplementation in the discharge conditions of the various catchment areas, which screws down the seasonal dependence of energy generation.

Utilization of the hydroplant, as well as availability of the established capacity, constitute a further important aspect of energy valency. The higher the plant factor, the more efficient is the utilization of the water resources. At the same time, availability of the capacity is restricted, which means a reduction of the energy valency. A measure to accomplish both targets is the temporary use of reservoirs, which makes an adaption to the day's fluctuations in energy demand possible. For long-term discharge balancing, large reservoirs are recommendable, such as have been built for irrigation purposes. Multipurpose utilization does not only allow an intensified reservoir utilization, but it also effects an essential rise in energy generation.

14.5.2 Micro-economic cost structures

Tables 14.2 and 14.4 show that the investment costs play a dominating role in the cost structure.

Investment costs consist of

- o estate costs, water title costs and any potential redemption costs
- o initial development costs at the project site, which means an additional burden
- o costs for planning and installation management for the hydroplant and the supply grid
- o transportation costs, which, in the case of remote project sites have to be considered at any rate, including sea freight, transportation, insurance, etc.
- o installation costs for the whole plant
- o costs for electric engineering equipment, including the local supply grid
- o dues to the public authorities (customs duties, fees, etc.)
- o interest coming due during the installation period
- o Incidence costs to cover any unforeseen expenses

A generally applicable, quantitative determination of the various shares is extraordinarily difficult, as the specific situation at the project site and the latter's location within the project area influences the costs to a high degree. In this connection, reference is made to the Nepal case study sion (contribution No.16), where the transportation costs are responsible for a high share.

On the basis of several Austrian small-scale hydroplant projects as well as the ITG (1979) and MAYO (1980) data, the following may be considered to hold true:

electric engineering equipment	40-60 %
installation costs	40-50 %
planning and management	5-15 %
preliminary costs, interests, dues	5-10 %

Installation costs for energy distribution constitute an additional share, with a wide scope (10-35 %). The above shares show that, by means of standardizing electric engineering equipment, a significant reduction of the total costs may be achieved.

Object of this standardization is a well-graded type program, similar to that of manufacture units, which entails a good utilization of the water resources.

The generation costs consists of the following shares:

- o interest rate for the capital invested
- o depriciation for plant parts according to their economic life. In line with the LAWA Working Group's guidelines, the average economic life would be

for structural plant parts	60 years
for engineering parts of the plant	40 years
for electric plant parts	30 years
for estates	100 years

- o salaries for staff in charge of operation and supervision
- o costs for repairs and spare parts
- o costs for consumption material
- o current dues and administrational costs

As saving is possible especially with regard to the expenses for salaries and administration, a stepped-up plant automatization will be aimed at in case of new plant installations. Further cost reductions may be achieved by a joint operation and maintenance of small-scale hydroplants or by means of a combination of small-scale plants and small-scale industrial plants. The latter one is a most promising form of energy generation, as the energy generated is to be substituted for the financial expenses for supply from elsewhere and distribution costs as well as salary expenses would be low.

Bearing in mind these individual items as well as the specific circumstances in each country concerned, the production costs may now be assessed, to which the plant factor α has to be added.

$$\alpha = \frac{\text{kWh per year}}{8760 \cdot L} \qquad \text{(L installed capacity)}$$

Starting with the investment costs, we shall attempt to make a rough estimate of the annual cost shares. Interest payment accounts for the main share and is determined by the interest rate, i, and the amortization duration, T. In table 14.7 the annual rate is expressed in percentages of the investment costs.

	T = 50 years	T = 25 years
i = 6 %	6.3 %	7.8 %
i = 8 %	8.2 %	9.4 %
i =10 %	10.1 %	11.0 %

Table 14.7 Annual payments in percentage of the investment costs

Annual costs for wages, operation and repairs are estimated at 1.5-2.0 %, while those for administration and other dues are 0.5-1.0 %. Interest payment, therefore, amounts to at least two thirds of the annual expenses.

With an accurate understanding for the cost structure, one may proceed to a comparison with caloric plants now. In the following we shall present an example in simplified form: Two small-scale plants with the same capacity and annual output, but with different economic life are to be compared on the basis of the Annual Costs Method, under consideration of the fuel costs.

	hydroplant	caloric plant
capacity	L	L
annual output	A	A
investment costs	I_w	I_k
specific costs	$i_w = I_w/L$	$i_k = I_k/L$
economic life	T_1	T_2
evaluation period	T_2	T_2
rest value	R_w	-
capital recovery factor (i, T)	WF	WF
modified capital recovery factor	WF_w	WF
maintenance and operation costs	$c_w \cdot A$	$c_k \cdot A$
fuel costs	-	e.K.A
duration of utilization	t_a	t_a

Table 14.8 Comparison of hydroelectric and caloric energy generation

The modified capital recovery factor comprises the rest value of the plant after T_2 years, K denotes the specific fuel costs and e accounts for the specific consumption. Suppose, the maintenance costs, c, are approximately the same, which will hold true to a limited degree only, the following will apply:

$$I_w \cdot WF_w + c_w \cdot A \leq I_k \cdot WF + c_k \cdot A + e.K.A$$

$$i_w \cdot WF_w + c_w \cdot t_a \leq i_k \cdot WF + c_k \cdot t_a + e.K.t_a \qquad (6)$$

$$i_w \cdot WF_w \leq i_k \cdot WF + e.K.t_a$$

14.5.3 Macro-economic costs structure

In the above assessment benefits and costs were expressed in monetary terms, with quantification according to the market situation. That is, all actions T (investments) influence the environment through the market and, on the analogy of this, prices are oriented at the market.

Additional effects, which were not comprised in the assessment form the entrepreneur's viewpoint, will be included in this chapter.

Evaluation bases, therefore, will change, and, in special cases, yield evaluation findings which are different from those of the micro-economic evaluation.

We must also differentiate between an analysis on the regional or national level. For illustration purposes, we shall describe both forms by means of a few catchwords.

On a regional level, the following criteria are of significance:

- o share of the investment costs brought to bear in that region
- o number of jobs created by the installation and operation of the plants
- o number of jobs created in consequence, e.g. by the installation of small-scale industrial plants
- o energy share to be utilized in that region, substituting energy from elsewhere
- o improvement of the infrastructure
- o improvement and balancing of the income and social structure within that region

Some of the quoted items may be quantified and explained by means of the multiplier effect of investments. Others, like improvement of infrastructure or the modified social structure, whose evaluation is described by BARTELS, are hard to quantify, or may be characterized in a qualitative way only.

On a national level, other aspects are in the foreground, as are briefly enumerated here:

- o For political economy, the multiplier effect of small-scale hydroplant construction

- o Relief of the trade balance by means of substitution of energy imports
- c Increased national supply reliability by means of utilization of domestic resources
- o Compensation of interregional differences in development and income
- o Improvement of the infrastructure
- o Improvement or consolidation of environmental quality

These items show evidently that one should not necessarily strive for a maximization of output, but that the general benefit, namely social welfare, ought to be the considered the main target of governmental undertakings.

According to comprehensive literature, reference is made to the studies of HOWE (1971) and DOE (1979), where special consideration is given to measures of water resources policy.

14.5.4 Macro-economic costs structure

For a regional as well as national assessment, costs are equally treated and consist of the following shares:

- o Design costs and other preliminary expenses include all costs, falling due before the project proper. Appointment of design groups, training of operation and maintenance staff as well as any potential measures concerning infrastructure, etc, would rank among these costs.
- o Construction and equipment costs. Here, major attention must be paid to financing by means of domestic or foreign capital, exchange rates and long-term interest levels, (shadow exchange-rates, shadow interest rates)
- o Operation and maintenance costs. Frequently, the current wage statements would fail to reflect the production factor of labour in a correct way. If, primarily, unemployed people and parttime workers are used for the implementation of a project, this will cause only small production losses on other production sectors of national economy, or none at all.
- o Opportunity costs, which make allowances for the slipped benefit in case of alternative use of the applied means.
- o Social and consequent costs cover the costs for the installation and operation of social utilities and improvements on the infrastructure sector, additionally caused by the project.
- o Allocated costs must be envisaged in case of multipurpose plants. As this item is of high significance for small-scale hydroplants, whose installation, in combination with projects of river engineering or water economy is highly recommendable, a simple example will be presented in 14.5.5.

Further cost shares may be caused on the grounds of encroachments or damages to the environment, which, however, is to be considered rather negligible in the case of small-scale power plants. This comparison of micro- and macro-economic factors clearly brings to the foreground the different decision bases. On the analogy of this, other techniques of decision finding, too, are being applied, which are discussed in 14.6.

14.5.5 Costs allocation in multipurpose projects

From an economic point of view, the use of multipurpose projects of water economy is highly promising, as cost sharing is possible with regard to planning, installation and further maintenance. This viewpoint bears high significance for Austria, where said combination of flood control measures (reservoirs, regulating devices) and small-scale hydroplants may be applied, as well as for non-European countries, which may, additionally, include irrigation projects, fish breeding, supply reservoirs, etc. As such a kind of planning would frequently affect different institutions and ministerial divisions, allocation of costs, according to their targets, will become necessary. LOUGHIN (1978) provides a description and analysis of the individual allocation techniques.

One of the above techniques will be presented by means of a simplified example, where flood control, an irrigation project and a small-scale hydroplant have their shares in the total project costs.

The Alternative Justifiable Expenditure Method at first compares the total project with the individual projects and, thereafter, allocates the individual shares to each project. These shares consist of the specific costs and the allocated costs, which are calculated in proportion to the individual share in the total benefit.

A water-related project has three objectives:

- o A, to improve flood control in that region
- o B, to irrigate a cultivated area
- o C, to generate energy for supplying an adjacent village

All values, transformed into annual payments, are figured in tab. 14.8.

	OBJECTIVES			
	A	B	C	Σ
1 Benefits from the objectives	o.8	5.8	4.2	1o.8
2 Alternative costs at the individual project	1.o	4.6	3.7	9.3
3 Justifiable costs	o.8	4.6	3.7	9.1
4 Specific costs	o.4	1.9	1.8	4.1
5 Remaining benefits (3-4)	o.4	2.7	1.9	5.o
6 Adjoint costs of the targets	o.19	1.3	o.91	2.4
7 Total costs of the targets (4+6)	o.59	3.2	2.71	6.5
8 Cost savings in percentage	8 %	54 %	38 %	1oo %

Table 14.9 Cost allocation in multipurpose projects in million Schillings per year

The benefits for the objectives A, B and C may be quantified, as are the total costs of the multipurpose project and the alternative costs in case of realization as an individual project. The costs shares (line 4) definitely reflect the cost shares to be allocated, while the rest, corresponding to the remaining benefit (line 5) is distributed to the targets.

14.6 ECONOMIC EFFICIENCY IN CASE OF AN EXTENDED EVALUATION SCOPE

Innumerous techniques are suitable for decision finding, which means the selection of one project from various other projects or the determination of the economic efficiency of a certain project. The hitherto described techniques, the Present Worth Method, the Annual Costs Method, Benefit-Costs Method, as well as the Rate of Return Method are applicable, if benefits and costs may be determined in monetary terms. Sensitivity and Risk Analysis offer extended opportunities.

14.6.1 Extended Benefit-Cost Analysis

If one can manage, even in the face of an extended assessment scope, to quantify all effects a measure has on national economy and, additionally to assess them in monetary terms, the cost-benefit analysis will be applicable also in future. This technique will solely assess the benefit of an investment for national economy.

Even if the analysis is carried out most thoroughly, government objectives like "general welfare" may, only to a limited extent, be assessed in monetary terms. Thus the application limits for this method are set.

14.6.2 Evaluation and decision in view of several objectives

Assessment of a project requires an accurate definition of the target aimed at. Thereafter one may find out, to what degree the various alternatives would come up to the set target. The degree to which a target would be accomplished, will be the basis for decision.

In case of a large-scale water-economy project, i.e. utilization of an extensive river system for energy production, several targets have to be accomplished, which, according to the design principles of the WRC (1973) may be classified into four main groups.

If "general welfare" is the objective, the

- o national economic development
- o regional structure and development
- o social situation
- o environmental situation

must be included into the evaluation. With each large-scale project the objectives, in correspondence to the above categories, are to be formulated in detail, and, if possible, quantified. If the measuring systems are irreconcilable, the effects of the project alternatives on the individual targets are to be examined. Eventually, the criteria for an accomplishment of the objective may be determined and balanced against each other.

Benefit Value Analysis and Cost Efficiency Analysis are suitable techniques, which are recommendable if benefit is hard to quantify. These analyses provide an extensive objectivation of the assessment and decision process, without anticipating the latter. It is merely possible to indicate the interaction of the objectives, and to exclude any unfavourable alternatives. Only by means of establishing a preference structure or a hierarchical classification of the targets, a selection of the individual alternatives is possible.

Even if these methods will still be discussed in detail and new techniques introduced, the above instruments offer designers the opportunity to arrive, in an operational way, at an assessment, which does not only take into account economic efficiency but also other, additional aspects of a measure.

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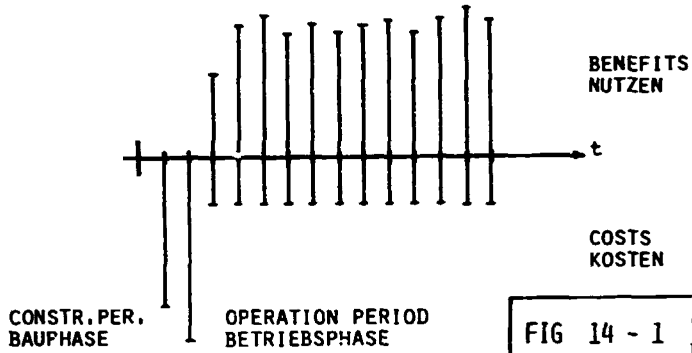


FIG 14 - 1 CASH FLOW DIAGRAM
DIAGRAMM DER ZAHLUNGSSTRÖME

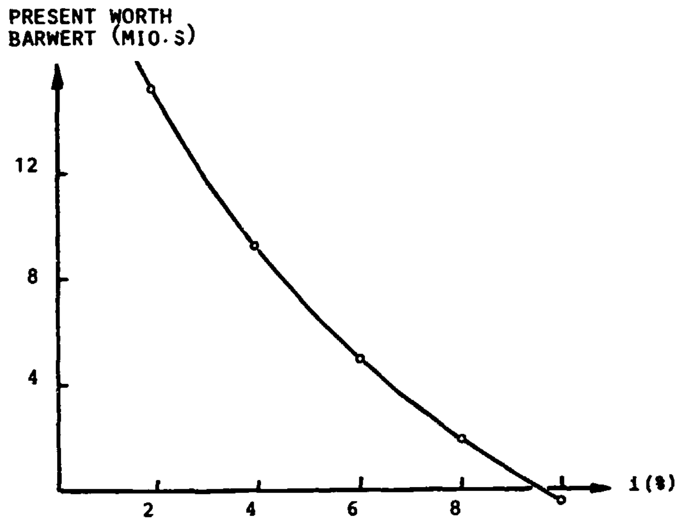


FIG 14 - 2 RATE OF RETURN METHOD
METHODE DES INT.ZINSFUSSES

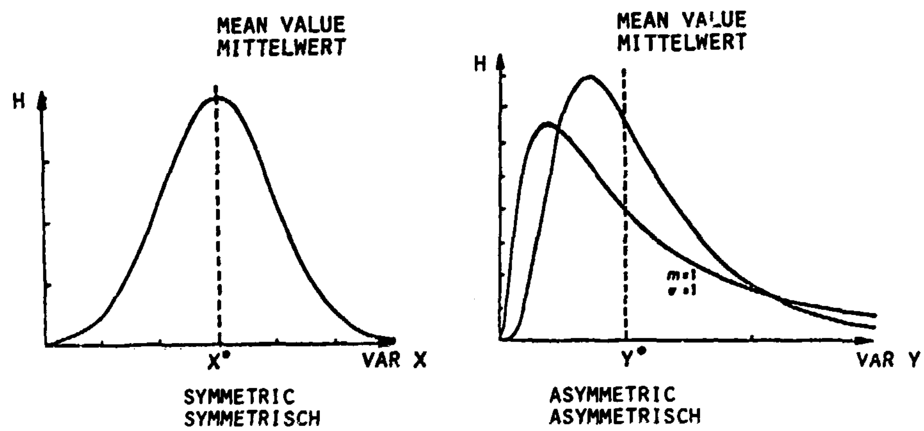


FIG 14 - 3 PROBABILITY DENSITY FUNCTION
WAHRSCHEINLICHKEITSDICHTE

14 ECONOMIC EFFICIENCY
WIRTSCHAFTLICHKEIT

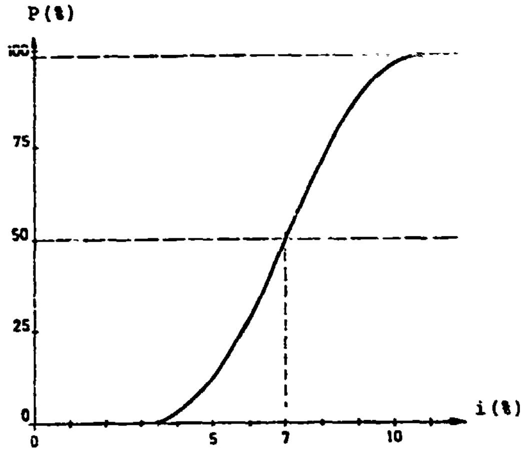


FIG 14 - 4 PROBABILITY OF INT.DISCOUNT RATES
WAHRSCHEINLICHKEIT INT. ZINSSÄTZE

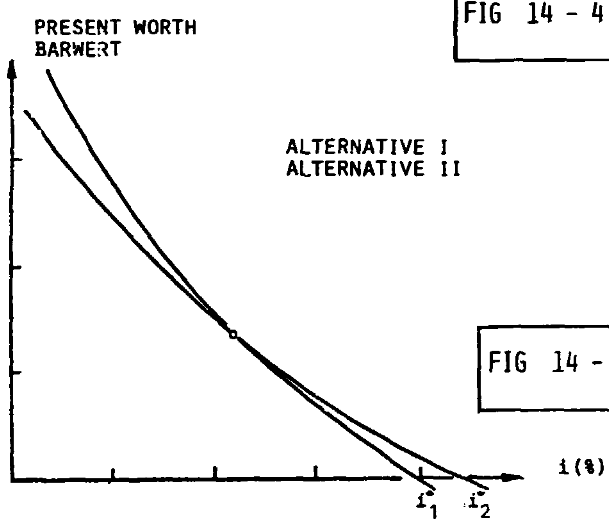


FIG 14 - 5 PRESENT WORTH VS. INTEREST RATE
BARWERT VS. ZINSSATZ

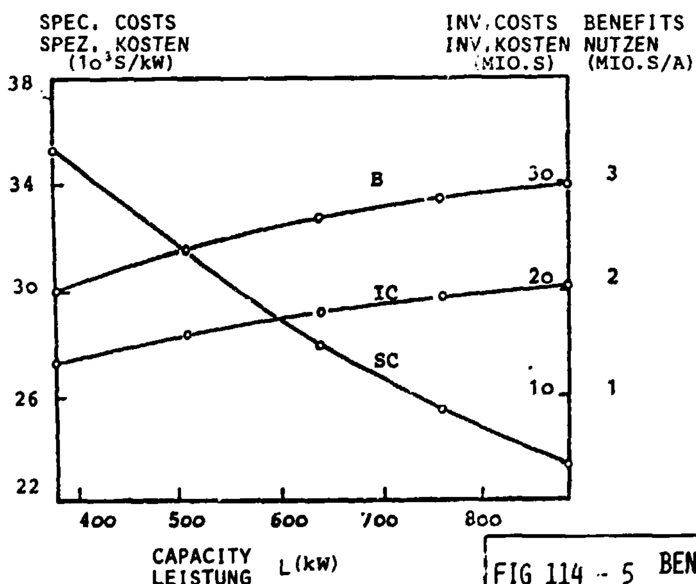
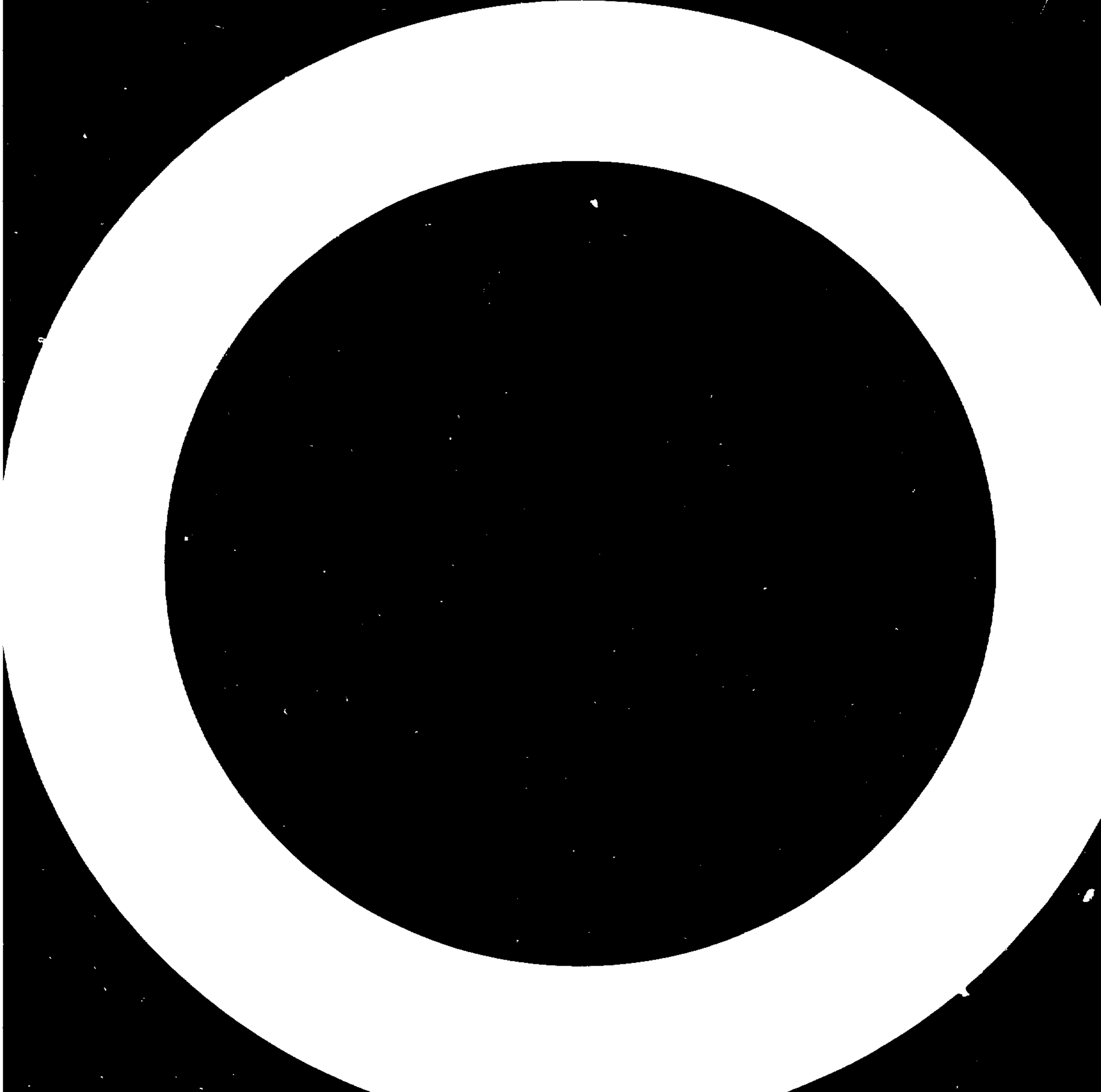


FIG 114 - 5 BENEFITS AND COSTS VS.CAPACITY
NUTZEN UND KOSTEN VS.LEISTUNG



15 CASE STUDY FROM EAST TYROL

SIMMLER H.

ABSTRACT:

The mini power scheme described in the following chapters is situated in a gorge cut by the Tauernbach stream, in East Tyrol, Austria. Working under a head of about 30 m and at a rated discharge of 2.0 m^3/s, the station is capable of 472 kW. The plant is composed of a crib dam, a sand trap, a 200-metre long penstock and the power station.

15.1 GENERAL

In a gorge of the Tauernbach stream in East Tyrol, there is an old mini hydro station originally constructed to develop a head of some 30 m. With a rated discharge of 1 m^3/sec, the installation was capable of an output of about 220 kW. After about 50 years' operation, the power station had reached a state that called for large-scale reconstruction and repair.

Considerations of power management and economy led to the decision to construct a new power station in addition to the old one, using the existing water intake. The two stations would have the power intake, the sand trap and the penstock in common.

The following data were developed for the new power station:

Rated discharge	3.0	m^3/sec	(for both power stations)
Gross head	30.75	m	
Effective head at rated discharge	28.80	m	(for the new power station)
Generator capacity	472	kW	(for the new power station)
Output of Francis turbine with governor	497.5	kW	
Turbine speed	750	RPM	

The old power station, which is now supplied through a branch from the newly constructed penstock, continues to operate with a generator capacity of 200 kW under an effective head of 26.95 m. In total, realisation of the project has made it possible to increase the winter energy from 0.990 GWh to 2.062 GWh and the summer energy from 0.990 GWh to 2.767 GWh.

15.2 POWER INTAKE AND SAND TRAP

The power intake as originally constructed, is a fixed-crest crib dam, part of which is masonry. The power water enters the penstock on the right-hand side of the dam over a horizontal rack ("Tyrolean Weir"). To handle the increased inflow of 3.0 m^3/sec in total, it has been necessary to heighten the existing dam by an average 50 cm, with the crest sloping from the left to the right-hand bank. The crest has been protected with stone slabs and a plank facing.

The existing intake has been enlarged from 1.8 m by 1.8 m to 1.8 m by 4.0 m. The intake rack has been given a slope of 6° to the downstream to improve bed load discharge. Immediately below the inlet is a control valve with a cross sectional area of 1.3 m by 1.8 m and an electric driving gear installed on a deck above flood level. The control valve limits the flow drawn in to avoid excessive loading of the sand trap.

The water intake and the underground sand trap are connected by a tunnel about 12 m^2 in cross sectional area. The sand trap has been provided with an overflow spillway. This is followed by a forebay, where the pressure pipe branches off. Flushing of the sand trap will be to the stream bed. The sand trap and the forebay will at the same time function as a surge tank. Hydraulic analyses have shown the water level to rise or to fall by a maximum $\pm$ 40 cm in the possible operating conditions.

15.3 PIPE LINE

The original timber pipe line has been replaced by a steel pipe line with a diameter of 1.25 m and a wall thickness of 6 mm. The steel pipe line is of the continuous type and has been embedded in concrete to save extension compensating members and to protect the pipe from falling rocks as well as to achieve greater safety against buckling from potential subatmospheric pressures. Inflow towards the new power station is through a branch from the steel pipe line.

15.4 POWERHOUSE

The new powerhouse has been constructed in a pronounced bay of the stream bed, at a much lower level than the existing powerhouse. Protection from floods is afforded by longitudinal embankments. Due to its location, it has been necessary to design the powerhouse structure as a watertight bunker.

The powerhouse is covered with a flat roof with a gravel layer on top to minimise interference with the natural landscape. Erection operations can be carried out through a hatch in the powerhouse roof. Access to the power station is extremely difficult. The low-voltage generator is coupled to a 380 V - 5 KV generator transformer from which the 5 KW cable leads to the substation in the old powerhouse.

15.5 BASES FOR THE DESIGN

- Preparation of a feasibility study and a power management survey
- Engineering report
- Hydraulic analysis and capacity computation
- Stability analysis and project drawings for submission to the water authorities for approval, as well as detailed drawings for construction.

The power station has been in operation for several years without experiencing difficulties of any kind. Only a better ventilation device has subsequently been installed because the generator temperature rose too high, which had an unfavourable effect on efficiency.

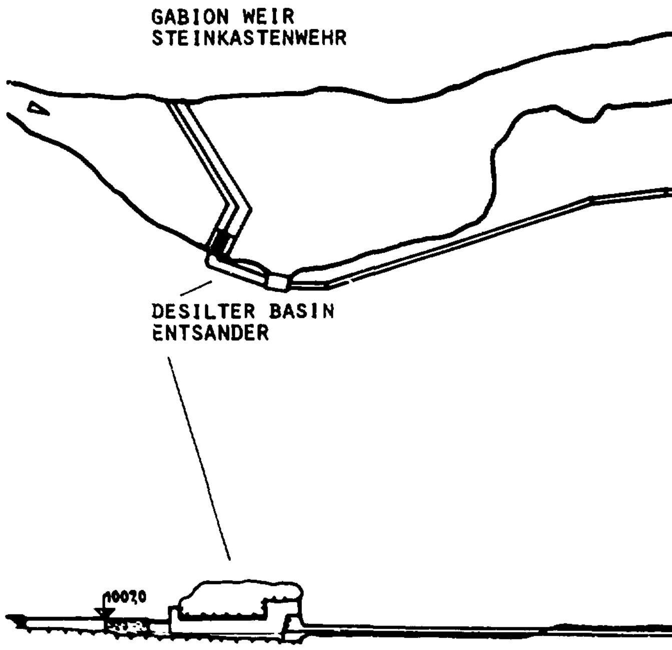
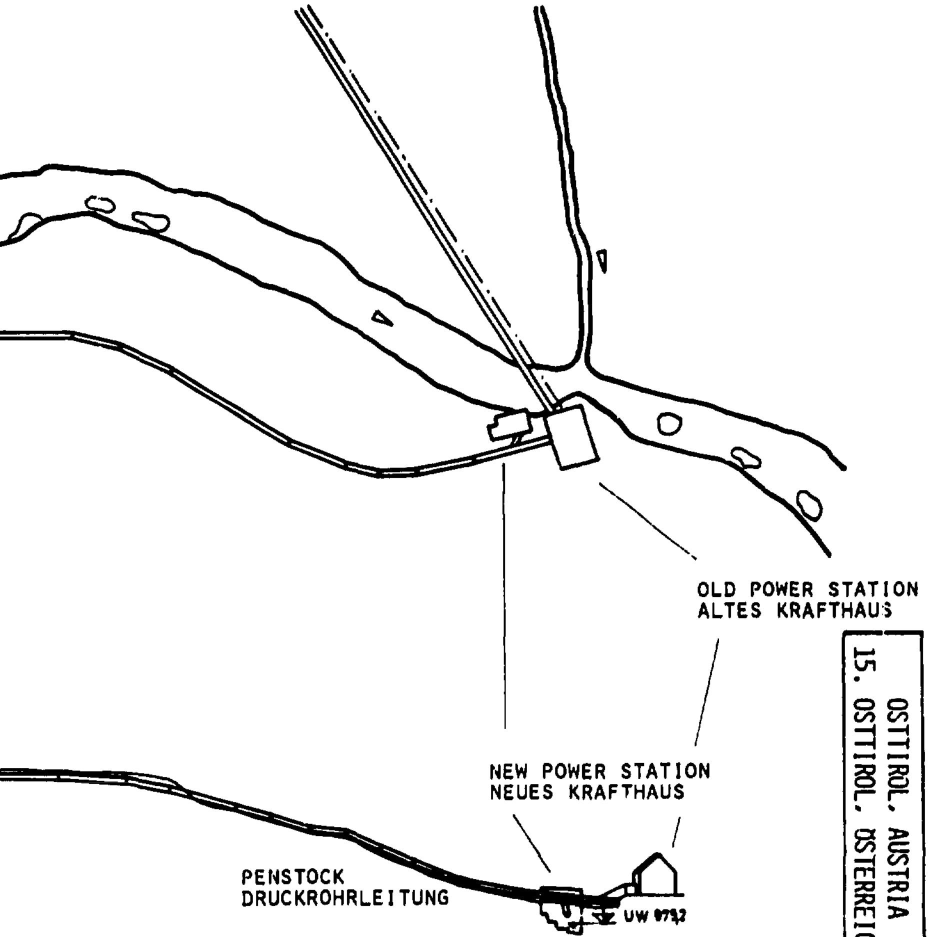


FIG. 15-1 PLAN VIEW
LAGEPLAN

FIG. 15-2 LONGITUDINAL SECTION
LÄNGENSCHNITT

OSTTIROL, AUSTRIA
15. OSTTIROL, ÖSTERREICH



15 OSTTIROL, AUSTRIA
OSTTIROL, ÖSTERREICH

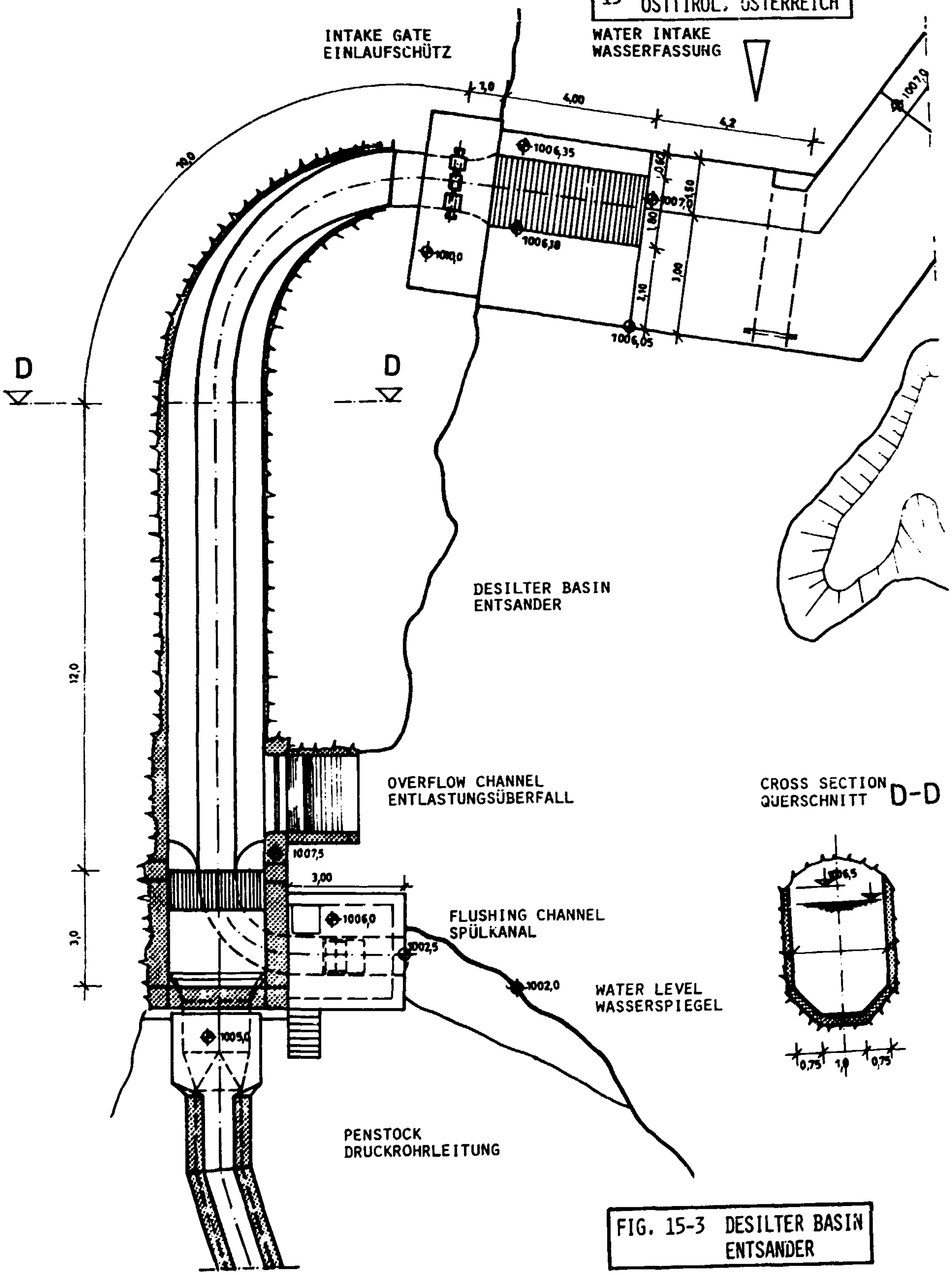


FIG. 15-3 DESILTER BASIN
ENTSANDER

15 OSTTIROL, AUSTRIA
OSTTIROL, ÖSTERREICH

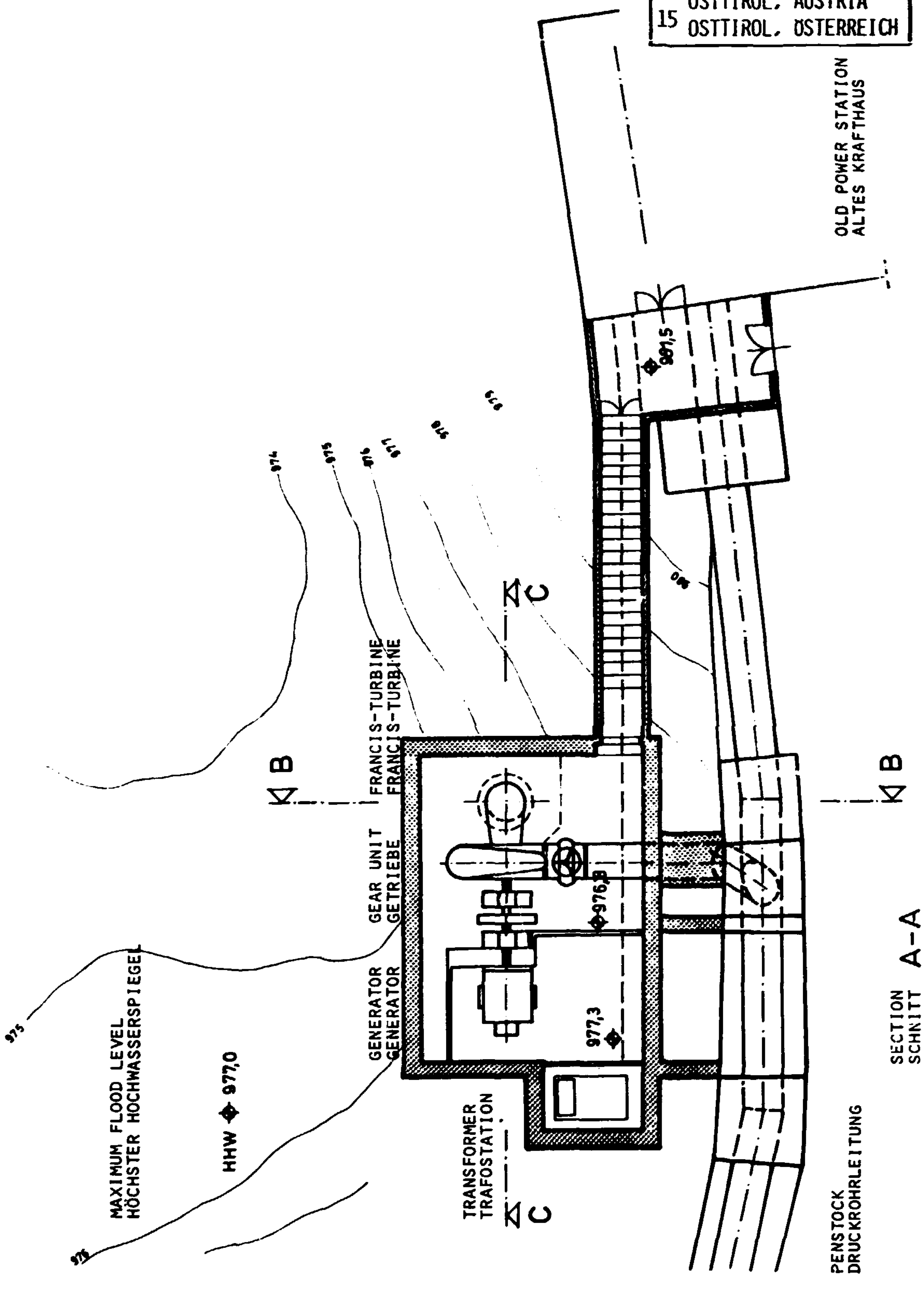
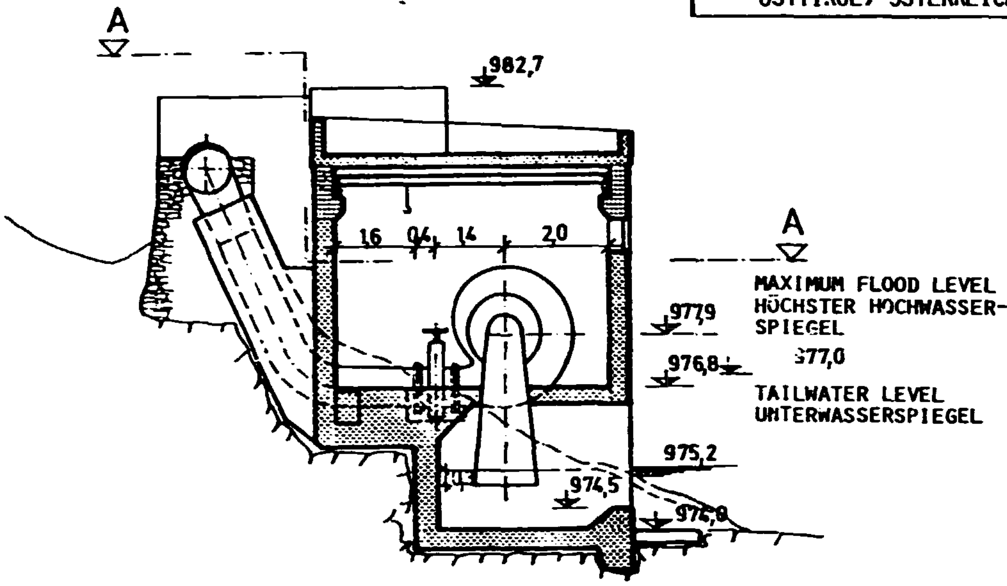
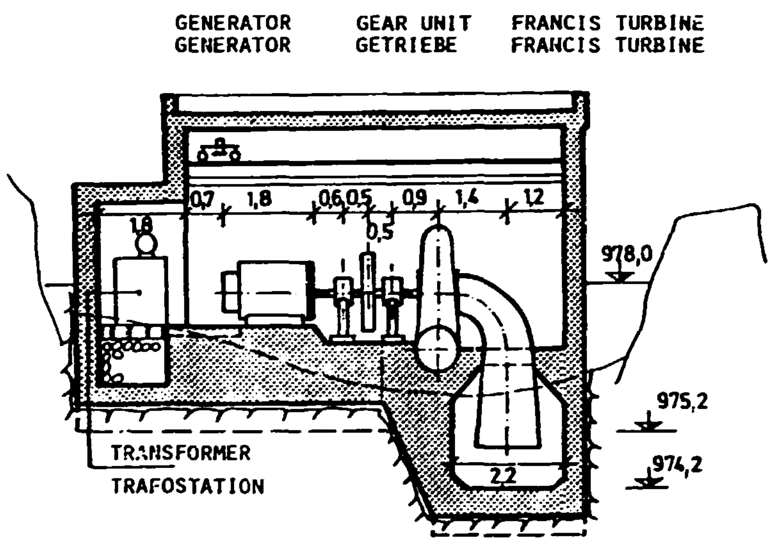


FIG.15-4 POWER STATION PLAN
KRAFTHAUSGRUNDRISS



CROSS SECTION
QUERSCHNITT B-B



CROSS SECTION
QUERSCHNITT C-C

FIG. 15-5 CROSS SECTIONS THROUGH POWER STATION
KRAFTHAUSQUERSCHNITTE

16 CASE STUDY FROM NEPAL

RADLER S.

ABSTRACT:

In the course of the Austrian development aid for Nepal the project, help with the construction and the supply of the electro-mechanical equipment for a small power project in the Khumbu region, Himalaya, was taken over (VERBUND-PLAN, 1978). Due to the altitude, the difficult access and the climatic conditions great problems arise in such an exposed situation, concerning the project studies, and especially the construction, above all as far as construction materials are concerned. The fact that in such an altitude no or only few meteorological and hydrological data exist, makes it even more difficult. With modern hydrological methods of data-generation and data-completion the necessary data for the project study and the construction have been gained by means of transfer from a neighbouring region. In view of the difficulties of transport, the construction had to be based on the use of materials that can be found near the site, particularly boulders, gneiss slabs and moraine layers. A cement-saving construction method and the use of gabions have ensured a minimal need of cement. Materials such as cement, steel and plastic pipes for the penstock will be flown to the next STOL-pist, only electro-mechanical equipment will be transported to the site by helicopter. As the diversion channel has been completed by now, the main constructions will be started this year.

16.1 INTRODUCTION

The development of tourism in Nepal, particularly in the Khumbu-region, causes great problems in regard to energy supply and environmental preservation. Therefore Austria, as a member state of the UNO, has granted a petition of the Nepalese government and has taken up one old wish of the Nepalese to construct a small power station in the Khumbu region, near Namche Bazar.

The trade centre Namche Bazar is situated in the centre of the Khumbu region, the homeland of the Sherpas, almost 4000 m above sea-level (Fig. 16 - 1). Namche Bazar is the key point for all expeditions that go to the hightest region of the Himalaya: Mt. Everest, Lhotse, Nuptse, Amai-Dablam, Cho-Oyu, Makalu, Pumori, etc. Sir Edmund HILLARY, who had conquered Mt.Everest for the first time in 1953 with his sherpa TENZING, started nere a largescale aid program from Newzealand, out of gratitude and attachement to the sherpa tribe. This program was above all concerned with the construction of roads and bridges, of so-called STOL-runways, with the building of a school and a hospital, and finally resulted in the foundation of the national park "Sagarmatha National Park".

This development is certainly to be welcomed, as it boosted tourism, that is to say trekking, but it could not keep pace with the demands of civilisation: with great expense diesel-emergency current aggregates were installed for the energy supply of the school, the hospital and the runway,and the hotel "Mt. Everest View", which had been built against the attempts of Newzealand to protect the landscape. Diesel oil has to be flown with special planes, which can still start and land at 4000 m above sea-level, over a distance of 150 km from Kathmandu - diesel oil gets to Nepal by a difficult, only poorly constructed countryroad of several hundred km, or by airway, too. Similar conditions are to be found in the case of water supply; only rivers which are supplied by the glacial region have a safe discharge, and these are situated in the deepsided valleys of the Himalaya. The lack of a sewage system causes great problems of hygiene, not only in the densely populated valley, but also in the villages situated high up.

The beginning of tourism has caused great problems here; the greatest is undoubtedly energy supply. Energy is needed above all for heating. The supply of fuel demands with wood found in the vicinity has already caused great damage and would inevitably lead to a complete deforestation and destruction of the wood zone, as in Nepal the wood zone extends to 4.500 m above sea-level, due to climatic conditions.

16.2 ENERGY SITUATION

Apart from already flourishing tourism Nepal has other immense possibilities of development by means of exploiting her water power potential. Nepal is the land with the largest energy potential. This fact ist due to the kettle position of Nepal. The variety of Nepal's topography corresponds to the climate, which ranges from the tropics to glacial zones. The course of the year is determined by the monsoon - 90 % of the annual rainfall is from the beginning of June to the end of September. The geographical distribution of rainfall is completely different in the Alps, where rainfall increases with altitude. The main range of the Himalaya forms a one-sided climatic limit for the weather to the south (Fig. 16 - 2).

Some geographical data about Austria and Nepal (RADLER, 1977) are supposed to show the high significance of water power in Nepal:

	Nepal	Austria
SIZE	141.000 km^2 (26-30° N 80-88° E)	84.000 km^2
INHABITANTS	> 10 Mio	7,5 Mio
ALTITUDE	75 - 8.800 m	140 - 3.800 m
MEAN HEAD	3.500 m	1.000 m
PRECIPITATION	1.800 mm	1.190 mm
RUN-OFF HEAD	1.210 mm	710 mm
SPECIFIC RUN-OFF	38,5 l/s, km^2	22,5 l/s,km^2
WATERPOWER POTENTIAL (area)	1.600 TWh	150 TWh

Apart from a bigger station at Sun Kosi and at Trisule, which were designed and contructed in India as development aid contributions of China, and some small hydro power stations, the water power potential in Nepal ist not exploited yet. The merely mechanical use of water power has been practised for years and is realised in the course of constructions of terraces together with irrigation works. Therefore it is all the more significant to start now with the construction of small hydro power stations. A list of the latest embankment and dam failures in Nepal shows that the enormous erosion and sedimentation problems have to be studied in detail before starting a water power project, in order to prevent further failures. A small power station offers, as is the case with all technological developments, the best opportunity to get familiar with the problems.

16.3 PROBLEM AND PRELIMINARY STUDIES

When mentioning the conditions of the site and situation of the project, we have already alluded to the difficulty of access, the untouched area, the dependence on the weather, and, consequently, the lack of any geological, meteorological and also hydrological data. The existence of very precise topographical maps - owing to the indefatigable work of many years of the Austrian cartographer Prof. Erwin SCHNEIDER - allowed, at least in this field, a very thorough evaluation of basic materials for hydrological investigations. But the untouched landscape, already considerably impaired by trekking, forbids any spoliation of the countryside. Any measures exceeding the minimal needs for trekking (such as access roads, cablecars, etc.) are not wanted because of the status of nature preservation and are also unattainable. These conditions involve considerable difficulties concerning the design as well as the preparation of the project, and particularly the construction, the supply of constructing materials and even later operation, especially for the supply of replacement parts and revision.

According to the energy demand in the vicinity of Namche Bazar for about the next 10 years, a river has been chosen which allows the construction for the calculated power of approximately 600 kW, even in the case of minimal run-off and in spite of the existing cross-head. It was all-important to choose a river whose catchment area was largely situated in the glacial region, in order to satisfy these demands. From experiences of many years it is known that in this region the night-temperature can be below zero throughout the year, the maximum daytime-temperature, however, is generally above zero. Consequently, there is a constant run-off of melted snow, even in periods of little precipitation.

In view of these aspects a skeleton investigation of the possible uses of water power near Namche Bazar was carried out in 1975 by the Norwegian Consulting firm "Norconsult". The demands mentioned above seemed to be met at Nangpo-Tsangpo (Bhote-Kosi). With a catchment area of about 400 km and an estimated run-off of 4 $l/s,km^2$ in dry seasons, a continuous run-off of about 1,5 m^3/s could be disposed of.

About 4 km north-west of Namche Bazar the most favourable situation for the project was found.

The river bed is filled over its whole length with river-transported debris and talus material, mixed with moraine material; the river passes through a very narrow eroded gorge, then crosses a wide talus slope of coarse blocks, and enters another long gorge to its junction with the river Dudh Kosi.

This opening of the gorge, 500 m long, should be used for the water intake, the headrace channel, the penstock and the powerhouse of this plant, making a cross-head of almost 50 m disposable.

It was of course all-important to choose a place safe from avalanches, and hang slides.

16.4 HYDROLOGY

16.4.1 Project area and reference area

The catchment area of the project, limited in the North by the Cho-Oyu and the Nangpa-La-passage, does not show any hydrological gauge. All specific hydrological data, such as precipitation, run-off, evaporation, bedload material, etc, had to be deduced and transferred, by correlation and regression, from a catchment area which is hydrologically known and similar in morphological and geological aspects. The area of the river Imja Khola, situated further in the east, with a catchment area of almost the same size, a similar distribution of altitudes and a similar glacier area (fig. 16 - 3) was considered to be suitable. This river has been the subject of hydrological investigations for many years. Flow-off measurements exist; they can, however, only be regarded as guidelines not only because of the short time of observation, but also because of the considerable modifications of the river morphology. The Himalaya is, as is well known, a very young mountain range, and erosion activities are much more intensive than in the Alps.

16.4.2 Precipitation

Almost 90 percent of the total annual rainfall is during the monsoon period (from June to September). The depth of the total annual precipitation is dependent on the altitude. It is greatest between 2.900 and 3.300 m above sea-level, its maximum decreasing with increasing altitude. The temperature pattern shows a similar dependence on the altitude (fig. 16 - 4).

One can deduce from the marked relationship of altitude and monthly precipitation that the distribution of altitudes in the catchment area is of high significance for hydrological facts, as the sections below 4.000 are primarily responsible for the transformation of precipitation into run-off. In view of a good correspondence between project area and reference area it seemed justified to transfer the data obtained in the area of Imja Khola to the project area without further corrections.

16.4.3 Run-off characteristics

According to the methods explained above (lowflow analysis) the value decisive for the power valuation was determined with the methods of GUMBEL and KATSCHMAREK, the most unfavourable value for a low-water occuring every ten years being taken as a basis.

After the short time of observation values of great dispersion were obtained to the different methods of flood statistics. The confidential range had to be restricted correspondingly. The final value of $HQ_{100} = 110\ m^3/s$ was assumed on account of trustworthy natives, who had observed the highest water levels in the past 50 years.

In view of the long construction period the diversion channel and the excavation were designed for decennial floods.

16.5 DESCRIPTION OF THE PROJECT

16.5.1 General

The planned Namche Bazar Hydro Power Station can be regarded as an extraordinary engineering project in every respect. This is due to the high altitude of the project site, which restricts construction activities to the periods between the monsoons, and to its difficult approach. Access to the site by land takes a 12 to 15 days' walk from the nearest point of the China Road and an about 2 days' march from the STOL-pist at Lukla, on which Twin Otters of the national Nepalese airline RNAC can land and which is in regular operation three to four months a year (in the periods between the monsoons). It is only from the Pilatus STOL-pist at Syang Boche, situated immediately above Namche Bazar, that it takes an only 3 hours' easy walk to reach the project site.

These fundamental conditions have had an important bearing on the choice of the design and the building materials. The construction methods adopted are all those allowing the use of materials found at the site. These are primarily boulders for massive structures, as impervious material, and wood. Only a minimum quantity of cement for the massive structures, structural steel for reinforcement, plastic pipe for the penstock, and the electrical and mechanical equipment such as turbines, generators and transformers as well as steel sections for hydraulic steel structures remain to be flown, or transported manually, all the long way from the pist at Lukla to the construction site.

For this reason certain operational safeties were dispensed with, especially as regards the removal of undesirable bed load deposits, which involves a small amount of clearing work towards the end of the summer monsoon.

16.5.2 Weir Complex

The weir complex is a wide clear-overfall weir arranged like a side weir, with the crest having about the same slope as the natural river bed. A crest length of 40 m has been adopted to satisfy the requirement of avoiding excessive overflow heights even during the maximum probable flood. In addition, the overflow length is increased in times of major water levels due to the presence of the embankment, which limits the TYROLEAN WEIR on the left bank (Fig. 16 - 5). This is a riprap-protected structure rising in the sense of flow.

The downstream face of the weir consists of large gneiss blocks with the spaces in between being filled with mortar. A certain roughness, with blocks of variable size, is, however, desirable to enhance energy dissipation in the case of floods. The side weir has 1:3 sloping faces, and the crest is designed to be some 1,5 m wide. The foundation is planned to extend some 1 to 2 m (fig. 16 - 6).

15.5.3 TYROLEAN WEIR and Settling Basin

The intake channel starts at the lowest point of the downstream weir face and runs in the sense of the flow, along the existing right-hand bank slope towards the TYROLEAN WEIR. The extended TYROLEAN WEIR is embedded between two large gneiss boulders. On the right bank a riprap-protected cut will be made for the channel, which will terminate with a concrete wing wall immediately upstream of the TYROLEAN WEIR. Only the box of the TYROLEAN WEIR is made of massive concrete. On the left bank gabions placed obliquely one above another lead into the downstream weir.

The rack area of the TYROLEAN WEIR is 2.0 m wide, 4.) m long and inclined at 20°. Bar spacing as well as profile will be fixed after determining the available material.

In addition to the access channel there is also a discharge spillway. A natural depression immediately downstream of the weir complex is used for the provision of a settling basin. Its capacity of 1.600 m^3 allows a passage time of 13 minutes at a flow of 2 16 m^3/s for sedimentation.

16.5.4 Headrace

On the basis of the results of an economic analysis, a trapezoidal section (see report 9, fig. 9 - 9) has been adopted for the headrace. The headrace is lined with gneiss slabs, placed in lean concrete and with the joints sealed with cement. Although the route is very direct, it can cross the talus slope with a minimum of material movement (maximum cut is 3.0 m, maximum embankment height is 2.5 m). The headrace 250 m long, terminates in the surge basin.

16.5.5 Surge Basin and penstock

The size of the surge basin has been guided by the constructional requirements for the six intake trumpets in the intake wall and the construction of a valley side discharge spillway. Due to the construction of a discharge spillway, no volume for swell protection was necessary, and the surge basin has to level only the starting surge apart from constructional requirements. The usable storage of 250 m^3 corresponds to a starting time of 2 minutes (fig. 16 - 7).

In view of the given transport possibilities plastic penstocks with a diameter of 350 mm have been chosen for every unit. For the 6 intake trumpets, which should be well designed hydraulically, but which could only be constructed with great difficulty at the site, a special construction has been de[illegible]ped with a mould clement each, whose endproduct is again used as mould [illegible]e intake wall (fig. 16 - 8).

The penstocks will be laid in a sand trench at a frost-resistant depth and will be backfilled. Their length will be 250 m.

16.5.6 Powerhouse

The location of the powerhouse is already given in front of the gorge entrance. Huge boulders border the river on both sides (fig. 16 - 9). The power station will be constructed parallel to the course of the river, sheltered by such boulders. The understructure consists of a concrete box section foundation, whose reinforced concrete ceiling has six circular openings for the introduction of the draft tubes and the placement of the power units (fig. 16 - 10). The concrete box is open on the south-southeast side and leads into the tailrace by means of a retaining wall and existing gneiss boulders, continuing into the natural river bed with a cavitation sill. The superstructure is covered by a lean-to roof resting on wooden cross girders. The powerhouse crane is suspended along the generator axis.

As far as the electro-mechanical equipment is concerned, 6 simple Francis-shaft-turbines of Austrian fabrication are provided for, with a rated discharge of 360 l/s each and directly connected generators, which will be placed on the machine room floor with anchor bolts. The extreme altitude of the site (3.400 m) also causes great problems concerning mechanical aspects: whereas the turbines provided for (specific rotation n_q = 50 U/min, crosshead h_K = 45 m) would allow a difference of altitude between runner elevation and lowest tailwater level of 6 m when located at an altitude of 400 m above sea level, it is only 2,3 m here. Trying to lay the foundation of the power house only as low as necessary, this value has been fully utilized.

16.6 CONSTRUCTION MATERIALS AND TRANSPORT

16.6.1 Construction materials

The main construction material consists of dressed and undressed blocks of massive granitic gneiss. They will be used as fill stones embedded in cement mortar, as facing material, packed stones (like dry masonry), riprap, and stone slabs for lining channels and basins.

Sand and gravel on the most varied grain sizes can be obtained from almost all sedimentation areas of the river course and can for the greater part be used as concrete aggregate. In the area of the settling basin a layer of silty and cohesive moraine material was found, which can be used as impervious core material. The use of wood for panelling and for special constructions (e.g. rafters) will only be possible under certain conditions, that is to say, after the permission of the National Parc administration.

16.6.2 Transport

A site at an altitude of almost 4000 m, which cannot be in operation from June to September because of the monsoon and from December to February because of the temperature, which furthermore has no access roads, only steep paths, causes great problems concerning the transport of construction material. The shortest distance between the site and a road practicable for lorries (the so-called Chinese Road) is about 80 km, the difference of altitude is approximately 2.500 m. This transport route, which should primarily be considered in an underdeveloped courtry like Nepal, with cheap labourers, had to be rejected due to the bad condition of the way and the long distance, all the more so as the beast of burden, the Yak, could not always be used because of the numerous suspension-bridges. The use of helicopters is only planned for special transport (turbines, generators, construction machines, etc.) because of the high costs. Therefore, there remained only the two-engined De Havilland planes of the type "twin Otter", normally used for trekking tours. These can fly loads of about two tons to the STOL-pist at Lukla, about 12 km south of Namche Bazar. From there the materials have to be taken to the construction site by a permanent crew of carriers.

The airport LUKLA, however, is the point of departure of almost all trekking tours in the Khumbu area and therefore too busy in the fine periods before and after the monsoon. The meteorological situation permits only one or two daily landing manoeuvres in the morning, even in periods of dry weather; the morning dew and the later winds make landing on the lawn pist, inclined up to 15 %, even more difficult. If 100 annual days for flying were assumed (with one flight per day), the transport of cement alone (400 tons according to the latest, most economical calculations) would last for two years. The need of carriers would be considerable, too, and will undoubtedly lead to an exorbitant demand of the carriers who are nowadays at the tourists' disposal: If one reckons 3 days (of carrying) for the transport of one sack of cement (50 kg), the result will be 24.000 days, and in the case of 200 days per year a continous supply of 60 carriers will be necessary for two years. The "Sherpa-trade union" is striving for restricting the weight limit to 25 kg, which would mean a further delay of transport.

16.7 DATA OF THE PLANT

Catchment area:		
	area	392 km^2
	glaciated	34 %
	mean altitude	5.200 m
	altitude of water intake	3.400 m
Water intake:		
	clear-overfall weir with TYROLEAN WEIR:	
	crest length (inclined)	40 m
	discharge capacity (1 m overflow)	80 m^3/s
	trashrack TYROLEAN WEIR	about 10 m^2
	rated for an intake of	2,16 m^3/s
	settling basin	1.600 m^3
Headrace:		
	headrace channel lined with gneiss slabs	250 m
	surge basin	500 m^3
	penstock 6 Ø 350 mm	250 m
	crosshead	45 m
Power station:		
	6 vertical Francis turbines 360 l/s each	125 kW each
Energy situation:		
	rated discharge	2,16 m^3/s
	crosshead	45 m
	power	750 kW
	annual output	6,35 Mio kWh

16.8 CONCLUSION

Preliminary work for this plant was already started 3 years ago. Troublesome detail work resulted in the construction of a construction store, a lodging and office room, the construction bridge, and the deviation channel. The real beginning of the construction is planned for the period before the next monsoon. An experienced Austrian expert will act as an adviser for the beginning of the construction. The plant will above all be constructed with materials which can be found near the site, and native staff will be trained during the electromechanical montage for later operation. Thus, self-initiative and achievement are supposed to be a return for the present of development aid. This kind of development aid is therefore the most constructive one; it does not supply a ready product, but the basis for individual development.

16.9 BIBLIOGRAPHY

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16 CASE STUDY NEPAL
FALLSTUDIE NEPAL

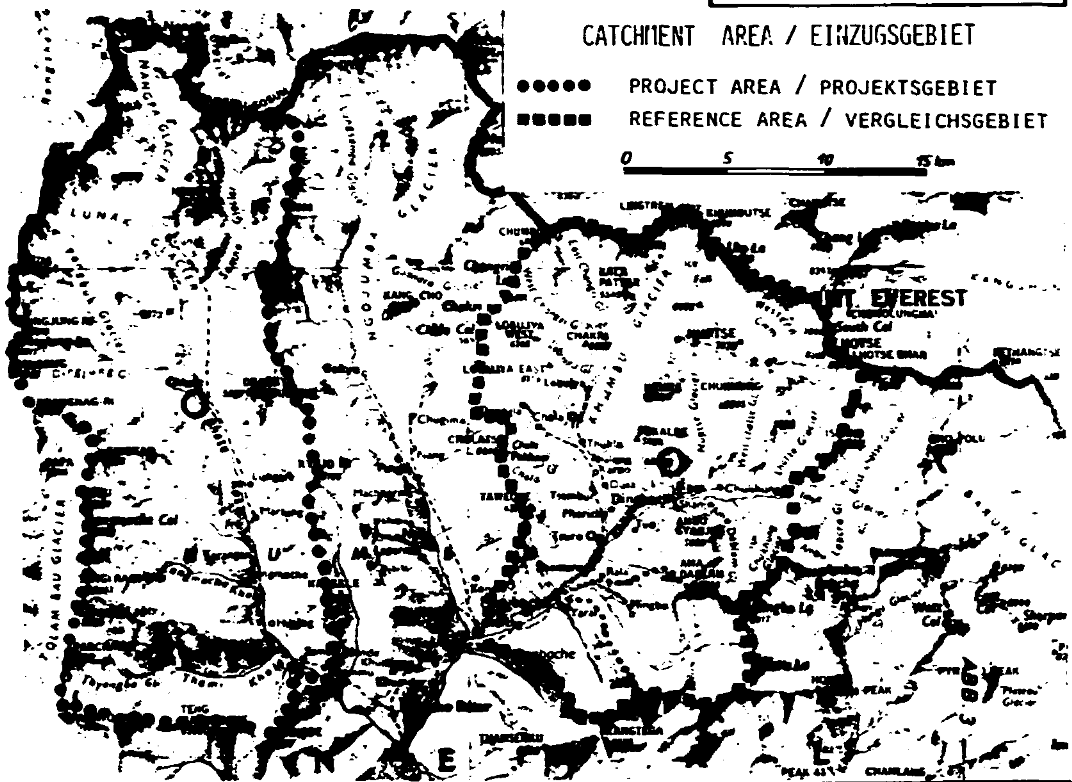

FIG 16 - 1 BASIN MAP
OBERSICHTSPLAN

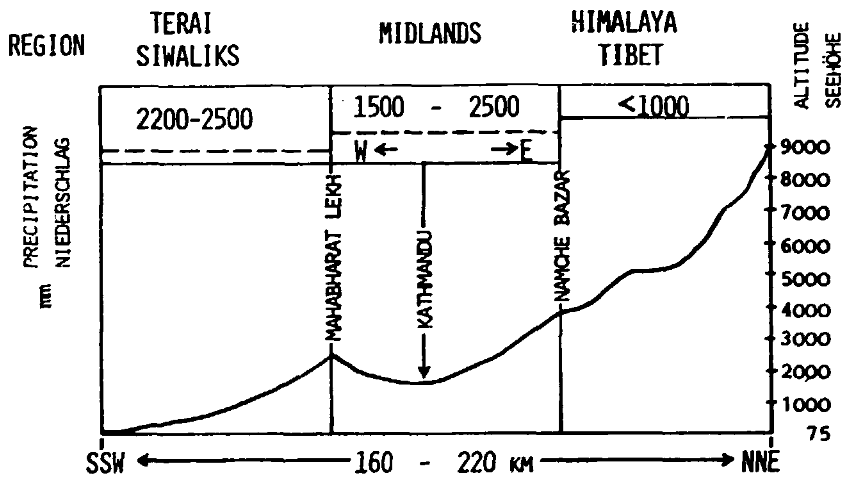

FIG 16 - 2 PRECIPITATION RELIEF
NIEDERSCHLAGSRELIEF

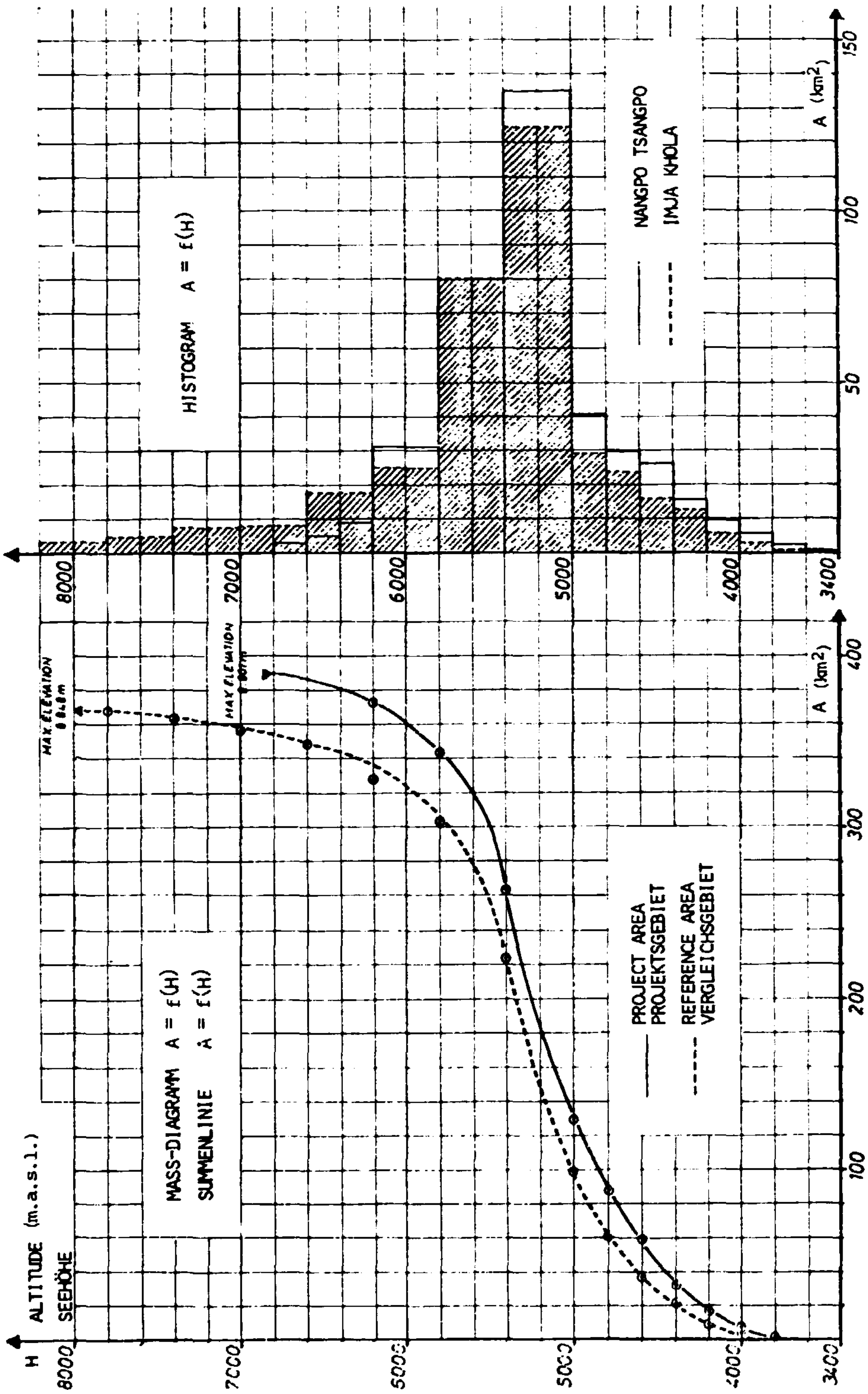

FIG 16 - 3 CATCHMENT-AREA CHARACTERISTICS
EINZUGSGEBIETSCHARAKTERISTIK

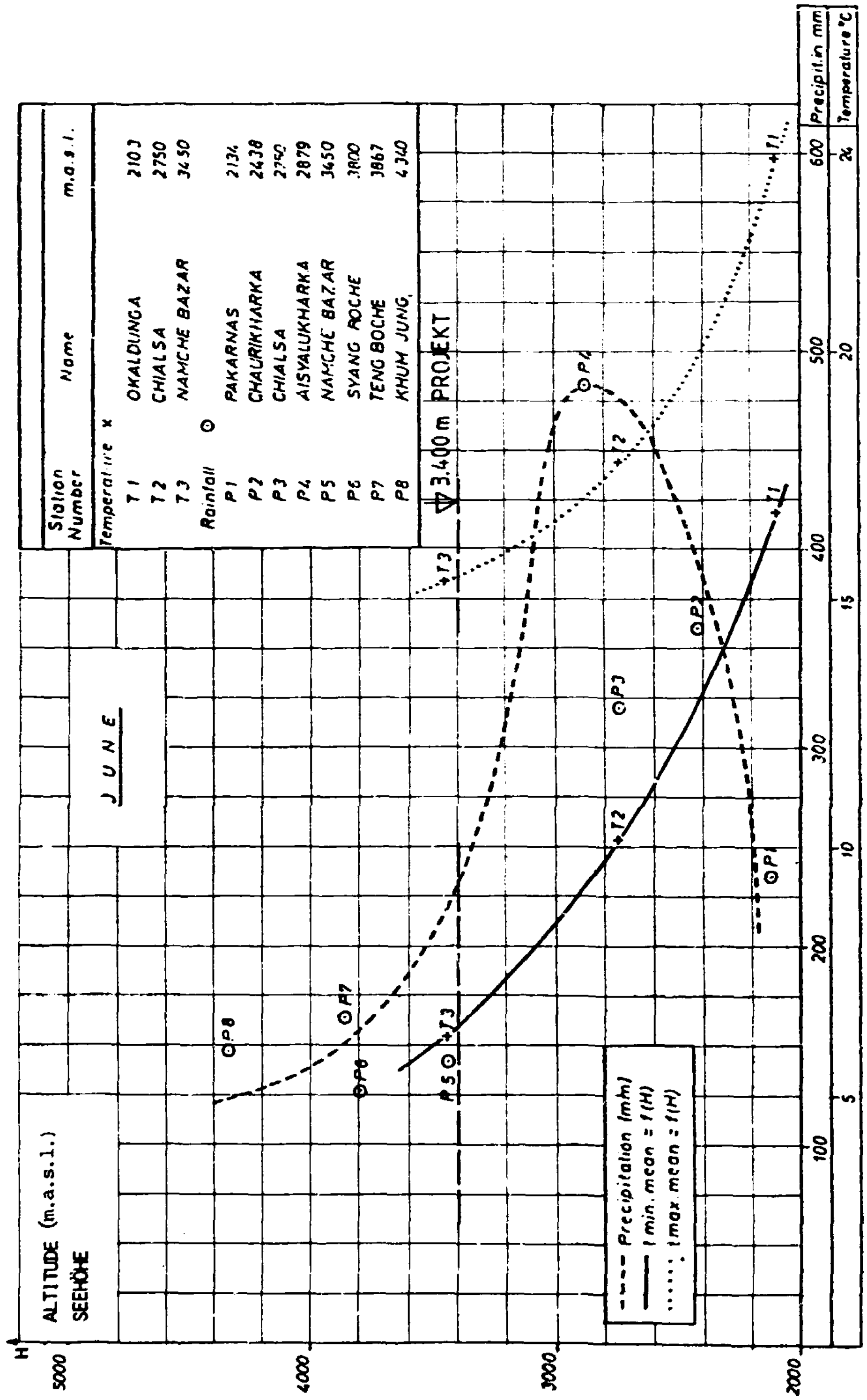

FIG 16 - 4 PRECIPITATION & TEMPERATURE = f(H)
NIEDERSCHLAG & TEMPERATUR = f(H)

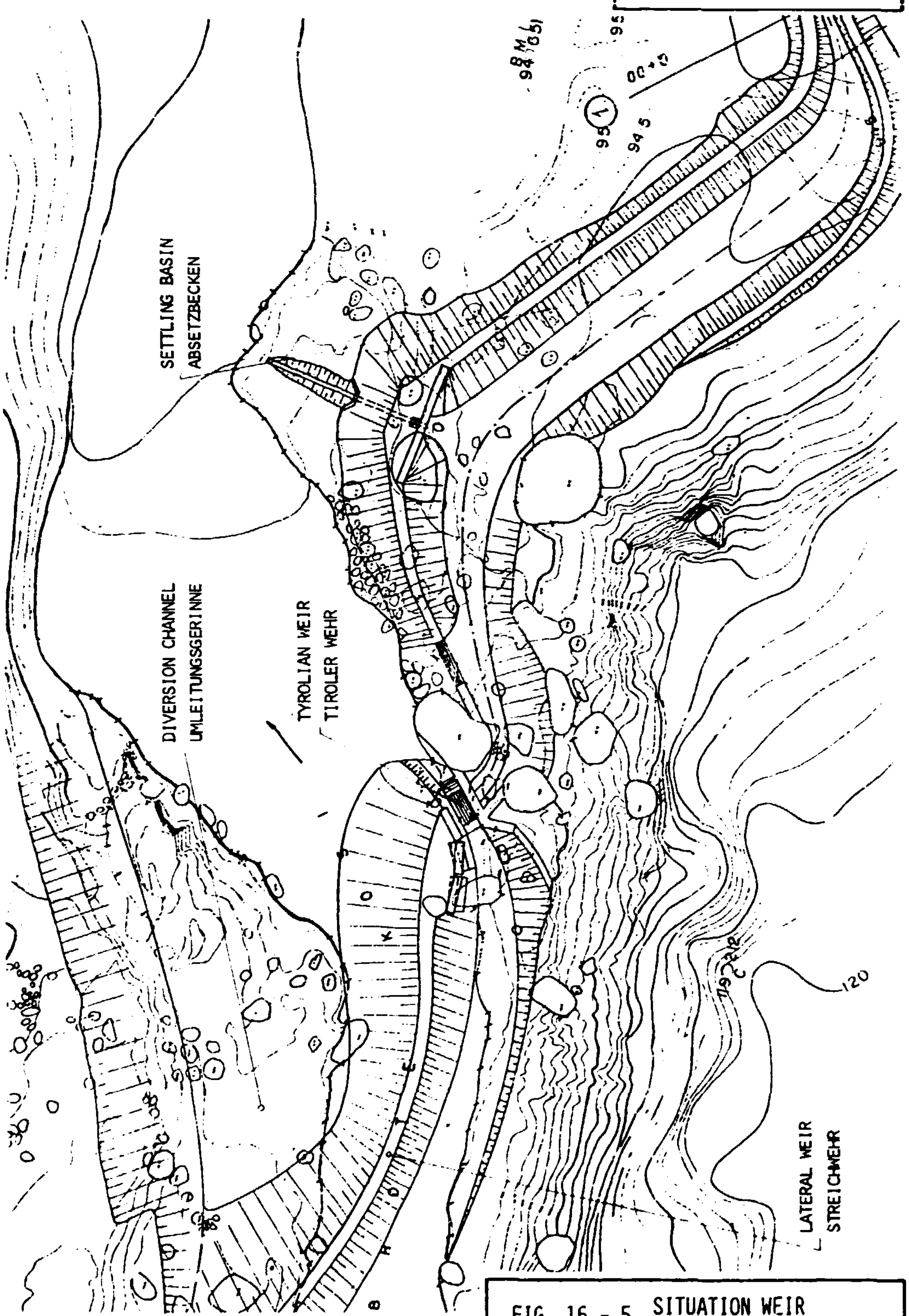

FIG 16 - 5 SITUATION WEIR
GRUNDRISS WEHRANLAGE

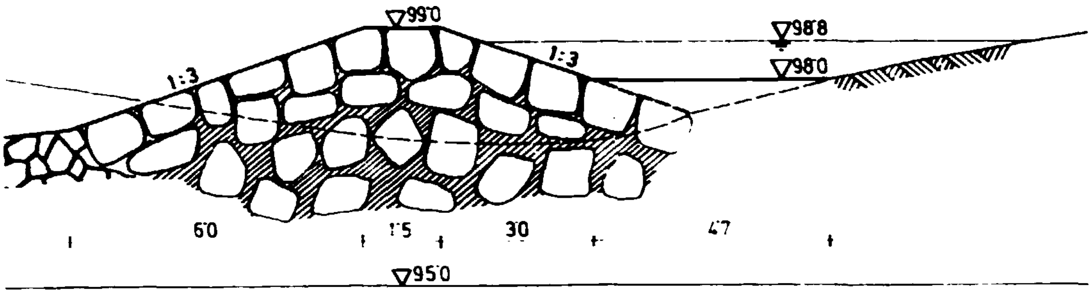

FIG 16 - 6 DAM SECTION
DAMMQUERSCHNITT

FIG 16 - 7 SURGE BASIN
AUSGLEICHSBECKEN

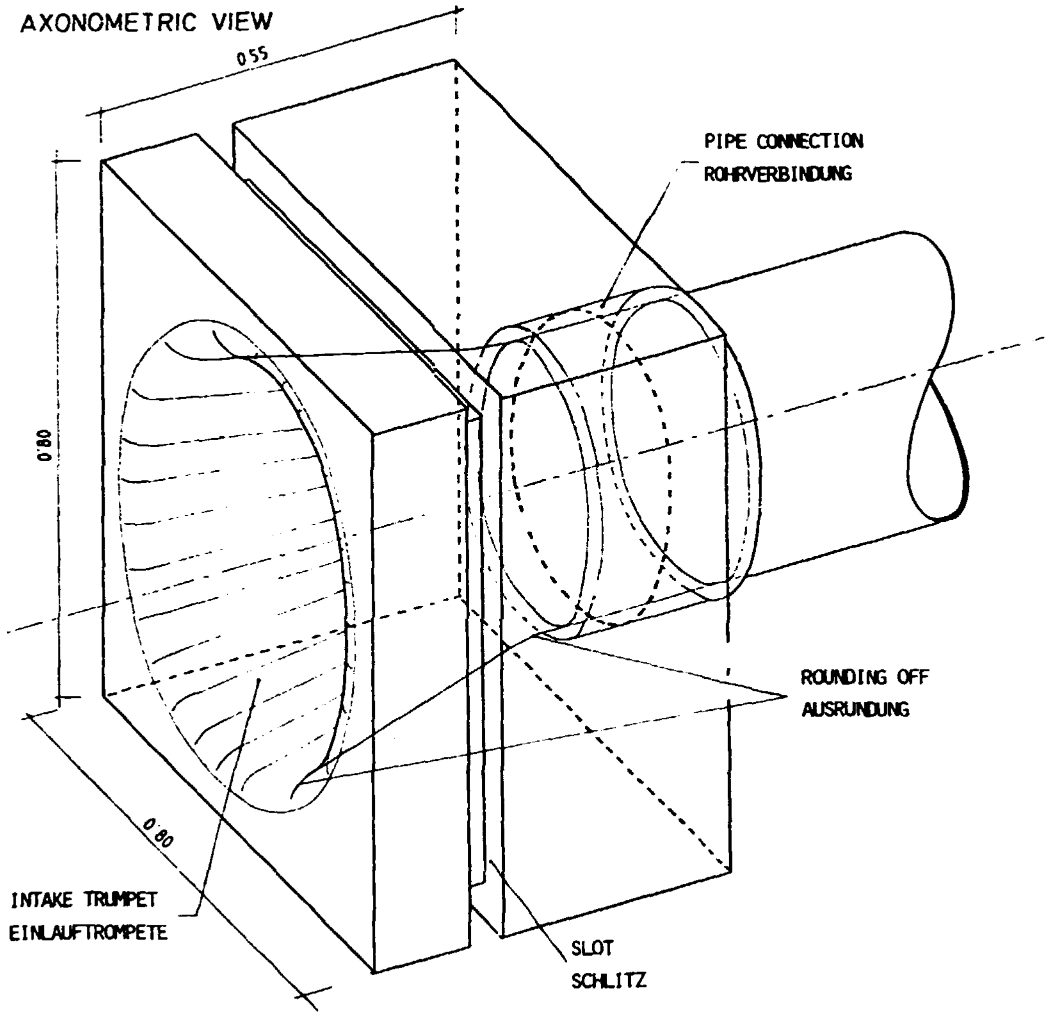

FIG 16 - 8 INTAKE TRUMPET
EINLAUFTROMPETE

16 CASE STUDY NEPAL
FALLSTUDIE NEPAL

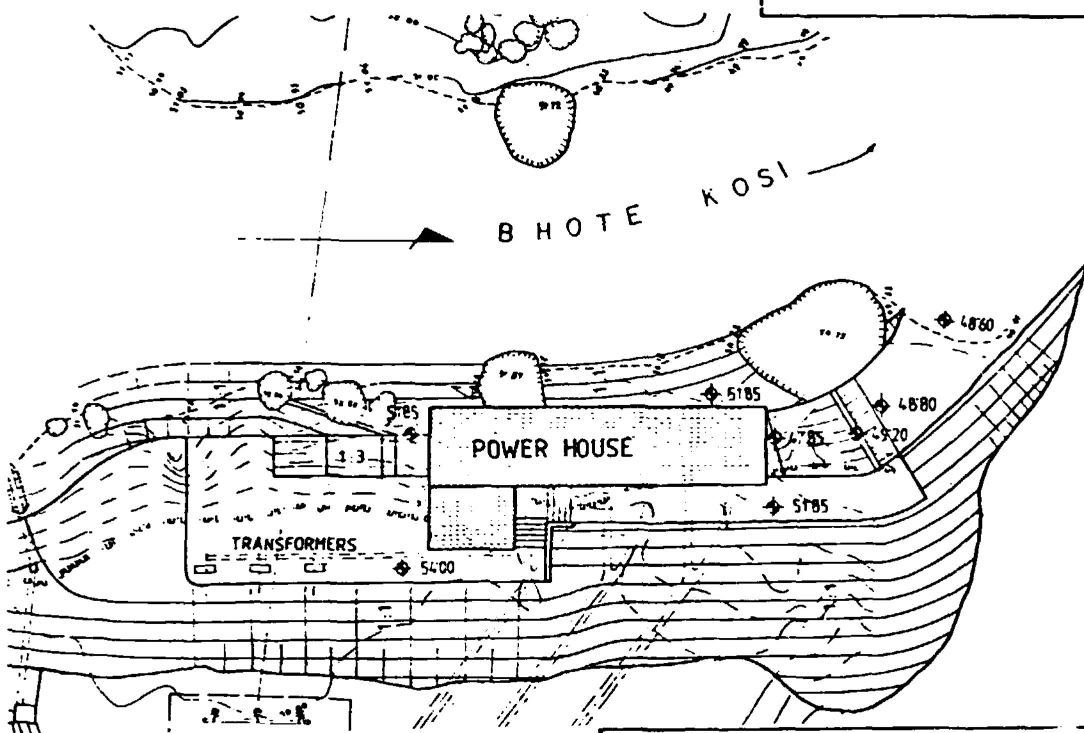

FIG 16 - 9 SITUATION POWER HOUSE
LAGEPLAN KRAFTHAUS

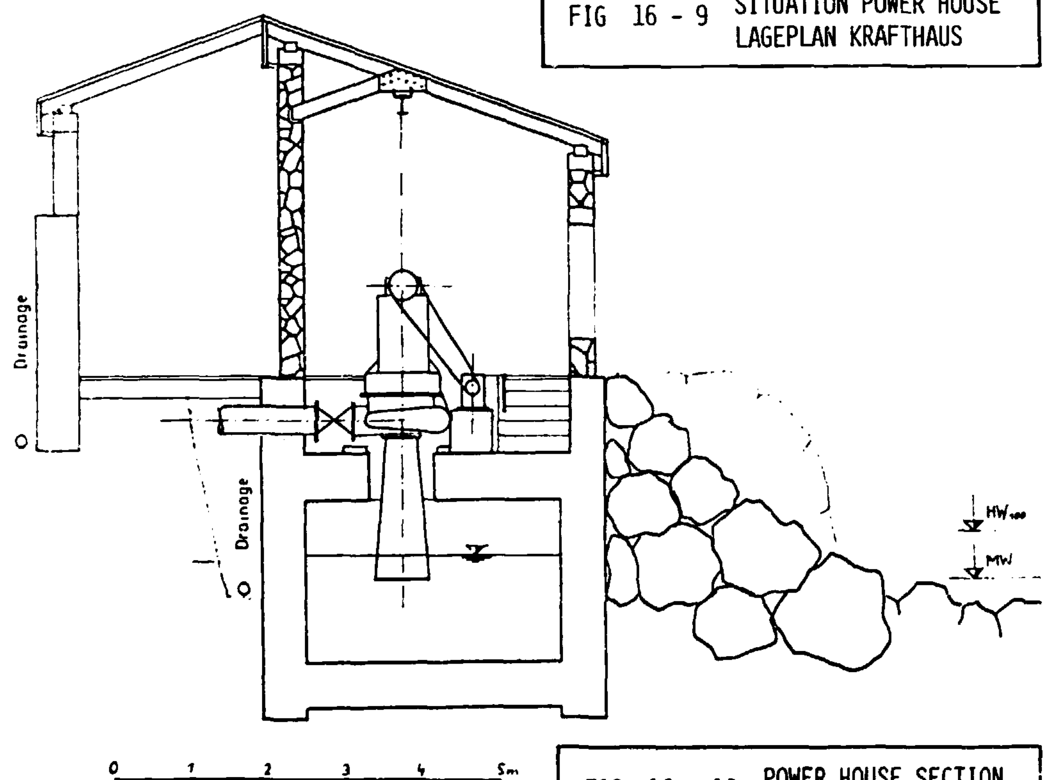

FIG 16 - 10 POWER HOUSE SECTION
KRAFTHAUS-SCHNITT

17 THE LEGAL SITUATION OF THE CONSTRUCTION OF SMALL HYDROPOWER PLANTS IN AUSTRIA

OBERLEITNER F.

ABSTRACT:

The complex relationships within the economics of water supply and distribution become apparent with the construction and the operation of small hydropower plants. The legal system within which these activities have to move must as much as possible take this complexity into account. Consequently sometimes rather considerable costs are involved with this or that project. Therefore, when judging the profitability of a small hydropower plant, the legal provisions in regard to the respective project have to be included into the considerations. The problems concerned and the different possibilities for solution in connection with the provisions of the Austrian water law are being described.

17.1 INTRODUCTION

The various human claims to the waters as well as the interferences with these waters necessitate some order which recognizes and safeguards the relationships and the far reaching scope of the economics of water supply and distribution. The Austrian water law regulates all branches in this field and has juridiction over all waters. It does not, however, cover navigation. The goals of the water law are

1) optimal water utilization with regard to public interests and the rights of any outside party, therefore
2) the protection of the water from man, and finally
3) the protection of man from harmful effects of water.

The consequences of human interferences are most of time not limited to the immediate field of hydrology. Therefore the Austrian water law is f.e. also concerned with building projects along water systems. The construction of small hydropower plants will serve as an illustration.

When constructing and maintaining small hydropower plants there are not only technical and economic issues to be solved. They are the foremost considerations, but also the legal issues which arise must not be overlooked as they might have a decisive influence on the technical and economic conditions under which a small hydropower plant can be built. Although the legal principles in the different countries, even within Europe, are hardly comparable and therefore legal regulations cannot be transferred to other countries, cultures or continents, a description of the legal situation as can be found in Austria, certainly can point at general problems and suggest possibilities for solution.

17.2 OBLIGATORY CONCESSION AND JURISDICTION

Austria is a Federal Republic. Legal jurisdiction and enforcement in some fields is in the hands of the Federation (federal government), in others it is in the hands of the federal provinces (subsidiary states). So is the water law within the jurisdiction of the Federation; nature- and landscape protection or the law concerning building are concerns of the federal provinces. Hydropower plant construction touches several legal spheres so that several authorities of the government and/or of the federal provinces are involved with this kind of project. Since the essential issues of our topic deal with the water law, some aspects of this field of law will be discussed in detail in the following chapters.

According to a fundamental principle of the Austrian water law the utilization of the waters is basically permitted to everyone, but in most of the cases a concession from the water law authority is necessary in order to safeguard public interests and the rights of any outside party.

Small hydropower plants are means for exploiting the mechanical power of the water. A concession from the water law authority is necessary for the utilization of the water, for setting up plants as well as for transforming installations of these plants which serve for the exploitation of the waters. No concession is necessary for private waters (i.e. small brooks) when they are used by the proprietor himself or used with his consent. This, however, is only permissable if there is no infringement on the rights of a third party or if no influence on the current of the water, on the river bed, on the quality of the water, or the water level of public or private waters is to be expected or if it is absolutely assured that the river banks are in no way endangered and the land of a third party not flooded or become marshy.

As a rule a concession has to be obtained from the water law autorities for a hydropower plant.*) Basically the district authorities have jurisdiction over these projects. If hydropower plants exceed a capacity of 200 HP (i.e. appr. 147 kWh) or if they are located at certain public waters or at boundary waters the jurisdiction is with the respective head representative of the federal province. The same applies to those cases when any local authority itself is involved in the proceedings as a party, or for plants that need for some reason or other an additional consent. If for the small hydropower plant a dam with a height of more than 15 m (embankment dam), or 40 m (arch dam and concrete dam) is built or if interstate negotiations are required the jurisdiction lies within the Ministry for Agriculture and

*) In some cases together with the concession from the water law authorities additional approvals from one or several other authorities might be necessary, such as the electric power authority, the building authority, the nature preservation authority, the forestry authority, the railway authority, the authority for commerce and trade etc. The preliminary requirement for the construction and operation of a plant is always that the approval of all necessary authorities have been called for independently of each other.

Forestry. The same applies to those projects which demand quick realization, in the interest of national economy and have been declared as preferential constructions. All bigger dams depend furthermore on the expert opinions of the "Staubeckenkommission", a special authority within the Ministry for Agriculture and Forestry whose suggestions and recommendations have to be the initial steps in the appropriation procedure.

17.3 DESIGNING AND CONFLICTS OF INTERESTS

Already before designing the project, the department for planning hydro-stations has to be informed of the intention to set up a small hydropower plant. In this way the coordination of related planning activities might be reached at the lowest costs possible and arising conflicting interests and goals can be avoided at an early stage. Also the people concerned with the designing will get the necessary information about the situation of supply and distribution and the overall physical planning of the projet site in order to help towards a realistic estimation of how the project can be materialized in the best way and how awing difficulties can be avoided. It might turn out that there is a conflict because of already existing water exploitation or that another project is being planned for the same stretch of river.

In these cases, if an agreement is not reached between the applicants, the water law authority has to go into proceedings of interference. Here the existing water rights have preference, if they are not limited or removed out-of-court or with compulsory measures and compensation. If there is a conflict with different projects among each other the Austrian water law gives preference to that application which is more favourable in view of public interests; if this consideration does not lead to a decision all equivalent claims - with special consideration of the water supply - are to be satisfied in the best suitable way. If this cannot be achieved either, those applications get preference where a better fulfillment of the goal or the least possible side-effects towards a third party are expected.

This testing and weighing of a conflicting situation in view of public interests by the authorities is rather complicated. Therefore the proceedings of interference are not only time consuming, but also because of the many necessary expert opinions, rather costly. The expenditure might be proportionally very high to the single project, so that the profitability of a small hydropower plant would be rather negatively affected.

In this context it should be pointed out that by an act of legislation the respective federal province itself has a right of entry. This regulation favours, however, only the federal province itself with a legal status, not for example , a power station, even if it should be owned by a federal province. But as the federal province hardly ever runs a power station this right of entry has little importance in practical life. It should, however, be pointed out that in 1947 the enterprises and plants for electrical energy supply and distribution were nationalized; excluded from nationalization are electrical energy enterprises with a capacity of a maximum of 200 kW as well as plants for self-supply if they do not supply to outside parties more than 100 000 kWh per year, or if a further supply does not go to a nationalized electricity distribution plant. It has sometimes been argued that this so-called "2nd nationalization law" allows small power plants only within the quoted exception. The water law has not to do with clarifying this situation.

An application for the consent of a small hydropower plant must include the dimensioning plans and all necessary technical data and comments by the experts. Along with that information must be given to make a judgement possible as to whether or not public interests or the rights of a third party are infringed. A statement must be added about the expected advantages from the projected plant or the disadvantages in case of renunciation. Furthermore one has to indicate whether the hydropower plant is for one's own supply or for an electricity supply and distribution enterprise; in the latter case an approval of the electricity authority has to be furnished. The requirement to indicate all people whose rights might be jeopardized makes it necessary for the applicant to contact the parties concerned for an agreement at an early stage. With such early considerations in the phase of designing serious conflicts can be avoided and sanctioned by the authorities and interference into the third party's rights can be reduced as much as possible.

17.4 SAFEGUARDING OF PUBLIC INTERESTS

If an application was brought in conform to regulations, the water law authority procedes to a prelimiary investigation of how far public interests are to be taken into account. In the case of hydropower projects the water law authority also examines whether or not a complete and economical utilization of the water power available can be expected. This investigation is important because a project infringing public interests is only permitted under certain conditions and if they are not sufficient to safeguard public interests, the project has altogether to be refused.

The water law names f.e. the following public interests:

- national defense
- public safety
- public health
- flood- and ice removal
- navigation and rafting
- existing or planned regulations
- condition of natural waters including their banks
- quality of the waters
- general usage
- necessary water supply
- national culture

- preservation of monuments
- preservation of the beauty of nature and settlements
- agricultural water needs
- economical distribution of water resources
- complete utilization of water power
- domestic water needs
- domestic economic interests

This enumeration of public interests to be safeguarded by the water law and contained in the national legislation is not complete. The water law authority has therefore also the jurisdiction over additional public interests.

This far-reaching competence is limited only in so far as the water law authority cannot disapprove of a project because of infringement of public interests; for the safeguarding of these a special approval is needed, (f.e. a procedure by the authority of the conservation of wild-life).

Among the numerous public interests only those will be discussed in detail which are of special importance for the construction of small hydropower plants: the requirement for retaining rest-water within the sloping zone and the estimation of complete and economic exploitation of the water power.

17.5 COMPENSATION-WATER PROBLEMS

As a rule small hydropower plants are set up along those waters whose water flow, by nature, is subject to serious fluctuations and where already proportionally small amounts of draw-off have very strong negative effects on the total water flow. Fishing, concerns of nature - and landscape preservation, water supply for district settlements, the quality of the water, concerns in the field of hygienics etc. require that rest waters are kept in much used stretches of waters. With that, along with the seasonal fluctuations, the amount of water withdrawn for utilization is quite limited. Moreover, small hydropower plants are mostly set up near a valley, so that the optical effects on the surroundings are more aggravating than those of high-altitude storage-plants. The retaining of restwater is essentially a concern of public interest. This is, of course, influential on the profitableness of small hydropower plants and therefore it is understandable that the problem of restwater is a decisive issue,especially with small plants.

17.6 COMPLETE AND ECONOMICAL UTILIZATION OF WATERPOWER

With the increase in utilizing the water powers, as a rule, also the specific building costs are rising. The water law authority must take care that with a smaller and less expensive project, an enlargement of the plant at a later date in view of a more complete exploitation of the existing water power does not present too many difficulties. The development of water power exploitation will have to follow the general economic development. Parallel to that there is another consideration known as "environmental protection" i.e. the recognition of ecological factors together with water supply and distribution.

The shortage of energy requires an optimal development of water power resources; the economic situation requires an optimally economical development of these and the ecological relationships require the greatest possible care with interferences into the natural environment. With the decision for a complete and economical utilization of the water power the expenditure for environmental protection, aspects of national economy and the time factor have to be included, not only in regard to the sloping part which is used but for the total catchment area. For the optimal utilization of domestic water power resources it is important to agree utilization potentials as to time and location. The development of water power serves for the guarantee of the water supply as a basis of existence in our country. The potential of small hydropower plants corresponds to appr. 8 - 11 % of the total consumption of electric energy (status of 1979). One has, however, not to forget that the actual water power potential is by far not completely usable. There are technical and local reasons for this. It depends on the profitableness of investments, on conditions of energy management, on business initiatives and on considerations related to nature- and landscape preservation, as to what extent the actual water power potential can be exploited. The profitableness of a small hydropower plant cannot be achieved at the costs of public interests, i.e. public charges. This shows that the construction of small plants cannot replace the development of large-scale hydropower plants, but they represent a good completion within the overall economic framework.

For the local co-ordination of power plant projects, the hydrological planning authorities have to be contacted and the previously mentioned administrative proceedings have to be taken into account. The time agreement with later executed projects can be reached with a time limit of the required water utilization right.

17.7 PROCEEDINGS, PARTIES, APPROVAL

If the water law authority has completed the preliminary examination of the project and has not found any reason for refusal, the proceeding is continued with an oral negotiation and with participation of all parties and experts concerned. The purpose of this oral negotiation is the clarification of all issues connected with the project. This means that the outside parties affected by the project have to raise their objections at the latest at this negotiation. Exceptions can only be made for those parties who were neglected to be asked for participation. These parties can raise their objections within 2 weeks from the date of knowledge or until the time of legal decisions of the matter concerned. The parties participating in this proceeding are together with the applicants for the concession, the owners of those properties which are affected by the project, or co-sharers of the water. If there is no agreement between the parties coercive rights can be decreed against compensation. Parties with authorization for fishing can claim the safeguarding against pollution, the arrangement for fishways and fish racks as well as a non-injuring regulation for the diversion of the brook. A compensation can only be required if these claims are not met.

The safeguarding of public interests is one of the concerns of the water law authority and is to be achieved together with the help of respective claims and charges; if this is not possible the concession has to be refused.

When the proceeding for the concession concerning the plant has been positively completed, the water law authority issues an official notice about the concession for the intended utilization. The concession includes the exact data about the plant and is combined with claims and charges for the safeguarding of public interests and the rights of a third party. At the same time possible objections against the project are refused, necessary sanctions admitted and related compensations stated. Mutual agreements are registered. In the notification the extent of the water exploitation and/or of the rest-water is also determined. The extent of water exploitation is dependent upon the needs of the applicant and the offered water volume and must not go to such an extent that the inhabitants of villages, single settlements and the like, suffer from lack of water necessary in case of fire, for public purposes or for any activities in the home or at the farm.

Construction time periods are determined as well as the length of the concession. Hydropower plants get a concession for no longer tha. 90 years. In order to co-ordinate timewise the construction of small hydropower plants with those of bigger projects, one has to apply to the Ministry for Agriculture and Forestry if concessions should surpass a period of more than 30 years.

When the construction of the plant is finished, the water law authority examines whether the installation is in accordance with the concession and checks whether the watermarks are duly fixed. Finally the right to use the water is entered into the register and in this way receives protecting publicity.

Now the hydropower plant is ready to be operated. It has, however, to be pointed out that a number of other approvals along with the concession are necessary without which the plant cannot be constructed or operated.

17.8 OBLIGATIONS DURING EXISTENCE

The contractor has to maintain his installations in accordance with the concession and has to observe the related conditions and charges. A violation of these obligations is not only followed by punitive sanctions and claims for compensations; the authority can abate nuisance and as the strongest measure can determine the loss of the decreed right. The contractor is also obliged to maintain the water-course in immediate vicinity in good condition, that is, the regular removal of detritus and spits as well as the good upkeep of the banks. Negative effects upon further water-courses are to be corrected and possible extra charges for protective- or regulation structures are to be met. These obligations also have essential influences upon the profitableness of a small hydropower plant.

The water law authority has to make sure that the lacking profitableness of a plant does not lead to the neglect of public interests and to the neglect of preservation of nature and landscape,it has to ascertain the prevention of negative effects on the quality of the water, on the ground water and on the run-off events,and that with the protective measures against floods no conflict arises between the requirements of environmental protection and flood protection. The water law authority has to guarantee the observation of these aspects - with control measures - even during the operation period of the plant.

17.9 EXPIRATION OF RIGHT

The right to use the waters expires f.e. with renounciation, expiration of a period, non-feasibility of a project within the arranged period of time, withdrawal and continous delapidation of the plant over more than three years. The water law authority has to make an official decision about the expiration and also has to give order about the measures the party entitled to the usage of the water has to take, in order to protect public interests or the rights of any third party. This means that also in the case of expiration of usage considerable costs are involved, f.e. for the removal of the plants, the filling up of canals, the construction of protective walls etc.

There is also the possibility that an interested third party takes over the plant. This party assumes then the rights and obligations of the predecessor, but with a renewed approval from the water law authority.

In the case of expiration of a period the continued operation of the plant requires likewise a renewed approval. The entitled party can apply for the renewal before the expiration of the period and has a legal claim to being granted anew the right of utilization.

In the case of destruction of plants the water law authority can grant a period of time for the reconstruction and can give assent to minor alterations.

17.10 FURTHER LEGAL PROVISIONS

The Austrian water law includes in addition to the above mentioned provisions some others which are relevant for small hydropower plants. Some of them will be discussed in the following:

The water law authority can claim, as far as this might be necessary in some exceptional cases, a reasonable and proportionate provision of security for satisfying necessary requirements, proper maintainance and/or removal of the plant.

Together with hydrological constructions of any kind the applicant has to take the necessary precautions in view of the security of people and properties and has to make sure that no traffic hold-up occurs.

If the utilization of waters is reached best with joint use of already existing reservoirs or storage plants, the water law authority can give its approval when questions of cost sharing and/or compensation have been settled. The water supply for agricultural purposes can be regulated in the same way.

When there is a shortage of water the water law authority has to distribute the existing supply in an adequate way.

The person (party) entitled to use the water is not only liable for those damages which he has caused but also for damages on which one did not count at all or of that extent, at the time when the authorization for usage was given. He is also liable for the damages caused by parties that were not named at the time of his application and therefore could not be called in to the preceeding.

Interferences upon the quality of the water require a previous approval from the water law authorities. This means in the case of hydropower plants that f.e. trash can principally not be put into the after-bay but has to be removed in an undetrimental way.

If protective - or control structures as well as works for the preservation of the water quality are undertaken which are paid for by the government or any of the federal provinces, the beneficiaries, that is, also the owners of a small hydropower plant, can be called in to make a contribution. On the other hand, if the person (party) entitled to use a certain water stretch takes advantage of the existence and/or the supply of the hydrological plant of a third party he has likewise to be approached for sharing the costs of maintainance.

If a noticeable improvement in the utilization of the water resources can be achieved after co-ordination of water use (plant operation), the water law authority is to pass order on any reasonable alterations in the operation or in the structure of the plant with a compensation of losses. The party entitled to use the water can be made obliged to make recordings of water flow changes, to set up and operate certain installations, to supply any observation data as well as give information about floods and must also allow any of such measures.

If designing or carrying out a project necessitates preliminary works or supplementary and aiding structures on the property of a third party, the water law authority can order the reluctant proprietor to tolerate this against compensation of detriments.

Co-operatives and associations can be founded for setting up, using and maintaining plants for joint usage.

In order to safeguard public interests as well as for the protection of a third party, preliminary legal provisions can be sanctioned. This may become necessary if a hydropower plant, because of insufficient maintainance, risks or endangers people's safety.

Damages of hydrological plants or installations, infringements of the water law regulations and non-observance of directives by the authorities are liable to prosecution. Independent of that, the water law authority can insist on the restitutionof the condition of a plant or installation as prescribed by the law.

17.11 CONCLUSION

The utilization of water power has a long tradition in Austria. After a certain time of stagnation the constantly rising prices for energy lead to increasing interests in small hydropower plants. The development of small-scale water power plants is definitely a trend towards optimal utilization of domestic energy sources. On the other hand there might be the case where public interests or the rights of a third party have to be safeguarded and where the construction of a small hydropower plant might be unfeasable or of hardly any profit. This proves that when designing a small hydropower plant, along with the technical and economic aspects, also legal aspects have to be considered, the neglect of which during the period of designing might lead to considerable difficulties later on. The legal provisions as a framework within a project which is to be planned and materialized are, however, not to be separated from ecological, economic, technical and social conditions.

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18 THE SUPPORTING FUNCTION OF THE UNIDO INVESTMENT PROMOTION SERVICE IN INDUSTRIAL CO-OPERATION PROJECTS

KNOTTER P. MAYERHOFER W.

ABSTRACT:

In their attempt to further the industrialisation efforts of developing countries, UNIDO (United Nations Industrial Development Organisation) launched some years ago a programme aiming at the promotion of industrial co-operation projects in developing countries. The main objectives of this programme are assistance in the identification of projects in developing countries and in the search for partners (mainly in highly industrialised countries) as well as consultation during the ensuing project development including the financing stage. In the practical realisation of the UNIDO promotion programme of industrial co-operation, the seven UNIDO Investment Promotion Services that exist in Brussels, Cologne, New York, Paris, Tokyo, Vienna and Zurich play an essential part. They act as an important link between the industrial sector of their respective host countries and interested parties in the developing countries, and assist both partners free of charge to the stage of realisation of co-operation projects, that is, industrial projects in developing countries with the aim of establishing a fairly close and long-term engagement of the partners, e.g. in the form of a transfer of technology (licences and know-how) with or without supply of machinery, management contracts and marketing arrangements. Some projects may even include a joint venture.

The following Paper will convey an idea of various aspects of the UNIDO programme for the promotion of industrial co-operation. Particular emphasis will be placed on a description of the possibilities available to the UNIDO Investment Promotion Service in Vienna in order to allow appraisal of its potential role in future industrial co-operation in the field of mini hydro power between interested parties in Latin America and in Austria.

18.1 THE UNIDO INVESTMENT PROMOTION PROGRAMME

The possibilities of co-operation in the realisation of industrial projects in developing countries cover a wide range including the training of local manpower and technicians in connexion with the establishment of turn-key or other installations, contracts for the provision of technical aid, management and marketing contracts, licence and know-how agreements and finally the closest form of co-operation, i.e. the joint venture. The prime and usually most difficult problem in most of the projects relating to the co-operation between firms from a developing country and an industrialised country, respectively, is the selection of, and the establishment of contacts between, suitable partners. The interested party, the individual enterprise in a developing country just as well as in a highly industrialised country will often not dispose of a promising possibility of making its wishes known across the oceans to a large enough number of potential partners, to say nothing of presenting its project in an adequate form and follow it up in the partner countries. In order to fill this gap, UNIDO has called to life an organisation that is available free of charge to all parties interested in industrial co-operation projects. Thus, in the UNIDO investment Co-operative Programme Branch, to which the joint World Bank/UNIDO programme is subordinate as well, and in the individual UNIDO Investment Promotion Services, a team of experts is endeavouring to improve the conditions required for the establishment of joint ventures and other form of industrial co-operation.

The first step is assistance in identifying a project and in many cases also in preparing the chief project informations necessary to enable a potential partner of co-operation to assess whether a project holds any promise of success. This is done by UNIDO experts in collaboration with the national governmental or private organisations (as e.g. ministries of industry, investment authorities, investment promotion services and development banks).

The next step consists in presenting these projects proposals from developing countries to an as large as possible group of interested parties in industrialised countries, through the following three channels: (a) Roster of Resources, (b) Investors' Forum and (c) UNIDO Investment Promotion Services.

18.1.1 Roster of Resources

UNIDO has so far pooled by computer the data of 1250 firms from 40 highly industrialised countries and newly industrialised countries, separated according to branches. All these firms have stated their basic interest in co-operation with developing countries and automatically receive all project proposals relating to the industrial sector specified by them.

18.1.2 Investors' Forum

Investors' fora are organised by UNIDO on the request of a country or economic region and afford an opportunity for a major group of interested parties from the industrial countries to meet project sponsors for thorough discussion for several days in the developing country itself. Thus, it is the intended purpose of such an event not only to provide information through papers and seminars on important aspects of co-operation such as e.g. the economic and industrial policies of a country (or region), investment laws, taxation and possibilities of financing, but also to enable interested local partners and potential interested parties from abroad to enter into bilateral negotiations on the individual projects. The interested party from abroad has an opportunity to obtain an idea of the infrastructure, working conditions, costs, available raw materials and the demand for products and finally on the partner himself. A disadvantage of such events results from the relatively high travel expenses incurred by persons from the industrialised countries.

18.1.3 UNIDO Investment Promotion Services

The UNIDO Investment Promotion Services (IPS) act as contact points for the UNIDO Investment Co-operative Programme in a number of industrialised countries. It is through the IPS that project proposals from the developing countries are brought to the notice of firms in the country for which the respective service is responsible. Red tape is avoided as much as possible. At present there are IPS in Brussels, Cologne, New York, Paris, Tokyo, Zurich and Vienna. Advantages of these Services are the thorough knowledge of firms, the direct contact with the firms to be considered for co-operation in the industrialised countries and the comprehensive and continuous expert advice offered free of charge and impartially to potential partners in developing and industrialised countries. Although the objectives and basic principles of work are fundamentally the same for all IPS (except for New York, whose main function is the training of staff from the developing countries according to a training-by-doing programme), the individual working methods show differences as they are adapted to suit the conditions of their respective host countries. The expenses of running these Services are normally met by a contribution made by the host country to the United Nations Industrial Development Fund.

18.2 UNIDO INVESTMENT PROMOTION SERVICE IN VIENNA

The UNIDO Investment Promotion Service in Vienna was founded on 1st February 1980 by UNIDO in collaboration with the Austrian government and the Austrian Federal Chamber of Commerce. Its objective is the encouragement of co-operation between Austria and the developing countries in the realisation of industrial projects in developing countries. It is concerned with industrial projects which require foreign capital and/or know-how and are suitable for assistance by Austrian firms. Such projects may relate to the establishment of new industrial enterprises just as well as to the extension of existing plants. The projects to be promoted should be economically attractive and correspond to the industrialisation objectives of the respective developing countries. Although the industrial plant and machinery required for industrial co-operation projects in developing countries can in many cases be supplied by Austrian firms, the aid offered by the UNIDO Investment Promotion Service in Vienna (and the other IPS) fundamentally differs from pure export promotion in that in projects furthered by UNIDO the transfer of technology is a prime consideration and close and major-term engagement of the partners is desired.

The services offered by the UNIDO Investment Promotion Service in Vienna include:

18.2.1 Information

Austrian firms are kept informed - selectively - about projects under consideration and projects in the stage of realisation in developing countries as well as about participation and co-operation possibilities. Government authorities, chambers of industry and commerce, development banks and enterprises in developing countries receive information on wishes to co-operate as well as technology offers and possibilities of Austrian firms.

18.2.2 Search for Partners

Search for know-how and investment partners in Austria for industrial projects on behalf of interested parties in developing countries.

Assistance to Austrian firms in their search for projects and partners in developing countries.

18.2.3 Contacts and Negotiations

Establishment of contacts between possible project and co-operation partners and assistance in the ensuing negotiation phases.

18.2.4 Structuring of Projects

Assistance in collecting and analysing data for the appraisal and realisation of a project. This includes general economic information, legal aspects (investment and corporate laws, taxation, tax privileges, transfer of profits and capital, social law), market conditions, import regulations, investment and manufacturing costs, cost and availability of manpower and raw materials.

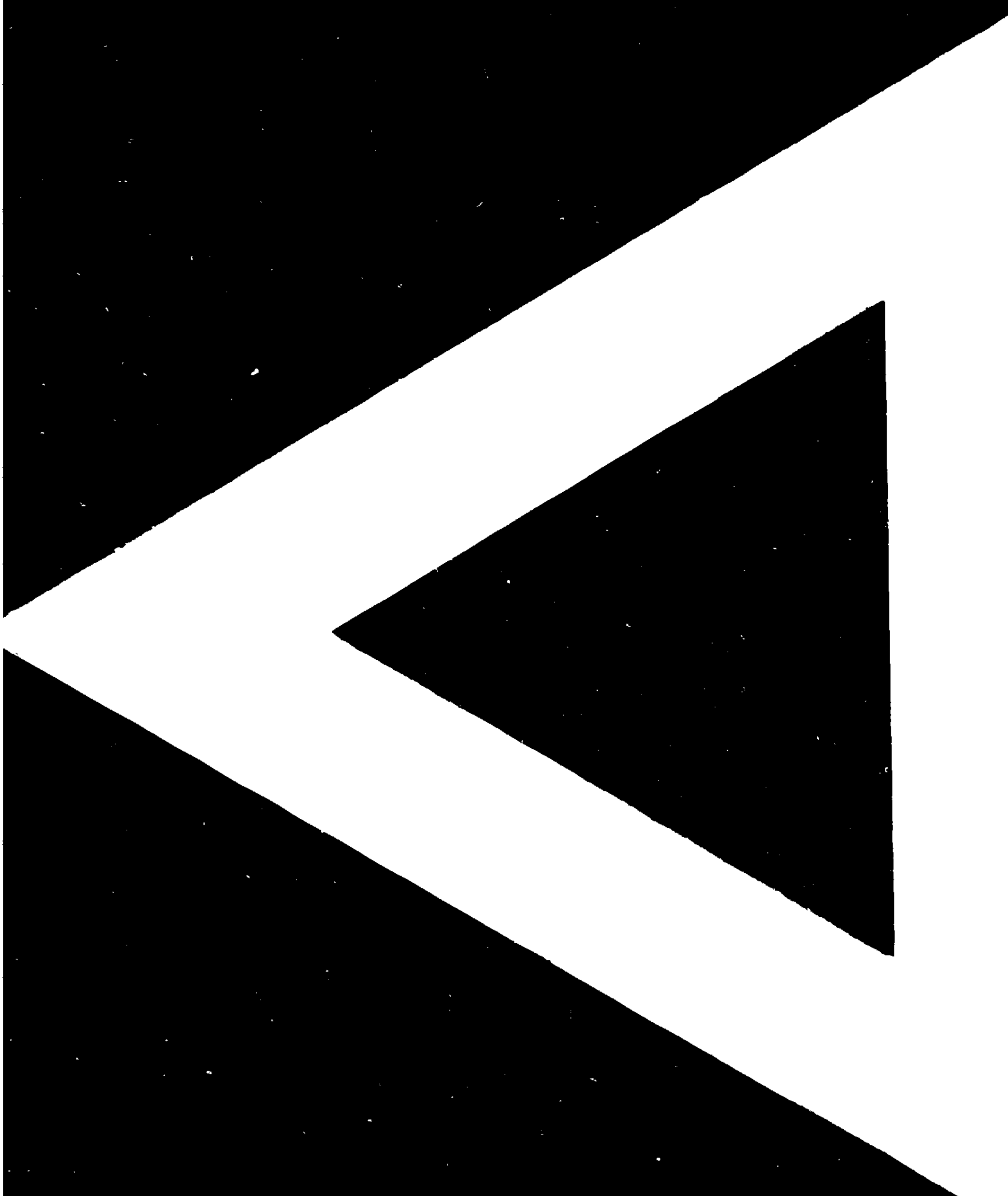
Project studies: Advice regarding the proper preparation, analysis and presentation of projects.

Project financing: Advice regarding the analysis and structuring of project financing. Advice regarding possibilities and sources of financing. Establishment of contacts with public and private investors and credit institutions both in Austria and in the developing countries concerned as well as on an international scale.

18.2.5 Work Approach

As follows from the list of services described above, the functions offered by the Investment Promotion Service extend far beyond the pure establishment of contacts. Its assistance can be called on in all the phases of project preparation and development. Its work is mainly governed by entrepreneurial principles and is available free of charge to public and private organisations, enterprises and financing institutions in Austria as well as in developing countries. The UNIDO Investment Promotion Service in Vienna is fully supported by UNIDO Headquarters in Vienna and has access to all UNIDO economic reports and project informations. In addition, it has close connexions with the World Bank, the International Finance Corporation and regional and local development banks.

UNIDO's objective is the promotion of industrialisation in the Third World. As part of this organisation, the Vienna Investment Promotion Service hopes to make at least a small contribution to the solution of this great task.



Naturally it is necessary to state probabilities of occurrence for cost figures as these are capable of reflecting cost as found in reality only with a certain probability. The following table shows the cost data underlying Fig. 13-4 as well as the interval limits for a certain probability of occurrence of the value to be found:

Turbine type	Cost function k=........AS/KW	Interval limits as percentages for a probability of occurrence of		
		95 %	67 %	50 %
Kaplan and bulb turbines	$78200.Q^{-0.35}.H^{-1.01}$	+ 33 - 26	+ 14 - 13	+ 10 - 9
Francis turbines	$158100.Q^{-0.46}.H^{-1.01}$	+115 - 54	+ 42 - 29	+ 27 - 21
Pelton turbines	$147300.Q^{-0.43}.H^{-0.83}$	+ 37 - 27	+ 15 - 14	+ 11 - 10

Table 13.7: Cost functions of turbines including gates and gearings for a certain probability of occurrence (Q in m³/s, H in m)

The above relationships will be explained with the help of a further diagram. For this purpose, we have taken a section along a 200-KW line on fig. 13-4. The result is shown on fig. 13-5. This clearly illustrates the above mentioned relationship between increasing head and decreasing specific cost for a constant total plant capacity. It is also seen from this figure that the transitions from one turbine type to the next are all marked by a major cost reduction, which can be explained by the fact that each following turbine type shows better adjustment to the respective head-discharge ratio. Fig. 13-5 indicates in addition the region of probability of occurrence of the specific cost forecasts for Pelton turbines, which suggests that the forecast value will fall within the hatched zone with a probability of 2 to 3.

13.4.2. Transformer Costs

Transformer costs are relatively easy to bring into a relationship as they are for the greater part dependent only on transformer ouput. The fundamental relationship is given by the following formula (based on 1978 prices):

$$\text{Transformer costs } k = \left(160 + \frac{41\ 000}{P(kW)}\right) \text{AS/kW};$$

P = active transformer output