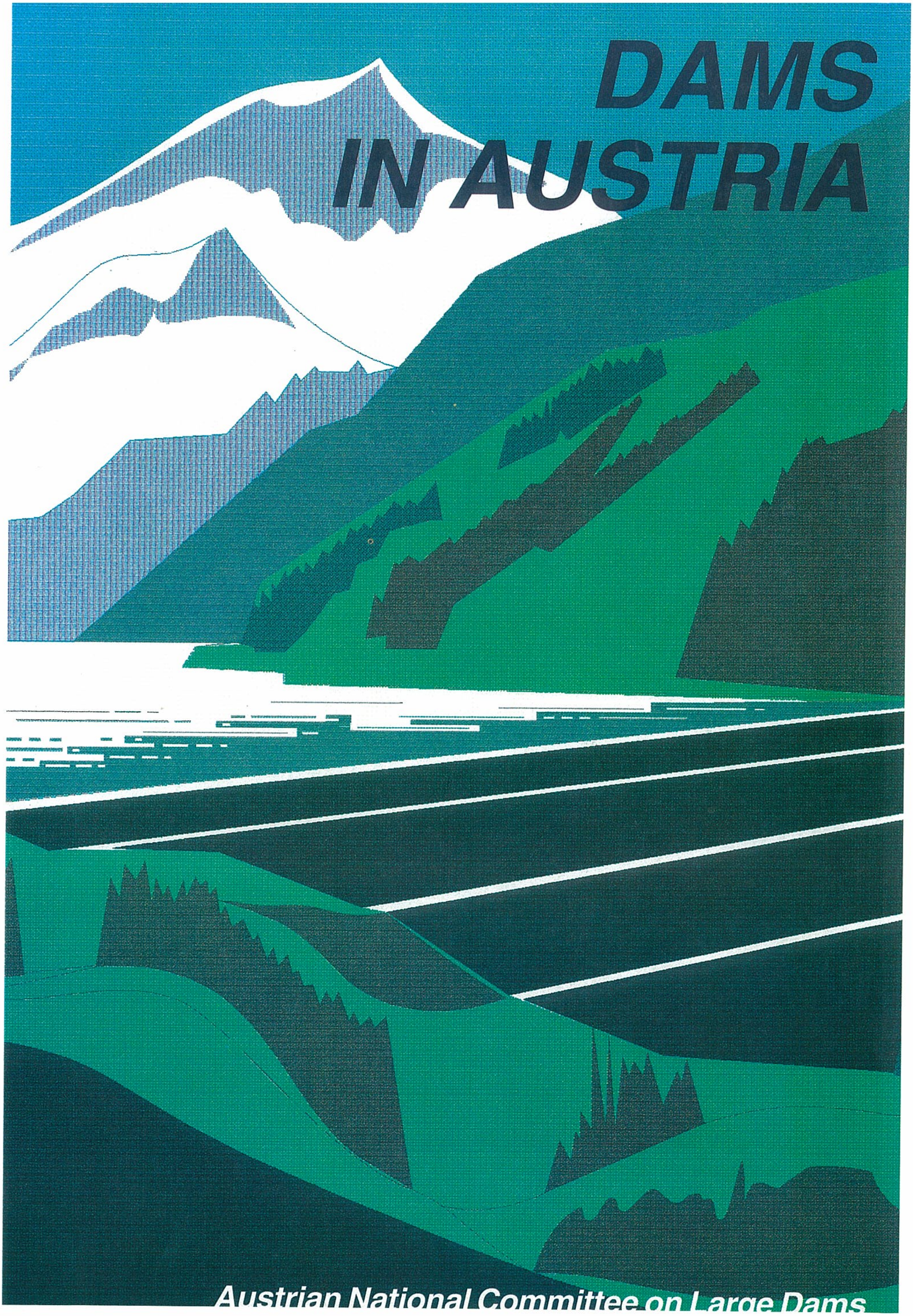
DAMS IN AUSTRIA



Austrian National Committee on Large Dams

DAMS (VA, PG, TE, ER)
RIVER BARRAGES (PG)
N
0
50
100 km
OBERAUDORF-EBBS
BOLGENACH (TE)
Rhine
VORARLBERG
Inn
SALZB
Ill
SPULLERSEE (PG)
FINSTERTAL (ER)
DURLASSBODEN (TE)
TAUERNMOOS (PG)
LIMBERG (VA)
MOOSER+DROSSE
LÜNERSEE (PG)
TYROL
STILLUP (TE)
ZILLERGRÜNDL (VA)
BOCKHARTSEE (ER)
KOPS (VA+PG)
VERMUNT (PG)
SILVRETTA (PG)
GEPATSCH (ER)
SCHLEGEIS (VA)
EAST TYROL

WELLACHER

OTTENSTEIN (VA)
SCHÄRDING-
NEUHAUS
ASCHACH
Danube
GREIFENSTEIN
VIENNA
MELK
YBBS-PERSENBEUG
TRAUN-
PUCKING
Traun
LOWER AUSTRIA
LOSENSTEIN
KLAUS (VA)
Enns
STYRIA
BURGEN-
LAND
Mur
TGULDENSEE (ER)
NBREIN (VA)
MÜHLDORFERSEE (PG)
PACK (PG+TE)
CARINTHIA
Drau
GABERSDORF
KORALPE (ER)
LAVAMÜND
VILLACH
EDLING
FREIBACH (TE)
DAMS DESCRIBED IN THIS VOLUME

DAMS IN AUSTRIA

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PREFACE

The Austrian National Committee on Large Dams has the honour and pleasure of hosting the 59th Executive Meeting and the 17th World Congress of the International Commission on Large Dams in Vienna in June 1991. In continuation of a long standing ICOLD tradition, this commemorative publication on dams and dam engineering in the host country is dedicated to the participants at these two events.

The 45th Executive Meeting in Salzburg in 1977 was also commemorated with a special publication, namely "Large Dams in Austria -1977", comprising a systematic description of all Austrian dams and river barrages in operation or under construction. In addition, since Austria's large dams and river barrages are constructed almost exclusively by the country's electricity supply companies, it was decided to mark the 15th World Congress in Lausanne in 1985 with a detailed presentation of all seasonal storage schemes and run-of-river plant – the two predominant types of hydropower development in this country – in a brochure entitled "Hydro Power Schemes and Large Dams in Austria - 1985". Both publications are still valid and useful reference works today.

Although this commemorative publication makes free use of data contained in the two earlier publications, it also contains much additional material and provides a completely fresh selection and restructuring of its subject matter. The authors have been at pains to offer a comprehensive overview of dams and dam engineering in Austria, its background and development, on the one hand, while restricting detailed description to a selection of dams that can be assumed to be of general interest in view of their height, design features or specific construction problems or experiences on the other.

Accordingly, the first five chapters (A–E) are devoted to the engineering objectives and natural conditions typically encountered in Austria (and especially the geology of the sites), the history of dam construction in its close interrelationship with general hydropower developments, approvals procedures and legislation pertaining to dam operation and surveillance, plus the environmental aspects of dams and reservoirs and their implications for the engineer.

This is followed, in chapters F and G, by detailed discussion of 16 concrete dams and 10 embankment dams, with a comprehensive introductory section in each case, while Chapter H is devoted to Austria's river barrages for run-of-river plant on her seven biggest waterways. A summary of dam engineering techniques and their development from the point of view of the Austrian construction industry is presented in Chapter J, and Chapter K comprises a new register with the characteristic data of all Austrian dams and reservoirs excluding river barrages. As an additional source of relevant data, the Austrian entries in the ICOLD World Register of Dams are included in the Appendix.

The Austrian Committee on Large Dams is indebted to many colleagues for their dedicated work in preparing and editing this book, and also to the sponsors from the construction industry and the electricity supply companies, without whom this work would not have been possible. We trust this publication will also be of interest to engineers from other countries as a source of information on the state of dam engineering in the host country of this 17th ICOLD World Congress and may perhaps serve as a stimulus for their own work.

In presenting the book, I should like to take this opportunity to welcome all colleagues from abroad on behalf of the Austrian National Committee and express our sincere wish that their stay in Austria will be most enjoyable.

Dr. Wolfgang Pircher
Chairman

Austrian National Committee
on Large Dams

Chapter A

OBJECTIVES AND BASIC CONDITIONS FOR DAMS IN AUSTRIA

CONTENTS

OBJECTIVES AND BASIC CONDITIONS FOR DAMS IN AUSTRIA

By W. Pircher *

1 HYDROPOWER - THE PREDOMINANT PURPOSE

Human needs and natural conditions guide and determine the design and construction of dams in every individual case. The following remarks briefly outline these factors in as much as they characterize the background for dam engineering in Austria in general. All the 131 Austrian dams which qualified for inclusion in the World Register of Dams according to ICOLD criteria have "Hydropower" as their main and very often only purpose (see Annex). For more than eight decades the construction of large dams and large river barrages in Austria has almost exclusively been undertaken by electricity supply companies which at present produce about 70% of the country's electric power requirements from hydro. It is only for the power stations on the Danube that the government pays a subsidy towards construction costs as compensation for the resulting improvements to navigation on this important European waterway. Apart from hydropower, multi-purpose benefits, in particular substantial improvements to flood control deriving from the large Alpine seasonal storage schemes, are provided free of charge. Only very few of the many structures built exclusively for flood retention or debris flow control would qualify as large dams according to ICOLD standards (see Chapter B, Table 4).

As hydropower generation is thus by far the dominant function of Austrian dams, the natural conditions for their construction must be viewed and assessed primarily from that point of view, too. The country's relief and resulting drainage network, the distribution of precipitation and related runoff determine Austria's hydropower capacities. The discrepancy between supply, in the form of runoff regime, and demand defines optimum reservoir size in terms of energy production. Topography, settlement patterns and the many and varied additional functions – including deliberate non-utilization – of potential reservoir sites, on the other hand, impose certain limits on reservoir size, and sometimes very narrow ones.

This had led to a particularly clear pattern in the siting of dams, which is characteristic of all Alpine countries and is especially marked in Austria, relating to two predominant types of hydropower plants. First, in mountain locations, we find medium to very high dams of various types, especially for the high-altitude seasonal storage reservoirs built for the high-head power plants located in the side valleys of the Central Alps. And second, there are the major river developments located in densely populated valleys with correspondingly limited scope for impounding, where power generation is almost exclusively by low-head run-of-river plants of the river barrage type.

To date, only about two-thirds of the 54 TWh of total estimated hydropower potential in locations considered technically and economically feasible is utilized in Austria. This is the product of a period of fairly constant development in hydropower since the end of the forties. It is only in the last few years that the pace of construction has been considerably reduced, in spite of the continued growth in demand for electric power. This reflects the increasing difficulty experienced in achieving a socio-political consensus for new projects at a time when people are showing a pronounced interest in conservationist ideas. Hence it is difficult to make predictions for the development of the remaining hydropower potential. With an undeveloped potential estimated on the basis of numerous studies at 19 TWh (including one third in the form of seasonal storage), one can therefore speak of considerable and reassuring hydropower reserves in a small country, whose other reserves of energy are very limited. At the same time, these hydropower reserves represent the future of Austrian dam construction.

2 RELIEF AND DRAINAGE NETWORK

2.1 Topography

With an area of 83 855 km^2, Austria is one of the smaller countries of Europe. Of a total east-west extension of approximately 570 km, 250 km take the form of the narrow band of land that constitutes the west of the country (i.e. Vorarlberg and Tyrol), with a north-south extension of between 60 and 90 km. The remaining 320 km are located in the main body of the central and eastern regions of the country, which measures between 200 and 250 km in the north-south direction. Austria is located at between 46 $^1/_2$° and 49° northern latitude.

In spite of its small area, the country has a ramified topography and varied countryside. As the satellite photograph on the front inside cover clearly shows, the Austrian share of the Eastern Alps accounts for 58% of total area and is naturally very mountainous in character.

The watershed formed by the highly glaciated main Alpine chain marks Austria's southern border in the narrow western limb of the country and extends into the wider central and eastern region as an obstacle to communications between the north and the south that reaches into the very heart of the country. The inner crystalline zone of the Central Alps, which includes the

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country's highest mountain, the 3 798 m high Grossglockner, is flanked on both sides – interrupted by narrow zones of schistous formations, which are lower – by wide belts of Calcareous Alps to the north and to the south, with peaks approaching or slightly above 3 000 m.

The major part of the Eastern Alps in Austria shows a distinct longitudinal structure, with mountain chains and main valleys located on a more or less east-west axis. Towards the eastern margin, the Alpine chain is broken down into more isolated and lower mountain massifs, where there are no more glaciers but extensive forests, and a Pre-Alp character emerges, until finally the 1 200 km long Alpine chain terminates with the foothills of the Wienerwald practically on the outskirts of Vienna.

On the northern slopes of the Alps, especially in the region of Upper Austria and Lower Austria roughly as far as the Danube, Austria occupies a part of the Pre-Alps, a wide band of gently undulating hills shaped by the glaciers of the Ice Age, where agriculture and silviculture flourish. More hills flank the south-east margin of the Alps in Burgenland and Styria. To the north of the Danube, with the exception of the area of Tertiary hills in the most easterly part, Austria occupies a part of the Bohemian Massif, which extends even south across the Danube at a number of points. Deriving from a very old crystalline nappe, the massif is a forested highland region with rounded peaks reaching just over 1 100 m in altitude and moderately deep valleys. The only significant plains to be found in Austria are the Vienna Basin (Marchfeld), the Tullner Feld on the Danube, and the depression of the Neusiedler See, which is part of the Little Hungarian Plain and, with its water level at 115 m above sea-level, represents the lowest point in Austria.

2.2 Drainage network

With the exception of Vorarlberg, which mostly drains into the Rhine, Austria is for the most part located in the basin of the *Danube*, which flows through Austria for 350 km from Passau on the German border to the Slovakian city of Bratislava, increasing its catchment from 76 597 to 131 338 km^2 in the process. With a considerable average gradient of 0.4‰ along its Austrian reach and a relatively regular flow over the year, the Danube is Austria's most significant hydropower resource, with 35% of total hydropower generation in Austria in 1989 produced by the run-of-river stations located along its course. It is also the country's only navigable river, and with the completion of the Rhine-Main-Danube Canal in a few years will form part of the most important European waterway, runnig diagonally from the North Sea across the European continent to the Black Sea.

The following is a list of the major tributaries of the Danube and the most important in terms of hydropower:

- The River *Inn*, with a length of 515 km, is the longest tributary, and with a drainage area of 26 130 km^2 also has the biggest runoff. Located in the upper catchment of the Danube, it links Switzerland, where it has its source, with Austria (196 km in the Tyrol, plus a 6 km border reach with Switzerland and 8 km with Bavaria), and Germany, before forming the border once again over a length of 70 km, this time between Upper Austria and Bavaria. Although its catchment area is only half the size of that of the Danube (54%) at its confluence with the Danube at Passau, the Inn carries 10% more water thanks to the predominantly Alpine character of its catchments. The Tyrolean reach of the Inn has a drainage area of 1 945 to 10 337 km^2, and the border reaches between Bavaria and Upper Austria 24 154 to 26 130 km^2. Its biggest tributary is the River *Salzach*, which is 225 km long and has a 6 722 km^2 drainage area, and forms the border between Bavaria in Germany and Salzburg and Upper Austria in Austria on the last 57 km upstream of the confluence.

- The 146 km long River *Traun* has a total drainage area of 427.4 km^2, with a runoff regime that is regulated by the big lakes of the Salzkammergut that were formed by the retreat of the glaciers.

- The 349 km long River *Enns* has a total catchment of 6 075 km^2 located in one of the areas of highest precipitation in the Alps and therefore has the highest mean specific runoff of all the tributaries of the Danube.

- The 147 km long River *Kamp* has a total drainage area of 2 134 km^2, and although it is not the biggest, its hydropower stations make it the most important of the northern tributaries.

- The River *Drau* rises just to the west of the Austrian border in Italy, and increases its drainage area from 140 to 11 955 km^2 as it flows 198 km through Austria (East Tyrol and Carinthia) before crossing into Yugoslavia where, after a further 749 km as the River Drava, it joins the Danube with a drainage area of 40 150 km^2. Its biggest tributary is the River *Mur*, which has a length of 350 km on Austrian territory and a drainage area of 10 313 km^2, followed by a 35 km border reach between Austria and Yugoslavia, before flowing through Yugoslavia and along the border between Yugoslavia and Hungary, where it joins the Danube after a total of 434 km.

The above mentioned longitudinal orientation of the Eastern Alps is also reflected in the drainage network, with most inner Alpine rivers flowing parallel to the main Alpine chain for a good distance, creating relatively good lines of communication through the Alpine area of Austria along the east-west axis, e.g. via the Inn, Salzach and Enns valleys, the Mur and Mürz valleys, and the Drau Valley in Carinthia. Tectonic lines running transverse or oblique to these main axes are followed by rivers flowing north and south into the Alpine foothills, which thus completes the communications network.

All the above rivers have been developed for hydropower generation using low-head stations with barrages, while the big seasonal storage power schemes are located in the side valleys of the rivers Rhine, Inn, Salzach and Drau.

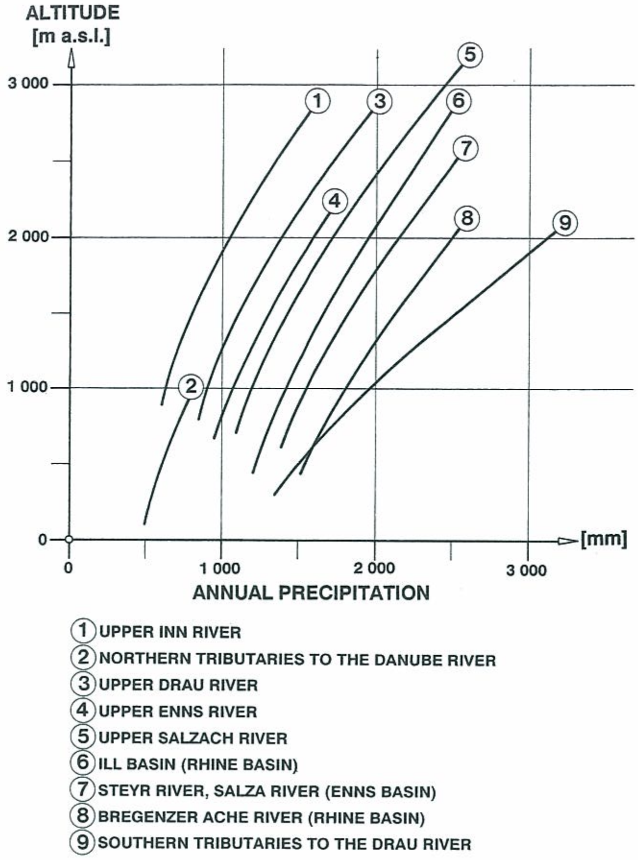


Figure 1
Variation in longterm average precipitation with altitude

3 PRECIPITATION AND RUNOFF

3.1 Hydrological and meteorological recording

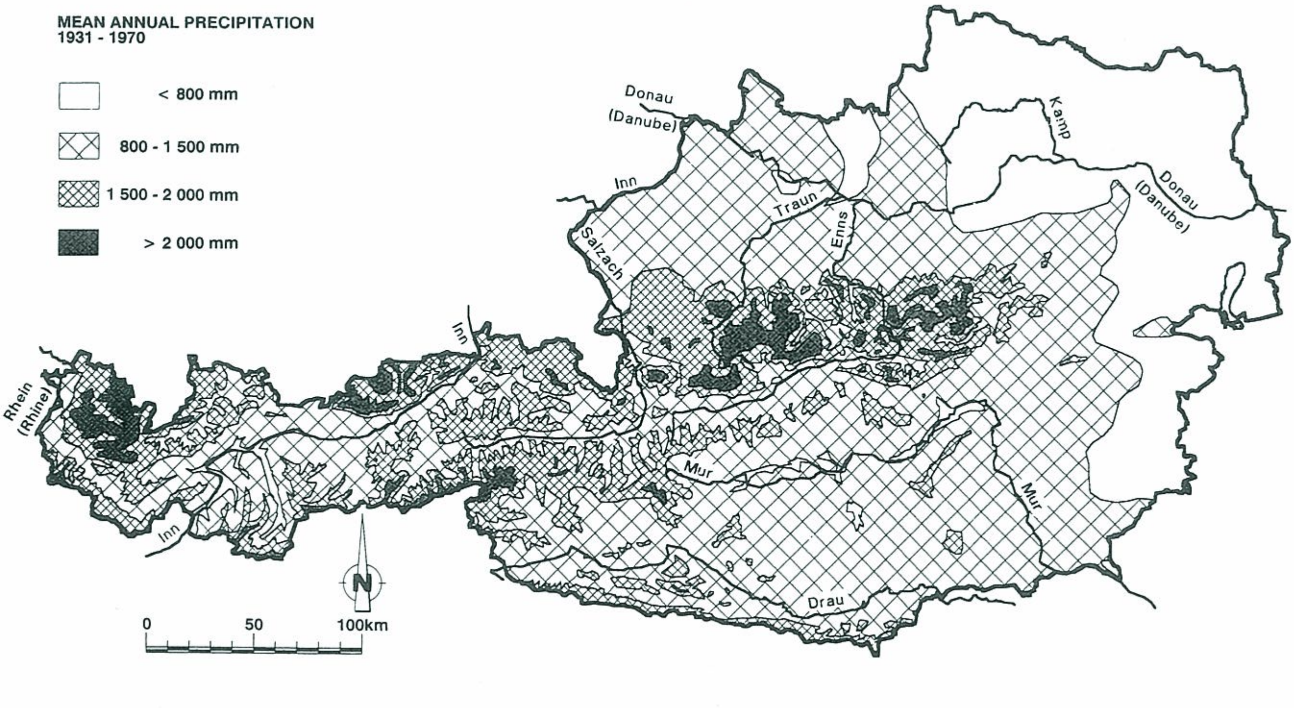
The hydrological basis of hydropower and dam engineering in Austria derives from a long tradition of hydrological and meteorological recording, dating back to the foundation of the Central Office of Meteorology and Geodynamics in 1851 and the Central Hydrography Office in 1893. These two institutions were responsible for the relevant basic research in Austria for many years and for the establishment of a monitoring system covering the whole of the Austro-Hungarian Empire. Gauges were in use even earlier (e.g. near Linz on the Danube starting in 1821), while some of the high-water marks are centuries old. In the last 40–50 years, the government agencies have received the support of the electric utilities, with their excellent monitoring facilities, which serve above all to improve observation of the high mountain regions, to elaborate mathematical runoff models for complete river systems with the help of correspondingly calibrated weirs and continuous records of the relevant operating data. This policy of collaboration between government services and the electricity generating industry has proved beneficial and successful for many years now.

3.2 Precipitation

With a mean annual precipitation of 1 190 mm, Austria is well endowed by Nature overall. However, the figures vary quite considerably for the individual locations. These differences relate not only to altitude but also to distinctions between locations to the north and to the south of the main Alpine chain and between the flanking chains. Superimposed on all this is a general pattern of decreasing precipitation from the west of the country, which takes the brunt of the fronts coming in from the Atlantic, to the east, which belongs more to the continental climatic zone (Fig. 2).

Apart from some local exceptions, mean annual precipitation at altitudes below 600 m (Fig. 1) is between 500 and 1 500 mm, and at 2 000 m above sea level it is between 1 000 and 2 500 mm. The highest annual precipitation figures relate to the weather side of the Alpine foothills to the north and south, whereas inner-Alpine locations can often be very dry. Precipitation is also very high along the main Alpine chain, reaching maxima of 3 000 mm in the

Figure 2
Depth of precipitation in Austria
(Based on Dr. O. Behr "Monographie der Donau", Vienna Technical University 1991)



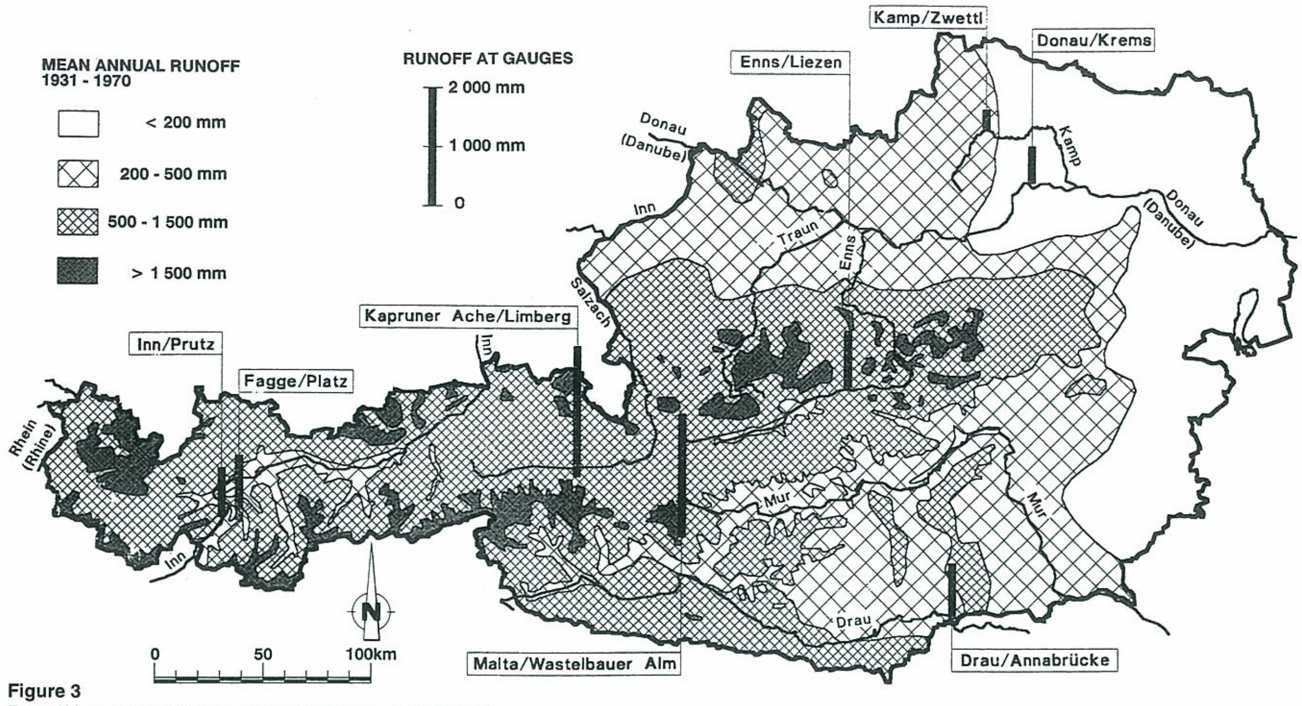
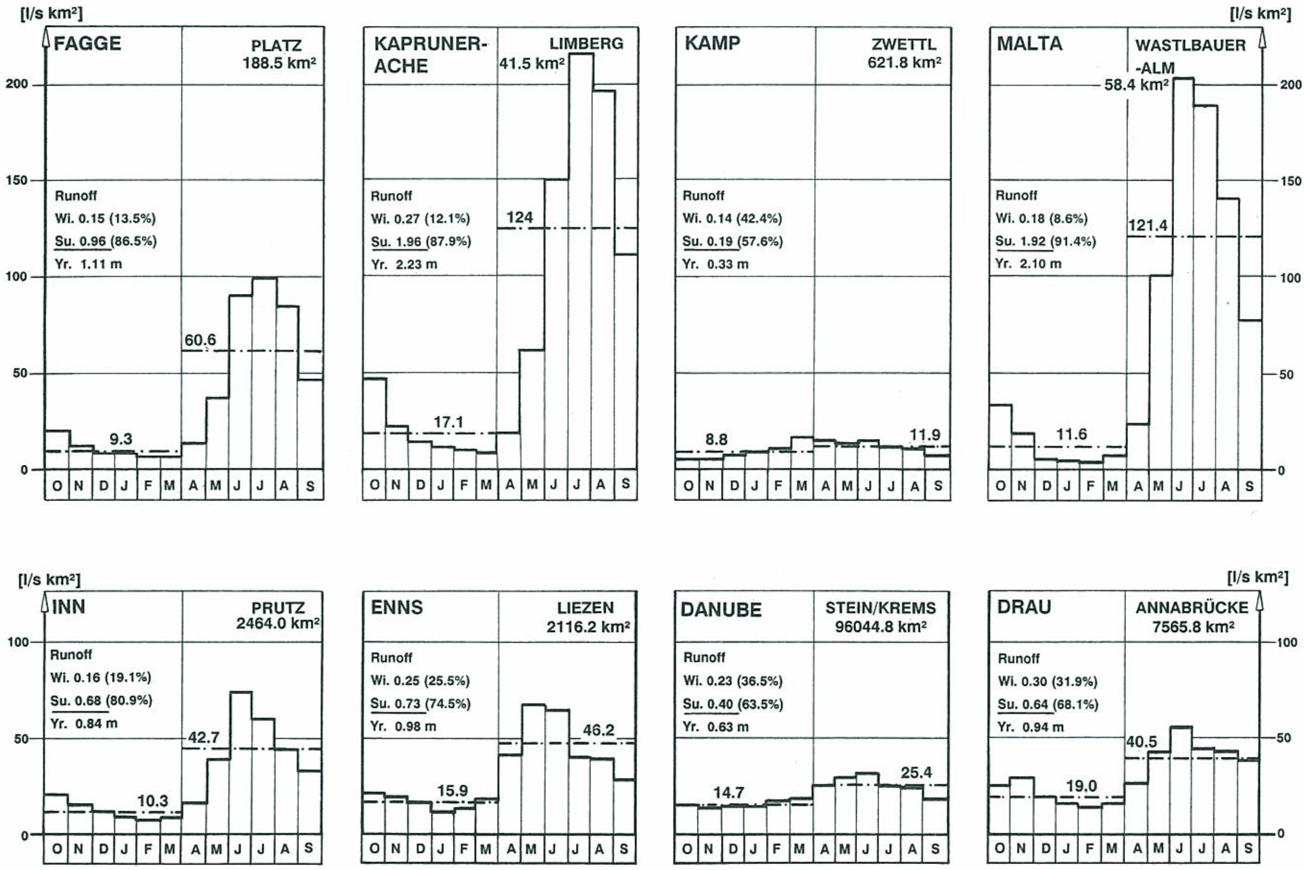


Figure 3
Runoff in Austria with reference to 8 gauges in figure 4
(Based on Dr. O. Behr "Monographie der Donau", Vienna Technical University 1991)

crest regions, while the lowest precipitation figures are found for the Alpine foothills to the east, decreasing to less than 500 mm for the area to the east of Vienna and north of the Danube (490 mm for the town of Retz).

For hydropower generation, the precipitation pattern over the year is obviously important, too, as is the proportion of winter precipitation that falls as snow. As a result of the prevailing north-west winds, precipitation is generally higher in summer than in winter and accounts for 55–65% of the annual total. In addition, a proportion of precipitation that increases with altitude falls as snow and does not drain until the snowmelt in spring. This means that, while total runoff tends to increase with increasing altitude of the catchment, the winter proportion decreases.

Figure 4
Examples of annual pattern of specific runoff in l/s × km², for 8 Austrian rivers and streams
(Location of gauges shown in figure 3)



3.3 Runoff

Average runoff in Austria is 670 mm for an mean runoff coefficient of 0.56, with pronounced scatter in the individual case for both values depending on location and altitude. Figure 3 shows the distribution of runoff by areas and also for a number of selected gauges, and Figure 4 annual regimes of specific runoff for eight of these gauges. As can be seen from the data, specific runoff – like precipitation – is higher for the western Alpine areas than for the eastern lowlands or the region north of the Danube, and naturally increases with altitude.

The mean runoff of the Austrian reach of the *Danube* is clearly lower than that of the Alpine rivers, but the great variety of its catchments makes for a balanced runoff regime that is remarkable for a river of this size. As a result, the Danube power stages built so far have a winter/summer power generation ratio of 43:57. The River Kamp, a northern tributary of the Danube, with its pluvio-nival foothill regime, also has high winter runoffs, including a maximum in March (Fig. 4).

The situation is quite different with regard to the typical Alpine runoff regimes illustrated by the remaining examples in Figure 4 for catchments of various sizes and altitudes. The decisive factor in the construction of high-head seasonal storage power schemes for small catchments is very low winter inflows of only 8 to 14% of total annual flows, followed by the snowmelt in spring, and in the case of catchments with extensive glacier cover, peak runoffs in June, July and August. This basic pattern is also found, albeit in less pronounced form, in the case of the bigger Alpine rivers. An additional increase in runoff in autumn can be seen in the case of the Drau, deriving from the influence of the low pressure zones over the Mediterranean that often form at that time of year.

With the exception of the Danube, with a mean discharge that increases from 1 400 to 2 000 m^3/s in Austria, and the lower reaches of the Inn, whose mean discharge along the border between Austria and Germany increases from 700 to 750 m^3/s, Austria has no major rivers. The Eastern Alps on Austrian territory are drained mainly by a number of rivers with very similar runoff patterns (Lech, the Inn in Tyrol, Salzach, Traun, Enns, Drau, Mur) and mean discharges of between 80 and 300 m^3/s.

Tributary lakes with a regulative influence on runoff are only to be found on the upper reaches of the Traun. On the other hand, significant areas of glacier cover can have a pronounced regulatory effect on runoff regimes in the case of small catchments.

Variations in total annual flow are not very pronounced in Austria. A thirty-year study of the Danube near Vienna, for example, shows a minimum mean annual discharge of 1 402 m^3/s, an annual average of 1 917 m^3/s and a maximum of 2 586 m^3/s, making a ratio of 0.73:1:1.40 between the driest, the average and the wettest year of this period.

3.4 Floods

A very significant factor in the design of barrages and dams is the flood water discharge capacity required for safe operation. The methods employed for design flood calculations are subject to a process of development and adaptation in the individual case. The general rule, however, is to provide the capacity to discharge a 5 000-year flood without operating the bottom outlet or power station conduits. This deterministic approach is based on ample data from long-term monitoring of the main rivers on the one hand, and on the other hand on the fact that spillway design floods for alpine reservoirs with their small catchments are usually relatively small even when based on assumptions which are definitely on the conservative side. Floods with shorter return periods must be controlled with n-1 gates open. On the Danube, the locks may also be used to pass exceptional floods.

On the Danube, barrages must be able to handle floods of over 11 000 m^3/s caused by summer storms, and especially by the snowmelt in combination with spring rains falling on soil that is still more or less frozen. A similar situation is to be found in the case of the Inn, requiring spillway capacities of 6 000–7 400 m^3/s.

In the Alps, dangerous floods are caused primarily by the arrival of cold fronts in summer. The pronounced differences in temperature along the fronts lead to the development of violent lines of thunderstorms with heavy rains, which can be intensified by the barrier effect of the mountains. In mountain areas, this pattern can be exacerbated by snowmelt over a considerable vertical height. In the case of the Drau, there is a further period of heavy rains in autumn, which can also cause major floods.

The barrages on the rivers Drau, Enns, the Inn in Tyrol, Mur, Salzach and Traun have spillway capacities of between 1 000 and 3 500 m^3/s, whereas spillway capacities on the dams built on the tributaries and minor watercourses in the mountains are hardly ever bigger than 500 m^3/s and usually significantly lower (see Table 5).

The impact of reservoirs on runoff is discussed in sections 4 and 6.

4 DAMS AND BARRAGES – FUNCTION, LOCATION, TYPES

4.1 General

The runoff regime of all rivers and streams in Austria is not constant over the year. It is also diametrically opposed to the pattern of demand for electricity, which is usually slightly higher in winter than in summer (53 to 47% in 1990). Moreover, the smaller the catchment and the higher the altitude, the greater is the variation in water discharge (Fig. 4). To compensate this imbalance, seasonal storage systems are therefore an increasingly urgent requirement. In the case of rivers with small catchments in the side valleys of the Central Alps, equalizing power generation in summer and winter presupposes a storage volume of 30–40% of annual runoff, depending on the degree of glacier cover in the catchments. This figure is much lower for the rivers of the main valleys. In the case of the Danube, 13.5% of annual flow would be adequate, although that implies a pondage of 8 000 hm^3, i.e. 4-5 times total active storage of all Austrian reservoirs at the present time of 1 485 hm^3.

And yet the Danube, like all the other major rivers in Austria, has either no live storage at all apart from some volume activated for flood management (Inn, Salzach, Mur), or only very limited pondage for periods of low flow (Drau, Enns). Moreover, human settlements, agricultural activity and communications systems, which have always tended to follow the course of rivers and are concentrated almost exclusively in the main valleys in the mountainous areas, make any significant pondage impossible as it would involve losses in cultivated land and cultural assets as well as considerable relocations which have never been considered acceptable in the past and are nowadays quite out of the question.

Table 1
Austrian reservoirs with more than 80 hm^3 gross capacity

A) River barrages

Name of dam	(river)	Gross capacity hm^3
Aschach	(Danube)	114
Altenwörth	(Danube)	93
Greifenstein	(Danube)	87
Edling	(Drau)	83

B) Other dams

Name of dam	(power stage)	Gross capacity hm^3
Kölnbrein	(Malta main stage)	205
Gepatsch	(Kaunertal)	140
Schlegeis	(Zemm upper stage)	129
Lünersee	(Lünersee)	94
Zillergründl	(Ziller upper stage)	90
Mooser + Drossen	(Kaprun upper stage)	87
Limberg	(Kaprun lower stage)	86

On the other hand, impounding space is available on tributary rivers in the mountainous regions – high above areas of human settlement – on such a scale that the 30–40% of total annual flow required for equalization of winter and summer generation can be achieved or even exceeded to such an extent that the natural drainage area can be considerably increased through the provision of extensive adduction systems. In this context it should be noted that the live storage of the five biggest storage reservoirs in Austria is only between 86 and 200 hm^3, but their catchments are also smaller than those of the major rivers mentioned above by twice the power of ten. In the case of the latter, although the ten biggest impounding areas created for the respective low-head power plants have a total storage volume of 50 to 114 hm^3, they do not provide active storage at all, or only for very limited pondage in exceptional cases (Table 1).

These general conditions have been responsible for the development of the two predominant types of hydropower plants in Austria mentioned in Section 1, with a clear geographical delineation of their locations corresponding to the type of dam involved (Table 2), namely

- low-head river barrage schemes with heads below 30 m, operating as run-of-river stations on the major rivers, and
- high-head seasonal storage schemes on alpine tributary rivers, with dams of various designs and usually medium to very great height, developing heads of between about 250 and 2 000 m in one, two or three stages.

Between these two predominant groups, the medium-head range is only sparsely represented in Austria, even though the corresponding type of power plant is the most common type worldwide and also accounts for the biggest plants, i.e. schemes on large rivers featuring a relatively high dam and a power station at its toe or immediately connected to it, which can be operated in response to demand largely independent of the variations in inflow with only moderate fluctuations in water level in the large reservoir.

Because of the above mentioned impossibility of damming up large rivers beyond the low-head range, this type of plant is not to be found in Austria except for two small plants on equally small rivers (Ottenstein on the Kamp, and Klaus on the Steyr). Instead, the few medium-head schemes that do exist in Austria derive their head mainly from diversions. These are either run-of-river power plants

Table 2
The role of dams in Austria – their purpose and position within the power scheme, 1991

Dams	Planned purpose			Total number of dams	Position of dam within power scheme			
					directly serving a powerplant, its head created			serving diversion to another reservoir
	Hydro-power only	Hydropower and navigation	Hydropower and flood-control		mainly by damming up	by damming up and diversion	mainly by diversion	
River barrages	52 (85.2%)	9 (14.8%)	–	61 (100%)	52 (85.2%)	5 (8.2%)	4 (6.6%)	–
Other dams*	69 (98.6%)	–	1 (1.4%)	70 (100%)	3 (4.3%)	5 (7.1%)	51 (72.9%)	11 (15.7%)
Total	121 (92.4%)	9 (6.9%)	1 (0.7%)	131 (100%)	55 (42.0%)	10 (7.6%)	55 (42.0%)	11 (8.4%)

*) including eight dams which form four reservoirs two by two

Basis: World Register of Dams 1988 plus Feistritzbach Dam (ER ia), completed in 1990. Rotgüldensee Dam built in 1957 replaced by new Rotgüldensee Dam (ER ia) in 1990.

With river barrages			With other dams		
without diversion			with diversion		
Dam (river)	T (MW)	AAE (GWh)	Power stage (dam)	T (MW)	AAE (GWh)
Altenwörth (Danube)	333	1 950	Malta MSt (Kölnbrein)	730	715
Greifenstein (Danube)	293	1 720	Sellrain Silz LSt (Finstertal)	488	459
Aschach (Danube)	286	1 648	Kaunertal (Gepatsch)	390	620
Wallsee (Danube)	210	1 320	Ziller USt (Zillergrund)	360	176
Ybbs-Persenbeug (Danube)	200	1 282	Zemm-Ziller MSt (Eberlaste)	345	613
with diversion			without diversion		
Dam (river)	T (MW)	AAE (GWh)	Power stage (dam)	T (MW)	AAE (GWh)
Schwarzach (Salzach)	120	480	Ottenstein (Kamp)	47	28
Walgau (Jll)	86	356	Klaus (Steyr)	20	73
Prutz-Imst (Inn)	82	537			

Legend: MSt = Main stage T = total generating capacity
USt = Upper stage AAE = average annual energy production
LSt = Lower stage

Table 3
Capacity of Austrian hydropower plants, by type of dam and creation of head

with minor pondage or small seasonal storage schemes with up to 120 MW capacity (Table 3).

4.2 Low-Head River Barrage Schemes

They are by far the commonest type for hydropower development on Austrian rivers, invariably featuring barrages, i.e. gate-structure dams for the passage of large floods, in combination with an adjacent powerhouse across the river, and frequently dykes along the banks of its impounded upstream reach. On the Danube, double locks had to be integrated, too. To an increasing degree such plants have come to form continuous chains on the developable reaches of seven Austrian rivers. They are described in detail by R. Fenz in Chapter H of this book. They account for no less than 47% of the Austrian dams included in the World Register of Dams. River barrages operate under a limited head, frequently between 8 and 16 m, and up to 27 m in reaches of favourable topography. Rated discharge is 1 750–3 150 m^3/s on the Danube, 600–1 000 m^3/s on the lower Inn, and 200–450 m^3/s on the lower reaches of the Drau, Enns, Mur and Salzach. The necessary head is derived solely from impounding, plus lowering of the downstream water level by excavating the bed in many cases.

Table 5
Spillway capacities of Austrian dams and river barrages

RIVER BARRAGES	(RIVER)	m^3/s	OTHER DAMS	(RIVER)	m^3/s
Melk	(Danube)	11 170	Klaus	(Steyr)	758
Altenwörth	(Danube)	11 170	Raggal	(Lutz)	615
Wallsee	(Danube)	11 100	Dobra	(Kamp)	580
Ybbs-Persenbeug	(Danube)	11 100	Lutz	(Lutz)	570
Greifenstein	(Danube)	10 750	Bolgenach	(Weissach)	530

The powerhouse is normally built adjacent to the gated weir. Some recent hydropower plants on the river Drau (and also on the Inn) represent a return to the pier head power station design that originated there (Lavamünd 1942–44), i.e. with gated bays alternating with piers housing one unit each. In spite of the low head, construction of these plants can involve considerable quantities of construction materials, e.g. for plants on the Danube up to 1.3 hm^3 of concrete, 24 000 tons of reinforcing steel and 12 hm^3 of fill material for lateral dykes.

4.3 Other Run-of-River Schemes

In some run-of-river stations the weir across the river only accounts for part of the head. The rest – usually the greater part – is created with the help of a diversion leading to the power station.

Table 4
Austrian hydropower plants, by type of dam and head utilized in a single stage

RIVER BARRAGES WITH DIVERSIONS		OTHER DAMS WITH DIVERSIONS	
Power plant (river)	head m	Power plant (dam)	head m
Strassen-Amlach (Drau)	370	Reisseck (Kleiner Mühldorfer See)	1773
Partenstein (Gr. Mühl)	176	Sellrain-Silz Lower Stage (Längental)	1258
Walgau (Jll)	162	Oschenik (Oschenik Lake)	1186
Prutz-Imst (Inn)	145	Malta Main Stage (Galgenbichl)	1103
Schwarzach (Salzach)	132	Lünersee (Lünersee)	975
Hieflau (Enns)	78	Kaunertal (Gepatsch)	895
		Kaprun Lower Stage (Limberg)	891
		Spullersee (Spullersee)	800
RIVER BARRAGES WITHOUT DIVERSIONS		**OTHER DAMS WITHOUT DIVERSIONS**	
Power plant (river)	head m	Power plant (dam)	head m
Annabrücke (Drau)	25.6	Ottenstein (Ottenstein)	60
Pucking (Traun)	25.5	Klaus (Klaus)	40
Feistritz-Ludmannsdorf (Drau)	23.7		
Grossraming (Enns)	23.5		
Edling (Drau)	21.1		
Ferlach-Maria Rein (Drau)	21.0		

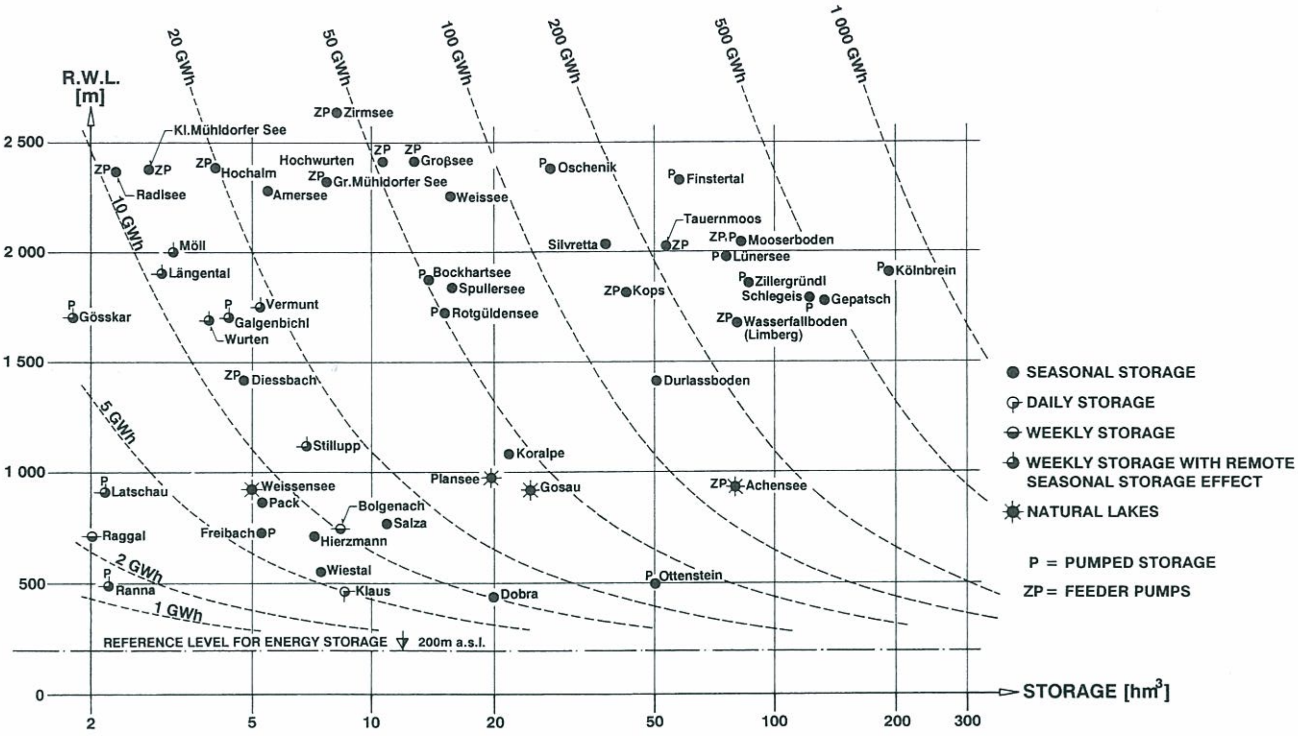


Figure 5 Retention water level, active storage and energy storage of Austrian reservoirs

Concrete-lined trapezoidal diversion canals, which are used only in low- head schemes, are to be found at a number of old installations on the Mur, plus one each on the Drau, Enns and Inn, where there are particularly favourable local conditions for this design. For conservation reasons, however, it has been general policy in Austria to develop major rivers for power generation with the chains of river barrages mentioned in the previous chapter, but without diversion canals.

More adaptable than diversion canals, diversion tunnels are to be found at quite a number of power schemes of over 20 MW capacity, from low-head plants (on the upper reach of the Enns) to high-head plants (on the uppermost reach of the Drau). The biggest installations of this type are to be found in the medium-head range with heads of between 80 and 150 m (Hieflau/Enns, Prutz-Imst/Inn, Schwarzach/Salzach, Walgau/Jll), with up to 20 km long pressure tunnels and pondage capacity at the end of the tunnel or at the intake. Most of them also benefit from their location immediately downstream from a seasonal storage scheme, and in some cases this was in fact a prerequisite for their construction.

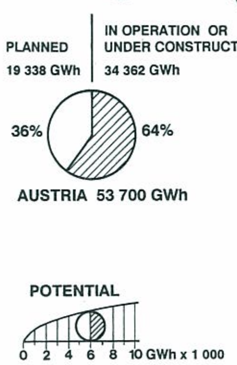
4.4 Seasonal storage schemes

Figure 5 is a graph of Austria's reservoirs showing live storage and elevation. The curves plotted across the graph relate to stored energy calculated for a reference level of 200 m above sea level, which is 60 m too high for the Danube (so that the figure for stored energy is slightly too low), and 140 and 195 m too low for the Drau and the Austrian section of the Rhine respectively (so that stored energy is indicated slightly too high). The following three preferred ranges of altitude may be distinguished.

1. An upper range situated between 2 200 m and 2 500 m a.s.l.: Mainly impounded alpine cirque lakes, used for seasonal storage. Most of them need pumping for seasonal filling because natural catchment areas are too small.

2. A medium range extending from 1 650 m to 2 050 m a.s.l.: Widened valleys of glacial origin at high altitude, which are used for most of the large seasonal storage reservoirs and for some of the associated intermediate storage reservoirs.

Figure 6
Austria's hydropower potential 1989
(Based on Götz-Schiller)



PROVINCE	POTENTIAL	(IN OP. OR U. C.)
TYROL	11 412	(5 252) GWh
LOWER AUSTRIA	10 319	(7 255) GWh
UPPER AUSTRIA	10 110	(9 024) GWh
CARINTHIA	6 986	(4 809) GWh
SALZBURG	5 174	(3 013) GWh
STYRIA	4 975	(2 543) GWh
VORARLBERG	3 754	(2 463) GWh
VIENNA	967	(-) GWh
BURGENLAND	3	(3) GWh
TOTAL	53 700	(34 362) GWh

3. A lower range between 400 m and 1 100 m a.s.l.: Reservoir sites mainly situated in the Pre-Alps, and natural lakes utilized by drawdown. These reservoirs include a small number of developed gorges on streams such as the Klaus reservoir on the Steyr, and the Ottenstein and Dobra reservoirs on the Kamp. (Klaus and Ottenstein, by the way, are the only real dam power stations in Austria, with the power station directly downstream of an arch dam, whereas all the other reservoirs discharge to diversion – type power stations with long power conduits.)

The reservoirs at the two higher ranges of altitude, all situated above areas of human habitation, where not a single hamlet had to be dislocated, account for the greater part of total storage capacity in Austria. They are all situated in Western Austria, with the Tauern Highway from Salzburg to Villach forming an approximate boundary (see Fig. 9). The tributary valleys in the partly glaciated Central Alps obviously afford the best conditions for seasonal storage. Among the eleven power schemes and groups of schemes of more than 80 MW capacity shown in Fig.9, only the Lünersee reservoir (part of the Upper Jll-Lünersee group of schemes) and the entire Achensee scheme are not located in the Central Alps, but in the adjoining Northern Calcareous Alps.

The 80 MW limit encompasses all major seasonal storage schemes, which account for about 95% of total plant capacity and 97% of the total live storage volume of all seasonal storage schemes in Austria. (Plant capacity of the daily and weekly storage schemes, not otherwise dealt with in this chapter, amounts to less than 10% of the seasonal storage schemes).

A detailed description of all these eleven seasonal storage schemes was presented, complete with illustrations and comprehensive tables, by H. Lauffer in the brochure "Hydro Power Schemes and Large Dams in Austria", a special issue of the series "Die Talsperren Österreichs" published on the occasion of the 15th ICOLD Congress in Lausanne in 1985. Table 6 compiles the main data of those seasonal storage schemes with more than 300 MW capacity.

They demonstrate the great variety of possible project configurations, especially of multi-stage schemes with several reservoirs, extensive adduction systems and additional pumped-storage facilities, in the attempt to achieve optimum solutions in terms of natural site conditions, power system requirements and the environment. But even the seven largest and most extensive groups, with their numerous reservoirs, compensating basins and power stations, provide only 500 to 1 000 GWh annual generation (without short-term pumped storage, but also without deduction of energy requirements for seasonal pumped storage). This is due to the ramified relief of the Alps, with very small catchment areas at high altitudes, which makes it very difficult and expensive to combine large areas for hydropower development. In view of the high specific cost of these schemes per kWh produced, in particular deriving from the high dams and extensive systems for water conveyance involved, such schemes must bring their generation to the power market solely as valuable peak and regulating energy. Hence, installed capacity is usually rather high, but is fully utilized only for 2 000 to 2 500 theoretical hours in older plants and for less than 1 000 hours in more recent developments. Installed capacities of the seven largest schemes range between 323 and 1 114 MW. Frequently combined with pumping facilities, this results in a total power range for system regulation of up to 1 670 MW (Table 6).

The largest reservoirs are those of Kölnbrein with 205 hm^3, Gepatsch with 140, and Schlegeis with 129 hm^3 (see Table 1). The highest heads developed in single stages are those of Reisseck, with 1 773 m (world record since 1957), Silz with 1 259 m, Innerfragant-Oschenik with 1 186 m, and Malta-Rottau with 1 103 m (see Table 4).

Above all, Austria's seasonal storage schemes have stimulated the country's greatest achievements in dam construction. Discounting river barrages from the 131 large dams that were listed in the 1988 World Register or have been built since, there are 70 dams proper. Of this total, 16 are over 60 m high, 10 over 100 m, and 4 between 150 and 200 m high. The highest are the Kölnbrein and Zillergründl arch dams (200 and 186 m respectively), and the Gepatsch and Finstertal embankment dams (153 and 150 m).

There are 46 concrete dams (discussed in chapter F by R. Widmann) and 25 embankment dams (discussed in chapter G by W. Schober and H. Schwab).

Amongst the concrete dams, 26 are gravity dams, some curved in plan and therefore, strictly speaking, arched gravity dams. Most of them were built before 1950, whereas most of the 19 arch dams were built later. There are no buttress or multiple arch dams in Austria. Nine of the 25 embankment dams are of the rockfill type, amongst them the two highest. Many embankment dams use asphaltic concrete as a sealing element, four as a core wall and ten as an upstream membrane. The 150 m high Finstertal Dam is the highest in the world with an asphaltic core (96 m high) and the 116 m high Oscheniksee Dam the highest with an upstream membrane (84 m high). Five dams have internal concrete core walls, but only one an upstream concrete face.

5 THE ROLE AND RANK OF HYDROPOWER IN AUSTRIA

5.1 Hydropower resources

Austria's total available hydro resources for both existing plant and future projects represent a potential generating capacity, on the basis of current technology and economics, of about 54 000 GWh. Almost 64% of this total, i.e. more than 34 000 GWh, has already been developed, with dams and barrages playing a key role as explained in the preceding chapters. How much of the remaining 36% will fulfill today's requirements in view of concerns about environmental impact and doubts about public acceptance is an open question, and the criteria involved are still highly controversial and unclear.

With 0.64 GWh/year per km^2, Austria has a lower average specific potential than neighbouring Switzerland with 0.86, but this average value is matched in the Tyrol (0.84) and even surpassed in Vorarlberg (1.44). As in Switzerland, hydropower resources are very unevenly distributed, mainly due to topography. Figs.6 and 7 show that they are mostly located in the western and southern areas of the Austrian Alps (with Tyrol leading with more than 11 000 GWh and the highest share of storage energy, followed by Carinthia and Salzburg), and also along the Danube in Upper and Lower Austria together with Vienna in the northeast. The Danube provides 54% of available and 62% of developed run-of-river potential in Austria.

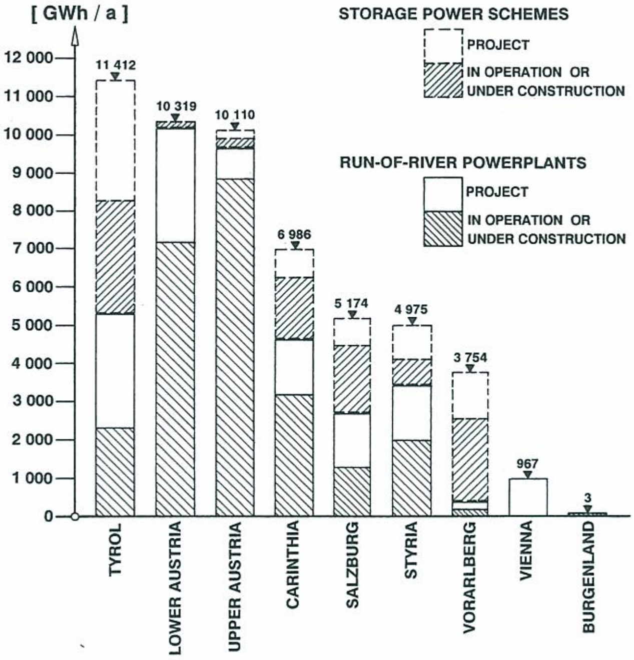


Figure 7 Austria's hydropower potential

The five northeastern provinces (Table 7) account for 72% of developed and 49 of future run-of-river-energy in Austria, whereas 89% of future energy from storage plant are concentrated in the four southwestern provinces. With regard to all existing powerplants, the energy content of a presently available total active seasonal storage volume in Austria of 1 485 hm^3 is the equivalent of 3 376 GWh, of which no less than 99% is located in the four southwestern provinces (Table 7). A comparison with neighbouring Switzerland, the classic country of Alpine hydropower, shows that Switzerland, with an area slightly half that of Austria, presently generates almost the same total of GWh from hydropower schemes with an insignificantly higher total installed capacity and in about the same winter/summer relation of 43/57. However, total storage volume and its energy content is more than twice as high. This is partly due to the topography of the Western Alps, which on average provide for larger reservoirs and higher total heads for hydropower development than the Eastern Alps, and partly to the fact that the developed share of total hydropower potential is already about 93% in Switzerland compared with only 64% in Austria. The identical winter/summer relation of hydropower generation in the two countries, which is astonishing given such a difference in storage capacity, is due to the large contribution made by the Danube, which has exactly the same winter/summer relation. Nevertheless, there is a definite demand for more storage capacity to cover the low-flow months from January to March.

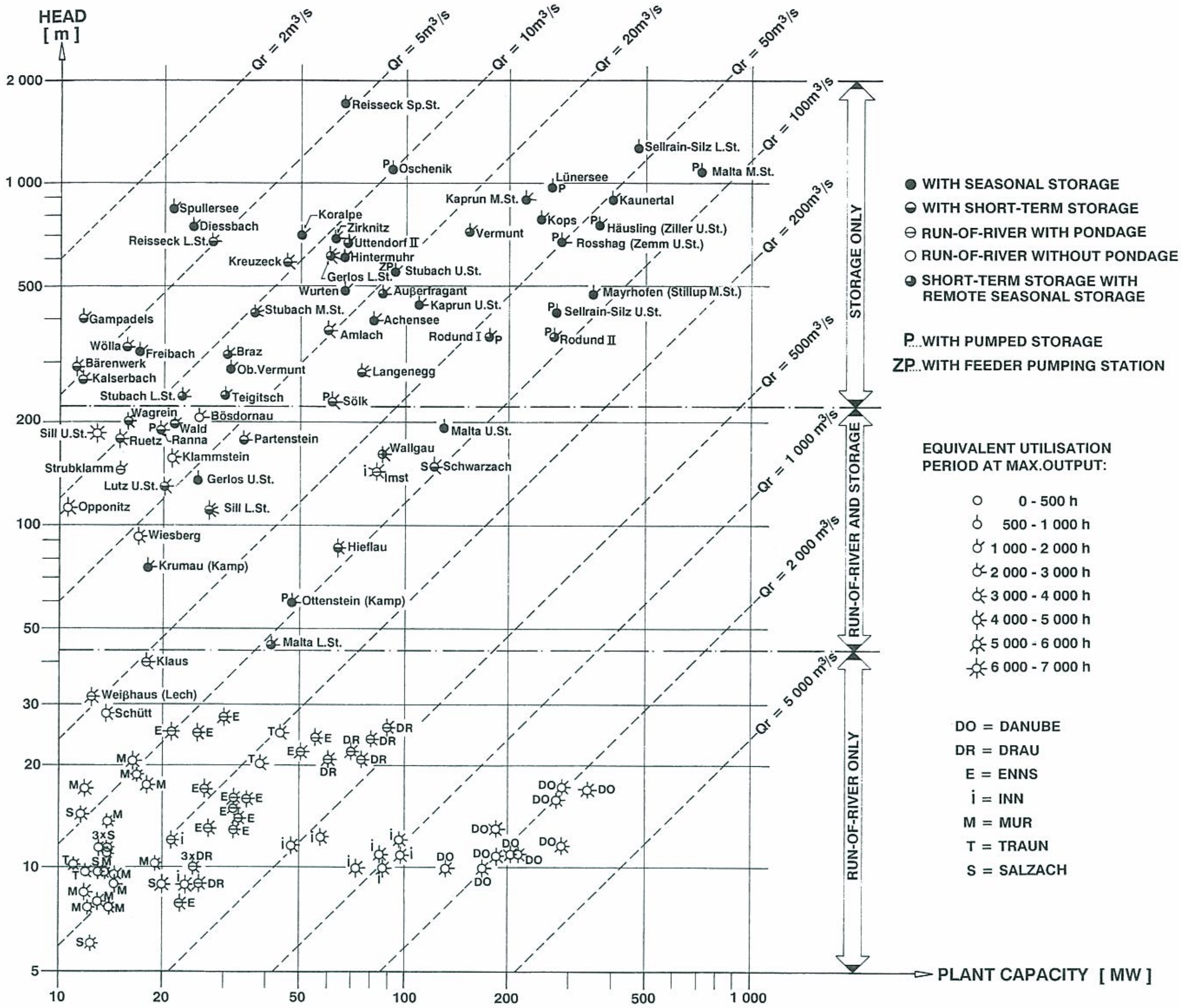
5.2 History and present situation

Hydroelectricity development in Austria dates back to the year 1882. By the beginning of the First Republic in 1918, annual generation was 1 765 GWh (895 GWh hydro, 870 GWh thermal). In the 1920s, the first major hydroelectric projects were initiated in the Tyrol and Vorarlberg to develop the obvious wealth of hydropower resources in the

Table 7
Geographical distribution by provinces of population, stored energy in seasonal storage reservoirs, electricity generation and consumption in 1988
A) without Austrian federal railways
B) with Austrian federal railways

A	Province	Population 10^3 (%)	Stored energy GWh (%)	Total electricity generation: Winter GWh (%)	Summer GWh (%)	Year GWh (%)	Electricity consumption GWh (%)
1	Upper Austria	1 337 (17.6)	16 (0.5)	3 900 (28.3)	5 124 (25.6)	9 024 (26.7)	10 059 (22.6)
2	Lower Austria	1 112 (14.7)	19 (0.6)	2 550 (20.7)	4 405 (22.0)	7 255 (21.5)	5 185 (11.7)
3	Vienna	1 774 (23.3)	– (–)	–	–	–	8 474 (19.1)
4	Burgenland	267 (3.5)	– (–)	1 (0.0)	2 (0.0)	2 (0.0)	809 (1.8)
5	Styria	1 167 (15.4)	12 (0.4)	1 000 (7.3)	1 543 (7.7)	2 543 (7.5)	6 696 (15.1)
1–5	Northeastern provinces % of total generation	5 657 (74.5)	47 (1.5)	7 751 (56.3) 41.2	11 074 (55.4) 58.8	18 825 (55.8) 100	31 223 (70.3)
6	Carinthia	543 (7.1)	944 (29.9)	1 500 (10.9)	3.242 (16.2)	4 742 (14.0)	3 627 (8.2)
7	Salzburg	461 (6.1)	441 (14.0)	1 360 (9.9)	1 422 (7.1)	2 782 (8.2)	3 045 (6.9)
8	Tyrol	612 (8.1)	1 256 (39.8)	2 300 (16.7)	2 765 (13.8)	5 065 (15.0)	4 473 (10.1)
9	Vorarlberg	317 (4.2)	468 (14.8)	860 (6.2)	1 489 (7.4)	2 349 (7.0)	2 020 (4.5)
6–9	Southwestern provinces % of total generation	1 933 (25.5)	3 109 (98.5)	6 020 (43.7) 40.3	8.918 (44.6) 59.7	14 938 (44.2) 100	13 165 (29.7)
1–9	AUSTRIA (% of total generation)	7 590 (100)	3 156 (100)	13 771 (100) 40.8	19 992 (100) 59.2	33 763 (100) 100	44 388 (100)
B		Population 10^3	Stored energy GWh	Total electricity consumption: Winter GWh (% of gen.)	Summer GWh (% of gen.)	Year GWh (% of gen.)	
	Austria % of total consumption	7 590	3 376	24 726 (179.5) 54.7	20 515 (107.6) 45.3	45 241 (134.0) 100	

Figure 8
Heads and capacities of Austria's hydro plants with indication of mode of operation and equivalent utilisation period at maximum output capacity



mountainous regions (Achensee powerplant 80 MW, and Vermunt 158 MW) in response to the distance between these most westerly provinces and the location of Austria's fossil fuel resources. It was not until much later, however, that a national grid was set up to provide access to the consumer markets of eastern Austria (a 110 kV transmission line in 1948, a 220 kV line in 1964, and a 380 kV line still under construction or partly in service). This in turn then led to a policy of co-operation with the southern and western German grids (exports and barter agreements with peak energy for base energy). During the Great Depression at the beginning of the 1930s, hydropower development in the First Republic ceased, and it was not until World War II and the difficult years of the reconstruction period that the hydroelectric industry was able to settle down to a healthy three and a half decades of uninterrupted and vigorous growth in the Second Republic. Growth rates at this time, however, were lower than in Switzerland, and work focused less on storage power schemes because of the higher capital costs in Austria and because of competition in meeting winter supply requirements from thermal power stations partly based on domestic fuels, but mostly on imported coal, natural gas and oil.

Since 1920, hydropower has consistently been used to meet 60–80% of Austria's electricity generation requirements. The present figure, with a total annual generation of 33 840 GWh based on 10 762 MW of installed capacity, is 71%, which makes Austria very much a hydropower country, ranking fourth in Europe in this respect, after Norway (99.6%), Iceland (94.2%) and Albania (86%). This electric output represents about one eighth of Austria's total energy consumption and 40% of total domestic energy production.

Fig. 10, however, shows that the share of hydropower in total electricity supply varies very much from summer to winter. Whereas summer generation has exceeded demand by almost 4% on average over the last five years, winter generation has not been able to contribute more than 54% on average in the same period. The winter deficit is mainly met by thermal generation and, to a much lesser degree, by power imports of winter base-load, which averaged 1 500 GWh, in contrast to average exports of 4 260 GWh of peak energy and summer surplus in the same years (Fig. 11).

Figure 9
Schematic map of western Austria showing locations of seasonal-storage schemes of more than 80 MW capacity and most of the other hydro plants of more than 10 MW capacity

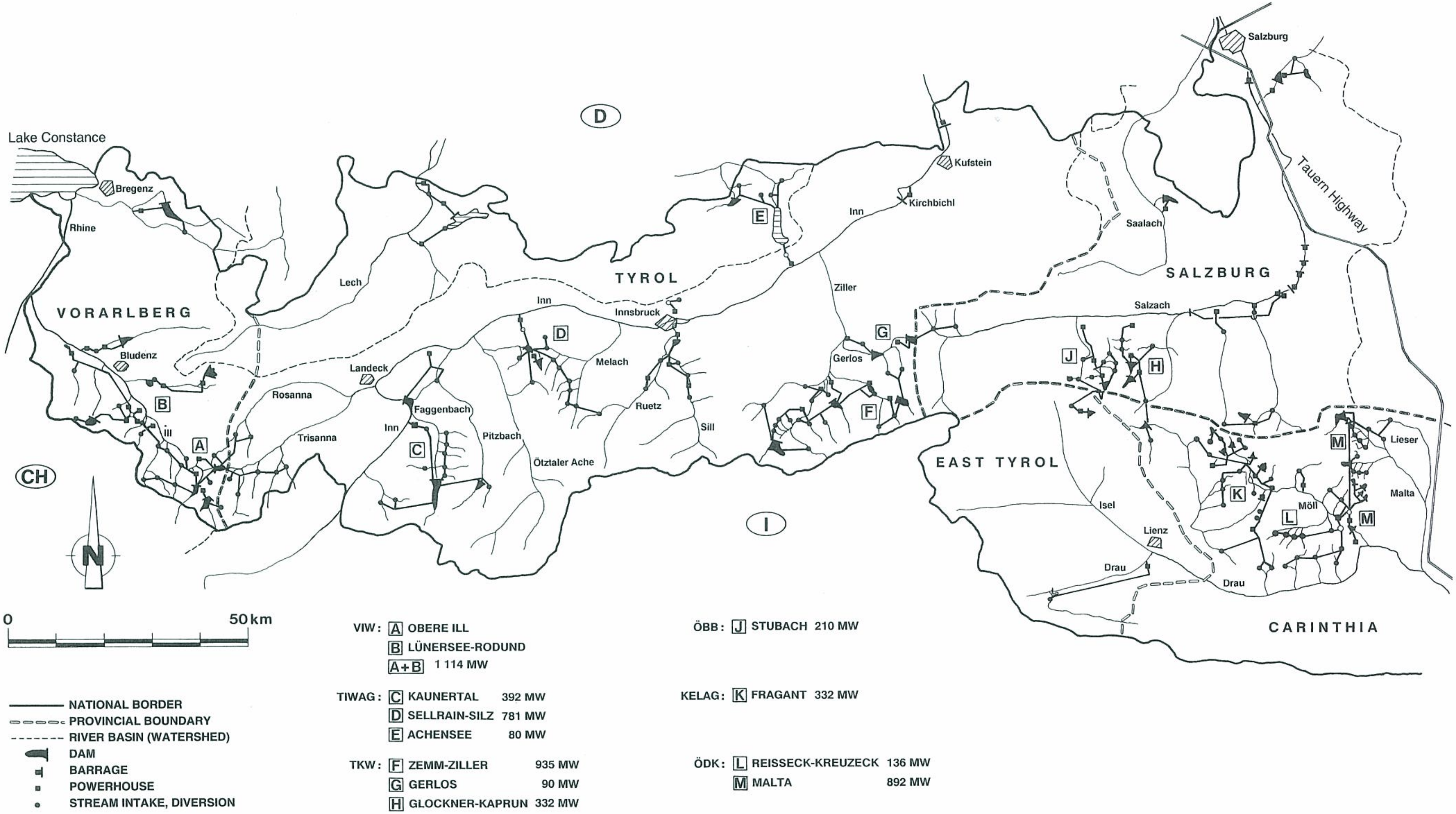
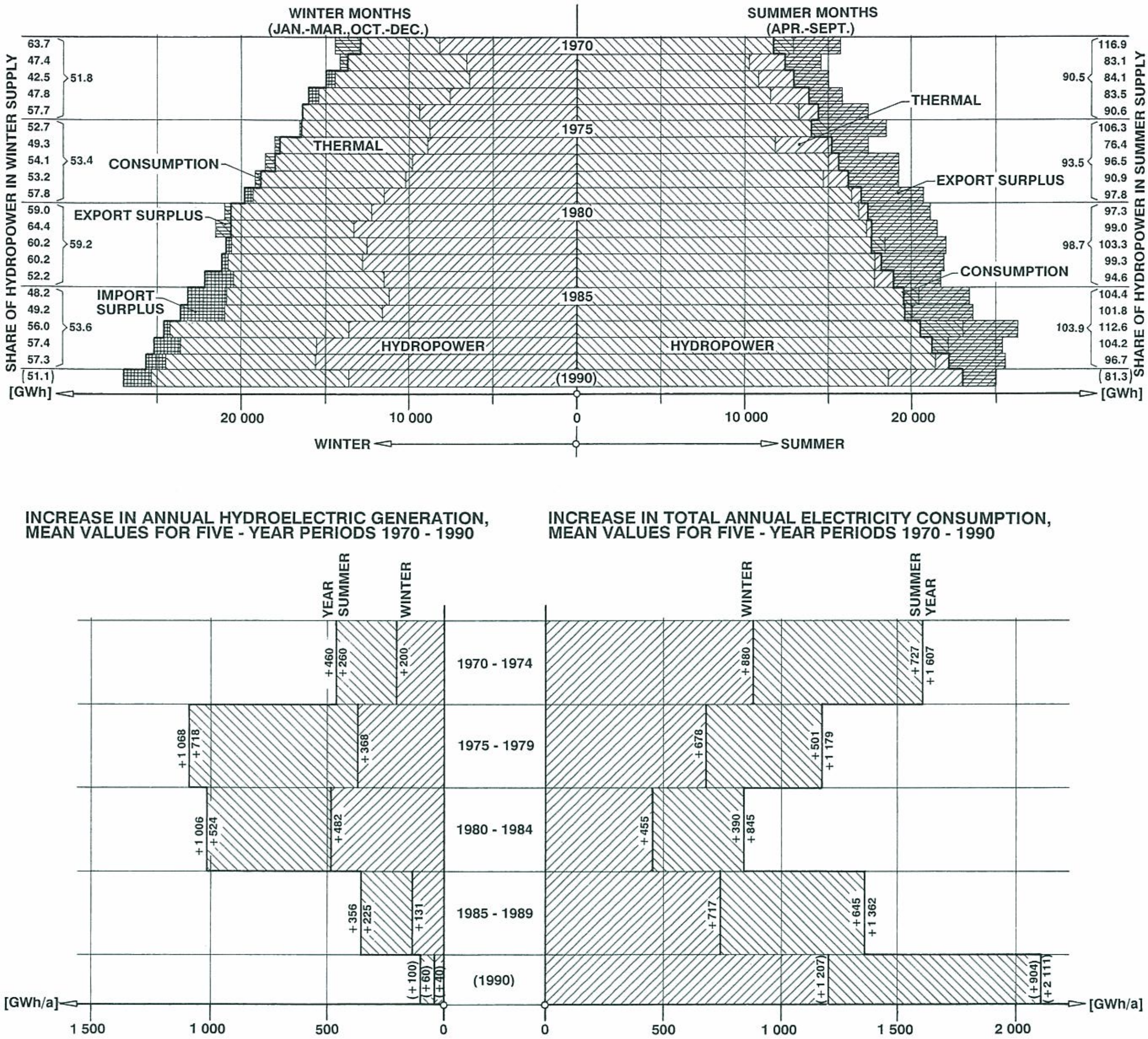


Table 6
Hydro power storage schemes of more than 300 MW capacity

SCHEME / POWER STAGE		NUMBER OF MAIN RESERVOIRS AND POWER STAGES: DAMS (SADDLE DAMS) AND RESERVOIRS					INTAKES AND SECONDARY RESERVOIRS FOR DIVERSION: CATCHMENT AREAS			SCHEME : OWNER: POWER STAGE (STATION)			ENERGY OUTPUT WITH SEASONAL PUMPING, WITHOUT SHORT TERM PUMPED STORAGE (WITH SHORT TERM PUMPED STORAGE)				
SYMBOL a)	YEAR OF COMPLETION	NAME	TYPE	HEIGHT [m]	RETENT. WATER-LEVEL [m a.s.l.]	ACTIVE STORAGE [hm³]	DIRECT [km²]	UPSTREAM POWER ST. [km²] (SYMB.)	DIVERSIONS (INTAKES/ PUMPSTATIONS) [km²] (NUMBER)	NAME	MAX. GROSS HEAD [m]	DISCHARGE TO	T / P T=TURBINING MODE P=PUMPING MODE [MW]	AAE AVERAGE ANNUAL ENERGY OUTPUT [GWh]	WINTER SHARE OF AAE [%]	AAE/T [h]	PUMP-ENERGY [GWh]
A 1	1950	SILVRETTA c) (BIEL)	PG (TE)	80 (34)	2 030	38.60	35	–	10 (1/–)	OBERVERMUNT	291	A 2	29 /–	45 (do)	45 (–)	1 551	–
A 2	1930	VERMUNT	PG	53	1 743	5.30	22	45 (A1)	50 (5/–)	VERMUNT (PARTENEN)	714	A 4 + B 1	156 /–	260 (do)	37 (–)	1 666	–
A 3	1969	KOPS c)	VA (PG)	122 (43)	1 809	44.00	7	–	163 (8/1)	KOPS (PARTENEN)	780	A 4 + B 1	247 /–	392 (do)	34 (–)	1 587	–
A 4	1969	RIFA	COMP.RES.		1 000	0.70	–	277 (A2+A3)	–	RIFA (PARTENEN)	34	A 4 + B 1	7 /8	– (8)	– (50)	– (890)	– (13)
B 1	1950	PARTENEN	COMP.RES.		1 025	0.10	22	279 (A2+A3)	155 (7/–)	LATSCHAU	28	B 3 + B 4	9 /–	22 (do)	34 (–)	2 440	–
B 2	1958	LÜNERSEE c)	PG	30	1 970	78.30	9	–	3 (1/–)	LÜNERSEE (LATSCHAU)	974	B 3 + B 4	232 /224	170 (371)	100 (78)	782 (1 656)	220 (541)
B 3	1952	LATSCHAU	TE fa	50	922	2.20	–	466 (B1+B2)	–	RODUND I	353	COMP.RES.	198 /41	274 (332)	41 (52)	1 383 (1 677)	– (58)
B 4	1976									RODUND II	354	RODUND	276 /260	127 (486)	41 (52)	1 460 (1 760)	– (359)
A+B		**5 MAIN RESERVOIRS, 8 POWER STAGES**					**22 INTAKES, 1 PUMPSTATION**			**OBERE ILL – LÜNERSEE: VIW**			**1 154/533**	**1 290 (1 916)**	**46 (53)**	**1 118 (1 660)**	**229 (980)**
F 1	1971	SCHLEGEIS c)	VA	131	1 782	127.70	58	–	66 (11/–)	ZEMM UPPER ST. (ROSSHAG)	674	F 2	230 /240	284 (534)	76 (59)	1 230 (2 320)	– (362)
F 2	1968	STILLUP b)	TE ia	28	1 120	6.90	61	191 (F1+F3)	138 (6/–)	ZEMM-ZILLER MAIN ST. (MAYRHOFEN)	476	TUX STR.	345 /–	613 (do)	51 (–)	1 760 (–)	–
F 3	1986	ZILLERGRÜNDL c)	VA	186	1 850	86.70	30	–	38 (4/1)	ZILLER UPPER ST. (HÄUSLING)	744	F 2	360 /360	176 (526)	83 (56)	490 (1 460)	7 (500)
F		**3 MAIN RESERVOIRS, 3 POWER STAGES**					**11 INTAKES, 1 PUMPSTATION**			**ZEMM – ZILLER: TKW**			**935/600**	**1 073 (1 673)**	**61 (55)**	**1 150 (1 790)**	**7 (862)**
M 1	1978	KÖLNBREIN	VA	200	1 902	190.00	51	–	–	MALTA UPPER ST. (GALGENBICHL)	198	M 2A	120 /116	76 (do)	94 (–)	630 (–)	65 (do)
M 2A	1974	GALGENBICHL	TE fa	50	1 704	4.40	7	51 (M1)	71 (15/–)	MALTA MAIN ST. (ROTTAU)	1 102	M 3	730 /290	715 (1 155)	83 (?)	980 (1 580)	138 (740)
M 2B	1975	GÖSSKAR	TE fa	55		1.80											
M 3	1978	ROTTAU	COMP.RES.		598	0.50	1 081	129 (M2)	–	MALTA LOWER ST. (MÖLLBRÜCKE)	45	DRAU-RIVER	42 /–	114 (do)	45 (–)	2 780 (–)	–
M		**3 MAIN RESERVOIRS, 3 POWER STAGES**					**15 INTAKES**			**MALTA: ÖDK**			**892/406**	**905 (do)**	**79 (?)**	**1 015 (1 510)**	**193 (795)**
D 1	1980	FINSTERTAL b)	ER ia	150	2 322	60.00	6	–	–	SELLRAIN-SILZ UPPER ST. (KÜHTAI)	440	D 2	290 /250	57 (261)	82 (74)	197 (900)	65 (331)
D 2	1979	LÄNGENTAL	TE fa	45	1 901	3.00	17	6 (D1)	116 (13/1)	SELLRAIN-SILZ LOWER ST. (SILZ)	1 258	INN-RIVER	491 /–	458 (do)	42 (–)	933 (–)	–
D		**2 MAIN RESERVOIRS, 2 POWER STAGES**					**13 INTAKES, 1 PUMPSTATION**			**SELLRAIN – SILZ: TIWAG**			**781/250**	**515 (719)**	**46 (54)**	**660 (920)**	**65 (331)**
C 1	1965	GEPATSCH b)	ER ie	153	1 767	138.30	107	–	172 (12/–)	PRUTZ	895	INN-RIVER	392 /–	620 (–)	59 (–)	1 582 (–)	–
C		**1 MAIN RESERVOIR, 1 POWER STAGE**					**11 INTAKES**			**KAUNERTAL: TIWAG**			**392 /–**	**620 (–)**	**59 (–)**	**1 582 (–)**	**–**
H 1	1952	MÖLL (MARGARITZE)	VA (PG)	93 (39)	2 000	3.20	44	–	28 (5/–)	MÖLL PUMP ST.	56	H 2	– /13	–	–	–	15 (do)
H 2	1955	MOOSER c) (DROSSEN c))	PG (VA)	107 (112)	2 036	85.50	22	72 (H1)	5 (5/–)	KAPRUN UPPER ST.	446	H 3	112 /130	152 (252)	57 (53)	1 360 (2 250)	– (156)
H 3	1951	LIMBERG c)	VA	120	1 672	82.80	15	99 (H2)	29 (11/1)	KAPRUN LOWER ST.	891	SALZACH-RIV.	220 /–	486 (do)	80 (–)	2 209 (–)	3 (do)
H		**3 MAIN RESERVOIRS, 2 POWER STAGES**					**21 INTAKES, 1 PUMPSTATION**			**GLOCKNER – KAPRUN: TKW**			**332/130**	**638 (738)**	**74 (75)**	**1 922 (2 223)**	**18 (174)**
K 1	1984	ZIRMSEE	ER fa	51	2 530	8.70	3	–	2 (2/–)	REMOTE RESERVOIR TO K2		K 2					
K 2A	1980	HOCHWURTEN	TE fa	55	2 417	12.70	7	6 (K1)	11 (5/4)	ZIRKNITZ	689	K 3B	32 /–	55 (–)	91 (–)	1 719 (–)	10 (–)
K 2B	1980	GROSSEE	ER fa	57		14.00											
K 3A	1980	FELDSEE **)	TE fa	17	2 217	1.60	2										
K 3B	1971	WURTEN	TE fa	51	1 695	2.70	8	26 (K2)	54 (7/–)	WURTEN (INNERFRAGANT)	490	K 6	66 /–	93 (–)	57 (–)	1 409 (–)	–
K 4	1979	OSCHENIK b)	ER fa	116	2 391	33.00	2	–	–	OSCHENIK (INNERFRAGANT)	1 186	K 6	108 /100*)	82 (–)	100 (–)	759 (–)	72 (–)
K 5	1968	HASELSTEIN	COMP.RES.		1 470	0.04	–	–	29 (1/–)	HASELSTEIN (INNERFRAGANT)	276	K 6	4 /5	8 (–)	33 (–)	2 000 (–)	
K 6	1968	INNERFRAGANT	COMP.RES.		1 201	0.20	–	156 (K3,4,5,7)	28 (6/–)	AUSSERFRAGANT	488	MÖLL-RIVER	96 /–	234 (–)	49 (–)	2 438 (–)	–
K 7	1984	WÖLLA	COMP.RES.		1 542	0.10	–	–	46 (5/–)	WÖLLA UPPER ST.	326	K 6	17 /–	40 (–)	25 (–)	2 353 (–)	–
K		**6 MAIN RESERVOIRS, 6 POWER STAGES**					**26 INTAKES, 4 PUMPSTATIONS**			**FRAGANT: KELAG**			**323/105**	**512 (–)**	**61 (–)**	**1 585 (–)**	**82 (–)**

a) SYMBOLS ACCORDING TO THE MAP OF WESTERN AUSTRIA (Fig. 9) IN ALPHABETICAL ORDER FROM WEST TO EAST b) DETAILS SEE CHAPTER G c) DETAILS SEE CHAPTER F
**) REMOTE RESERVOIR TO K 3B *) PUMPING FROM K3 AND K5

Figure 10
Austria's electricity supply, winter/summer balance 1970 - 1990



5.3 Future prospects

The increase in electricity consumption over the last 20 years has been rather irregular from one year to the next, reflecting variations in economic conditions as well as climatic fluctuations (Fig.10). Five-year-averages of annual increase range from 850 to 1 600 GWh, with 1 250 GWh as the long-term mean. In the last few years, however, the pace of hydroelectric development has fallen increasingly short of this rate of growth in demand. Whereas hydropower plant totalling 4 200 GWh annual generating capacity was under construction in 1982 (enough to meet the increase in demand, given an average construction period of four years), the corresponding figure in 1986 was 876 GWh and is currently (summer 1991) little more than 350 GWh. There is no prospect of achieving a substantial improvement on this figure in the next few years to provide for the steady rise in demand predicted for the next decade.

Therefore increased imports of electricity as well as coal, natural gas and fuel oil for thermal generation will be inevitable. Energy savings, the much quoted "Negawatt power plant" will not solve the problem of future electricity supply; they can only contribute towards a solution. As in its neighbouring countries, considerable economic growth has been achieved in Austria with the total annual energy input remaining almost constant since 1973 – in itself proof enough of energy savings achieved. Demand for electricity, however, accounting for 18% of Austria's total energy supply, has grown at a similar or even slightly higher rate than GDP, because use of electricity helps save total energy.

The regrettable and illogical present situation, with hydropower development greatly reduced while generating requirements continue to increase, derives mainly from the sudden rejection of further hydropower projects for the Danube, where such good progress had

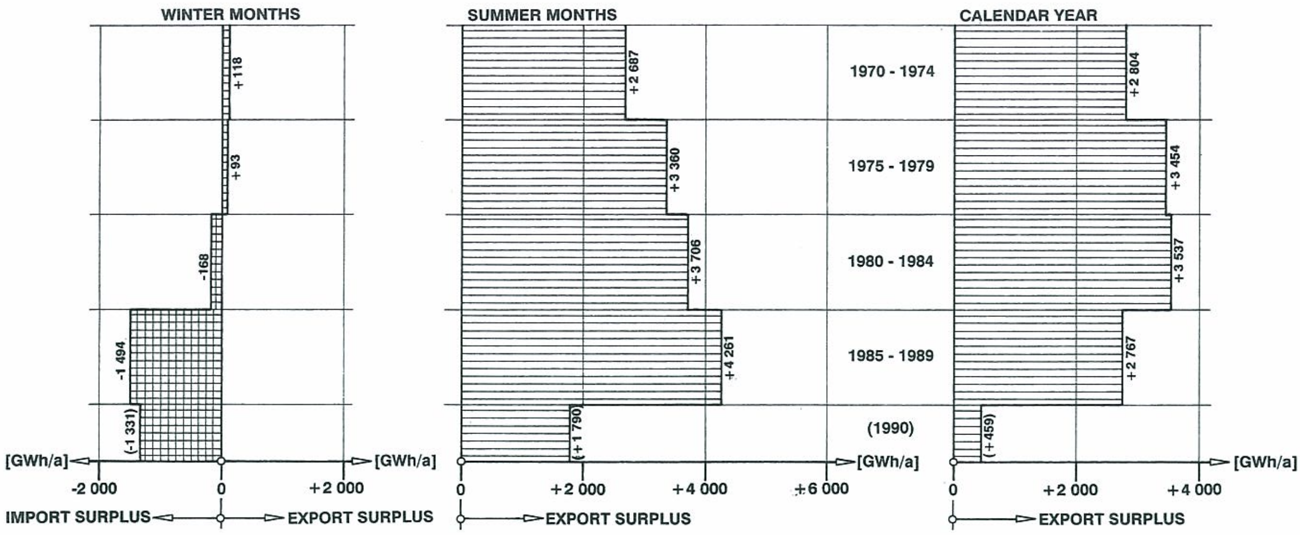


Figure 11 Balance of Austria's electricity imports and exports; mean values for five-year periods 1970 - 1990

been made in the past. The movement away from hydropower was clearly signalled at the end of 1984, when conservationist groups succeeded in stopping tree-felling work which had begun in preparation for the construction of Hainburg power station (360 MW, 2 075 GWh), for which the approvals had already been obtained. In the following years a number of other projects had to be postponed or abandoned, too, including the large 900 MW Dorfertal-Matrei storage scheme with the well over 200 m high Dabaklamm dam.

None of the very few projects presently under construction or nearing completion is of more than 70 MW capacity. The largest is Oberaudorf-Ebbs, a binational barrage-type run-of-river station on the Inn river (60 MW, 262 GWh shared with Bavaria, planned for commissioning in 1992). Alberschwende (28 MW, 102 GWh, also to be commissioned in 1992) is of the medium-head diversion type with a basin for daily compensation. Nearing completion is the Hintermuhr scheme (65 MW, 71 GWh) on a tributary to the Mur river, which replaces an older, much smaller scheme featuring a dam with Austria's first asphaltic concrete core built in 1957. The 45 m high new Rotgüldensee rockfill dam incorporates the old one in its upstream shoulder and has a vertical asphaltic core membrane. Impounding is now in progress at another dam of this type, the 88 m high Feistritzbach Dam in Carinthia, with its reservoir designed to provide seasonal storage for the high-head Koralpe power scheme (50 MW, 83 GWh including a 20% share for Yugoslavia). In addition, the power conduits of the lower stages of two existing storage schemes (Uttendorf in the Stubach scheme, and Gerlos) are being replaced and their installed capacity considerably uprated.

The consolidated ten-year development programme drawn up annually by Austria's federal and provincial electric utilities presently lists 35 projects totalling 1 350 MW for 6 000 GWh of average annual production. Although this corresponds to a growth rate for supply which is only about half predicted growth in demand, it might still turn out to be an unduly optimistic assessment of the next few years. In terms of energy produced, half of the projects relate to the further hydroelectric development of the Danube east of Vienna, although only one of the two or three stages possible there, namely the upper stage at Freudenau (141 MW, 907 GWh) has so far been made definitive. On the river Inn, applications for planning permission have been lodged and legal procedures started for new plants at Martina-Ried (binational, 100 MW, 424 GWh including a 14% share for Switzerland) and Langkampfen (27 MW, 158 GWh), and additional developments to exploit resources of up to 1 400 GWh have been shown to be technically feasible. The other projects included in the above ten year-plan are on the rivers Drau (450 GWh), Salzach (370 GWh), Traun (260 GWh), Möll (160 GWh) and JII (140 GWh), as well as on a number of minor rivers and streams.

At the present time it is not possible to predict whether, and if so when these projects will complete what are now very lengthy approvals procedures. Promoting new hydropower projects is now so difficult because the engineering works involved impact the increasingly burdened and congested space that is our biosphere at a point where it has hitherto been best preserved in its natural state, i.e. in the mountains and along the rivers and streams. And the environmental impact of man's technologies – the source of a level of affluence we now take too much for granted – is a subject on which people have become very sensitive in the last few years, perhaps even over-sensitive; and people now rightly pay critical attention to any further products of modern technology and to the question whether and under what conditions they are compatible with the environment and its needs. As far as approvals procedures are concerned, these questions and concerns will be addressed in future in the framework of an environmental impact assessment, although – as in many other countries – the corresponding legislation has not yet progressed beyond the drafting stage in Austria. In the meantime the present regulations and procedures for nature conservation will continue to apply, involving criteria that unfortunately are neither clear nor free from contradictions, and it will take years before the new steps in the approvals procedures can be handled with the routine in terms of form and contents that has been built up for the other steps.

Apart from that, if we take a holistic view of the concept of the environment, relating not just to the individual situation but to man's biosphere in general, and if environmental protection is interpreted in the sense of our obligation to hand down this biosphere and its resources intact to the next generations, then hydropower should have no difficulty in finding the necessary level of acceptance for the future, because so far it has been the only "soft" technology for large- scale energy production. It is self-replenishing and draws its energy from the solar-powered global circulation of water, with extremely high conversion rates, without exhausting irreplaceable resources and without emitting waste heat, pollutants or carbon dioxyde – the latter dreaded now so for its contribution to the green-house effect. The environmental impact of hydropower and of dams in particular (see Chapter E) is far less problematical than those of thermal or nuclear energy, in particular if not only direct generation is taken into account, but also the processes involved in the production, transport and storage of fuels as well as in the treatment and disposal of waste.

Therefore, in line with the recommendations of the World Energy Conference, which are based on similar arguments, there are significant arguments advocating the future development of some more of the substantial hydropower resources still available in Austria, which are estimated on the basis of thorough investigations to amount to no less than 19 000 GWh.

6 MULTIPURPOSE BENEFITS

6.1 Storage power schemes and flood control

Although the primary goal in the construction of all storage power schemes in Austria, and the only source of subsequent earnings for the hydropower companies, is of course electricity generation, such schemes also involve a whole series of additional utilities that accrue to the general public free of charge. As a rule, significant improvements in flood control are the most important of these spin-offs, although benefits to the local infrastructure and tourism as well as a significant increase in flow during the winter period are not to be underestimated, either.

The fact that the retention volume of a reservoir is a significant flood-risk reducing factor for downstream areas is obvious enough and is appreciated by the people who benefit directly, but this function has not yet been given due recognition by a wider public, who frequently pass unfavourable judgment on projects for further hydropower development nowadays.

At any station along a river downstream of a storage reservoir, the degree of improvement in flood control is determined by two parameters:

a) the ratio between the storage volume available for retention at the time of a flood and that part of the flood which contributes to peak outflow, and

b) the percentage of total catchment area which is controlled by the reservoir.

Criticism is sometimes made of the apparently contradictory strategies of providing either for a full energy reservoir or for empty flood storage. Theoretical studies and long-term operating records, however, show that for conditions common with seasonal storage reservoirs in the Austrian Alps, firstly, even a relatively small retention volume can drastically reduce a flood, and secondly, such a retention volume will in all probability be available at the time of extreme floods.

Available retention volume is determined by the volume between actual and retention water level, plus an additional flood surcharge above the latter, i.e. above the crest of the predominantly used free overflow spillway. This flood surcharge volume would also be available in the case of flood discharge into a full reservoir, but that is an extremely unlikely event given the normal filling cycles of Austria's seasonal storage reservoirs. Extreme flood events, and especially the biggest which occur in July and August, in all probability do not coincide with maximum storage in the reservoirs, whose filling, as a rule, is not completed until September or October, so that only heavy rains in September can still pose hazard to a certain extent. In any case, statistics show that maximum reservoir level is only reached once every five years on average, and that this condition is maintained for only a few days so as to avoid spillage losses.

Given the considerable size of Austria's seasonal storage reservoirs relative to the runoff from their catchment areas, even an available retention capacity of only a few percent of reservoir capacity is sufficient to effectively reduce peak discharge. An analysis of data taken from decades of operating experience with over thirty Austrian reservoirs suggests that the spillways of reservoirs with an active storage equal to at least 40 (60)% of annual inflow will only be activated by floods with return periods of more than 10(25) years. No side valley in Austria that is controlled by a seasonal storage reservoir upstream has ever suffered flood damage since the construction of the reservoir, and peak flows in the main valleys have been significantly reduced since then as well.

Fig. 12 shows one of the many examples that could be quoted in this context. On 19 July and 25 August 1987, the Stubai and Ötz Valleys southwest and west of Innsbruck in the Tyrol (Fig. 9) were severely damaged by floods. On these two days the Kauner Valley, which is located only 10–12 miles to the west of the Ötz Valley and runs parallel to it, suffered no damage at all – due to the effects of the Gepatsch reservoir. The gauge at Platz, with a 189 km^2 catchment area 54% controlled directly by the Gepatsch reservoir and 25% indirectly via diversions, recorded a peak discharge of only 5 m^3/s on both days, which is a normal figure for a summer day (Fig. 12b). Without the reservoir, peak discharge would have been 62 and 50 m^3/s for these two days, figures that correspond to a more than one hundred-year flood and would certainly have been higher than peak discharge for the 1960 flood whose devastating effects have not yet

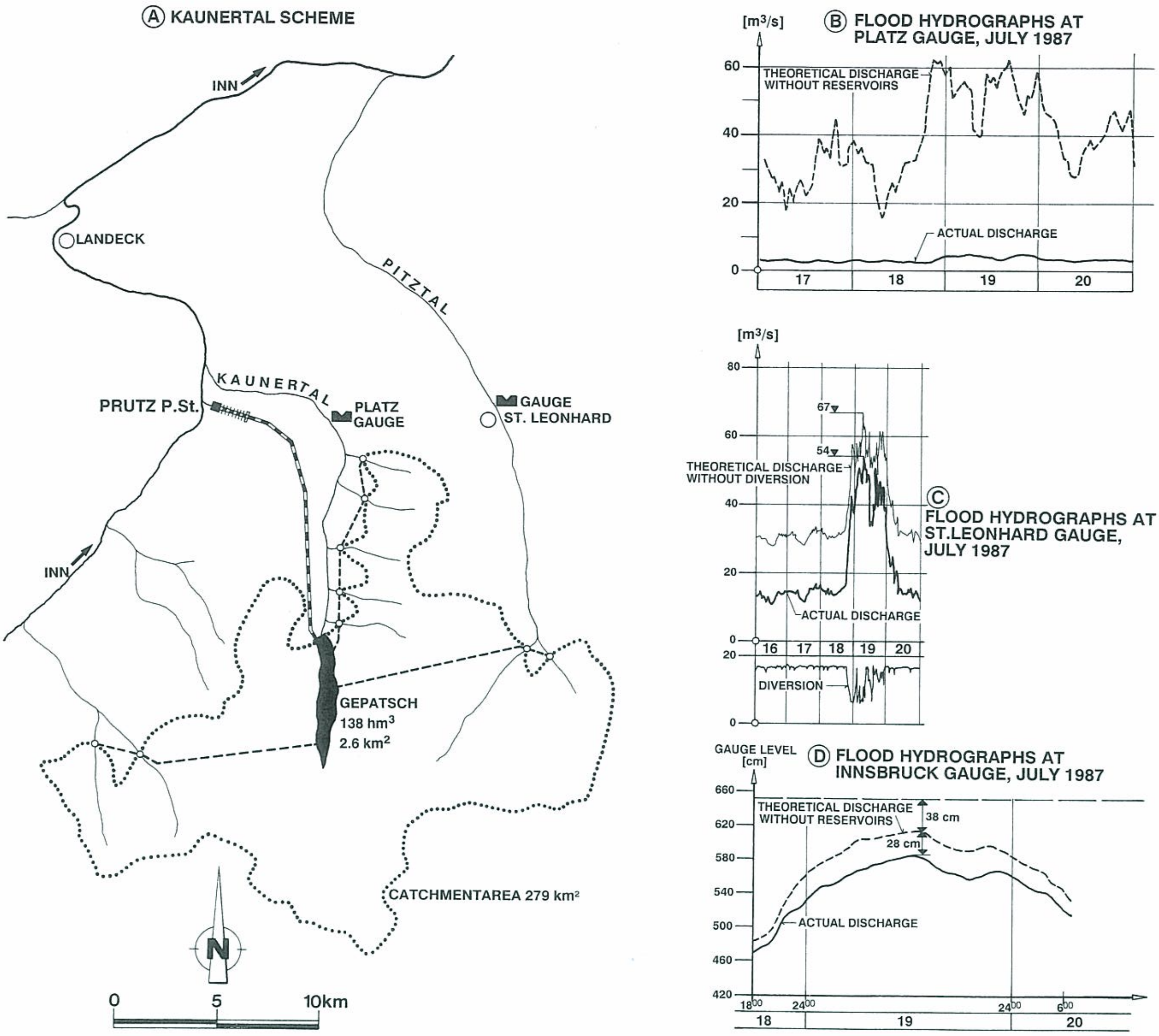


Figure 12 The effect of the Gepatsch reservoir on the july 1987 flood

been forgotten by the people of the Kauner Valley. The result would have been flooding across the full width of the valley in the flatter sections where most people live. Instead, the river has not once burst its banks since the Kaunertal scheme went into operation in 1964.

Short-term storage reservoirs, in spite of their limited active storage relative to flood flows, can also significantly reduce discharge peaks. In addition, a marked reduction in flood flows has even been observed downstream of mere water intakes. Depending on their design capacity, this effect relates above all to minor and medium floods, but (Fig. 12c) on the aforementioned 19 July 1987 the diversion from the upper Pitz valley to the Gepatsch reservoir reduced peak discharge at the St.Leonhard gauge from an approximately eighty-year flood to the magnitude of a thirty-year event.

Remote reservoir effects from Gepatsch and two other seasonal storage reservoirs with a total active storage of 362 hm³ markedly reduced peak discharge on 19 Juli 1987 on the main river, too. Based on a carefully calibrated mathematical flow model, analysis showed that peak discharge at the Innsbruck gauge, with a 5 794 km² catchment area, decreased from a 200-year flood to a 40-year flood (without reservoirs, which retained 13.4 hm³), saving the Innsbruck city precincts from partial flooding (Fig. 12d).

6.2 Other benefits from storage reservoirs

Reservoir draw-down during winter tends to balance or frequently even reverse the natural winter-summer relation of flow downstream of storage power schemes in side valleys. This is accompanied by considerable variations in the daily flow pattern due to electricity demand, but these variations are more and more mitigated and compensated along the main rivers, where the substantial increase in winter runoff benefits their biology and purity and improves the operating efficiency of run-of-river power plant.

The 9 313 km^2 catchment area of the Kirchbichl run-of-river plant on the River Inn in the Tyrol, for example (Fig. 9), is influenced by eight reservoirs, whose total active storage of 717 hm^3 accounts for about 31% of natural winter runoff volume. Hence, the resulting increase in winterflow is almost one third, and over half in the coldest months, while the 10% retention in summer is negligible. Even on the Danube, the favourable effect of winter storage on navigation and power production is clearly measurable (e.g. 5.5% average increase in winter flow downstream of the mouth of the River Kamp).

In the last few decades, there have been many examples of mountain areas threatened by rural exodus which were then revitalized by the decision to construct a storage reservoir and power plant. The actual hydropower construction work was always accompanied by significant improvements to the local infrastructure through highway construction, and support with water supply and waste water management, by major torrent and avalanche control measures, and by annual, index-linked contributions to local community funds.

With regard to tourism, storage reservoirs usually become an additional attraction for the region, as the visitor statistics for existing schemes clearly show, including some that attract hundreds of thousands of visitors per year. Like natural lakes, reservoirs are considered an asset for the countryside as long as they are more or less full. Admittedly, it is a normal aspect of their primary function that they should be only partly filled for longer periods of time and temporarily even empty, but in terms of annual operating cycles we know by experience that the reality is considerably more attractive than is often assumed. Hydrographic records show that Austria's seasonal storage reservoirs are normally more or less full from the middle or end of July to the end of the year, and during subsequent drawdown the slopes of the reservoirs are covered with snow until the end of May and longer, so that there are only two months in early summer when the aesthetic aspect is really unsatisfactory, and these months are not part of the main tourist season in the Alps.

Given the very cold water temperatures of high-altitude reservoirs, aquatic sports are unfortunately out of the question there, whereas the few low-altitude reservoirs in Austria have become popular recreational facilities.

6.3 River barrages and flood control

Run-of-river power plants also contribute to flood control, although not in the same way as seasonal storage schemes. As is well known, peak discharge is conveyed faster on a developed reach than on a river without pondage, and the gradient of the discharge peak declines at a slower rate. This effect decreases with increasing magnitude of the flood, as the ratio between flow depth in rivers with and without pondage tends towards 1, especially towards the upstream end of the backwater curve. Nevertheless, this is a basically negative effect of run-of-river power plants, albeit one that is not only compensated through a whole series of structural and operational measures but is actually reversed by them, so that in the last resort one can still speak of a significant contribution to flood control. Apart from that, the stipulations to be met for the official approvals also guarantee that there will be no detrimental impacts in the backwater and downstream areas compared to the situation before construction.

In the normal case, various structures and measures are employed, e.g. dykes and dyked enclosures, bank protection, raised banks, and lowering of the riverbed downstream of barrage by dredging, to ensure practically complete flood protection for all major settlements, industrial plants and all other objects and zones of any importance. In the case of agricultural land, flood control is at least improved to the point where only the very big floods, which are correspondingly rare (calculations are based on a return period of 10 to 30 years), can lead to inundation.

Given today's civil engineering capabilities, it would be possible and even commercially viable in many cases to include any degree of continuous flood control for the upstream banks of all hydropower projects. Such measures, however, would accelerate and increase peak discharge for the adjoining downstream reach at the same time, i.e. the risk would merely be transferred, and improved flood control for one section would be achieved at the price of a deterioration in the situation of the next section. It is therefore necessary to continue to permit the bigger and rarer floods to inundate designated flood retention areas and thus lose a part of their flow volume.

The construction of lateral dykes alone would cut the river off from its existing flood retention areas and considerable retention volume would be lost. To prevent this, the dykes must therefore be provided with overflow sections designed to permit the river to inundate the flood retention areas once a certain high water mark is reached. Drainage is normally provided in the form of natural and artificial channels draining into the downstream section of the stage involved. In the case of riverine woodlands used as retention areas, it is in fact ecologically desirable to provide for annual inundation.

A relieable monitoring network with telemetry for the most important data and subsequent processing in a mathematical flow model is the basis for a flow prediction capability that can be constantly improved with growing experience. Such monitoring systems permit not only the establishment of an effective flood warning service but also a certain degree of flood management, especially in the case of a chain of run-of-river power plants. Admittedly, the scope for intervention is much more limited than in the case of storage power schemes. However, the closer the overflow embankments are located to the power station, the greater the scope for flood management with the aim of influencing the timing of inundation of the flood retention areas. Delaying inundation so as to have adequate retention volume to cope with the actual discharge peak is achieved by means of anticipatory drawdown. Where there is a correspondingly large backwater area, this strategy provides additional retention volume in the river channel, too. Such drawdown operations, however, presuppose a high standard of flood prediction, efficient plant control, and independence from peripheral restraints relating to such factors as navigation requirements.

At all events, today's power plants, with their monitoring services and mathematical flow models – in collaboration with meteorological and hydrographic services, civil defence and the waterway authorities – have proved their usefulness in the elaboration of improved flood control systems.

ACKNOWLEDGEMENT

The preparation of most of the figures and tables relating to this chapter by Dr.Dr.h.c. H. Lauffer is gratefully acknowledged.

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Chapter B

THE HISTORY OF DAM CONSTRUCTION IN AUSTRIA

CONTENTS

THE HISTORY OF DAM CONSTRUCTION IN AUSTRIA

By W. Pircher*

1 INTRODUCTION

All major dams and river barrages in Austria were primarily built for hydropower generation, as discussed in the previous Chapter A and as indicated in several sections in Chapters B, H and confirmed by the statistics in Chapter K. The history of dam construction in Austria is therefore inseparably linked with the development of the public electricity supply, to which hydropower has always made a very significant contribution in Austria, meeting between 60 and 80% of demand in the last fifty years and currently about 70%. Needless to say, the progress made in the electricity supply industry has always been closely related to developments in the economy as a whole, and they in turn have been dependent on the twists and turns of the country's political history.

The first use of hydropower for electricity generation goes back to 1882 when a small power station was built to produce electricity for street lighting in the town of Steyr to mark an electricity exhibition held in 1884, and the first public electricity supply system in Austria was built in 1886, when a small power station was constructed on the Erlauf to generate electricity for the town of Scheibbs. Electricity in Austria thus has an over one-hundred year history, a history that is characterized by vicissitudes which have left a clear imprint on the development of hydropower, with periods of growth alternating with periods of stagnation, as in the case of dam construction engineering, too. Three decades spent in the big economic space of the Austro-Hungarian Empire, a period of initial progress and what were major achievements by the standards of that time, were followed by a further three, extremely difficult decades, bringing a lack of sociopolitical and economic cohesion that was highly detrimental to an industry like hydropower development, with its overriding need for continuity, and including three radical about-turns in the political system, one in 1918, the second in 1938, and the third in 1945, plus two world wars, with the Great Recession in between and Austria's temporary loss of sovereignty. All the greater were the achievements involved in the construction of those plants that were completed, at a time when – under the influence of these negative factors – growth in the industry generally lagged far behind expectations. It was not until after the first trying post-war years and the reconstitution of Austria as the Second Republic that hydropower development, and thus dam construction was able to enjoy a return to more orderly structures and significant growth rates in a period of continuous progress that lasted for almost four decades and has only been checked in the last few years (Figs. 3 and 4).

It is therefore logical to divide the following survey of the history of dam construction in Austria into four main sections defined both by the decisive historical events in which they are embedded and by marked differences in the related political systems and the structures of the organizations responsible for hydropower development and dam construction in the country. Subsequent chapters are therefore based on the following chronological classification:

a) Early period: structures used for a variety of purposes prior to electricity generation from hydropower

b) 1882–1918: from the first use of hydropower for electricity generation to the end of the First World War and the dissolution of the Austro-Hungarian Empire

c) 1918–1938: the time of the 1st Republic

d) 1938–1945: the time of the *Anschluss* and the Second World War

e) 1945–1988: following three difficult post-war years needed to make good ravages of war, the boom years of hydropower development and dam construction in the 2nd Republic

f) 1988–date: the last few years and future prospects

2 THE EARLY PERIOD IN THE CONSTRUCTION OF DAM STRUCTURES

There have been no finds of ancient dam structures on the territory of modern Austria of the type with which we are familiar in the arid regions of the Middle East. In view of the more moderate climatic conditions of central Europe, there was no need to store large volumes of water in reservoirs for use during long months of drought.

Springs and wells were abundant everywhere, and in the Eastern Alps the period of maximum precipitation coincides with the growth period of the vegetation.

Even in those Alpine valleys in the lee of the high mountain chains where *irrigation* was traditionally practised, there was no need to build reservoirs, as the streams fed by the waters stored in the glaciers would be fullest in the hot summer months, and the farmers built systems of open channels, which were often complex and negoti-

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ated difficult rocky terrain. And in the wide flat valleys and lowland plains, the ground water level was usually so high that drainage was more important than irrigation. It is only with the widespread implementation of river control works and the reduction in bedload of most rivers that this situation has been reversed for some regions in recent years.

With regard to *drinking water*, Austria has so far been able to maintain supplies from springs and ground water, without the need to fall back on surface water. Vienna's mountain spring water, for example, which is famous for its excellent quality (annual consumption in 1988: 147 hm^3), is piped almost exclusively from karstic springs located in the practically uninhabited massifs of the Rax-Schneeberg and Hochschwab mountains. The 120 and 181 km pipelines built in 1873 and 1910 respectively to transport the water to the city deliver the water to underground tanks of up to 0.6 hm^3 capacity, so that prior storage in reservoirs is superfluous.

One frequent reason for impounding water in the Middle Ages, on the other hand, was to create *fish ponds* to provide the supply of edible fish needed to observe what were then strict religious rules for fasting. The usual technique was to build low retaining walls of earth around flat-bottomed depressions well away from the main waterways. One of the most famous of these old fish ponds was built in 1460 by the Tyrolean Duke Sigismund the Rich, who had an 8 m high and 250 m long earth embankment built near Tarrenz in the Gurgl Valley, remnants of which could still be seen in 1960.

The more direct precursors of today's dams are the *logging dams* built to float the newly cut timber through the inaccessible forested mountains. These logging dams which can be traced back to at least the 13th century, were built where the stream had too little flow to float the logs unaided. The water was repeatedly impounded and suddenly released to produce a flood wave, which would carry the logs down to the end of the logway section, where they would be stopped by a wooden rack and hauled ashore.

Originally these logging dams were timber structures, as in the case of the 12 m high Archduke Johann Dam in Tyrol, which could discharge a total pondage of 230 000 m^3 within an hour (Fig. 1). Today, with the forests now served by their own forestry track networks, log floating has become more or less extinct. A few of the old logging dams have been preserved for their historical interest, while others have been converted to other uses, such as the Preszeny logging dam on the Salza in Styria with a storage reservoir of 0.65 hm^3, which is now utilized by a small power station.

Mountain torrent control structures are not unlike dams in their design, either. In Austria these structures, often 15 m high and more, are built by the Office of Mountain Torrent and Avalanche Control, which was established in 1884. Since their purpose is not to impound water but to provide temporary or permanent restraint for the bedload, they are not included in the dam statistics. Nevertheless some of the earliest structures were very interesting feats of engineering, such as the Madruzza sediment retention dam built in a narrow gorge on the River Fersina near

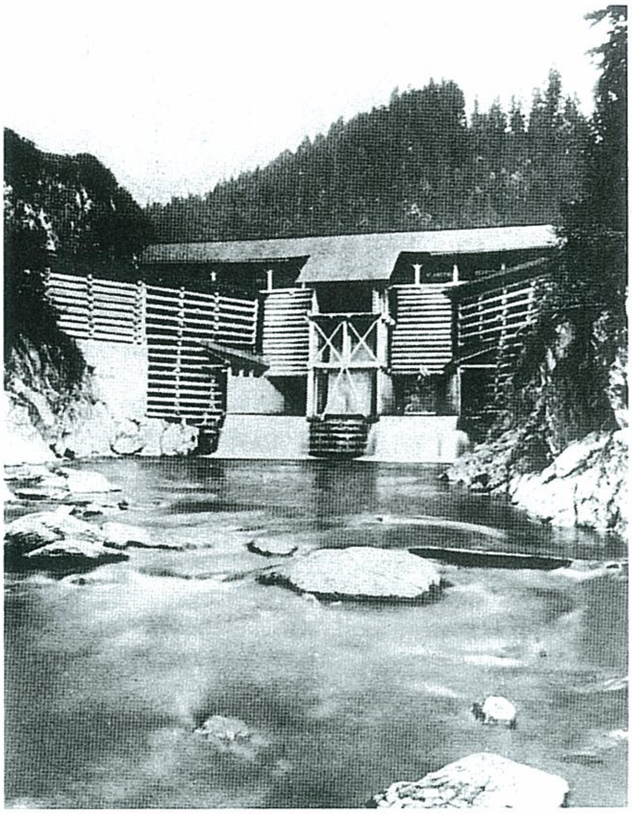
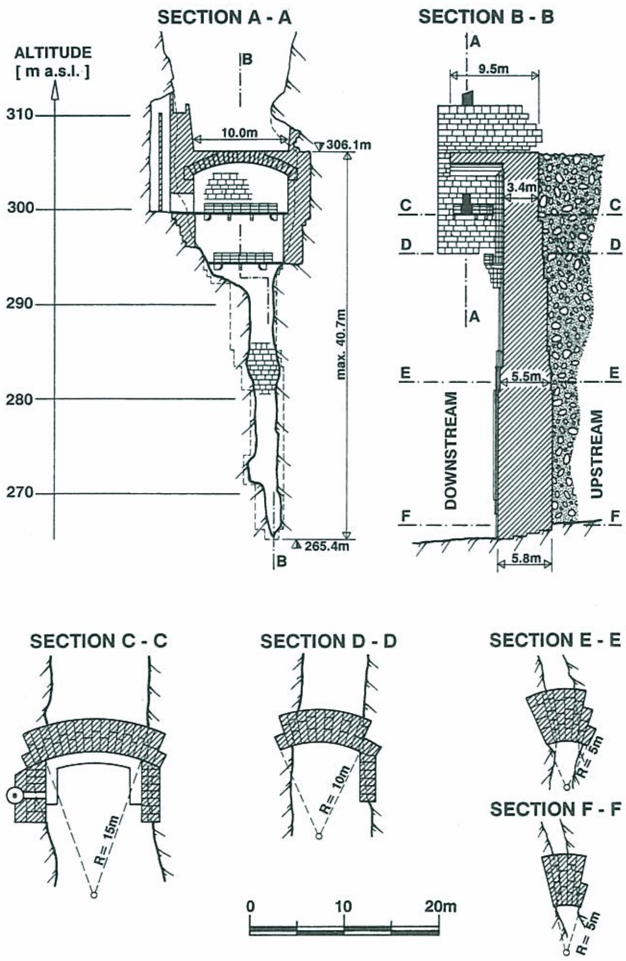


Figure 1 Archduke-Johann log dam

Figure 2 Madruzza dam (1885/86)



Trento (Italy) by what was then the Austro-Hungarian waterway authority in 1885/86. The 41 m high structure was an arch dam of variable radius, decreasing from 15 m at the crest to 5 m at the base (Fig. 2).

A number of dams were also built at a relatively early date to form flood retention basins. The most dangerous of the flash floods of that time were caused by the unpredictable discharge of a number of notorious glacier lakes which formed – at a time of generally more extensive glaciation – wherever the runoff from a high-level valley was blocked by a barrier of ice formed by a glacier advancing from the side and were emptied without warning when the waters finally penetrated the ice. Following a sequence of such catastrophic flash floods, the then Austro-Hungarian waterway authority built a 332 m long masonry dam at Zufallboden in South Tyrol (now Italy), which had a retention capacity of 0.72 hm^3. Similarly, at about the turn of the century, the Vienna municipal authorities built a chain of flood retention basins of 1.6 hm^3 total capacity on the Wien river, with a continuous flood wall separating them from the river channel. Only very few of the dams built in combination with flood retention basins today satisfy the ICOLD criteria for large dams with regard to their height. They are filled very rarely, if at all, and then only partly and briefly, and for that reason they have not so far been included in the ICOLD register (Table 4). In comparison with the far-reaching contribution to flood control made by seasonal storage reservoirs built for hydropower generation, such flood retention basins are of purely local significance in the Pre-Alp zone.

Pre-industrial hydropower utilization in Austria, such as the use of watermills, which can be traced back to the 8th century, did not involve the construction of major dams. The use of water power, as the only source of energy that was independent of man and his animals, early became very widespread.

Given the limited scope for engineering works at that time, the use of water power was restricted to streams and small rivers which permitted the construction of diversions to feed run-of-river plants. A good site for water power for direct conversion to mechanical energy, e.g. on the streams from the Calcareous Alps (with large springs that also provide adequate winter flows) soon attracted a variety of commercial activity, including mills, sawmills, and iron forges, and later also paper and textile manufacturing plants, and the sites of many of today's big industrial complexes bear witness to these more modest beginnings.

One historical structure that is of special interest is the town canals and mill race chains deriving from the Middle Ages, which were usually built along the natural side arms of the main rivers. They were often of considerable length and comprised several stages, each fitted with a waterwheel for optimum hydropower utilization. One such structure is the mill race chain built on the right bank of the Mur upstream from Graz, with a length of no less than 20 km and a runoff of 11 m^3/s. This structure is still in use today, although the original 33 stages, with heads varying between 0.6 m and 2.2 m and a total capacity of about 3 MW, have since been combined to form a smaller number of bigger stages. Another example is the Alm canal. Started in the 10th century, it has conveyed 5.5 m^3/s of water since 1280 from the Königseeache to the city of Salzburg, where the water is distributed through secondary canals. At the turn of the century 63 waterwheels with a total of 1 900 hp were in operation along the 11.8 km long main canal.

3 THE PERIOD FROM 1882 TO 1918

As mentioned in the introduction, 1882 was the year of the Austrian debut for hydropower generation, the location being *Steyr*, a city with a long tradition in mechanical engineering and armaments production. This was followed in 1886 by the first public electricity supply system, a 130 kW installation for the town of *Scheibbs* in Lower Austria. In 1888 the entrepreneur Anton Rauch had a small hydropower station built for his mill in *Innsbruck*, which is still in operation today, and in 1889 the Innsbruck municipal authority built the city's first electric power station on the same stream.

With the invention of three-phase current and the construction of the first long-distance electricity transmission line by Oskar von Miller in 1891, carrying energy at a voltage of 25 kV a distance of 175 km from Lauffen to Frankfurt, developments started to gather speed, driven as they were by the pioneering spirit of private entrepreneurs, big companies and progressive municipal authorities. Many electricity generating plants were built at that time, some of which still exist in modernized and extended form, including such milestone projects as the power station built on the Töll to supply the town of Meran in 1896 (South Tyrol, now Italy), an 8 MW plant built on the Sill (Tyrol) in 1898 to supply an electrochemical processing plant, a power station built on the Alm Canal for Salzburg, and a further generating plant on the Sill with a 13 MW capacity built to supply Innsbruck in 1903.

All these plants were constructed without reservoirs. Only the *Andelsbuch* power station on the Bregenzer Ache, whose 7.4 MW capacity made it a major plant by the standards of the time, included a reservoir built for daily storage between the intake tunnel and the penstock. But the first real dams were not constructed until almost a quarter of a century after the beginnings of electricity generation. The two very first dams for hydropower generation were commissioned for an organization that was to play an important role in subsequent developments, namely the Austrian Railways, whose management was quick to appreciate the benefits of hydropower and in 1905 established a research team to study and record the hydropower potential of the Austro-Hungarian Empire. In 1908–13, to generate electricity for the Mariazell Railway, two slightly curved gravity dams were built on the Erlauf in Lower Austria, namely at *Wienerbruck* and *Erlaufklause*, with heights of 13 and 35 m respectively. The bigger of the two was a masonry structure. At almost the same time, in 1910–11, the *Gosau Dam* was built by a private electricity generating company to impound a natural lake for long-term storage for a small power scheme with several stages. This was a 17 m high earthfill dam with a masonry core wall as a sealing element, which unfortunately just failed to connect with the impervious ground moraine. In 1913, the 28 m high *Wiestal Dam*, also a masonry gravity dam with a slight curvature on plan, was constructed to supply the city of Salzburg with electricity. The first major river barrage was built in 1914–18 in *Faal* (Fala) on the Drau (Drava) in what is now Yugoslavia. With its 35 MW capacity and 220 GWh annual energy, this plant is now integrated into a complete chain of power stations extending into both countries. By the end of the First World War in 1918, the hydropower plants built in the Austro-Hungarian Empire – and located primarily on the territory of present-day Austria – by the electric utilities and private industry in approximately equal measure, had a total annual generating capacity of 1 280 GWh. Absence of a national grid system, however, meant that capacity utilization was inefficient. In 1918 actual electricity generation from hydropower totalled 895 GWh and met 51% of consumption, the remaining requirement being met from thermal power stations.

4 THE INTER-WAR YEARS FROM 1918 TO 1938

In addition to Austria's thermal power stations, the eastern part of the country in general, and especially Vienna and the surrounding industries, had relied for decades on supplies of coal from Moravia and Silesia (to such an extent that planning work had already begun on increasing transport capacities either by converting the northern railway to a four-track line or by building the Oder-Danube Canal). With the disintegration of the Austro-Hungarian Empire after World War I, the residual territories of Austria were cut off from these traditional sources of coal, and Austrian industry and the railways were deprived of their very means of existence (deliveries from the succession states being much too small in the first post-war years). Out of necessity, a keen interest therefore developed in the country's water power potential, and a spate of planning and construction activity followed, especially on the part of the Austrian Railways, and also in the individual provinces, where provincial utility companies had been founded which, with slight modifications, have remained the backbone of the country's electricity generating industry and hydropower development up to the present day. The more immediate result of the post-war situation was the construction of a number of run-of-river power plants, including some with short-term storage, the details of which are beyond the scope of this chapter, and also several seasonal storage plants which brought real progress in dam construction.

The first plant at *Spullersee*, was built by the Austrian Railways in 1921–25 as part of the electrification scheme for the Arlberg line, Austria's rail link with the west, for use in combination with the Schönberg run-of-river plant to the south of Innsbruck, which had been built in 1912. Located in a chalky limestone depression on the watershed between the Danube and the Rhine, the Spullersee, a natural lake with a gross head of 800 m to the valley, was raised by means of two gravity dams slightly curved on plan, the bigger of which, to the south, is 36 m high and 280 m long, while the north dam is slightly smaller. When the Spullersee dams were built – and the same applies to the 37 m high Strubklamm Dam built by the Salzburg municipal authorities in 1920/21 – concrete technology was still in its infancy. Nevertheless, the facing concrete used is rich in cementitious material while the hearting concrete, with 15% boulders, is low in cementitious material. Also, Bavarian trass (a pozzolanic material) was added, quarry-run aggregates used, and, with growing confidence, the usual masonry facing was omitted. The concrete was poured from a scaffolding bridge via delivery chutes and compacted by tamping. The concrete was of medium strength, and thus impervious and frost-proof, and after forty years it was still in such good condition that it was possible to raise the dam by 4.60 m using prestressed anchors.

Following Spullersee, the next project in the Austrian Railway's electrification programme, in 1926, was hydropower development of the Stubach Valley in the Hohe Tauern, for which the relevant rights had been acquired in 1914. Completed in 1929, the old *Tauernmoos Dam*, which was also built to raise a natural lake, was a concrete gravity dam with a masonry facing built to a height of only 28 m to provide seasonal storage for the Enzingerboden power plant. In 1941 and 1956 two lower stages were built, while additional upstream diversions were also added to further increase overall capacity of the scheme, involving the construction of small dams in some cases like the Weissee gravity dam completed in 1953. In 1973 the original Tauernmoos Dam was replaced by a higher structure, and at the present time work is in progress to replace the two lower stages with one combined stage.

Just how convenient the solution of converting natural lakes into seasonal reservoirs is, is perfectly illustrated by the case of the *Achensee* in Tyrol, which is operated by drawdown alone without any additional impounding structures. With a live storage of 80 hm³, the Achensee remained Austria's biggest seasonal reservoir until 1950. The Achensee power station was built by TIWAG (Tiroler Wasserkraftwerke AG) in 1924–27. This 80 MW seasonal storage power plant was a feat of engineering at the time, with caisson foundations for the intake, a steel-lined pressure shaft with partial load bearing by the surrounding rock, and the use of diversions to increase the size of the catchment area. Annual energy greatly exceeded demand in the Tyrol, so that the Austrian Railways could also be supplied, and negotiations were held with the Vienna municipal authorities on power supplies in connection with the construction of a 110 kW transmission line. The Vienna authorities, however, were reluctant to rely on such a remote plant, while the miners and coal merchants, who were worried about their market, also brought their influence to bear. As a result, an agreement was signed in 1928 with Bavaria, the Tyrol's northern neighbour, which is still valid today and has been followed by several similar agreements. The 110 kV line that was then built via Innsbruck to the Walchensee power plant was the first binational line at this voltage level in the whole of the area occupied by today's European grid.

The grid system set up together with southern and western Germany as a result of the lack of transmission lines with eastern Austria greatly stimulated hydropower development not only in Tyrol but also in Vorarlberg, and these two most westerly of the Austrian provinces subsequently took the lead in this sector for a number of decades.

In 1928–31 the Vorarlberger Jllwerke built the 53 m high *Vermunt Dam* as the first power stage in a scheme that has since been continually extended. This Partenen power stage which has a head of 700 m and a 90 MW capacity, also laid the foundations for a grid with Baden-Württemberg and the Rhineland. The concrete engineering employed for what was to remain the biggest dam built in the inter-war years, with a volume of 142 000 m³, was not up to modernday standards, of course, but with an 80 mm maximum grain size, 30 cm layers and 2.50 m lifts, it was a step in the right direction. To compensate the lack of compaction with pneumatic stamp hammers, a reinforced gunite membrane was placed on the upstream facing concrete. In 1988–89, as a result of increasing percolation flows and saturation of the dam body, repairs were carried out in the form of a 60 cm facing of impervious concrete.

Similar repairs have also been performed recently on the 33 m high *Pack* gravity dam built in 1929–31 by the Styrian utility STEWEAG. The combination of inadequate imperviousness achieved with the concrete with quarry-run aggregates and slightly aggressive water had necessitated repeated sealing and grouting works. The dam was built to form the bigger of two reservoirs operated by STEWEAG for its Arnstein high-head power plant on the Teigitsch to the east of Graz, which was begun in 1923 as an addition to the run-of-river and thermal plants. In 1950 the 58 m Hierzmann arch dam was built to provide a third reservoir.

In addition to the above structures, a whole series of other dams were built in the Twenties and others refurbished or enlarged. In addition many small power plants were constructed at that time, especially by local communities with rapidly growing power requirements, which had not yet been linked up to a provincial grid and had adequate funds from their logging operations to finance such projects.

1929, however, with the Great Depression and the growing political confusion in the country, marked the beginning of the end of this period of busy construction acitivity, and by 1932 hydropower development had practically ceased. In 1938 Austria still had a developed potential of only 2 400 GWh, meeting 91% of demand for electricity at that time.

5 THE WAR YEARS FROM 1938 TO 1945

The *Anschluss* and the incorporation of Austria in the German *Reich* brought with it a complete turnabout in political, economic and organizational terms, which at first gave promise of a period of exceptional progress in water power development. Germany's rapid arms build-up in 1938–39, however, benefited thermal power plants, which could be built more quickly, while the outbreak of the Second World War was followed by rapid and almost total paralysis. In this situation much work was begun, sometimes prematurely, and some projects were completed. Only a small number of the really big projects, however, ever left the drawing boards or progressed beyond the first stages of construction.

A number of important run-of-river power plants did come on stream during the war, however. Three were located on the Inn, namely *Kirchbichl* in Tyrol and two much bigger plants at *Ering* and *Obernberg* on the border reach between Upper Austria and Bavaria. On the Drau three run-of-river plants were completed: *Schwabeck*, which started operation in 1942 and was to become a main pillar of Austria's electricity supplies in 1945, *Lavamünd* (1942–44), the first pier head power station in the world, and one more of the same type in what is now Yugoslavia at Dravograd (construction of a fourth at Maribor, had been started further downstream). Construction also began on four power stages on the Enns, one of them to supply the iron and steel works in Linz – later to become the VOEST company – but work was not completed until after the war. In the case of the *Ybbs-Persenbeug* power station begun in 1938, also to improve navigation on the Danube, only the cofferdams for the construction pit had been completed by the end of the war, and the actual plant was not built until 1954–59.

In 1937–40 the Austrian Railways built the middle stage of the Stubach scheme in the Hohe Tauern, with the 29 m high *Enzingerboden* gravity dam placed on landslip boulders sealed with an 18 m deep cut-off wall. In 1939–45 TIWAG constructed the *Gerlos* power plant in the Ziller Valley under the most arduous conditions. The 39 m high arch dam built to provide short-term storage for the high-head stage was the first of this type in Austria, with a large thrust block placed on the left abutment to achieve the symmetry that was then considered essential. In order to prevent sedimentation of the small reservoir, a weir was built at the end of the backwater curve to intercept the bedload, which was sluiced along a concrete pipe and tunnel to the tailwater. In 1959 the first signs of creep were detected in the imbricated structure of the slope downstream from the left abutment. This was followed by rockfalls in 1963–64, and the decision was taken to incorporate a foundation block to provide mutual support to both slopes and to place a gravity dam in front of the arch. Further repairs are now required owing to swelling in the concrete caused by a weak alkaline reaction with the aggregates.

In 1940 work began, in a second development phase for the Jllwerke in Vorarlberg, on the construction of two dams on the saddle of the Bieler Höhe to form the Silvretta seasonal storage reservoir. The western structure is the 80 m high *Silvretta* gravity dam, which, together with a small adjoining dam, has a length of 572 m. Requiring a total concrete placement of 425 000 m³, this is an impressive structure even by today's standards, though construction was otherwise unremarkable. The eastern structure, the *Bieler* embankment dam, is considered the first major fill dam in Austria, with a length of 733 m, a core height of 25 m and a dam height above the upstream toe of 35 m. The gravel fill was placed on extremely hard and impervious ground moraine material, and the sealing element, a reinforced concrete core wall with a pattern of vertical and horizontal contraction joints, extended 4–6 m down into the ground moraine. The two dams were not completed until 1948, although power generation with partial filling was begun in 1943.

The most valuable heritage among the unfinished wartime projects was without doubt the torso of the *Glockner-Kaprun* power scheme, an advanced design that Hermann Grengg had managed to defend in the face of other plans that were typical of the megalomania of that time. By the end of the war one of the turbines for the main stage was operating, but without pondage. The five big dams for the two seasonal storage reservoirs and a storage reservoir for the biggest of the diversions were not built until after the war. In the case of the Limberg Dam for the lower reservoir, the original plan, under the influence of the Old Dixence Dam, was to build a buttress dam, and A. Stucky was consulted. As a result of the destruction of the Möhnetal Dam by special bouncing bombs, however, it was decided to build an arch dam instead, and since then no buttress dams or multiple arch dams have ever been built in Austria.

In 1944 all hydropower plants in operation on the territory of present-day Austria generated a total of 4 033 GWh, covering 82% of total consumption (the figure for 1945 being statistically meaningless because of the total collapse at the end of the war).

6 THE PERIOD FROM 1945 TO 1986

6.1 Overview

The four and half decades that have elapsed since the end of the Second World War have witnessed the construction of no fewer than 115 of the 132 dams listed in the 1988 World Register of Dams plus two completed after its publication. This long period of time, in which hydropower development and dam construction have flourished, is not easy to subdivide for lack of truly decisive events that might offer a logical framework. On the contrary, the post-war period is characterized by continuous economic development and political stability. Only two or three years after the total collapse that came with the end of the War in 1945, the Austrian economy was well on the way to recovery. This is clearly illustrated by the development of electricity generation (Fig. 3) and reservoir volume for seasonal storage (Fig. 4), two excellent indicators of hydropower development and dam construction.

Both curves show a significant upturn in 1948 that marks the beginning of a development that was to continue unabated until about 1986, with average annual growth rates for seasonal storage reservoirs of approximately 35 hm³ (Fig. 4). Nevertheless, the curve does alternate between steeper and flatter sections, relating to periods of more concentrated acitivity and also to gaps in the years of completion as listed in the Register. And it is on this basis that the period from 1945 to the present, in which 87% of all Austrian Dams have been built, has been subdivided for greater clarity:

a) 1945–1950: economic reconstruction, completion of dams begun during the war, and construction of a number

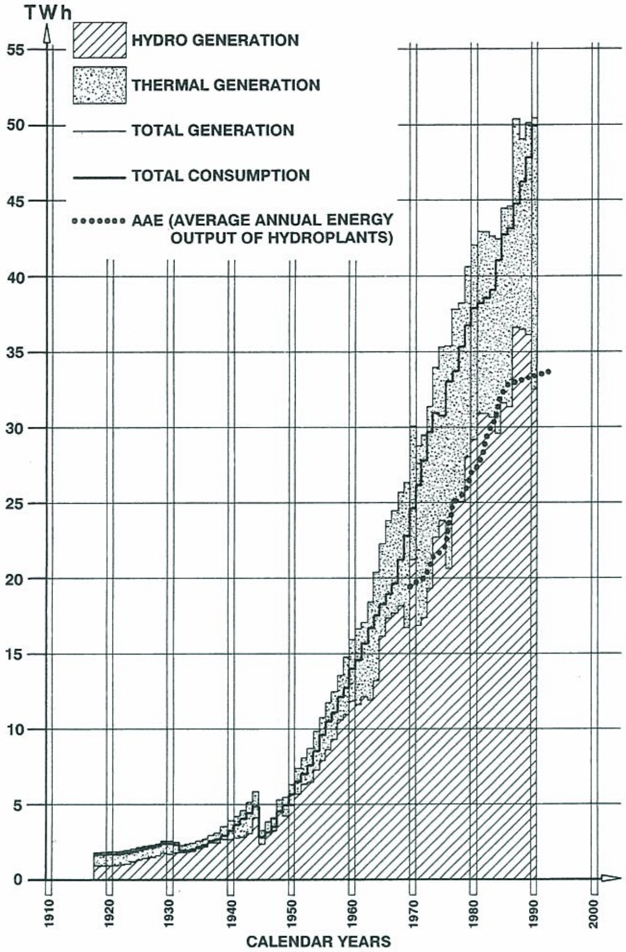


Figure 3 **Austria's electricity production and domestic consumption 1920–1990**

of smaller ones; work started on the first arch dam over 100 m high; a total of 12 dams completed, including 4 barrages

b) 1951–1960: the most active period of dam construction in Austria to date, with a total of 27 dams completed, including 7 barrages

c) 1961–1970: a decade dominated by river barrage construction, with 14 out of a total of 20 dams built; construction of what were then the two highest ever dams; international standards reached in large fill dam engineering

d) 1971–1980: another particularly productive decade, with 27 dams constructed, including 11 river barrages

e) 1981–1986: a further period in which the river barrages dominated, accounting for 12 out of a total of 18 dams built, all of them except one less than 50 m high. The year in which the one higher dam was completed – and with a height of 186 m it is the second highest in Austria – coincides with a levelling off in the growth curve for hydroelectricity generation and storage volume as environmental problems start to make themselves felt.

The last few years, with their clear downturn in dam construction activities, are dealt with in Chapter 7, which also includes a brief review of future prospects.

6.2 The post-war years 1945–50: reorganization and recovery

Of fundamental importance for all hydropower development and dam construction after the war was the complete reorganization of Austria's electricity supply industry under the Second Nationalization Act of 1947, amended in 1987. It distinguishes the following groups of companies (also quoted as owners of dams in the tables in Chapter K):

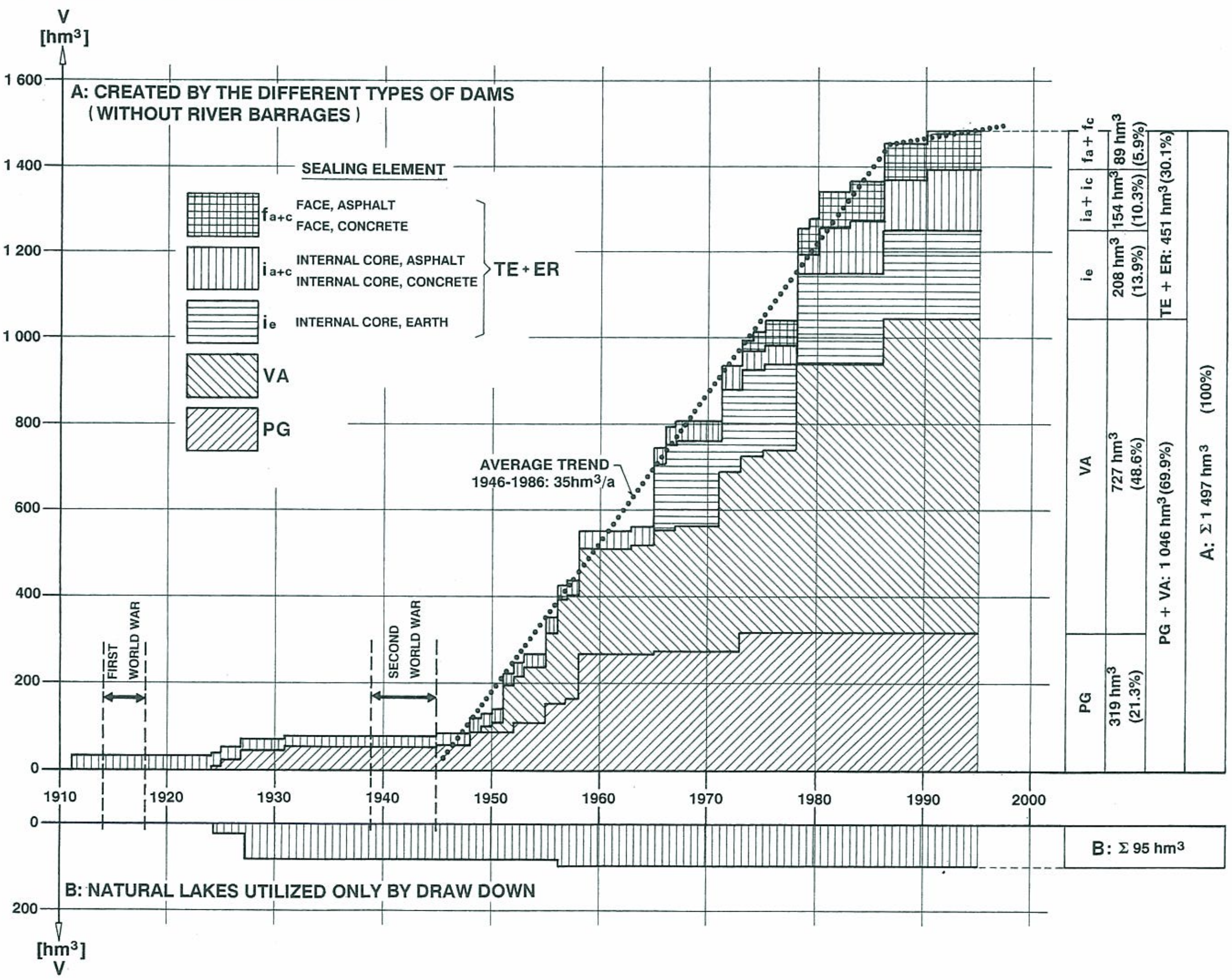
a) Special hydroelectric companies („Sondergesellschaften", e.g. DKJ, DoKW, EKW, ÖBK, ÖDK, TKW, VIW) engaged in the construction and operation of large power plants or schemes along a particular river or in a particular area allotted to them, but not in energy distribution to consumers. These companies are at least 50 or 51% owned by the Österreichische Elektrizitätswirtschafts-

Table 1 Development of total reservoir volume (A = created by dams, B = created by drawdown of natural lakes)

Year	Volume A hm³	Shares of dam types as % of A: PG %	VA %	PG + VA %	TE + ER %	ie %	ia + c %	fa + c %	Volume B hm³
1918	34	28.2	–	28.2	71.8*	–	71.8*	–	–
1928	72	65.8	–	65.8	34.2	–	34.2	–	90
1938	84	70.4	–	70.4	29.6	–	29.6	–	90
1948	124	74.6*	0.7	75.3	24.7	–	24.7	–	90
1958	562	49.1	43.4	92.5*	7.5	1.0	6.5	–	90
1968	818	34.3	35.3	69.6	30.4	24.4*	5.4	0.4	95
1978	1 267	25.1	50.3	75.4	24.6	16.4	3.5	4.7	95
1988	1 463	21.8	49.7	71.5	28.5	14.2	8.2	6.1*	95
1991–95	1 497	21.3	48.6	69.9	30.1	13.9	10.3	5.9	95

* Maximum increment per decade

Figure 4 Development of total reservoir volume



Aktiengesellschaft (Verbundgesellschaft) which itself is at least 51% owned by the Federal Republic of Austria, and is also responsible for the construction and operation of the Austrian supergrid.

b) Regional hydroelectric companies („Landesgesellschaften", e.g. EVN, KELAG, OKA, SAFE, STEWEAG, TIWAG, VKW) at least 51% owned by the regional authorities of the Federal Provinces. These companies are responsible for the electricity supply of their respective regions, i.e. for the construction and operation of power plants as well as for energy distribution, and they are allowed to possess shares in the "Sondergesellschaften" and in power plants constructed by them.

c) Municipal utilities („Stadtwerke", e.g. EWI) owned by the municipal authorities of the capitals of some of the Federal Provinces.

In addition to the above groups, there are the Austrian Federal Railways (ÖBB = Österreichische Bundesbahnen) which also operates a separate network at 16 2/3 c.p.s, industrial self-suppliers, and smaller utilities.

For planning the dams as well as for supervising construction work, the companies in groups a) and b) mainly make use of their own personnel.

Thus they can rely on their experience with existing plant and their profound knowledge of local details, while the disadvantage of an irregular workload is largely compensated by frequent sharing of know-how and reciprocal assistance among the companies. Sometimes, e.g. for special tasks, consulting engineers are also assigned. Construction work is allocated to contractors by tender. Gates or steel linings as well as mechanical and electrical equipment for the power stations is bought by tender directly from the manufacturers. Construction and assembly is coordinated and supervised by the owners.

In the framework of the general reconstruction effort in the aftermath of the severe damage caused by aerial bombing and the general ravages of war, energy requirements soon started to increase rapidly, requiring completion of those plants that had not been finished during the War, and especially the two dams for the Silvretta reservoir (see Chapter 5) and the first of four barrages on the Enns (*Staning, Mühlrading, Ternberg and Grossraming*).

Working under great difficulties and in austerity conditions that are almost unimaginable today, four small arch dams were constructed on the cement-saving Gerlos model. The 53 m high *Salza Dam*, with an overflow spillway covering the full length of the crest, serves a small independent seasonal storage plant in Styria. The 45 m high *Ranna Dam* was built to provide weekly storage for an existing run of-river plant with pumped storage from the Danube. It was the first non-Alpine dam to be built in Austria. The 58 m high *Hierzmann Dam* for the Teigitsch scheme to the west of Graz was remarkable for the design effort required to cope with a very unsymmetrical valley cross-section. The 34 m high *Bächental Dam*, whose reservoir has since silted up, was built not for storage but merely to provide the head needed for a diversion to the Achensee. It is nevertheless of engineering interest as a very slender cupola dam that was prestressed through pressure grouting the radial joints to reduce bending tensile stresses. The experience gained with these small dams and the success of the design had an impact on what was by far the most important project started in this period, namely the 120 m high *Limberg Dam*. The decision already taken in 1944 in favour of a slightly unsymmetrical double-curvature arch dam requiring 450 000 m³ of concrete was confirmed in 1948, and construction began.

6.3 The decade from 1951 to 1960

With the completion of the Limberg Dam in 1951, Austrian dam engineering had caught up with the international state-of-the-art in the design and construction of large arch dams, from calculations and concrete technology to dam instrumentation and monitoring. The next step was construction of the other four large concrete dams in the Glockner-Kaprun scheme. In 1952 the two dams for the intake reservoir for the Möll diversion were built, which supplies the scheme with over half of total inflow. The *Margaritzen Dam* is a 40 m high slightly curved gravity dam, functioning as a cheap spillway at the same time. The 93 m high *Möll Dam* is a more interesting design as a relatively thin arch dam placed on a 36 m high foundation block. Because of delays experienced in the excavations, the upper part of the foundation block was concreted onto the debris as a vertical arch and the underlying gorge cleared down to the bedrock while the concreting work began on the dam above. In 1951–55 two dams were built, one on each side of a huge rock formation, to form the upper of the two seasonal storage reservoirs for the Kaprun scheme. The *Drossen Dam*, with a height of 112 m the higher of the two, is a constant-angle design with varying thickness along horizontal sections. Construction was preceded by excavation of an unusually large volume, namely 310 000 m³, of debris and rock for a dam volume of 350 000 m³. The 104 m high *Mooser Dam* is an arched gravity dam. The ridge on which the dam is located was of poor material and was also rather narrow for the required height. For that reason the downstream face is steeper than in the case of a pure gravity dam, so that the slight arch effect was essential for the stability of the design. An inspection gallery concreted straight onto the foundation, as is the case with all the Kaprun dams, is designed to intercept uplift pressure. Both dams feature extensive grout curtains over 120 m deep. A considerable improvement to concrete technology was the introduction of the Rheax procedure for hydraulic separation of the sand fraction finer than 1 mm and control of any excessive silt content. With the visual impact of its large dam structures, the Glockner-Kaprun hydropower scheme became the symbol of recovery in the Second Republic for an entire decade, and for many years beyond it formed the backbone of the country's electricity supply system.

Less prominence was given to the only non-Alpine group of reservoirs in Austria which were built at about the same time, namely three dams on the River Kamp in Lower

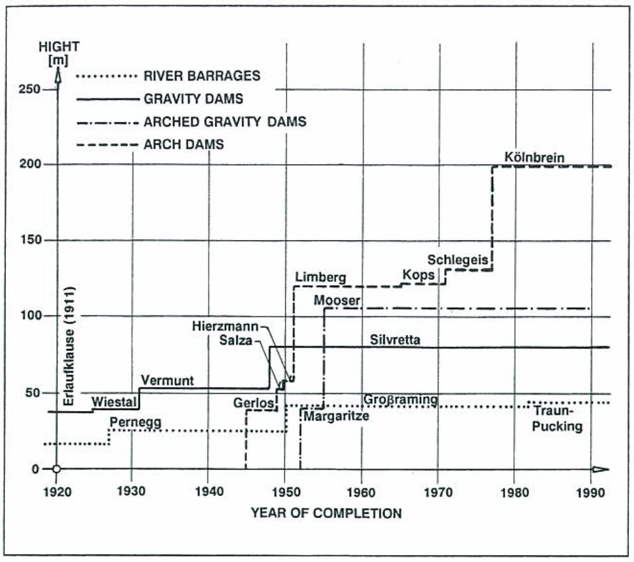


Figure 5 Development of highest concrete dams by types

Austria, which retain about 68% of total annual flow for winter generation. In combination with pumped storage, this enables the regional electric utility to cover the winter peaks.

The *Dobra Dam*, which was built in 1950–52, was the first constant-radius arch dam to be built in Austria. The geology at the right abutment featured severely folded and crushed gneisses, with a high proportion of mica schist and loam-filled fissures, which could neither be consolidated nor sealed and therefore necessitated deep excavation. Subsequently the dam was prestressed using the radial joint pressure grouting method developed at the Bächental Dam. In December 1953 there was an incident involving the bottom outlet tunnel in the left embankment, which has taken on a particular significance in retrospect (see Malpasset). The power conduit had been closed to permit repairs to be carried out on a pipe bridge, and water was discharged into the tailwater via the bottom outlet instead, which is never otherwise under load. Leaks in the bottom outlet tunnel developed and led to the build-up of water pressure in the rock joints, which – after ten days of operation – caused shear failure in a rock slab measuring about 30 x 30 m. This resulted in the development of an 80 cm crack, through which water was lost at a rate of 100 m³/s until the reservoir was empty. To solve the problem, the bottom outlet was subsequently lined with steel.

Thurnberg-Wegscheid, which was built at about the same time, is only a small dam, with a 26 m high concrete gravity section serving as a free overflow spillway, but its 200 m long embankment section features the only upstream concrete face in Austria. The *Ottenstein Dam* built in 1953–57, on the other hand, is a 65 m high constant-angle dam. Also, the powerhouse, which is equipped for pumped storage operation, is located in the immediate vicinity, making this one of only two dam power plants in Austria. In addition, Ottenstein was the first dam in Austria in which coils were embedded in the concrete for artifical cooling. Following the incident with the rock slide at Dobra, great care was taken to provide adequate drainage for the rock spur that forms the right abutment.

From 1954 to 1958 four smaller dams were built for the Reisseck-Kreuzeck scheme in Carinthia. The attraction for the hydropower engineer was the presence of four small cirque lakes at high altitude in the Reisseck mountains, which provide an immense head for a very short horizontal distance of the waterways. (The single stage head of 1 772 m from the Kleiner Mühldorfersee reservoir to the Kolbnitz power station is a world record that has stood since 1957.) As the catchments are very small, they are filled by pumping from diversions at lower horizons, actually including transbasin diversions from the Kreuzeck mountains across the valley.

The biggest of the four structures is the *Grosser Mühldorfersee Dam*, a 47 m high gravity dam. It is curved on plan to suit the rocky ridge on which it is sited and which originially impounded the natural lake. A large base gallery with a sloping elliptical cross-section provides relief from uplift pressure and also permitted 11% savings in concrete to be achieved – an important feature in view of the 65 km hauling distance from the Drau Valley for all aggregates. Prefabricated vacuum concrete slabs served both as formwork during concrete placement and as a permanent facing. The joints between the slabs were only sealed successfully after repeated problems with ice impacts. The dams built for the *Kleiner Mühldorfersee* and *Hochalmsee* are very similar in design but somewhat smaller (while the fourth, at *Radlsee*, is a rockfill dam). A base gallery was also incorporated in the Bemposta arched gravity dam in Portugal (1964) and in the new Tauernmoos Dam built in 1973.

In 1950–58 Austrian Railways increased the seasonal storage volume of their Stubach scheme by impounding three high-altitude lakes, namely *Weissee, Amersee* and *Salzplattensee*, by means of small gravity dams.

Another natural lake that was enlarged by impounding, but this time of a quite different magnitude, was *Lünersee* in Vorarlberg. The lake had first attracted the attention of hydropower engineers back in the Twenties. In 1920, lake level drawdown was prepared by driving an outlet tunnel sited at a depth of 50 m, and in subsequent years the narrow and steep rock sill composed of dolomite forming a natural barrier to the north was systematically investigated and sealed. The Lünersee gravity dam was eventually built in 1955–58 to service a high-capacity high-head pumped storage station needed to increase total generating capacity of the Upper Jll scheme. On plan, the dam, with a maximum height of 28 m, follows an irregular multiple curve along the line of the narrow rock sill with its steep northern face.

In addition to the concrete dams which were still dominant during this period, the first modern earthfill dam, the 41 m high *Freibach Dam*, was also built – with a central core, transition zones and shell zones – for a small peaking plant with additional pumped storage in a side valley of the Drau in Carinthia. Site geology precluded construction of a concrete dam and required extensive sealing works in the left abutment, with a concrete cut-off wall constructed with the help of mining techniques and completed with a grout curtain. Another structure worthy of mention is the *Rotgüldensee* rockfill dam built in 1956–57 to impound a small natural lake in the province of Salzburg. Although only 18 m high – and now incorporated in the upstream shoulder of the new Rotgüldensee dam, which was completed recently – it was nevertheless an innovative project at the time, with a hand-placed inclined asphaltic concrete core as a sealing element.

The river barrages constructed in this period (see Chapter H, Fig.3) include the first two built for hydropower development on the Danube. They are located on the reach where the Danube has cut a narrow valley through the granite gneisses of the Bohemian Massif. *Jochenstein* (132 MW) was built as a binational plant on the border reach with Germany, mainly at the instigation of the German Rhein-Main-Donau AG, and went on stream in 1955. It was followed in 1954–59 by *Ybbs-Persenbeug*, after permission had been granted by the occupying powers, who finally withdrew from Austria in 1955, to utilize the beginnings made to the cofferdams for the construction pit in 1938. These two barrages marked the start of a step by step programme of development of Austria's most important hydropower resource, which continued unopposed until 1984 and is still far from complete. Also, hydropower development of the border reaches of the Inn between Austria and Germany was continued by the binational company set up for that purpose, with stages at *Braunau* (1953, 96 MW) and *Schärding* (1961, 96 MW).

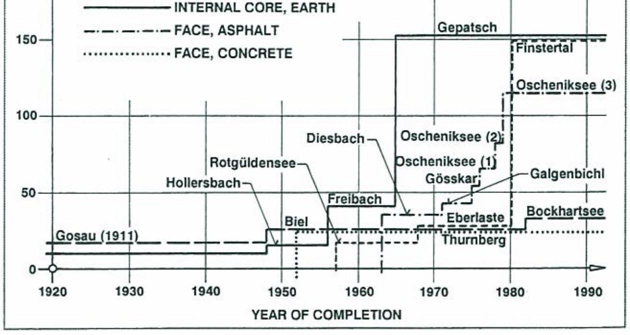
This decade also saw the construction of a few schemes of a type that is rare in Austria, namely medium-head plants incorporating a weir without a reservoir that only just meets ICOLD's 15 m height criterion, plus a long power tunnel as a diversion to provide the necessary head. The first in fact even qualifies for a high-head classification, namely *Braz* (30 MW), which was built by the Austrian Railways in 1953 as the lower stage for the Spullersee seasonal storage power plant. It has a head of 314 m and features Austria's first underground powerhouse. It was followed in 1955 by a plant of twice the generating capacity at *Hieflau* (63 MW) on the Enns, which features a daily storage reservoir at the end of the pressure tunnel, and *Prutz-Imst* (82 MW) on the Inn in Tyrol, which was built in 1956 with the second underground powerhouse. To cope with the difficult conditions encountered in the construction of the pressure tunnel, the basic principles of what has come to be known as the New Austrian Tunneling Method (NATM) were developed under the guidance of L. v. Rabcewicz based on the combined application of shotcrete and rock bolts to activate the supporting effect of the surrounding rock. H. Lauffer also developed his rock classification system based on stand-up time as a function of the span of the unsupported mass. *Schwarzach* (120 MW), on the Salzach, which was completed in 1958, involved the construction of a 17 km long pressure tunnel, which represented a further step forward in the development of the NATM. The plant benefits from the large seasonal storage reservoirs of the Kaprun and Stubach schemes and features a compensating basin of 1.5 hm^3 active storage at the end of the pressure tunnel.

6.4 The decade from 1961 to 1970

In this decade run-of-river power plants dominate the statistics, including two on the Danube, namely *Aschach* (1963, 286 MW) and *Wallsee* (1968, 210 MW). After an interval of more than fifteen years, hydropower development also continued on the Drau, at *Edling* (1962, 70 MW) and *Feistritz* (1968, 80 MW), while development of the lower reaches of the Inn was completed with the construction of a binational plant at *Passau-Ingling* (1965, 86 MW) slightly upstream from the confluence with the Danube. On the Enns, no fewer than six new stages came on stream, and work began on the last of a total chain of 14 power stages. Smaller plants were built on the Mur, Salzach and Traun.

As far as storage dams are concerned, this decade marked the coming of age of embankment dam engineering. Following the sporadic use of this dam type for a number of minor structures (Gosau 1911, Bieler 1947, Hollersbach 1949, Rotgüldensee 1957, and Radlsee 1958) and the still relatively small precursor of Freibach (1958), the construction of the *Gepatsch Dam* in 1961–64 for the Kaunertal power station in the Tyrol – the tenth highest embankment dam in the world when completed – represented a vast step up in terms of size, and brought Austria up to the international state-of-the-art in embankment dam engineering. With three extensive diversions to provide additional inflows, the Gepatsch reservoir, with an active storage of 140 hm^3, was Austria's biggest reservoir for two decades, and the dam is still the highest fill dam in the country. Mainly for reasons of economy, preference was given not to a concrete structure, but to a rockfill dam with steeply sloped shoulders and a relatively slender central core sited on excellent foundation rock. The dam is interesting for its core element of processed kiln-dried talus material, with added bentonite on the upstream side, for the decision to permit winter working on fill placement, and for its generous instrumentation as a source of valuable information on the behaviour of high rockfill dams. A well known incident occurred shortly before completion of he dam during first filling in the summer of 1964, involving a slide affecting a one-kilometre length of the left reservoir slope in the proximity of the dam. The successful investigation of the causes, the control measures adopted and the stabilization of the movements after a total horizontal displacement of 11 m

Figure 6 Development of highest fill dams (earth- and rockfill) by type of sealing element



are discussed at length in the literature. Overshadowed, as it were, by the Gepatsch dam, the 36 m high *Diessbach Dam* was constructed at about the same time in the Calcareous Alps of Salzburg Province to provide seasonal storage for a small high-head stage. This dam is of interest because it was the first to incorporate an upstream membrane of bituminous concrete as a sealing element, which had previously been used only for a number of artificial reservoirs, headrace canals and dykes along backwater areas. Such bituminous concrete membranes were to play a significant role in dam construction in the Seventies.

Another noteworthy project at that time was the *Durlassboden* earthfill dam, which was built in 1965–66 to provide seasonal storage plus a small upper stage for the Gerlos power plant which was built during the War. It thus put into effect an idea that was thirty years old, which could not be tackled earlier, however, because of the very problematical foundations at the dam site. The left valley flank is composed of green schist and phyllites, while the right flank is formed by an early rockslide mass, and the river channel is filled with very heterogeneous material over 130 m deep in the middle of the valley, fortunately with a silt layer at a depth of 30–50 m. Taking account of experience gained with the big grout curtains installed in alluvial and moraine materials at Sylvenstein, Serre-Ponçon and Mattmark, it was possible to seal this difficult foundation with a grout curtain comprising up to eight rows of boreholes, plus pressure relief wells and a heavy weight berm of the downstream side.

In spite of its height of only 28 m, the *Eberlaste* earth dam, which was constructed in 1966–68 for the Stillupp reservoir forming part of the Zemm-Ziller scheme in the Tyrol, is also of particular interest for the solution adopted to overcome difficulties with the foundations. At the only feasible site for a dam, between steep valley flanks, the valley is filled to a depth of over 150 m with alluvial and lacustrine deposits. In the immediate vicinity of the dam foundations, there is silty sand located in the middle of the valley, with talus material and boulders towards the steep valley slopes. This zone was sealed with a slurry-trench cut-off wall with a depth of up to 52 m to provide adequate sealing to avoid any risk of erosion and to reduce seepage losses to an acceptable level. Because of the great heterogeneity of the subsoil materials and the anticipated settlement, a slurry-trench cut-off was preferred to a grout curtain. The vertical asphaltic concrete core of the dam connects with the foundation cut-off. The core has withstood differential settlements of up to 2 m without damage, thanks to the additional provision of pressure relief wells and a weight berm.

In order to add an additional reservoir to the Upper Jll scheme in Vorarlberg, the *Kops Dam* was built in 1961–65, comprising two structures, namely a 122 m high arch dam incorporating a 50 m high thrust block for the left abutment, which at the same time forms the right abutment for an adjoining 33 m high gravity dam. Apart from the fact that the arch dam was the highest concrete dam in Austria at the time, it is also remarkable for the horizontal geometry based on parabolas of the 4[th] order, plus very generous instrumentation using modern telemetry systems.

The 48 m high *Raggal* gravity dam, also in Vorarlberg, which was built in 1965–67, was the first major dam to be embedded into the flysch in Austria and also the first to be constructed using a special cement with 55% interground blast-furnace slag.

Experience gained with this cement proved very useful for the concreting work involved in the construction of the *Schlegeis* arch dam, which was begun in 1967 and completed in 1971 to form the bigger of the two seasonal storage reservoirs for Austria's second biggest storage power scheme in terms of capacity and annual generation, namely the Zemm-Ziller scheme in the Tyrol. (The

Table 2 Distribution of Austrian dams by type, height and period

Period of completion	Before 1929		1930–1949			1950–1969					1970–1989					1990 or n.c.			Total completed or n.c.					
Dam-type / Height m	15 to 29	30 to 59	15 to 29	30 to 59	60 to 99	15 to 29	30 to 59	60 to 99	100 to 149	>150	15 to 29	30 to 59	60 to 99	100 to 149	>150	15 to 29	30 to 59	60 to 99	All heights	15 to 29	30 to 59	60 to 99	100 to 149	>150
River barrages	1	–	7	1	–	16	6	–	–	–	22	8	–	–	–	1	–	–						
	1		8			22					30					1			62	47	15	–	–	–
Gravity	4	3	2	2	1	2	6	–	–	–	1	2	–	–	–	–	–	–	23	9	13	1	–	–
Arched, gravity	–	–	–	–	–	–	1	–	1	–	–	1	–	–	–	–	–	–	3	–	2	–	1	–
Arch	–	–	–	2	–	–	5	2	3	–	–	4	–	1	2	–	–	–	19	–	11	2	4	2
Concrete dams (without riv. barr.)	4	3	2	4	1	2	12	2	4	0	1	7	–	1	2	–	–	–	45	9	26	3	5	2
	7		7			20					11													
Embankment dams, Sealing element: ia	–	–	–	–	–	1	–	–	–	–	–	–	–	–	1	–	1	1	4	1	1	1	–	1
ic	1	–	–	1	–	2	–	–	–	–	–	1	–	–	–	–	–	–	5	3	2	–	–	–
ie	–	–	1	–	–	–	1	1	–	1	–	–	1	–	–	–	–	–	5	1	1	2	–	1
fa	–	–	–	–	–	–	1	–	–	–	–	8	–	1	–	–	–	–	10	–	9	–	1	–
fc	–	–	–	–	–	1	–	–	–	–	–	–	–	–	–	–	–	–	1	1	–	–	–	–
Embankment dams	1		0	1	1	0	4	2	1	0	10	9	1	1	1	0	1	1	25	6	13	3	1	2
	1		2			9					12					2								
Total	6	3	10	6	1	22	20	3	4	1	23	24	1	2	3	1	1	1	132	62	54	6	6	4
	9		17			50					53					3								

second storage reservoir, Zillergründl, was built 15 years later). The crest length/height ratio of 1: 5.6 of the double-curvature arch dam reflects the unusual length of span for this type of structure, which – with a concrete volume of 980 000 m³ – was considerably more economical than a gravity or embankment dam. The horizontal arches were designed as conic sections with a transition from a hyperbolic shape in the upper part to ellipses with their longitudinal axes transverse to the valley at the lower level.

6.5 The decade from 1971 to 1980

Following completion of the Schlegeis arch dam in 1971, the next large concrete dam was the new *Tauernmoos Dam* built by Austrian Railways in 1969–73. This more than doubled live storage in the biggest of the seasonal reservoirs in their Stubach storage scheme, permitting transfer of 3/4 of annual inflow to the winter six months. The new dam was simply located across the original stream channel downstream from the existing dam as integration of the old structure in a new one 20 m higher was not considered advisable. Although only 55 m high, the new dam, which is 1 100 m long, is remarkable for its unusual plan, which is at best comparable with Lünersee. The choice of a gravity dam was dictated by the requirements of the site, and the dam also features an enlarged base gallery on the model of the dams in the Reisseck-Kreuzeck power scheme. In the main dam section spanning the valley, grouting was performed on the radial joints to activate the arch effect. Alignment of the right lateral dam section smoothly follows the rocky ridge that closes the lake basin.

Table 3 Austrian dams higher than 80 m in order of height, with rank in terms of dam volume and waterload

Order of height	Name of dam	Type	Height m	Concrete volume 1 000 m³	(rank)	Fill volume 1 000 m³	(rank*)	Waterload 1 000 MN	(rank)
1	Kölnbrein	VA	200	1 580	(1)	–		54	(1)
2	Zillergründl	VA	186	1 370	(2)	–		41	(2)
3	Gepatsch	ER ie	153	–		7 100	(1)	26.7	(4)
4	Finstertal	ER ia	150	–		4 500	(2)	11.9	(5)
5	Schlegeis	VA	131	960	(3)	–		19.5	(3)
6	Kops	VA	122	663	(5)	–		12.4	(6)
7	Limberg	VA	120	446	(6)	–		9.2	(7)
8	Oschenik	ER fa	116	–		2 300	(4)	5.0	(9)
9	Drossen	VA	112	355	(8)	–		7.5	(10)
10	Mooser	PG-VA	107	665	(4)	–		10.1	(8)
11	Möll	VA	93	35	(–)	–		0.8	(–)
12	Bolgenach	TE ie	92	–		1 350	(6)	4.0	(14)
13	Feistritzbach	TE ia	88	–		1 600	(5)	3.8	(12)
14	Durlassboden	TE ie	85	–		2 520	(3)	8.1	(11)
15	Silvretta	PG	80	407	(7)	–		5.8	(13)

* without river barrages

The *Kölnbrein* arch dam, whose 200 m height makes it the highest dam in Austria, was built in 1974–78 for Austria's biggest seasonal storage reservoir, with an active storage of 200 hm³, which forms part of the big three-stage Malta storage power scheme (with additional pumped storage for the upper and main stages). Since first filling in 1978, the very slender arch dam (1 525 000 m³ for a hydrostatic load of 52 000 MN) has suffered problems with cracking in the lower-most part, high uplift pressures and seepage losses. Complex repair works lasting a number of years have reduced the uplift pressures and seepage, thus permitting the continuation of safe operation with only a slight reduction in retention water level, but a permanent satisfactory solution to the problems has not been achieved. Investigation of the causes of the damage (high shearing stress in combination with residual stresses resulting from the construction processes leading to oblique principal tensile stresses and cracking) has been largely completed, thanks above all to the installation of excellent instrumentation systems during construction and repeated subsequent additions. On the basis of the results, a 70 m high supporting structure, with adjustable Neoprene bearing pads, was constructed in 1989–91 to absorb the shear forces in the lower section of the arch dam. The literature includes detailed discussion of the dam with regard to construction, faults and repair works.

Of the small concrete dams constructed during this period, the *Klaus* arch dam, built in 1975 with a small power station located at the dam toe, and the *Sölk* arch dam, built in 1978 with an overflow spillway, are worthy of mention.

Table 4 Flood retention basins meeting ICOLD criteria but not registered

Name	(Province)	Year	Stream	Dam type	Height m	Res. cap. hm³
Wernersdorf	(ST)	1986	Weisse Sulm	PG (VA)	24.5	0.27
Teufenbach	(ST)	1985	Thadabach	PG	23	0.21
Langenlois	(N)	1959	Loisbach	TE ic	17	0.50
Altmannsdorf	(O)	1984	Pram	TE ic	15	1.1
Lichtenwörth	(N)	1985	Leitha	TE	11	2.2

The embankment dams built in the Seventies include a group of seven dams with upstream membranes of asphaltic concrete. Located at altitudes of between 1 690 and 2 400 m, the group comprises the *Wurten, Oscheniksee, Hochwurten* and *Grossee Dams* in the Fragant storage power scheme in Carinthia, the *Galgenbichl* and *Gösskar* in the Malta scheme and the dam for the *Latschau II* compensation reservoir in the Upper Jll scheme, followed in 1980 by the *Längental Dam* for the Sellrain-Silz scheme (and finally, in 1983, by the *Zirmsee Dam*, located at an altitude of 2 530 m, for the Fragant scheme). Of these dams, the Oscheniksee is of particular interest in view of the fact that it was raised in four stages to its projected height of 116 m above the downstream toe, with a height of 61 m for the upstream membrane. This technique is advantageous for dams located at high altitudes where suitable core material is lacking. In particular it permits the dam to be operated at an early stage during construction, with subsequent raising of the dam; the technique utilizes the entire dam body for support, and protects it from wetting-drying cycles in the case of small reservoirs with frequent lake level oscillations. Valuable experience has been gained on the susceptibility to damage of such membranes and on suitable repair methods in the extremely severe conditions encountered at high altitudes.

Bolgenach Dam, built in 1976–78 on the Weissach in Vorarlberg, is a 92 m high gravel and rockfill dam with a central moraine core, providing short-term storage for the medium-head Langenegg power station. The central core is vertical over its upper portion, while the lower part is inclined towards upstream and widens downwards. This ensures connection with a sloping impervious marl bed intercalated between solid sandstones, which makes special foundation treatment superfluous.

In 1977–80 the *Finstertal Dam*, for the seasonal storage reservoir of the Sellrain-Silz power scheme, was built in the Tyrol at an altitude of 2 325 m where the severe climatic conditions only permitted placement on about 120 days a year. The dominating feature of this very steeply sloped 150 m high rockfill dam is its impervious core membrane of asphaltic concrete, inclined over its full height of 96 m, which is still the second highest such core membrane in the world (but located in the highest embankment ever to be sealed in this way). The dam also incorporates many other interesting features, such as the artificial roughening performed on some extremely smooth rock surfaces which would otherwise have coincided with critical failure surfaces, the comprehensive instrumentation of the core area in particular, including an accessible shaft over the full height of the dam, and excellent compaction of rigorously controlled quarry-run rockfill material placed in layers of 0.7 and 1.0 m thickness producing a very stiff dam body (confirmed by crest settlement at first impounding of a mere 10 cm, with 14 cm horizontal displacement).

The river barrages built in this period include three power stations on the Danube, namely *Ottenstein* (1973, 179 MW), *Altenwörth* (1976, 335 MW) and *Abwinden-Asten* (1979, 168 MW), all with bulb turbines, while work was also begun on a fourth, at Melk. On the Drau, run-of-river plants at *St.Martin-Rosegg* (1973, 80 MW, the only stage on the Drau with a headrace canal) and *Ferlach* (1975, 75 MW) came on stream, and a third, Annabrücke, was started. On the Enns, the last of the 14 stages, *Schönau,* was also completed, and a number of smaller installations were built on the Mur, Salzach and Traun.

6.6 From 1981 to 1986

Only two years after completion of the *Melk* run-of-river power plant (1982, 187 MW), the last hydropower station to be built on the Danube to date, *Greifenstein* (1984, 293 MW), went on stream on completion of a project in which great attention had been paid to preserving the extensive, periodically flooded riverine woodland. In spite of that, work on the next stage, Hainburg, located to the east of Vienna and the biggest of all potential Danube hydropower plants in Austria, with 360 MW and 2 075 GWh, had to be stopped as politicians bowed to the pressure of the environmentalists concerned with the fate of the riverine woodlands there when tree felling started in 1984. That brought hydropower development on the Danube, which had made such excellent progress hitherto and today meets one quarter of Austria's electricity requirements, to a sudden end. On the Drau the run-of-river plants at *Annabrücke* (1981, 90 MW), *Villach* (1984, 24 MW) and Kellerberg (1986, 24 MW) were completed and work was begun further upstream at Paternion, the latter three being the first pier head stations in Austria since Lavamünd was built in 1944. Hydropower development on the Mur, Salzach and Traun continued with a number of smaller run-of-river stations (See Chapter H, Fig. 3).

A lower stage was added to the Upper Jll scheme, namely the *Walgau* power plant (1984, 86 MW), a medium-head stage with a head of 162 m and a 21 km power tunnel that was geologically very challenging.

Otherwise, the only dams for storage reservoirs constructed during this period are one very large dam and five small ones with heights of 23 to 44 m. Of the small dams, the most interesting is the 33 m high *Bockhartsee* rockfill dam, which was completed in 1982 to provide seasonal storage for a planned three-stage power scheme with a total generating capacity of 91 MW. The sealing element is a 25 m high vertical concrete core wall, the first to be built after Bieler Dam at the end of the War. In this case, however, to minimize sliding friction and the consequent build-up of perpendicular forces in the core on settlement of the surrounding fill, and also as an additional seal, a bituminous slip layer (on 4 mm geotextile) was bonded to the core wall on both sides. In order to study the behaviour of this design for incorporation in future projects, including higher dams, full testing was performed in the laboratory and on the site itself.

The 44 m high *Zirmsee Dam* in the Fragant scheme has already been mentioned in the previous chapter, leaving just three small concrete dams serving individual stages, namely *Paal* (arch), and *Verwall* and *Bodendorf* (gravity dams).

By far the largest dam constructed in this period is the *Zillergründl* arch dam, which was built in 1980–86 to provide impounding for the second biggest annual storage reservoir in the Zemm-Ziller scheme (The Francis turbines in the Häusling power station, which is directly supplied from Zillergründl and is also equipped for pumped storage, operate under a maximum gross head of 744 m, which is presently a world record for this type of turbine). For a dam volume of 1 373 000 m^3 of concrete, unusually extensive excavations were required, namely 1 100 000 m^3 of alluvial material from the 40 m deep overburden at the valley bottom, and 610 000 m^3 of rock, especially from the right flank. In view of the problems with cracking experienced at the Kölnbrein Dam, a more sophisticated design was adopted for the lower portion of the upstream dam toe, extending well into the flanks, namely a concrete slab connected to the dam by means of a special sealing element, with the grout curtain at its upstream edge, and a circumferential joint in the arch dam body from the upstream face to the control gallery. Portland cement was blended with 33% fly ash, and part of the water was added as ice to keep placement temperatures down to 5–8°C. This large and impressive dam marked the end of a forty-year period of consistently high growth rates in hydropower development in Austria, which brought an increase in annual average generation from 7 000 to 33 000 GWh, and an increase in storage capacity from 160 to 1 460 hm^3.

7 RECENT YEARS AND PROSPECTS

Since 1986 there has been relatively little hydropower development and dam construction activity in Austria. In addition to a number of smaller run-of-river plants on the Salzach and Mur, a further pier head power station (*Paternion*, 1988) was built on the Drau as the upstream continuation of the existing chain, plus a high-head stage on the upper reaches with daily storage and a 22 km pressure tunnel (*Strassen-Amlach*, 1989). In addition to one small arch dam (*Ginau*, 1987), two rockfill dams with vertical asphaltic core membranes were also built, based in their design and construction on the Finstertal model. The 88 m high *Feistritzbach Dam* (1988–90) provides seasonal storage for the Koralpe high-head plant (50 MW) in Carinthia. The old Rotgüldensee Dam with the inclined asphaltic concrete core wall, which was completed in 1957 (see section 6.3), was integrated into the upstream shoulder of the new 45 m high *Rotgüldensee Dam* built in 1988–90. At the same time the new high-head stage with underground powerhouse at Hintermuhr (65 MW) also replaced the earlier, smaller plant.

The situation in Austria today, with total seasonal storage volume now at 1 485 hm^3 and total stored winter energy at 3 376 GWh, ist that not a single storage dam is under construction – the first time such a state has existed for 52 years. Nor have approvals procedures been completed for any such dam at the present time. Several projects have had to be postponed or abandoned, including the Dabaklamm Dam (in its most recent design alternative a gravel fill dam with a central earth core and a height of 260 m above the downstream toe), because the necessary political support and social acceptance have been lacking in the face of growing environmental awareness. Present activities in hydropower construction are limited to a binational river barrage on the Inn, namely *Oberaudorf-Ebbs* (60 MW, of the pier-head type with bulb turbines, to be completed in 1992), a medium-head plant on the Bregenzerache, *Alberschwende* (28 MW, also to be completed in 1992), and some enlargements and refurbishing of existing plants.

As explained in section 5.3 of Chapter A, this regrettable caesura in the development of Austria's hydropower resources, following decades of progress managed with remarkable and meaningful continuity, is in clear contradiction to continued growth in the demand for electricity, which economic forecasts generally expect to continue in the foreseeable future. Accordingly, there will soon be no alternative but to turn once again to what is fortunately a very considerable undeveloped hydropower potential in Austria, estimated at about 19 000 GWh on the basis of serious studies and research. Given a holistic view of the environment as man's biotope, such a development will truly be environment-friendly and will also have the backing of Austria's many decades of positive experience with hydropower. The one third share of this undeveloped potential that would require storage power plant construction (representing about 70% of the annual energy available from existing such plant) is a guide to the magnitude of the scope for further feats of engineering in the future of dam construction in Austria.

ACKNOWLEDGEMENT

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Chapter C

GEOLOGY AND DAM CONSTRUCTION IN AUSTRIA

CONTENTS

GEOLOGY AND DAM CONSTRUCTION IN AUSTRIA

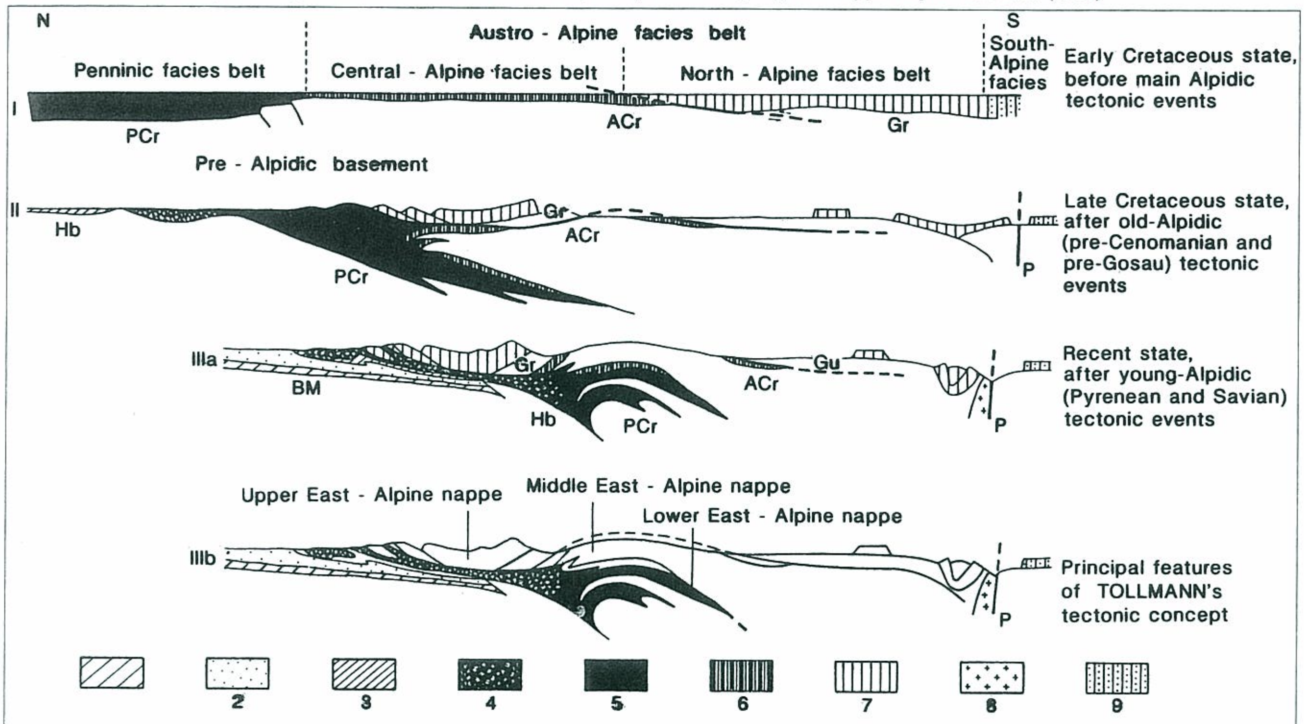
By W. Demmer *

1 A SURVEY OF GEOLOGY IN AUSTRIA

Austria lies for about two-thirds of its territory in the Alps and for its remaining one-third in the so-called Bohemian Massif. Whereas the Alps are a very young mountain system, the Bohemian Massif in the northern part of Austria forms part of an ancient crystalline basement. This difference in ages within Austria's geological structure manifests itself in the morphology of the country. Steeply rising mountains and deep valleys are characteristic features of an Alpine landscape, while the Bohemian Massif is a plateau with rounded hilltops and gentle valley forms. Another difference is that in a very recent geological past (Quaternary), during a total of four great ice-ages, the Alps except for their eastern foothills were covered by a continuous ice shield, while the Bohemian Massif remained free from ice (Fig. 2).

The Eastern Alps extend in a generally east-west direction and show a fairly symmetrical geological make-up. To the north and south of an axial zone which largely consists of crystalline or metamorphic rocks are large limestone chains. These rocks are former marine sediments deposited in a sea-basin south of the present main chain of the Alps. It was during the Alpine mountain-building process in the Tertiary that parts of this ocean bottom were progressively uplifted in the course of a south-to-north compression process, while other parts were subducted and swallowed altogether. Growing compression finally led to a great variety of strata piling up like sheets, or nappes (Fig.1). As part of

Figure 1 Hypotheses of the tectonic evolution of the Eastern Alps (sections I, II and III a; modified by A. Matura 1980 after E. Clar, 1973) and a schematic section (III b) showing the subdivision of the "East-Alpine" (= Austro-Alpine) Unit support by A. Tollmann (1963)



Legend for the sections I, II, and III a:

1 Tertiary rocks of the Molasse Zone
2 Extra-Alpine post-Variscan sedimentary rocks
3 Helvetic Zone and Klippen Zone
4 Flysch Zone
5 Permomesozoic of the Penninic Zone
6 Permomesozoic of the Central-Alpine facies belt
7 Permomesozoic of the North-Alpine facies belt
8 Periadriatic Intrusion
9 South Alpine facies

BM Bohemian Massif
Hb Basement of the Helvetic Zone
PCr Crystalline Basement of the Penninic Zone
Gr Paleozoic rocks of the Gray Wacke Zone
Gu Palaeozoic rocks of the Gurktal Sheet
ACr Crystalline basement of the Austro-Alpine Unit
P Periadriatic Lineament

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this process, the Northern Calcareous Alps overrode the Central Alps from their site of deposition south of the central chain of the Alps.

A remarkable feature is the so-called Tauern window, where mainly three cores of granitic gneiss have risen in a geologically very recent past, piercing the overlying covering rocks. These primarily consist of former marine deposits of Mesozoic age, which are summarized under the term "Schieferhülle" (slate mantle). During the Alpine mountain-building processes, they temporarily sunk to greath depths and were metamorphosed. Much older crystalline rocks of Paleozoic age overlie the geologically young Schieferhülle and especially to the east and west of the Tauern window extend over large areas. They are an impressive manifestation of the Alps being built of a pile of sheets.

2 RECENT TECTONIC AND SEISMIC ACTIVITIES (FIG. 3)

The sheet-thrusting processes appear to have reached a state of relative inactivity in the Eastern Alps, although increased horizontal tectonic stresses in a north-south direction have been established at many locations.

shown neither damage nor deformation and are evidence of a state of relative tectonic inactivity in the Eastern Alps.

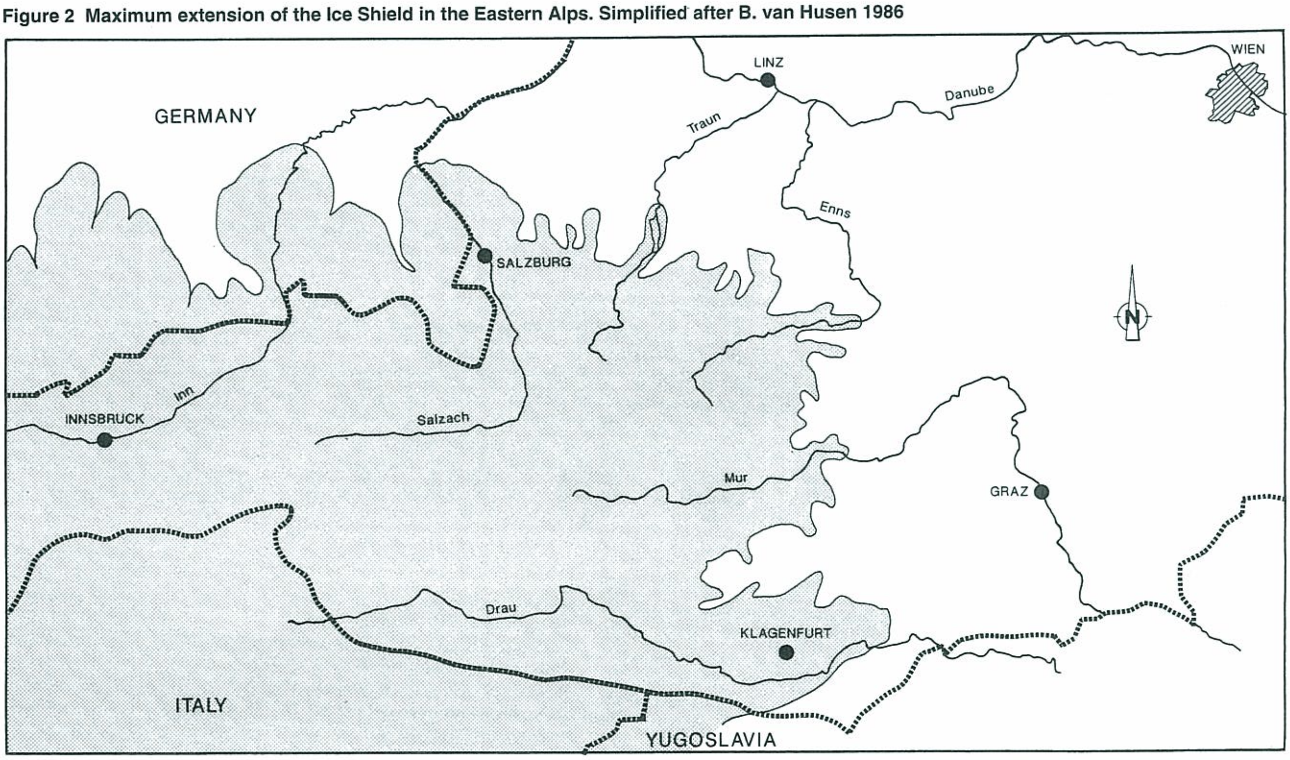
As to seismic activities, statistical evaluation of all the earthquakes recorded in the past in the Eastern Alps shows a zonal distribution of earthquake centres which at first sight does not coincide with the east-west extension of the geological units of the Alpine system of nappes (Fig. 3).

Actually, there is a zone with a large number of epicentres extending from the Vienna basin in a south-westerly direction with the largest intensities being reached near Villach. A second location of high intensities lies in the Inn valley.

Comparison with the main known tectonic fault structures will show that only few large faults exhibit seismic activity.

Although earthquakes with substantial epicentral intensities have been recorded in some regions, the Eastern Alps are considered as having moderate seismicity by international standards, in the light of the low frequency of earthquake events. For this reason, the Austrian authorities impose in most cases a

Figure 2 Maximum extension of the Ice Shield in the Eastern Alps. Simplified after B. van Husen 1986



Vertical tectonic movements have, however, persisted to the present day. They are assumed to amount to 1 or 2 mm per year. Recent vertical movement at the southern edge of the Hohe Tauern mountains is even responsible for damage on the pressure tunnel lining of the Malta hydroelectric scheme.

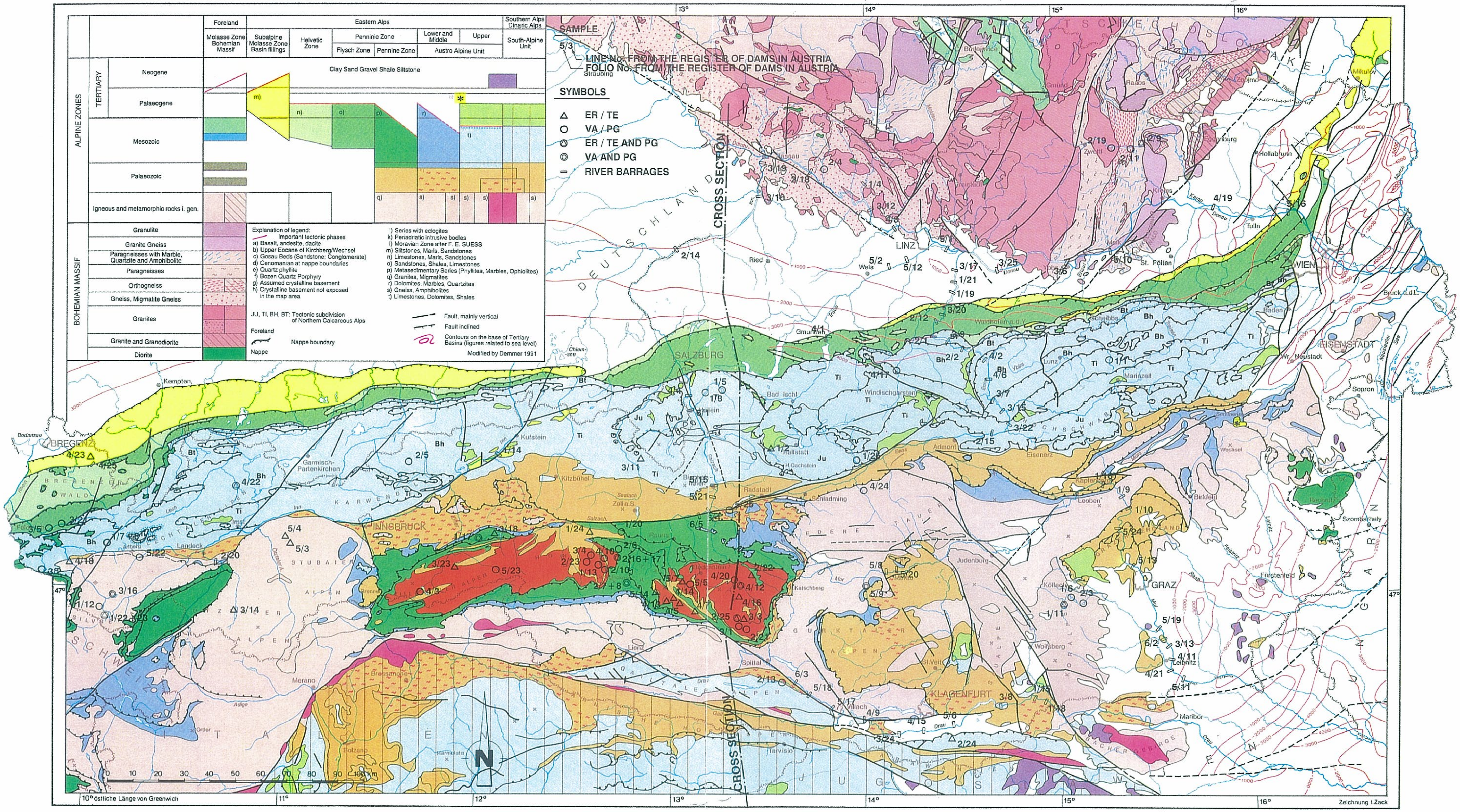
But a large number of engineering structures, as e.g. tunnels, bridges and especially large dams, which were constructed across pronounced fault zones have

horizontal acceleration of 0.04 g to be allowed for in the stability analysis.

The dam nearest to a zone of seismic activity is the Wiederschwing arch dam (25, 2/13) in the south of Austria. It was constructed between 1951 and 1953 and was founded on chlorite-bearing quartzitic phyllites. The original stability analysis did not allow for an earthquake loading case. In the light of present-day knowledge, the design would have to be based on an

GEOLOGICAL MAP OF AUSTRIA 1:1,500,000 (WITHOUT QUARTERNARY)

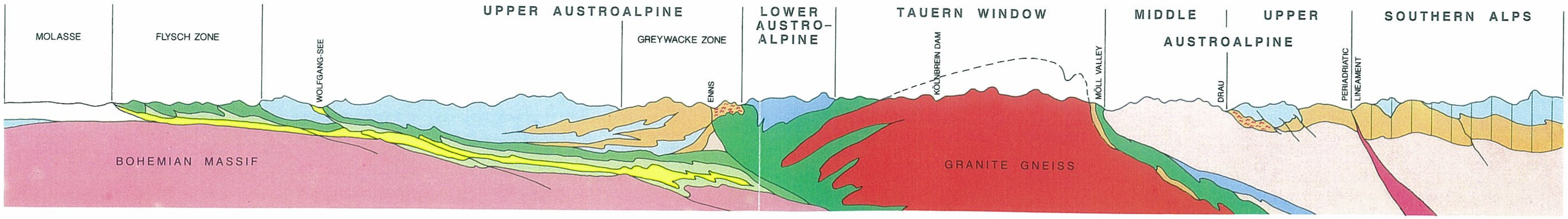
Compiled by P. BECK-MANNAGETTA (Eastern Alps) and A. MATURA (Bohemian Massif)
Modified by W. DEMMER 1991
Edited by Geologische Bundesanstalt, Vienna 1980



Layout and reproduction technique by Geologische Bundesanstalt

SCHEMATIC CROSS-SECTION OF THE EASTERN ALPS

Modified after S. PREY 1976



LARGE DAMS IN AUSTRIA
(including large river barrages)

1/1 ERLAUF-KLAUSE	2/1 TERNBERG	3/1 KLEINER MÜHL-DORFERSEE	4/1 GMUNDEN	5/1 ABWINDEN-ASTEN	6/1 HALLEIN SOHLSTUFE
1/2 GOSAU	2/2 GROSSRAMING	3/2 LÜNERSEE	4/2 WEYER	5/2 MARCHTRENK	6/2 LEBRING
1/3 WIESTAL	2/3 HIERZMANN	3/3 RADLSEE	4/3 SCHLEGEIS	5/3 FINSTERTAL	6/3 PATERNION
1/4 LANGHALSEN	2/4 RANNA	3/4 SALZPLATTEN	4/4 URSTEIN	5/4 LÄNGENTAL	6/4 ST. VEIT
1/5 STRUBKLAMM	2/5 BÄCHENTAL	3/5 LUTZ	4/5 WURTEN	5/5 NASSFELD	6/5 WALLNERAU
1/6 LANGMANN	2/6 LIMBERG	3/6 YBBS-PERSENBEUG	4/6 SCHÖNAU	5/6 ANNABRÜCKE	
1/7 SPULLERSEE SOUTH	2/7 MÖLL	3/7 ESSLING	4/7 OSCHENIKSEE	5/7 BOCKHARTSEE	
1/8 SPULLERSEE NORTH	2/8 MARGARITZE	3/8 EDLING	4/8 OTTENSHEIM	5/8 BODENDORF	
1/9 PERNEGG	2/9 THURNBERG	3/9 LOSENSTEIN	4/9 ST. MARTIN	5/9 PAAL (BODENDORF)	
1/10 MIXNITZ	2/10 WEISSEE	3/10 SCHÄRDING-NEUHAUS	4/10 TAUERNMOOS	5/10 MELK	
1/11 PACK	2/11 DOBRA	3/11 DIESSBACH	4/11 GABERSDORF	5/11 SPIELFELD	
1/12 VERMUNT	2/12 ROSENAU	3/12 ASCHACH	4/12 GALGENBICHL	5/12 TRAUN-PUCKING	
1/13 ENZINGER-BODEN	2/13 WIEDER-SCHWING	3/13 GRALLA	4/13 GROSSEE	5/13 WEINZÖDL	
1/14 KIRCHBICHL	2/14 BRAUNAU-SIMBACH	3/14 GEPATSCH	4/14 HOCHWURTEN	5/14 ZIRMSEE	
1/15 SCHWABECK	2/15 GSTATTER-BODEN	3/15 GROSS-REIFLING	4/15 FERLACH	5/15 BISCHOFS-HOFEN	
1/16 MÖTSCHLACH	2/16 MOOSER	3/16 KOPS	4/16 GÖSSKAR	5/16 GREIFENSTEIN	
1/17 GERLOS	2/17 DROSSEN	3/17 THURNSDORF	4/17 KLAUS	5/17 VILLACH	
1/18 LAVAMÜND	2/18 JOCHENSTEIN	3/18 DURLASSBODEN	4/18 LATSCHAU	5/18 KELLERBERG	
1/19 STANING	2/19 OTTENSTEIN	3/19 PASSAU-INGLING	4/19 ALTENWÖRTH	5/19 MELLACH	
1/20 BÜRG	2/20 RUNSERAU	3/20 GARSTEN	4/20 KÖLNBREIN	5/20 ST. GEORGEN	
1/21 MÜHLRADING	2/21 GROSSER MÜHL-DORFERSEE	3/21 RAGGAL	4/21 OBERVOGAU	5/21 URREITING	
1/22 SILVRETTA	2/22 ROTGÜLDEN-SEE	3/22 WANDAU	4/22 ROTLECH	5/22 VERWALL	
1/23 BIEL	2/23 AMERSEE	3/23 STILLUPP (EBERLASTE)	4/23 BOLGENACH	5/23 ZILLERGRÜNDL	
1/24 HOLLERSBACH	2/24 FREIBACH	3/24 FEISTRITZ	4/24 SÖLK	5/24 RABENSTEIN	
1/25 SALZA	2/25 HOCHALM-SEE	3/25 WALLSEE	4/25 SUBERSACH	5/25 GINAU	

earthquake assumption of 0.08 g because of the nearness of the Villach zone of seismic activity. Check analyses have proved the stability margins included in the design to be large enough even for this additional earthquake loading.

The formulation of an extrapolated extreme earthquake is being discussed.

As to the run-of-river stations on the Drau which are also situated near Austria's earthquake centre at Villach, 0.02 g was assumed in the design of the Villach station and 0.15 g for the Kellerberg and Paternion stations situated a little farther away.

No damage due to an earthquake has so far been recorded at the Austrian dams.

3 GEOLOGICAL ASPECTS IN THE SELECTION OF DAM TYPES

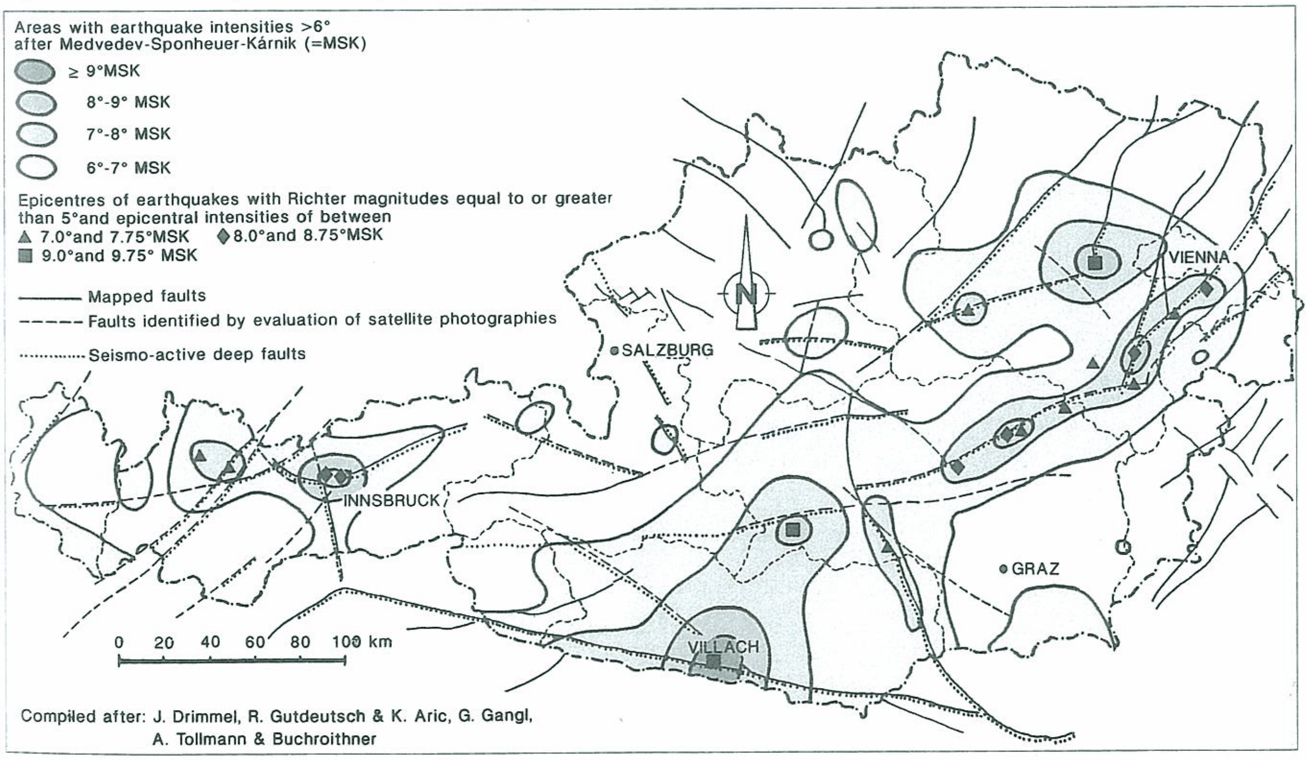
The variable geological history and the complex sheet structure of the Alps has had a great influence on large dam construction in Austria. By their erosional action, or exaration, the glaciers in the Alpine valleys have stripped off the whole Tertiary waste mantle in such a way that sound bedrock is exposed. On the other hand, however, these valleys have been widened to large U-shapes, where embankment or gravity dams have often represented the most suitable dam types for creating artificial lakes, while sites with a morphology lending itself to thin arch dams have been rare. Especially the high arch dams have called for a number of special engineering measures to compensate for the geological and morphological shortcomings of the sites.

In those areas which have remained unaffected by glaciation, that is, the eastern part of the Alps and the Bohemian Massif, dam construction has been faced not only with a waste mantle attaining substantial depths, which has greatly affected site selection and foundation depth, but often also with a lack of suitable aggregates. The gentle relief and the absence of glacier-carved valley basins have generally prevented the deposition of natural gravels. In addition, the eastern foothills of the Alps are mainly composed of metamorphic crystalline rocks whose mica content makes them a material that is not very well suited as concrete aggregate.

For the construction between 1948 and 1950 of the 58-m high Hierzmann dam (17, 2/3), the only thin arch dam located outside the formerly glaciated region, concrete aggregate was brought over a distance of 39 km from low-level sources in the Graz basin. But for the nearby Pack gravity dam (10, 1/11), 33 m in height and constructed in 1929–30, quarried and processed local material was used. The resulting poor concrete quality has meanwhile called for remedial action by means of an upstream facing wall.

The majority of the embankment dams are directly connected to bedrock by inspection galleries or cutoff walls. Actually, in many cases site geology would have allowed the construction of conrete dams instead of the embankments. But dam type selection has generally been governed by considerations regarding availability of construction materials and, hence, economy. Over the last decades, aspects of landscape preservation have increasingly entered planning and design considerations. At present it is an embankment

Figure 3 Maximum earthquake intensities, epicentres of important earthquakes, and main faults in Austria



designed to blend harmoniously with the surrounding landscape that is more readily accepted in Austria than a concrete dam.

It is interesting to note that it has extremely rarely been necessary in Austria to construct a large dam across a fault. This applies both to the dams in the crystalline rocks of the Central Alps and to the few dams in the Calcareous Alps, in the Flysch Zone and in the Bohemian Massif.

The most outstanding exception is the 122-m high Kops arch dam in the western part of Austria, where a 20-m wide shattered zone with fault-gouge inclusions is present in the valley floor. The fault is not active and has been bridged by the arch dam in such a way that there have been no objectionable reactions whatever from dam or foundation. Another fault, although inferior in width, in the left abutment near crest level at the same dam site called for the provision of an artificial abutment block.

4 DAMS IN THE BOHEMIAN MASSIF

Trapezoidal valley cross sections in the Bohemian Massif have led to the selection of arch dams that are either thick arch dams or arch-gravity dams. Examples of these are the dams at Ottenstein (27, 2/19) 69 m in height, Dobra (22, 2/11)with a height of 52 m and Ranna (18, 2/4) with a height of 45 m.

5 DAMS IN THE CENTRAL ALPS

Within the formerly glaciated region of the Alps, the majority of articifical reservoir lakes are situated in the Central Alps. In terms of morphology, this is the area of the highest peaks, which, reaching altitudes above 3 000 m a.s.l. (Grossglockner, 3 789 m a.s.l.), rise to glaciated levels even today. The region of the Central Alps is made up entirely of crystalline rocks.

In this central crystalline zone of the Eastern Alps lie both the highest arch dams, as Kölnbrein (200 m; 54, 4/20) and Zillergründl (186 m; 60, 5/23) or, further to the west, Kops (122 m; 38, 3/16), and the highest embankment dams, as Gepatsch (153 m; 39, 3/14) and Finstertal (150 m; 57, 5/3), as well as the highest gravity dams, Vermunt (9, 1/12) and Tauernmoos (46, 4/10), 53 m high each. In terms of engineering geology the crystalline rocks of the Central Alps present no great problems regarding reservoir watertightness or dam foundation treatment. In a few cases, as at the Gepatsch (39, 3/14) and Durlassboden (42, 3/18) embankment dams, there was a risk of the reservoir slopes being unstable. In actual fact, during first filling of the Gepatsch reservoir, a large slide occurred, which was successfully kept under control by engineering measures. This slide has been repeatedly described in the relevant literature. Landslide risks in this area are a result of the fact that after the retreat of glacier ice, which had had a stabilizing effect on the flanks, zones

Figure 4 Foundation zone for supporting structure viewed from the crest of Kölnbrein Dam

of instability developed in many Alpine valleys. Postglacial slides are also responsible for many narrow passages in what are in fact U-shaped valleys. Even today, slopes tend to be subject to creep, especially in phyllitic zones.

Apart from the valley configuration, the geological structure of the foundation has an essential bearing on dam type selection. Thin arch dams, being the most exacting dam types with respect to foundation properties, have preferably been adopted for sites where hard gneisses, granites and amphibolites prevail. These rock types are the only ones to have withstood erosional attack in such a way that relatively narrow valley sections have locally remained besides basin-like widenings. However, these very hard and massive rocks tend to involve problems regarding combined dam and foundation behaviour.

Aggregate for the concrete dams in the Central Alps was generally obtained from the sandy gravels of valley fills. Exceptions are the Kölnbrein dam (54, 4/20) and the 53-m high New Tauernmoos gravity dam (46, 4/10) with a crest length of 1.1 km – a length which is unusual by Austrian standards –, where concrete aggregate was quarried from the granitic gneisses of the basement zone of the 'Tauern window'.

Embankment types had to be adopted for sites where bedrock is covered with thick overburden. A classical example is the 83-m high Durlassboden earthfill dam (42, 3/18), which was constructed on valley fill consisting of unconsolidated sediments of extremely heterogeneous composition with repeated alternations of moraine, ancient landslide material, former lacustrine deposits and fluvial deposits with fans of talus material descending from the flanks. Particularly careful foundation treatment by means of borehole grouting was required to make up for these shortcomings.

The case of the 28-m high Eberlaste embankment dam (44, 3/23) may be mentioned as a classical example of dam underseepage. Preliminary geological studies penetrated to depths of up to 124 m without reaching bedrock. The deep U-shaped valley carved by glacial erosion had subsequently filled to substantial depth with relatively pervious fluviatile gravels. Although ground moraine is thought to be present underneath, it was not possible to include this in the foundation treatment scheme. Slurry-trench cutoffs up to 52 m deep were provided to lengthen seepage paths. The lateral contacts were sealed by grout holes.

Complex foundation conditions are also present at the Freibach embankment dam (37, 2/24). Apart from the grout holes, concrete cutoffs, cutoff trenches filled with natural material as well as a subsequent thin diaphragm had to be provided to ensure imperviousness. But this dam is not situated in the crystalline zone of the Central Alps but in the Southern Calcareous Zone at the boundary with Yugoslavia.

Zillergründl dam (60, 5/23) is a limit case in Austria with respect to valley fill thickness in combination with a concrete dam. Mainly fine-grained sandy to silty overburden material had to be excavated to a depth of about 40 m in the face of most difficult geology in order to reach bedrock. Excavation slope dewatering and stability problems called for large-scale engineering measures. Zillergründl is also a clear exception among Austria's concrete dams with respect to depth and volume of rock excavation.

6 DAMS IN THE CALCAREOUS ALPS

Whereas there is a single dam – the above mentioned Freibach embankment dam (37, 2/24) – in the Southern Calcareous Alps, there exist several small to medium-sized dams, mainly concrete dams, in the Northern Calcareous Alps. Among them are also the oldest dams in Austria, the 37-m high Erlaufklause gravity dam (2, 1/1) founded on dolomite, which was constructed between 1908 and 1911, and the Gosausee embankment dam (3, 1/2) constructed in 1910–11.

As anywhere else in the world, dam design studies for limestone locations in Austria focused on the permeability problem for the reservoir basins. Detailed hydrogeological studies preparatory to dam design allowed the 53-m high Salza arch dam (15, 1/25) and the 36-m high Diessbach rockfill dam (40, 3/11) projects to be successfully completed in spite of the visible presence of karst phenomena.

On the other hand, however, one of the early dam projects was a clear failure. The reservoir created by the 37-m high Strubklamm gravity dam (5, 1/5) constructed between 1920 and 1924 is so permeable that the dam can only be used for flood retention. As seepage flows escaping from the reservoir through karst fissures have for a long time been utilized for electricity generation by several small enterprises in the neighbouring valley, sealing measures have so far not been taken to avoid potential legal dispute.

The treatment of dam foundation zones has always given satisfactory results at the limestone locations, provided these are situated in rock. It has either been possible to benefit from the natural imperviousness of clays or marls intercalated between the limestone strata in places, as at the Spullersee dams (6a, 6b; 1/7, 1/8), or foundation grouting has been successful. An outstanding example is the foundation treatment scheme developed for the 55-m high Klaus arch dam (50, 4/17). Preliminary geological investigations using water pressure tests in core holes had given extremely high permeability rates in some regions of the dolomitic dam foundation. On the other hand, a perched ground water table was found to exist at different levels. This information was fully allowed for in the design of the foundation treatment scheme, so that only a single line of contact grouting holes were sunk to a maximum depth of 15 m. The idea was that the perched water table should remain unaffected by the drillings. This scheme was successful.

Some minor seepage has been recorded at the Gosausee (3, 1/2) and Freibach (37, 2/24) embankment dams, which are both founded – at least partly – on unconsolidated sediments. Subsequent foundation treatment, though having reduced seepage, has not been able to stop it entirely.

7 DAMS IN THE FLYSCH ZONE

Geologically the Flysch Zone may be regarded as a body of debris from the Central Alps. Hard sandstone beds alternate with soft but usually impervious marls and shales. This sequence from the Cretaceous and in places of early Tertiary age was overridden by Calc-Alpine thrust-sheets, or nappes, in the Tertiary. The dams constructed in the flysch have met with no serious permeability problems due to the presence of shale intercalations. Geological investigation of the shales has mainly been focused on potential sliding risks on the reservoir slopes and abutments.

The largest dam constructed in the Flysch Zone is the Raggal gravity dam (41, 3/21), which is 48 m high and curved in plan.

8 DAMS IN THE SUB-ALPINE MOLASSE ZONE

The Sub-Alpine Molasse is part of the Molasse Zone and forms the northern boundary of the Alps. Its stratigraphy is in places much like that of the Flysch Zone, being composed of clearly stratified sandstones, siltstones, marls and conglomerates. These are marine fluviatile and lacustrine deposits from the Tertiary which then took part in the folding and thrusting processes during the late phases of Alpine orogeny. The 92-m high Bolgenach embankment dam (58, 4/23) is the only Austrian dam in this zone. The design of this structure is exemplary in making careful allowance not only for the available construction materials but for the closely spaced alternation of different beds in the foundation. A main subject of the geological investigations was the stability problem of the reservoir slopes.

9 RIVER BARRAGES

River barrages have been founded on a great variety of different geological strata. On the rivers Danube, Enns and Inn, the majority of the dams have been founded on rock or strongly consolidated sediments from the Tertiary. Similarly favourable locations have been found for the barrages in the lower course of the river Drau and partly for those on the Mur. But the five barrages that have so far been placed in operation on the river Salzach between Schwarzach and Bischofshofen have all been founded on young valley-fill gravels, as has also been inevitable at most of the project sites on the Middle Drau. Excavation dewatering during construction was a main geological concern. Major problems were encountered in this respect at Annabrücke (..., 5/6) on the river Drau, where inclined beds of young sediments were quite unexpectedly encountered which preliminary investigations had failed to identify. The sediments were deposited during the disintegration of an ice-age glacier, which lasted for several centuries.

On the Upper Drau, power schemes even had to be founded on relatively soft silts. With a cellular arrangement of slurry trench cutoffs it was possible to make up for the shortcomings of the ground in respect of the reduced loadbearing capacity and safety from sliding.

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Chapter D

CONSTRUCTION OF DAMS IN AUSTRIA

AUTHORIZATION PROCEDURE

CONTENTS

CONSTRUCTION OF DAMS IN AUSTRIA

AUTHORIZATION PROCEDURE

By F. König and E. Schmidt*

1 LEGAL BACKGROUND

1.1 Water Law

Dams typically carry a high risk of loss and damage in case of failure. Even small dams may put human lives at risk. Accordingly it is necessary to ensure that they conform to the highest possible level of operational safety. In former times the Water Code did not make any provision for examining or evaluating water utility structures, apparently because they had been tested over centuries of use on a modest scale. Technological progress since the beginning of this century, however, has advanced water utilization schemes to the limits of what is technically feasible, so that issues of design safety and protection of downstream populations have become crucial to authorization procedures under Water Law. Responsibility for the safety of dams was vested in the Water Authorities from the very beginning by the Imperial Water Law, which was passed in 1869 and remained effective for more than five decades without any amendments. Together with the implementing statutes passed by the provincial diets soon after, it greatly promoted the development of water resources. Under the constitutional amendment of 1925, authority and responsibility were conferred upon the national authority alone, with the result that the Federal Water Law of 1934 was enacted as one body of legislation that was applicable to the whole territory of Austria.

The Water Authorities are part of Austria's general administrative system, with responsibility divided among the district administrations, provincial governors and the Federal Ministry of Agriculture and Forestry, depending on the size and capacity of the structure involved. The actual division of responsibilities has since been changed several times.

Until 1947, the Federal Ministry of Agriculture and Forestry had authority over dams over 15 metres in height above their foundation. This figure was increased to 25 metres for gravity dams in 1947. Under the Water Law of 1959, the Federal Ministry of Agriculture and Forestry was made responsible for all embankment dams over 15 metres in height above their foundation and for concrete dams exceeding 40 metres in height above their foundation or with a storage capacity exceeding 5 million m³. The 1990 amendment finally defined the relevant height at 30 metres for both embankment and concrete dams.

The Water Act of 1934 provided a detailed definition of the various types of dams, but the result was found to be too confining in view of ongoing progress in dam construction, and it was therefore abandoned in the amendment of 1947, with the result that no legal definition now exists for these terms.

1.2 The Austrian Commission on Dams

The "Commission on Dams", which was first appointed in 1918 following the failure of a dam, was conceived as a governmental advisory body charged with the study and assessment of reservoir design safety from an engineering and economic point of view.

The 1934 Water Law then provided the legal basis for issuing the "Ordinance on Dams and Reservoirs". The present ordinance, issued by the Federal Ministry of Agriculture and Forestry on May 14, 1985, defines the authority of the Commission on Dams, which becomes active at the request of the Water Authorities and whose purpose it is to eliminate safety risks to the maximum possible extent.

In its function in support of the Water Authorities, the Commission has been given a number of responsibilities, e.g.:

- technological and economic evaluation of designs for new dams and reservoirs, for alterations to existing structures or structures currently under construction;
- filing of recommendations and submission of expert opinions on general measures and regulations concerning dams and reservoirs;
- commissioning or execution of investigations into engineering aspects of dams and reservoirs;
- collection, updating and analysis of information on the state of Austria's dams and reservoirs over their entire life spans and on their operational behaviour, and if necessary elaboration of proposals for corrective maintenance and rehabilitation schemes.

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The Commission consists of 26 experts in dam construction and related sciences, who serve on an honorary basis. At least five of the members must be university professors. Open interdisciplinary discussions within the Commission are designed to ensure adequate containment of the safety risk, in particular by producing a stability evaluation of the structure with regard to such factors as foundations, structural calculations, flood control, etc. For this purpose the Commission is made up of the head of the Division for Water Management and Hydraulic Engineering in the Federal Ministry of Agriculture and Forestry and his deputy, who act as chairman and secretary respectively, three senior civil servants with an engineering background – one from the Federal Ministry of Public Economy and Transport, one from the Federal Ministry of Economic Affairs, and one from the Federal Ministry of Defense – and experts from the fields of dam and barrage engineering, structural calculations, soil mechanics and foundation engineering, geology, rock mechanics, asphalt and concrete technology, meteorology, seismology, hydraulic engineering, hydrology and mechanical en-gineering. The Commission has held a total of 58 sessions on the evaluation of dams since 1948. Until 1962 it also served as the ICOLD National Committee, be-fore the National Committee was enlarged and reconstituted as an association. The members of the Commission on Dams are at the same time active in the ICOLD National Committee.

2 AUTHORIZATION PROCEDURES

Under the Austrian Water Law, formal approval procedures for a dam project focus on the issue of adequate safety requirements – a process that is frequently arduous for both the applicant and the authority.

Procedures are founded on the provisions of the Water Law, and to a lesser extent on the Austrian General Administrative Procedures Law. They are governed by the ex officio principle, i.e. the government authority itself determines the facts, consulting appropriate experts.

2.1 Technical requirements

The Water Law stipulates which data and drawings, calculations, verifications and design details are to be included in an application. The detailed project submitted to the government authority has to describe the functions of the various parts of the structure, safety measures and the impact of the project on third parties. Depending on the type of barrage (embankment or concrete dam), project documentation must provide information on the following subjects.

2.1.1 Hydrology, hydraulic engineering

Hydrological examination of the catchment area to calculate the design flood at the barrage for the purpose of dimensioning the spillways and freeboard depending on the type and function of the barrage concerned (embankment or concrete dam).

The lack of hydrographical data in the past frequently made it necessary to dimension spillways on the basis of estimations. In many cases, calculations were based on flood return periods that were too short, with the logical consequence of inadequate discharge capacity. Today, spillways are designed for a 5 000-year flood discharging into a reservoir filled to retention water level, while the discharge capacities of the bottom outlet and turbine flow are not included in the calculations. In dimensioning the freeboard, consideration must also be given to the overtopping behaviour of the embankment or concrete dam.

These requirements derive from the fact that, although Alpine reservoirs for seasonal storage do not normally operate at maximum water level at times when large-scale floods may be expected, owners still insist on being able to manage their reservoirs without restrictions and solely in accordance with energy requirements.

Experts are currently discussing whether to make dimensioning rules even more stringent in view of ongoing deliberations concerning the "Maximum Probable Flood" (M.P.F.).

From the point of view of operational reliability, preference is given to all types of fixed-crest weirs because they have no movable parts and consequently require no operation/control and special maintenance.

Where the catchment area is heavily wooded, the effect of log jams must be considered as well. The owner must provide evidence of measures taken to catch and remove floating logs without damaging the structure.

The discharge capacity of the bottom outlets must be dimensioned to permit drawdown in case of danger, with due regard to be given to the discharge capacity of the waterway downstream of the dam.

A proposal must furthermore be submitted for the minimum flow requirement needed to preserve the ecological viability of the developed water course.

2.1.2 Geology and rock mechanics

Mandatory procedures include geological investigation of the dam foundation, reservoir basin and surrounding slopes, including investigation of the foundation contact area and its immediate vicinity for faults, large fissures, loose strata and horizons of increased permeability, using geological surface reconnaissance, exploration galleries and drilling, plus geological mapping of the terrain to locate fault planes and subsidences, and possible sources of landslides, rockfall, mudflows and bedloads.

These works are to be followed by analysis of the deformation and bearing behaviour of the dam foundation and the shear strength of the rock types with the help of in-situ and lab testing, geoseismical tests to determine the rock-mechanics parameters for calculating the dam body and for stability analysis of the foundation, plus calculation of the impact of loading on the foundation and backwater area as functions of dam height and impounded water volume.

2.1.3 Seismicity

Dam developers must obtain a seismological expertise for the relevant area, prepared by the Austrian Central Institute of Meteorology and Geodynamics and containing, at the very least, information on maximum response acceleration (b_{max} as a percentage of acceleration g) and earthquake occurrence probability. Where adequate data are available from seismic monitoring, the expertise should also include indications of the seismic acceleration and velocity ranges and duration of the tremor peak phase. Calculations on the impact of earthquakes on a dam should be based on a minimum horizontal acceleration of 0.04 g, a figure which is also recommended as the standard value for preliminary studies.

In designing dams, the impact of earthquakes may be limited to the horizontal direction, generally parallel to the valley, except where the design of the dam demands that another, particularly unfavourable direction be considered or where the seismological expertise requires inclusion of the vertical component. The expertise must also examine the possible effects of an earthquake on the foundation and identify the geological conditions under which the volume of impounded water would trigger an earthquake.

2.1.4 Structural calculations for concrete dams

Stability must be demonstrated using accepted methods of calculation based on parameters determined on site and in the lab for the dam foundation and concrete, with due consideration to be given to changes in the deformation modulus caused by long-term loading and possible differential settlement in the foundations.

The following load cases must normally be investigated:

- normal load case for empty reservoir (dead weight);
- normal load cases for intermediate and full storage levels, taking into account uplift pressures over properly sealed foundations and normal temperatures;
- rare load cases caused by increased uplift and extreme temperatures;
- exceptional load cases caused by surcharge of the design flood water level and extreme uplift;
- earthquake load case superimposed on the normal load cases. Load assumptions for earthquakes must be documented by an official seismological expertise.

It must also be established that abutment forces are reliably transmitted to the dam foundation, taking into account rock structure and deformation behaviour. To evaluate force dissemination in the foundation of arch dams, the minimum dissemination angles are normally calculated.

Adequate slide stability must be established for the foundation contact area.

For large-scale concrete dams the responsible authority may furthermore order verification of the results of stress calculations by means of stress and failure tests performed on a scale model.

2.1.5 Concrete engineering

Concrete tests must be performed with the binders and aggregates scheduled for use, comprising compressive strength, flexural and splitting tensile strength, adiabatic temperature change during curing, and the modulus of deformation and elasticity as a function of the number of load cycles and of load duration.

2.1.6 Structural calculations for embankment dams

Stability calculations are based on soil parameters determined by testing. Computation of the safety factors is normally performed in accordance with Fellenius' rule, or at the least an unequivocal relationship to such values must be established. The Fellenius values must not be lower than the minimum safety factors given in the table below.

Load case	Minimum safety factor
I. Normal operating loads at flood (HQ 100), rapid discharge at partial impounding	1.3
II. Feasible individual disaster load cases, maximum possible flood (HQ 5 000), design earthquake, damage to diaphragm (facing)	1.2
III. Most disastrous combination of feasible individual disaster load cases as in II, e.g. combination of HQ 5 000 flood with design earthquake	1.1

Prevention of hydraulic rock fissure and retrogressive erosion must also be demonstrated. It may be necessary to study dam material behaviour using scale models. Dimensioning and arrangement of the foundation sealing works and drainage must be based on the results of permeability tests in the exploration boreholes.

2.1.7 Monitoring equipment

When damaged or destroyed, dams may cause great devastation and will be out of operation for long periods of time. In order to keep them operative while subjected to constant and temporary forces and impacts, it is necessary not just to choose an appropriate design but also to ensure efficient surveillance through continuous monitoring and observation of potentially harmful events. This requires maximum reliability and precision in monitoring the behaviour of dams and their environments, involving the installation of a wide range of equipment. A typical feature of dam construction is that each dam must be conceived as an individual case in view of its specific location so that measuring equipment must always be individually tailored.

Principally, the following requirements should be met:

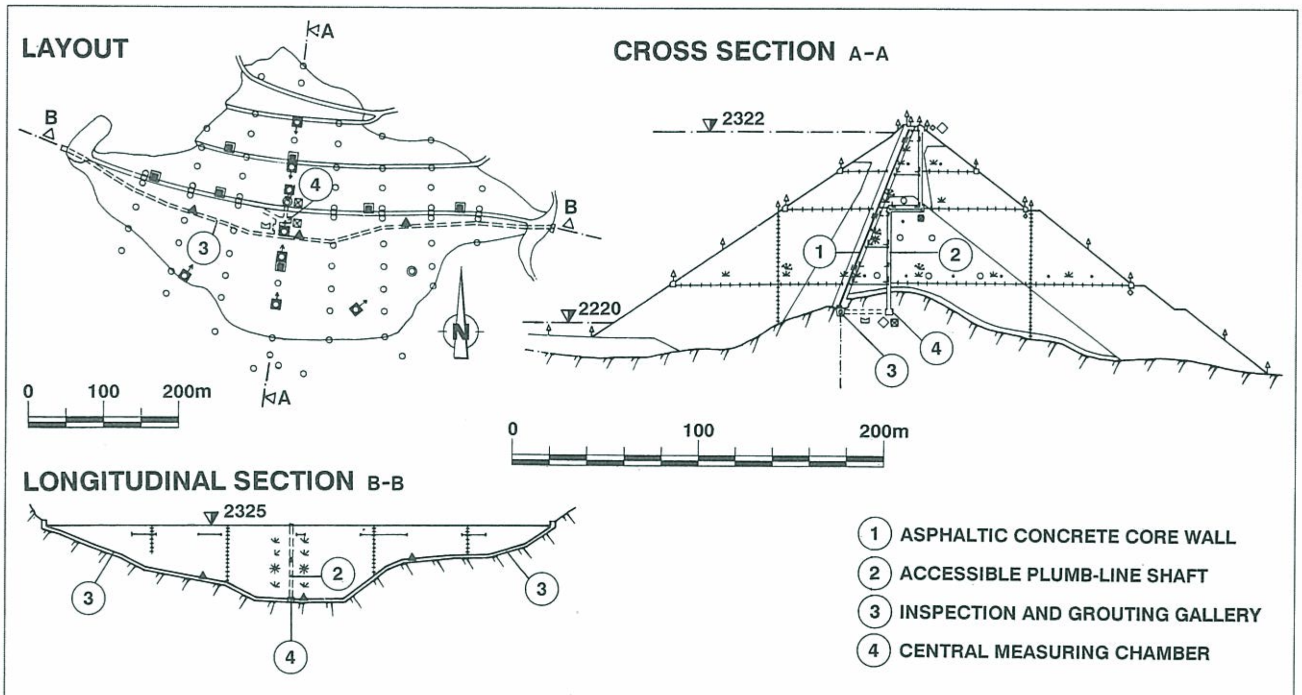
- assessment of overall dam and foundation behaviour;
- prompt availability of the data required to evaluate dam stability through the use of telemetry;
- accurate monitoring of foundation contact area behaviour;
- early installation of monitoring equipment in order to record the effects of dam construction;
- monitoring equipment configurations offering ease of comparison with the results of stability calculations.

Deformations and displacements of dams are usually identified by pendulums and clinometers or by geodetic surveying methods such as traverse, leveling, alignment, etc. Stress and temperature are measured by instruments embedded in the dam body (telepress and teleform meters, temperature sensors). Extensometers and sliding gauges are used to record spatial displacement vectors and foundation deformation. Piezometers supply data on uplift and rock water pressures. Seepage measurement points with flow gauging weirs monitor seepage volumes. In earthquakes-prone areas the instrumentation is rounded off by seismographs.

2.1.8 Surveillance

An early warning system utilizing such key data as

Figure 1 Finstertal dam – monitoring equipment



Symbol	Monitoring equipment	Number
	TWO DIMENSIONAL GROUP OF EARTH PRESSURE CELLS	92
✳	THREE DIMENSIONAL GROUP OF EARTH PRESSURE CELLS	18
○	PORE PRESSURE CELLS	12
●	RESISTANCE THERMOMETERS	9
◇	PEAK RECORDING ACCELEROGRAPHS	3
◇	STRONG MOTION ACCELEROGRAPHS (THREE DIMENSIONAL)	2
▲	PIEZOMETER IN BOREHOLE	13
	RECORDING OF SEEPAGE WATER LOSSES	3

Symbol	Monitoring equipment	Number
○	SURFACE MONUMENTS	110
	HORIZONTAL PLATE GAUGES	8
	VERTICAL PLATE GAUGES	7
	EXTENSOMETERS	13
	GROUPS OF 4 STRAIN METERS	4
	FLUID LEVEL SETTLEMENT DEVICES (PERMANENTLY INSTALLED)	9
⊠	PLUMB-LINES	2
	MEASURING DEVICES FOR DETERMINATION OF CORE-WALL THICKNESS VARIATIONS	3

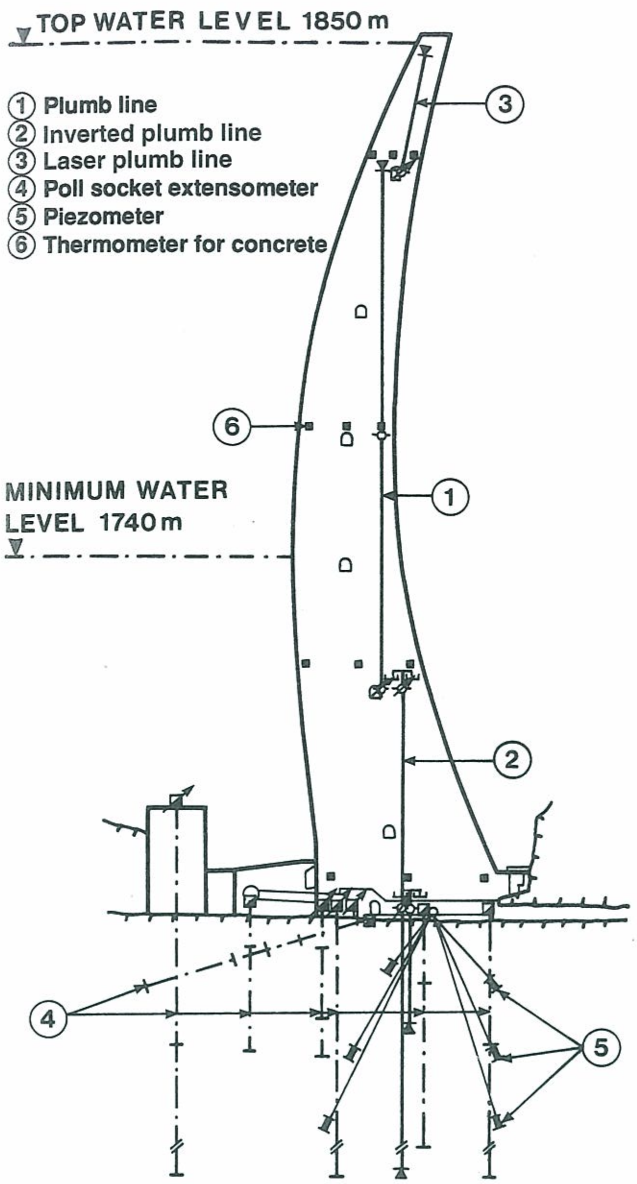


Figure 2 Zillergründl dam – monitoring equipment

foundation deformation from deadweight, temperature changes, shrinkage in concrete dams and deformation in embankments must be provided in the form of a monitoring schedule defining the type and frequency of the readings and their limit values.

The monitoring devices normally used for modern large dams are illustrated in Fig. 1 (for a rock-fill dam) and Fig. 2 (for an arch dam). Telemetry is typically employed to transmit the results to a central control room.

2.2 Licencing procedures

2.2.1 Preliminary examination

Under the Water Law, a project application must first be filed with the Water Management Planning Board established at the appropriate Provincial Government Office. The Board is responsible for coordinating all planning measures at the provincial level which impinge on water management.

The application is then subjected to a "preliminary examination" by the Water Authority relating to the impact of the project on *public interests*. Experts, public authorities and agencies, local government agencies, and representatives of any persons or groups affected by the project are invited to contribute.

The term "public interests" covers the following functions and factors: defense; public safety; health; flood and ice control; existing or planned control structures; course, depth, gradient and banks of the water courses involved; water quality; public use; water supply; cultural factors; monuments of historic, artistic or cultural importance; natural monuments, appearance of villages and towns, scenic beauty, agriculture; wastage of water; routing of water into neighbouring countries to the detriment of one's own country; economic interests; ecological viability of the water course, etc.

Special emphasis is placed on public interest with regard to safety of downstream settlements. Except in the case of imminent danger, the Water Authority must obtain an expert opinion on safety aspects from the Commission on Dams for all dams which are subject to the authority of the Federal Ministry of Agriculture and Forestry or which feature extraordinary foundations, unusual construction methods or special load requirements.

The work of the Commission on Dams begins with a study by the Executive Board of the technical assessability of the project submitted by the Water Authority.

A working group consisting of experts from the relevant fields is appointed for each project. The members prepare their individual expert opinions with due regard to interlinkages with other fields, in coordination with the other members, and taking account of the latest research and technological advances. After harmonizing the individual expertises, the working group drafts a joint expert opinion, which is then discussed by the Commission. In the final decision-making process, the Commission may then approve the project and submit recommendations for its implementation.

Should it be found in the "preliminary examination" that the project is inadmissible for reasons of public interest, the application is rejected. Other reservations must be communicated by the Water Authority to the applicant, who is then given a deadline to clarify or modify the project. If no further information is received from the applicant upon expiry of the deadline, the application is deemed to have been withdrawn.

If the application is not rejected out of hand and not withdrawn as a result of the objections lodged, a hearing has to be scheduled to ensure that the official report is legally binding.

2.2.2 Water Rights Hearing

Invitations to the hearing are once again issued to the relevant federal and provincial government authorities and agencies, to the representatives of local government and interest groups concerned by the project, as well as to the parties involved. The latter comprise the applicant, all persons of whom a performance, sufferance or forbearance is requested or whose rights (in particular lawfully exercised water rights and freehold rights) are affected, persons who hold fishing rights, local governments (for large-scale projects), etc. The hearing offers an opportunity for all authorities and parties to make statements, raise objections and request consideration of specific demands. Experts consulted by the Water Authority review the project documents submitted as well as any demands and statements filed by other participants in the hearing. The Water Authority calls in officially recognized experts on hydraulic engineering and hydrography, and appoints experts for specific fields (geology, foundation engineering, soil mechanics, tunneling, rock mechanics, structural calculations, structural steel engineering, hydraulic steelwork engineering, concrete technology, meteorology, mechanical engineering) from among a group of relevant university professors or consulting engineers.

The Water Authority identifies the public interests involved at the preliminary examination stage. Whenever opposing public interests are at stake it is necessary to strike a balance between them, conceivably after consulting relevant experts. While the Water Law does not lay down any hierarchy of public interests, public safety obviously enjoys absolute priority.

Issues arising in this process are mainly of a technical nature and cannot be solved by legal consideration alone, with the consequence that engineering experts today find themselves in a key position of great responsibility with regard to approval procedures. They must examine the stability and operational safety of the dam with a view to defining operational requirements in the official notice of approval and later in the operating regulations. They are assisted in their task by the Commission on Dams with its pool of technological knowledge and practical experience.

Another major aspect apart from safety is the minimum streamflow requirement. This must be assessed so as to ensure ecological viability of the water course downstream of a dam on impounding. The original legal priority given to the complete and economic harnessing of water power in Austria was eliminated in the 1990 Amendment to the Water Law.

In view of the constitutional division of responsibilities between the national authorities (e.g. Water Law) and the provincial authorities (e.g. environmental protection) it is conceivable that the Water Authority may mandate a different minimum flow than the Office of Environmental Protection. No senior authority is provided for under the division of responsibilities to coordinate the decisions of the two government agencies. In order to avoid a situation that would be detrimental to either the public or the applicant's interests, the two authorities contact each other in the course of their respective procedures so as to settle the question of minimum streamflow jointly.

With regard to interference with the rights of private parties, it is incumbent upon the applicant to make every reasonable effort to achieve an amicable settlement. If no agreement can be reached, the Water Authority will make a compulsory order obliging the parties concerned to suffer certain encroachments upon their rights and the applicant to pay reasonable compensation assessed by the Water Authority itself.

2.2.3 Notice of Approval

Upon completion of the approval procedures, the Water Authority must issue a *notice of approval*, including the conditions to be met in the public interest and in the interest of private rights, and ruling on objections which have been judged to be of insufficient merit. In the event that any rights should have to be enforced, the authority must balance the interest in the development project against any disadvantages suffered by the party against whom the right is to be enforced.

Each party may appeal against the notice of approval within six weeks of its receipt, either to the Constitutional Court (for infringement of rights guaranteed under the Constitution) or to the Administrative Court (for infringement of other rights). The appeal does not per se suspend the effectiveness of the notice, i.e. the res judicata effect of the notice is not affected by the appeal, but the appellants may petition in their complaint to have the effect of the notice suspended. The petition is normally granted in the case of large-scale projects in view of the fact that changes created by constructional activities are usually irreversible.

When all approvals required for the dam (under the Water Law, Forestry Law, Environmental Protection Law or Building Law, where applicable) have been finalized, and when all parties involved have reached an agreement, the applicant may begin to implement the project.

3 CONSTRUCTION, SUPERVISION

For all large-scale and complex structures, the Water Authority appoints an official supervisor, who is responsible in particular for regular monitoring of project implementation, for verifying observance of the stipulations set down in the Water Law licence, and for filing regular reports on the progress of the works.

Supervision is also provided in the form of "expert meetings" held to deal with individual issues developing from specific on-site factors in the course of construction work. At these meetings, the Water Authority experts review the situation in detail on site and provide guidelines for further procedures.

4 PRELIMINARY TECHNICAL ACCEPTANCE

Upon completion of the construction works, the equipment and structures which are to be submerged in the course of first filling are inspected by the Water Authority. Under the so-called "preliminary technical acceptance" procedure, which includes all public authorities involved in the project, the affected components are examined on the basis of the working drawings and certificates issued for lab tests performed during construction, the operating equipment is tested for proper functioning, and a filling schedule and "preliminary operating and monitoring rules" are established.

The *operating and monitoring* rules are typically revised repeatedly to accommodate experience acquired during filling and operation before they can be given final approval after several years of operation. They cover operating principles and regulations, individual responsibilities, preventive maintenance, ongoing monitoring on the basis of a surveillance schedule, and maintenance intervals for the operating equipment.

To keep the risk levels for a population as low as possible in cases of natural disasters or unpredictable events, it is incumbent upon the dam's owner to develop an *emergency flood plan* in coordination with the appropriate authorities, and to provide for and operate the requisite alarm systems.

The emergency flood plan established on the basis of *flood wave calculations* lays down the measures to be taken to deal with extraordinary events and imminent dangers.

In order to draw up an emergency flood plan it is necessary to identify the height, spread and discharge of a flood wave caused by an assumed breach. The two-step calculations, i.e. determining the size of the flood wave from the reservoir as a function of the type of breach and then calculating the progress of the wave downstream, give the time required to warn the population and the flood level to be expected for each point of the valley.

The calculations and the assumptions on which they are based must be coordinated with the chief water engineer at the Water Authority. They are then checked for their accuracy and plausibility by the Water Authority and filed with the provincial government agencies responsible for civil defence.

4.1 First filling

Unlike other structures, a dam cannot be subjected to a test load. Consequently, first filling is carried out in a number of stages which may be spread over a number of years. Initial filling is naturally accompanied by continuous monitoring and tests, whose results are checked against the relevant calculations. At each stage, the Water Authority will only allow filling to continue if the dam behaves exactly as predicted.

If test results deviate substantially from the calculations, filling is stopped and the reservoir level may have to be lowered until the causes of the irregularity have been identified and eliminated.

Large-scale dams require individual approvals for partial filling levels before being granted approval for maximum water level. The latter approval is usually limited in time, and unrestricted operation is allowed only after several years of successful operating experience.

5 FINAL ACCEPTANCE

It is only after a number of "trial years", with dam behaviour conforming to expectancy, and upon completion of all attendant works necessary for utilization of the reservoir that the structure is considered completed and ready for final acceptance under Austrian Water Law. For this purpose, the owner must submit to the Water Authority, among other things, detailed geological mapping of the foundations, results of material tests performed with the material used in the dam, stability calculations based on monitoring data derived in the course of filling, final operating and monitoring rules including a surveillance schedule, etc., together with the as-built documentation. The main item in the acceptance procedure is a hearing where all authorities and parties involved in the project and heard during the licencing procedure are once again invited to participate.

This phase of the procedure is limited to a review of whether the actual structure conforms to the licence as granted; minor deviations from the approved project can be allowed subsequently provided that they are not detrimental to any public interests or third-party rights

Table 1 Approval procedure under Austrian Water Law

Step	Authority	Action	Result
1	Competent Authority	Preliminary examination: embankment and concrete dams height > 30 m volume > 5 million m³ difficult foundations special constructions	Referral to Commission on Dams
2	Commission on Dams	Engineering and technical-economic examination:	Acceptance or rejection
3	Federal Ministry of Agriculture and Forestry	Licensing procedure: hearing, expert opinions, parties	Notice of permission
	Provincial government		
	District administration		
4	According to step 3	Regular construction supervision	
5	Federal Ministry of Agriculture and Forestry	Preliminary acceptance: filling schedule, preliminary operating and monitoring rules, emergency flood plan	Partial filling, full filling
	Provincial government		
	District administration		
6	Federal Ministry of Agriculture and Forestry	Final acceptance: acceptance hearing, execution in conformity with plans, stability	Notice of acceptance
	Provincial government		
	District administration		

or provided that the parties concerned give their consent.

This acceptance procedure is the final step in the sequence of licencing steps prescribed by Austrian Water Law.

6 OFFICIAL MONITORING AND INSPECTION OF DAMS

Responsibility for dam maintenance, i.e. safety and operation of dams, rests primarily with the holder of the water rights (= owner). The owner must ensure that the dam is maintained in a condition corresponding to the licencing provisions. The Water Authority, however, is authorized to stipulate additional requirements reflecting the state of the art in dam engineering.

Austrian Water Law furthermore mandates a supervisory duty on the part of the authorities with regard to waterways and engineering structures. Responsibility for this task rests primarily with the provincial government, although the Federal Ministry of Agriculture and Forestry is empowered to take over this responsibility should the need arise. Government supervision, however, in no way relieves the owner of its responsibilities.

Mandatory monitoring and surveillance and other responsibilities undertaken by the owner are set down in the operating and monitoring rules, which must be approved by the Water Authority.

Dam monitoring and inspection is performed at several levels.

– Owner of the dam

From among its technical staff the owner must appoint a senior engineer (or commission a consulting engineer) to act as its "safety engineer". He is responsible for proper preventive and corrective maintenance, for observance of the stipulations and requirements laid down by official agencies, and for measures to be taken in the event of an emergency. For this purpose, the safety engineer must be vested with suitable authority by the owner. The safety engineer and his deputy must be named to the Commission on Dams and the District Commissioner. The owner must log the findings of his inspections and reviews in a "dam record" and submit an annual report summarizing the findings to the supervisory authority and the Commission on Dams.

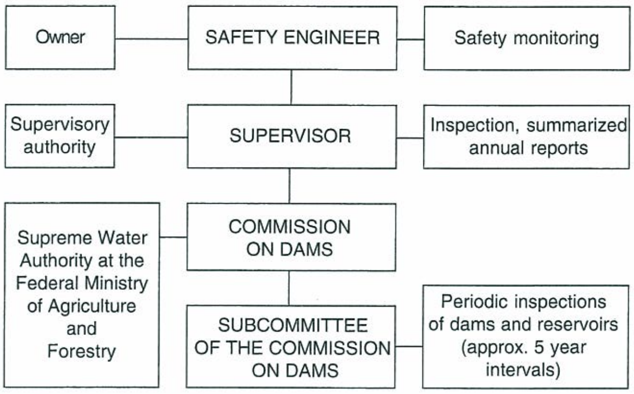
– Supervisory authority

The government body responsible for the country's water resources appoints a senior engineer in the civil service to supervise dams and reservoirs. The appointment is made by agreement with the Executive Board of the Commission of Dams and notified to the Supreme Water Authority. The supervisor verifies the monitoring results produced by the owner and reports his findings to the Water Authority, the Commission on Dams and his own office at least once a year.

– Commission on Dams

In the mid 1960s, a "Subcommittee on Dam Inspection" was constituted within the Commission on Dams, consisting of experts in the field of stress calculation, dam engineering, geology, rock mechanics, monitoring of dams, and hydraulic engineering. From the mid 1970s, the Subcommittee was commissioned by the Supreme Water Authority to inspect existing and accepted dams

Table 2 Operations monitoring dams >15 m in height (excluding run-of-river power stations) of >500 000 m³ in volume



at 2–5 year intervals and to submit appropriate reports. The Supreme Water Authority must then order the owner to institute any corrective measures which are necessary to prevent imminent danger or which are in the public interest (human life and health, severe damage to the national economy). The procedure for revising valid approvals was considerably facilitated in 1990.

7 CONCLUSION

Dams will normally produce extensive damage in the event of failure. The Austrian authorities are consequently doing everything in their power in engineering, legal and organizational terms to ensure maximum safety for the population as well as progress in dam engineering within the framework of orderly construction, administration and monitoring procedures. This guiding principle pervades the modern Austrian Water Law and is reflected in the activities of the Commission on Dams, the Water Supervisory Authorities and the management teams of the country's power utilities.

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Chapter E

ENVIRONMENTAL ASPECTS OF HYDRO-POWER DEVELOPMENT IN AUSTRIA

CONTENTS

ENVIRONMENTAL ASPECTS OF HYDRO-POWER DEVELOPMENT IN AUSTRIA

By R. Widmann*

1 INTRODUCTION

Man has tried for thousands of years to replace his own muscular force by that of slaves or domestic animals in order to make it easier for him to satisfy his needs. For hundreds of years he has utilized the mechanical power of water by means of water-wheels, as for mills. In the last century, with the beginning of the industrial age, man started to use machines to do his work. But the machines needed driving agents. Several possibilities were developed. An outstanding example is the steam engine, which converts thermal energy from the combustion of various fuels into mechanical energy, a system underlying all types of thermal power plants. In a second step, the mechanical energy is converted into electrical energy in the same way as in hydro power plants.

It was certainly since the publication of the first report by the Club of Rome that the notion of our globe as a spaceship has met with general acceptance. The idea is to demonstrate that there are limits to the areas and mineral resources utilized by man and that unlimited growth and unlimited consumption of mineral resources are not possible. Space-ship earth, as it were, receives its fuel - solar energy - from outside as a basis for the eternal cycle of nature. The energy supply from outside remains nearly constant over geologic spaces of time and has not been used up during the past ages. Surplus energy has been stored in the form of coal, crude oil, natural gas, etc. Over the last two decades the energy requirements of part of mankind have grown enormously, undoubtedly due also to the population increase, and this has marked the beginning of an era characterised by the exhaustion of energy resources which, though having accumulated during thousands of millions of years, are ultimately limited in quantity. Obviously, the only way out will be the direct or indirect utilization of solar energy.

Several possibilities of direct utilization have been studied, but they all need further development, mainly because of the large cost involved at present and because of the large amounts of energy required for the fabrication of solar equipment. Therefore, it is no doubt necessary to intensify research efforts in this field. In view of the present state of the art, however, the climatic conditions prevailing in Central Europe hold little promise of the direct utilization of solar energy gaining any major importance in the near future.

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The indirect utilization of solar energy through the combustion of biomass has been practised to a limited extent ever since the invention of fire. But large-scale utilization is likely to meet with serious difficulties, as e.g. the wearing out of soils by quick growing energy forests (Gepp, 1986) or the residues of combustion contributing to global air pollution, very much like those resulting from the combustion of fossil fuels. While the efficiency of solar facilities ranges around 15 or 20 %, combustion processes utilize about 40 or 50 % of the primary energy, with no allowance being made for the energy required for the winning of the fuel and its transport to the power plant. Another aspect to be considered in this context is the amount of energy spent for the construction of the energy-generating facility as compared with the amount of energy generated during the service life of the plant. Relevant studies have shown that at present it is possible to produce approximately

- 1 to 5 times the energy input by flat collectors,
- 5 to 10 times the energy input by crystalline solar cells,
- 50 times the energy input by coal-fired power plants, and
- 100 times the energy input by hydro plants.

Even if aspects of safety are included in the comparison of the different types of power production, the generation of electricity by means of hydro power – with account being taken of large dams – comes out much better than thermal power if account is taken of fuel production and transport.

The indirect utilization of solar energy for electricity generation through the development of the mechanical energy inherent in water has reached a level of great perfection. Efficiency is almost 90 %, the water is the same in quality and quantity as it was, when it is returned to the eternal cycle of nature, and there are no waste products risking to affect air or soil. What then are the problems of hydro power development?

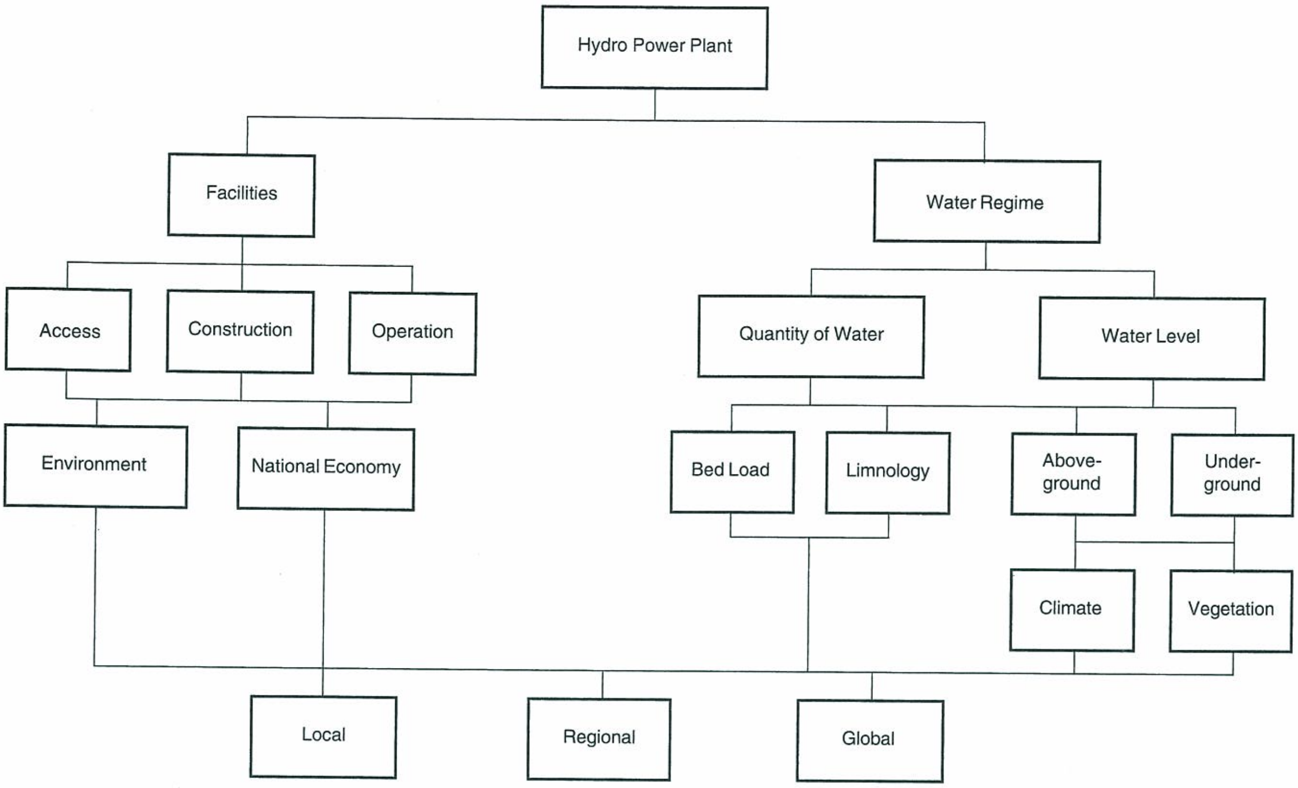
Like any other human activity, the construction and operation of hydro power facilities may influence the environment. These influences come from two main factors that are essential to hydro plants (Fig. 1):

- Various engineering structures of different sizes, partly above-ground, partly underground and consequently invisible.
- Changes in the natural stream bed which are neces-

sary for the utilization of the mechanical energy of the water and which depend as to nature and amount on the type of hydro plant provided.

the capacity rating of small power stations. As large-scale development of Austria's hydro resources actually commenced immediately after the end of the Second World War, the experience gathered in this field extends over

Figure 1 Environmental implications of hydro power plants



In order to identify changes in the natural environment that result from the construction of hydro power facilities, fact-finding procedures have been imposed for several decades in Austria. For example, several years prior to the commencement of construction, spring yields, vegetation, fog formation, etc. within the area of assumed influence are observed and recorded. When the plant is in operation, the corresponding data are collected once more and compared with the original data so as to obtain an objective idea of potential changes. In response to the growing sensitivity to environmental issues, an environmental compatibility check was introduced a few years ago in order to estimate and evaluate expected effects. This check includes any potential changes resulting from the construction of a hydro power plant and should not be limited to conceivable ecological effects of changes resulting from the construction and operation of hydro power facilities, but should be supported by information available from the fact-finding procedures and from subsequent supplementary investigations. In addition, the environmental compatibility check should also include aspects of national economy and realistic alternatives so as to afford a comprehensive view of the problem, as is often claimed, especially by ecologists.

Construction of hydro power developments in Austria started already towards the end of the 19th century. As early as 1908, Innsbruck's Obere Sill power station exceeded the legal 10-MW limit now in force in Austria for several decades. The following paragraphs are an attempt to report on this experience and the conclusions to be drawn.

2 THE POWER FACILITIES

Hydro power facilities at Alpine locations normally require the following main features:

- Reservoirs created by dams for collecting water from direct inflow and usually also from several stream diversions;

- Galleries, shafts and ducts to convey the flows from stream intakes to the reservoir, from the reservoir to the powerhouse and finally from the powerhouse back to the nearest natural stream bed;

- The powerhouse and appurtenant installations where the mechanical energy of the water is converted into electrical energy.

2.1 Relationships with the environment

Where the construction of a power development is planned, it is necessary at first to provide for suitable access to the site. Depending on the nature and size of the structure, this may be accomplished either by unmade roads, single or double lane roads, or even by areal ropeways or inclined hoists. As the transport capacities required for these access facilities are usually larger during construction than is needed subsequently during operation for supervision and maintenance, the newly constructed facilities may either be removed or may be made available to tourist trade. Actually, the provision and operation of such traffic facilities are subject to the same criteria as comparable normal traffic facilities. At any rate, it is up to the discretion of the invidiual communities whether to use the new facilities for tourist trade after the completion of construction work, which in fact involves major traffic loadings at times while the amount of traffic needed for purely operational purposes is negligible.

The individual features of a power development may take several years to construct. During this period, the environment is certainly affected to some degree, as at any construction site. For major construction sites at remote locations, it will be necessary to provide camps to accommodate staff, primarily to avoid the daily trips to and from the site. Such camps are equipped e.g. with their own sewage treatment works. The fuels required for driving construction equipment must be stored in accordance with the relevant regulations. Tunnel or service water as e.g. from aggregate preparation plants must be cleaned before being returned to the natural stream bed. Tunnel spoil or other excavated material must be dumped in a stable condition and the surface shaped to blend with the surrounding terrain and planted after completion of the work with plants selected to harmonise with the local natural vegetation. Many examples demonstrate that this is possible even at high-level mountain locations (Fig. 2).

Special care should be taken already at the stage of design to make the visible structures of the project blend with the surrounding landscape. Naturally, while the small structures for stream intakes will hardly be noticed provided they are adequately designed (Fig. 3), large dams of whatever design will always be impressive structures. In fact, embankment dams do allow a certain measure of natural appearance to be reached by an adequate shaping of the abutments and by suitable planting on the downstream slopes. But concrete dams, in particular arch dams, are large technical structures which are recognised by the majority of the visitors as being impressive engineering feats and a manifestation of man's creative power.

The provision of access facilities for large dams, especially at high-level sites in the Alps, may become a basis for tourism in that they greatly reduce the time and efforts needed to reach high-level destinations, as evidenced by the growing numbers of visitors to mountain huts situated above reservoir lakes.

There are basically two powerhouse types – aboveground buildings and underground structures. The choice between the two is in many cases less of a cost problem than a function of the local geology. The power stations of large hydro developments may reach substantial dimensions, but given adequate architectural design may be made to blend relatively well with the surrounding landscape or village. Indoor switching stations, although higher in cost, are increasingly provided nowadays so as to allow the reduction in size of the switchyards, which are almost always considered a disturbing sight (Fig. 4).

During construction, any site will convey the impression of completely destroyed nature to the outsider. However, suitable measures carefully designed and carried out to clear and seed the construction site will bring about, usually within a surprisingly short time, a landscape that differs from its original appearance, but harmonises with its surroundings. The cost involved hardly ever exceeds a few percent of the total construction cost and is, therefore, justified even from an economic point of view.

During operation, there is hardly any influence from the individual elements of a hydro development on the environment. Pollutant emissions or toxic sewage are absent altogether, and the noise nuisance is negligible.

2.2 Aspects of national economy

The construction and operation of hydro developments, like any industrial enterprise, have multiple effects on a country's economy. Only a brief outline of these effects can here be given.

An entrepreneur will establish an industrial plant only if he is sure that he will be able to sell the product he plans to manufacture. For the moment the annual electricity consumption growth in Austria equals the annual energy from a power project on the Danube, and hydro power plants can meet only about two-thirds of the country's total requirements - the remaining one-third coming from thermal power plants running on fuels the greater part of which must be imported. As long as this situation prevails, the construction of new hydro power plants seems to be justified from an economic point of view.

There is no doubt that the construction of a power scheme has favourable consequences in terms of employment, with jobs becoming available at the construction sites and with all the suppliers of electrical and mechanical equipment, especially where large power developments are concerned. That means in fact that the consequences are supra-regional. The employment aspect has been established to be two man-years per million of Austrian schillings invested, with account being taken of all subcontractors and the service industry. The communities in the areas concerned benefit not only from the access facilities provided for the power schemes, but also from increased tax revenue, which is used for improving infrastructure, and this in turn improves the living conditions of the local population and renders the area more attractive to tourist trade. Naturally, the lower the standard of infrastructure in these communities prior to the construction of the project, the larger are these local effects. Typical examples are the Montafon valley in Vorarlberg after the end of the First World War when construction was commenced on the Upper JIl project, and Kaprun after the Second World War when the construction of the Glockner-Kaprun project was started. But even communities with a highly developed infrastructure, as e.g. Mayrhofen prior to the construction of the Zemm-Ziller development, have made good use of the additional revenue.

3 CHANGES IN WATER REGIME AND THEIR CONSEQUENCES

Any hydro scheme is bound to cause changes in the water regime. What we have to find out is whether these changes are desirable, admissible or inadmissible. It is a fact that the flow utilised is returned unchanged in quality and quantity to the natural cycle. But there are differences depending on the type of power plant concerned:

- Run-of-river and dam power plants are normally situated at low-level locations, with the heads being created by impounding the river within its bed. The water remains in the natural river bed and flow undergoes no change in time.

- Diversion-type power schemes are generally situated at medium high and high levels, where the flows of several streams are collected in a reservoir and released as required, so that their return to the natural stream bed is delayed.

Some of the main experiences in the operation of such schemes will be discussed in the following paragraphs.

3.1 Run-of-river and dam power plants

This type of power schemes, as e.g. Klaus or Ottenstein is characterised by the natural water level being raised and the flow velocity being reduced within the reservoir or backwater area. Inadmissible rise of the water table in the adjoining areas can be prevented by appropriate engineering measures. However, the reduction in flow velocity within the reservoir results in several changes.

- Within the backwater area a fish population tends to develop that is larger than before, but is adapted to the lower flow velocity. Interruption of upstream migration of fish can in some cases be avoided by providing fish ladders.

- Depending on the amount of bed load and suspended sediments in a river, sedimentation may result and, when having reached an inadmissible magnitude, calls for flushing in periods of major flow.

- The reduced self-purifying capacity of the slowly flowing river is felt only where the water quality is worse than grade II. In such cases, water purification facilities will have to be provided upstream of the project in order to accomplish the desirable water quality, a requirement that is in fact independent of the power project.

- The potential effects on the microclimate from the enlarged water surface have been the subject of several studies, which have shown that Austria's topography would not allow changes in water surface that are large enough to be of any significance as compared with the natural fluctations.

These aspects of the environmental implications will be dealt with in greater detail under D.

3.2 Diversion-type power schemes

Prominent examples of diversion-type power schemes in Austria are the large Reisseck-Kreuzeck, Glockner-Kaprun, Zemm-Ziller, Sellrain-Silz and Upper JII developments. This type of power scheme may cause various changes in the water regime, which will be discussed in the following for each of the four main sections of the waterway, in the light of the experience gained so far.

3.2.1 Reservoir

Water storage is needed where natural flow with its daily and seasonal fluctuations is used to meet water requirements that follow different rhythms. As electricity cannot be stored in large amounts, it must be produced at the moment it is needed. Storage reservoirs should mainly be considered as energy reservoirs to be drawn upon at times of peaks in electricity demand and failure of other power stations, so as to guarantee a reliable electricity supply.

As demonstrated by the comparison between natural and artificial lakes in Austria and all over the world (Table 1), the surface areas of the large Austrian reservoirs are smaller than one thousandth of the world's largest reservoirs and smaller than one hundredth of the large natural lakes in Austria. The total surface area of all the artificial lakes in Austria corresponds in magnitude to that of lake Attersee. This comparison alone shows that Austria's reservoirs cannot possibly cause climatic changes of any kind that extend beyond the immediate shores, as has also been confirmed by many different studies. Therefore, the following comments may be confined to the much-discussed topic of local effects.

Table 1 Comparison of the size of reservoirs and natural lakes all over the world and in Austria

	Large Lakes				
	All over the world			In Austria	
	Name	Country	Area km	Name	Area km
Reservoirs	Akosombo High Aswan Bratsk Cariba Cabora Bassa	Ghana Egypt USSR Zimbabwe Mozambique	8 482 5 900 5 470 5 100 2 580	Ottenstein Gepatsch Kölnbrein Schlegeis Tauernmoos	4.5 2.6 2.55 2.2 1.89
Natural Lakes	Lake Superior	Canada	82 414	Lake Constance	538.5
	Lake Victoria	Uganda Kenya Tanzania	68 800	Neusiedlersee	276.4
	Lake Aral Lake Huron Lake Michigan	USSR USA/Canada USA	62 000 59 586 57 994	Attersee Traunsee Wörthersee	45.9 24.5 19.3

Within the section of reduced stream flow downstream of the reservoir, the stream recovers along with increasing distance from the reservoir and can be compared with a spring rising at the same level and being unaffected by annual floods or bed load supply from the catchment above the reservoir. The effects of reduced stream flow will be discussed under 3.3.

Reservoir sedimentation is mainly a function of the geology of the catchment area directly above the reservoir. In Austria, erosion in these catchments reaches a magnitude of between 0.1 and 1.8 mm p.a. Based on the ratio between reservoir volume and catchment area, this would result in sedimentation periods of several hundreds to several thousands of years without calling for artificial measures. Sedimentation periods are shorter for the smaller schemes and may necessitate occasional flushing and clearing (Widmann, 1988).

The retention of flood waves undoubtedly counts among the desirable effects of seasonal storage in the Alps. A large measure of flood control is already ensured by normal reservoir operation. Detailed statistical studies have shown that reservoirs with a net storage larger than 60 % of the annual volume of water yield are capable of retaining flood waves with a return period of about 25 years in their entirety. This affords substantially improved flood control to downstream populations, which however decreases along with increasing distance from the reservoir. Here, too, experience has shown that the reduction of the flood peak is larger than would have been expected from the relationship between the catchment areas. For example, the reservoirs of the Zemm-Ziller development, which collect the runoff from a catchment of 173 km^2, have led to an about 25 % reduction in flow peak for flood waves in the Ziller at its junction with the Inn so as to avoid the flooding of the valley floor, although the catchment above that point is six times as large.

Discussion of the limnological aspects of reservoirs in the Alps should be based on the normal fluctuations of the reservoir water level in the course of a year, on the nutrient supply dependent on the high-level location, on the water temperature also dependent on the high-level location, and finally on the conditions prior to the construction of the reservoir. Recent studies have shown that only a few of the high-level lakes in Tyrol have fish populations, which moreover may have been put in over the last centuries. Undoubtedly a reservoir, despite its restrictions, offers a larger living space to fish than existed in the natural stream bed prior to the construction of the reservoir, which may just as well be utilized as a managed piece of nature. The use of such artificial lakes for fishing, however, should not be considered in terms of nature preservation, but from the points of view of sports, commerce and tourist trade.

Underground water levels in the flanks above the reservoir area remain practically unchanged. Hardly any microclimatic effects have been established even in the immediate vicinity of the lake shores. Therefore, the vegetation above the maximum water level in the reservoir also remains unchanged during normal operation, as has been confirmed by a great number of studies.

As to tourist trade, experience has shown that the large Alpine reservoirs have become outstanding centres of attraction. The reservoirs at Lünersee (Vorarlberg), Schlegeis (Tyrol), Mooserboden (Salzburg) and Malta (Carinthia) have been visited by many more than 200 000 persons every year during the summer months ever since their construction, to name but a few examples. This great attraction is a result of the easy access afforded by the construction of traffic facilities for the project sites, and of the fact that the reservoirs are practically full during this season. When the reservoirs are more or less empty towards the end of the winter season, they are hidden under a snow cover, which does not melt until May or June, especially at high levels, so that the slopes below top water level look hardly different from those above top water level (Fig. 5).

Aspects of safety certainly play an essential role, in particular where high dams are concerned. A cursory look at the world statistics will show that about 1% of the existing large dams have failed. Looked at more closely, the statistics reveal that while almost 5% of the large dams constructed before 1900 failed, this percentage has decreased steadily for the dams constructed in our century and has dropped to less than 0.2% for the dams constructed between 1970 and 1980 (Widmann, 1990). It should be mentioned in this context that the main causes of failure are excessive floods – which at the more recent dams can be safely handled by appropriately dimensioned spillways – and incidents during first filling of reservoirs, often in the absence of suitable supervision. The improvement in design and construction methods and in particular in supervision methods over the last few decades has contributed a great deal to dam safety.

3.2.2 Reduced-flow sections

At the stream intakes, inflow is diverted – wholly or partly – and conveyed to the reservoir. The reduced flow in the affected stream bed section has certainly an influence on life in the water and on the bed load situation. The new spring developing below the intake, unlike reservoirs, may be affected by annual floods, whereas flushings of stream intakes and sand traps risk to cause definite sedimentation, if any, only over short stream bed sections of a few hundreds of metres.

As concerns life in the water, potential changes should be classified as follows:

- Changes concerning microorganisms, which can live on very small quantities of water.

- Results of investigations carried out so far have proved that the diversity of species is preserved above the section of reduced flow and that within that section – perhaps with the exception of the section down to the first tributary stream coming from the valley slopes – the diversity of species gradually reaches its original level along with increasing distance from the intake, although the number of individuals may be lower due to the smaller amount of water available. The interruption caused in the stream – as is actually also the case for cascades, waterfalls and torrent control structures –

Figure 2a 570 000 m tip of excavated rock at 1 800 m a. s. l. three years after completion (Zillergründl)

Figure 2b 210 000 m tip of excavated rock in the Salzach valley, twenty years after completion

Figure 3 Hierzbach stream intake, 1.8 m /s, in design flow, Glockner-Kaprun development

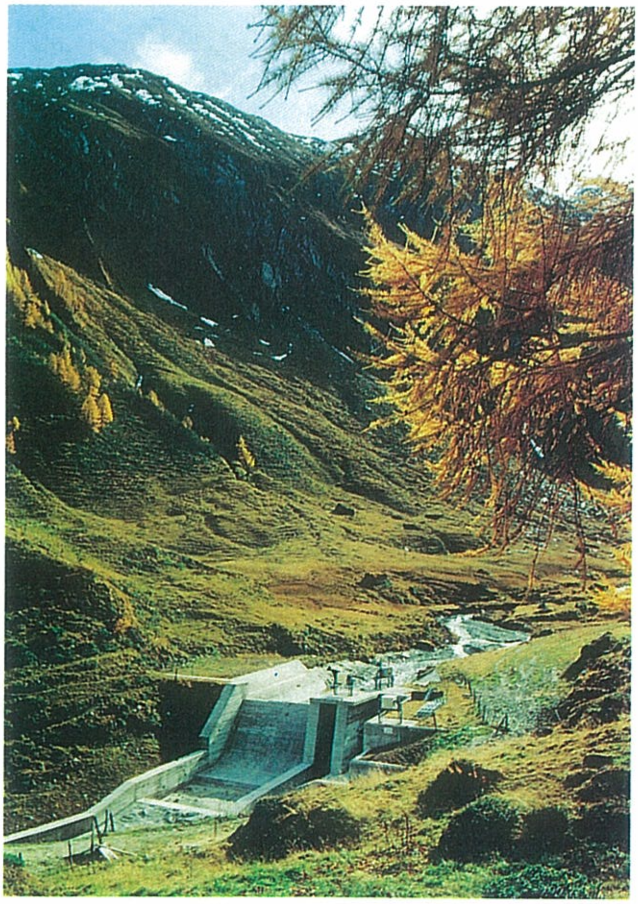
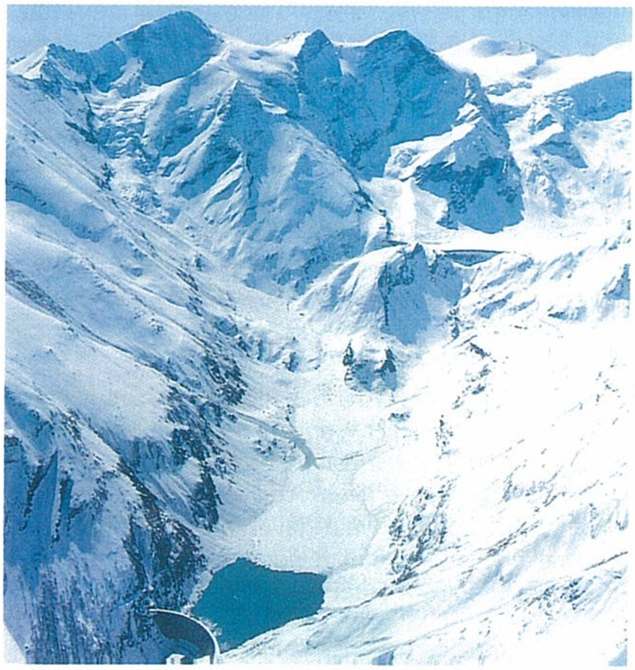


Figure 5a View of Mooserboden and Wasserfallboden reservoirs, Glockner-Kaprun development, in April



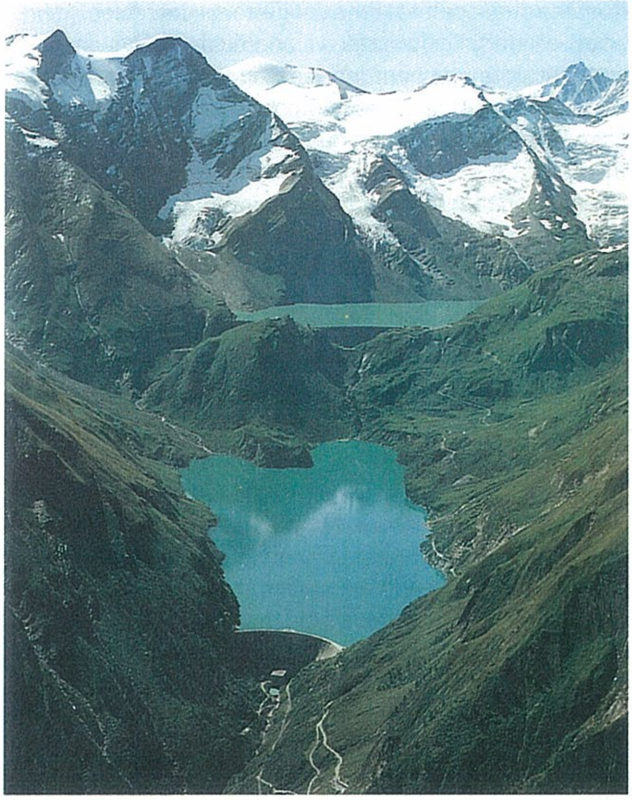


Figure 5b View of Mooserboden and Wasserfallboden reservoirs, Glockner-Kaprun development, in August

Figure 4a Rosshag power station, Zemm-Ziller development

Figure 4b Mayrhofen power station, Zemm-Ziller development

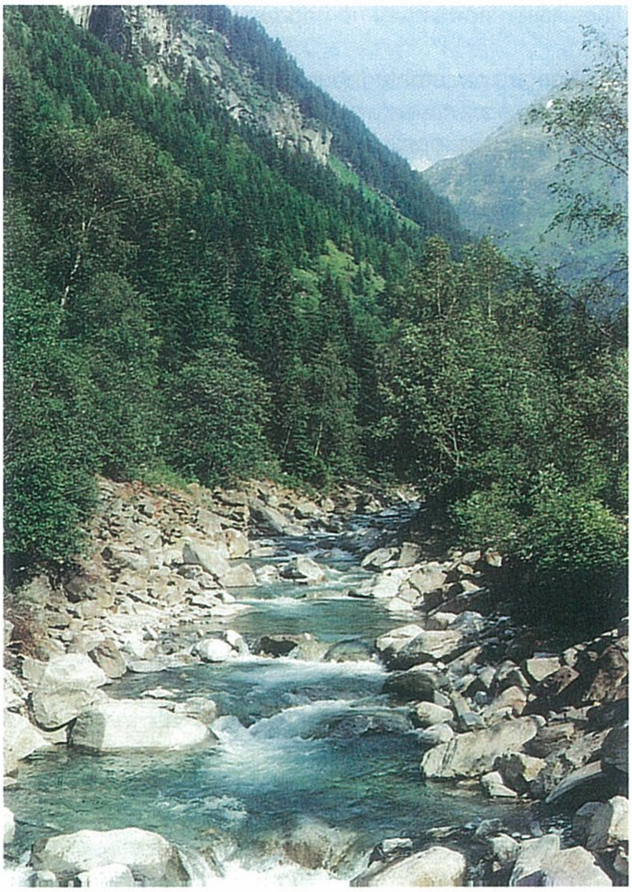
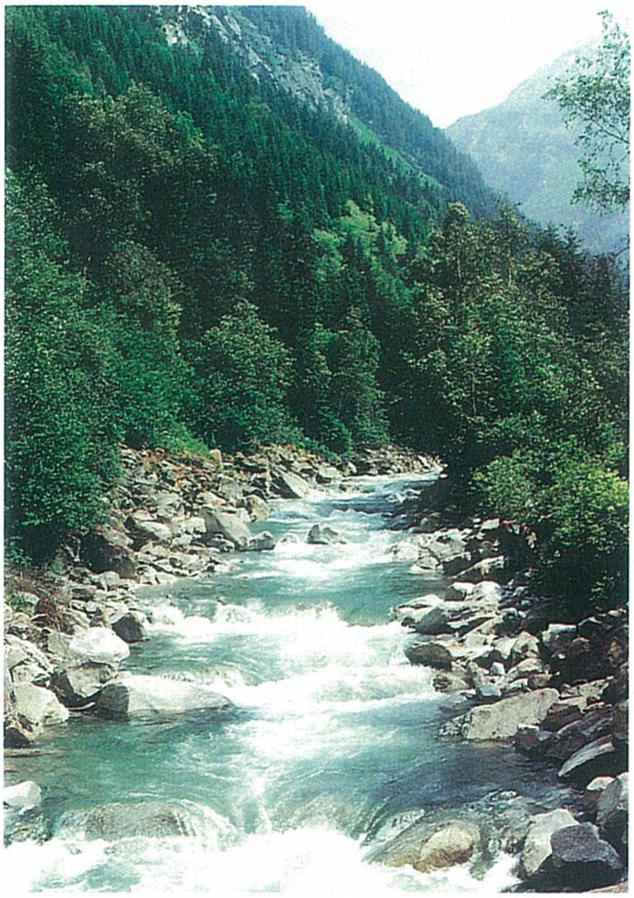


Figure 6a View of Ziller stream 1.2 km downstream of the stream intake, 1.2 m /s

Figure 6b View of Ziller stream 1.2 km downstream of the stream intake, 4.4 m /s



are assumed to have no appreciable effect.

- Changes concerning the colonisation by fish, unless this has not anyway been hampered hitherto by steep grades or sections occasionally falling dry.

- Many protective measures for fish, however, are taken not so much out of ecological concern, but to protect sports and economic interests of fishery, which in fact ought to be rejected for the sake of animal protection and should be regarded only in terms of economic use. It is this economic use that forms the background for the measures taken in most of the fish waters to increase the stock of fish, often by putting in young fish alien to the location. In order to maintain a certain – although reduced – stock of fish, a relatively large flow is needed, which is generally not reached until several kilometres downstream of the stream intake, when stream flow has normalised thanks to the tributaries from the valley slopes.

- Apart from the above ecological changes, there is the change in the general setting that should be mentioned in this context. A stream bed abounding in water is certainly an impressive sight to the alpinist, enriching the landscape at the points accessible to him. Evaluation of this optical change is, however, a difficult task. Where the surrounding landscape is included in the consideration, the change will be small. Where the stream only is considered, the change will be substantial.

In order to minimize even the above effects, the release of compensation water at the stream intake is usually imposed, although the exact amount of compensation water to be released is difficult to determine, as evidenced by the great number of formulas available for this purpose. In several instances, the release of compensation water has been imposed even for the winter season for limnological reasons. Actually, the determination of the amount of flow to be left in the stream bed to answer fishing or esthetic requirements still involves problems. Naturally, the power development owners will be in-terested in minimizing compensation water claims, as these reduce the amount of diverted flow and electricity generation and, hence, increase the cost of electricity generation for a constant cost of construction. Ecologists in turn will want a maximum of flow to remain in the stream bed. This problem is aggravated by the fact that there are only few scientifically supported results available to throw light on the true ecological effects of water abstraction or reduced flow. The small number of limnological studies undertaken in reduced-flow sections of mountain streams appear to confirm the assumption that even without the release of compensation water at the intakes, stable and limnologically satisfactory conditions remain below the intakes, and that there are no provable changes above the intakes. The same is true of the vegetation on the banks of streams.

Normally a relatively small flow is sufficient to maintain the aspect of a true stream. The exact flow rate required is determined with the help of a series of photos taken from the same point at times of different flow rates. In fact, above a certain flow rate, in particular in the case of waterfalls, the outward appearance of a stream has turned out to remain almost unchanged (Fig. 6). In many cases, therefore, the amount of compensation water releases has been determined on the basis of such photographs to the effect that water must be released at points that are considered important to tourism and at those times where they are likely to be visited by tourists.

Contrary to the general opinion, the bed load regime in reduced-flow sections is little influenced. Stream and river courses in the high mountains are rarely in a state of equilibrium. Actually, the construction of hydro power facilities has been used as a ready opportunity to provide stream engineering structures in the area concerned as a stabilising measure where the natural conditions tend to be critical. Even in natural streams, the bed-forming flow rate beyond which bed load transport begins is exceeded only on a few days a year, that is, at times of major flows, and these are little reduced by the diversions. Erosional action on the stream banks is certainly reduced by the lower flow rate in the stream bed. Whereas mudflow from the tributary streams is independent of power scheme operation, the evacuation of the mudflow material in the stream bed may become more difficult due to the reduced flow rate. It will have to be decided in every individual case whether to provide debris dams to keep off bed load material, or whether to provide for the clearing of the stream bed after bed load has been deposited. Both alternatives will have to allow for aspects of safety and ecology.

The design studies for the earliest hydro schemes in Austria already considered the bed load question, and appropriate measures were taken where necessary. In this respect, our large hydro developments in the Alps have never presented any problems that have not been successfully handled by the operators of these facilities with the help of the devices available. Thus it can be stated that on principle bed load problems are not caused by hydro facilities and arise only in those stream sections where the natural bed load regime is unstable. In such cases, however, the necessary measures taken during the construction and operation of the hydro development will constitue an improvement over the original condition.

3.2.3 Power conduit

The water conveyance structures leading from the power intake to the reservoir, from the reservoir to the powerhouse and in some cases also from the powerhouse back to the stream bed include several galleries and shafts. As in the case of tunnelling for traffic facilities, the excavation of underground openings for hydro schemes risks to affect the underground water level. When water-bearing joints or springs are encountered during heading driving, these must be sealed off as rapidly as possible, if only to enable construction activities to be continued. A lowered underground water level resulting from the water draining into the tunnel might affect springs utilized for drinking and service water supply.

Thus, fact-finding procedures preparatory to hydro projects have always included a survey being made of the springs situated within the assumed sphere of influence of construction operations. The annual patterns of yield are recorded in agreement with the proprietors. As many springs depend on the amount of precipitation, it is useful to have records extending over several years so as to establish weather-dependent variations in yield. If these measurements are continued after the completion of the project, which is usually done only where there is reason to suspect the presence of an influence, it is possible to arrive at an objective appraisal of potential changes. Where such changes result from tunnel construction, the owner is obliged to provide for compensation equivalent both in quality and quantity.

As the underground water level is often several hundreds of metres below the ground surface even without influence from the construction activities, except at the points of emergence of springs, there are no direct effects on alpine vegetation, which is dependent on precipitation and capillary water.

Finally it should be mentioned that water emerging from the tunnel during construction should be examined for potential contamination and the appropriate treatment plants provided where necessary so that only sufficiently purified water is returned to natural stream. The shaping of material dumps has been dealt with under 2.1 above.

3.2.4 Flow conditions below the powerhouse

This refers to the natural and artificial channels and ducts below the powerhouse. The distance between the powerhouse and the natural stream bed is usually covered by an open ditch or, in the case of underground power stations, by a gallery.

Consistent with the purpose of storage schemes, full-load admission on the power units is often required within a few minutes. This is followed by an equally rapid increase in flow in the outlet conduits. In order to prevent the water level in the stream course to rise too rapidly as a result of the increase in flow, compensating reservoirs have been provided below several powerhouses. For reasons of safety, they are designed so that the water level rise in the stream does not exceed 10 cm per minute (Mayrhofen, Uttendorf).

The power units of storage schemes run as required, usually during several hours on weekdays. As, however, the compensating reservoirs only slow down the water level rise, but cannot equalise the entire discharge wave, there are daily water level fluctuations in the stream bed below the outlet works which usually exceed the natural water level fluctuations during that particular period. However, the water level fluctuations due to power station operation decrease in amplitude along with increasing distance from the power station as a result of the natural retention capacity of the stream, the increasing basic flow and the increasing width of the stream. As detailed investigations performed in the Ziller valley have shown, the influence of these water level variations on the water table in the valley bottom decreases rapidly with increasing distance from the water course, as in the mountain valleys the water table is characteristically more affected by the water from the flanks than by the stream. At any rate, no effect on the vegetation of the valley floor has ever been found to exist, as actually the water table is usually low enough not to have a direct influence on vegetation.

In fact, the increased flow below the outlet works of storage schemes during the low-flow winter months is a most welcome condition helping to ease the sewage problem, and may be a definite advantage over the natural conditions, especially in areas where tourist trade concentrates during the winter season. The corresponding flow reduction during the summer months is practically not felt.

4 SMALL POWER SCHEMES

Under Austrian law, power schemes with rated capacities smaller than 10 MW are termed small power schemes. Using the corresponding rated discharge as a first measure of potential ecological implications, a rated discharge of 10 MW results in

a rated discharge of 125.0 m^3/s for a head of 10 m,
a rated discharge of 12.5 m^3/s for a head of 100 m, and
a rated discharge of 2.5 m^3/s for a head of 500 m.

This means that the diverted flows from stream intakes for small power schemes may be of the same magnitude as that of diversions for large power schemes and should also be studied for ecological effects. Power intakes for small diversion-type schemes are designed for a specific rated discharge of 25 to 40 $l/s.km^2$. This means that the flow remaining in the stream during the high-flow summer months is certainly larger than in large power schemes, whose intakes are designed for specific rated discharges of between 150 and 300 $l/s.km^2$. During the low-flow winter months, however, the conditions in the reduced-flow sections are comparable, and all the changes resulting from the interruption of the stream course are very much like those occurring in large schemes. As to the surrounding landscape, the effects of small power schemes, given an appropriately unobtrusive outward appearance, are generally negligible. But as soon as the benefit in terms of power economy is included in the comparison, the following aspects should be borne in mind:

- Savings in power lines by small power schemes are possible only if the facility is not connected to the public system, that is, if there are no provisions for evacuating surplus power or receiving supplies in case of deficiency. This will often imply not only a lower level of economy but also a lower level of security of supply, which should be taken into account by any company operating a power scheme.

- Naturally, the smaller size of the power scheme implies lower expenses for access facilities, construction and operation, which is to be welcomed with respect to the environmental implications. On the other hand, however, this means that any compensating measures that may become necessary to improve environmental conditions, infrastructure, etc. will be difficult to finance for economic reasons and will hardly bring any appreciable additional tax revenue to the community concerned.

- There are at present about 280 small power stations in the province of Tyrol. Assuming that these are all situated on different streams, it should be pointed out that utilization of 280 streams corresponds to only 79 % of the annual energy and only 48 % of the winter energy generated by the Zemm-Ziller group of power schemes, although this develops only 18 streams.

Therefore, the construction of small power schemes will be reasonable only where a connection to the public system is not possible or not required, but small power schemes should not be allowed to prevent a much more efficient use to be made by large-scale development of the flows concerned.

5 SUMMARY AND CONCLUSIONS

Satisfying human needs, as those of food, accommodation or e.g. the freedom of travelling, causes changes to the environment which grow along with increasing population density and increasing demand for luxury. Thus, the demand for energy, resulting more or less from an endeavour to satisfy these needs with a minimum of muscular strength, can be met only at the expense of certain changes in the environment which are hard to reconcile with the fundamental idea of a preserving protection of nature. Electrical energy is both the most comfortable and the cleanest form of energy and is first in versatility of use. Therefore, the demand for electrical energy will continue to rise even if savings in overall energy consumption are achieved.

When trying to appraise the ecological repercussions of man's interference with natural systems, and thus also the various methods of electricity generation, we should differentiate between temporary and lasting effects, as well as between local, regional and global effects. Temporary effects from construction activities can be remedied by careful rehabilitation and planting after site clearing. Lasting regional or global effects from hydro developments of a magnitude feasible in the Alps do not result. Lasting local effects have been discussed above. The conclusion that can be drawn from the foregoing is that lasting local effects are confined to inevitable changes

- of the village or landscape concerned due to visible buildings and structures and

- of flow in certain stream or river sections.

Effects beyond the above, whether in the form of air or water pollution or only a local change in climate or vegetation, will certainly never occur. The effects from changes in the flow regime can be limited to the stream bed itself and within the stream bed can be reduced to a minimum by taking the appropriate measures. In order to achieve this, it is necessary that cooperation be maintained between

- the experts in the various ecological questions who have the necessary understanding of technical problems and requirements and

- the experts in the design, construction and operation of hydro power schemes, who should include environmental aspects in their considerations for the solution of technical problems.

Given an understanding cooperation between the two parties, it will probably always be possible to arrive at a satisfactory solution in the sense of a creative preservation of nature.

The construction and operation of power schemes is dictated by a growing energy consumption. Any effort – understandable as it may be – to prevent developable sections of streams and rivers from being utilised for the generation of electrical energy and to leave them in their natural condition should at the same time make allowance for the fact that every kWh from hydro saves a kWh from thermal or nuclear power plants and thus contributes to the reduction of regional or even global repercussions.

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Chapter F

CONCRETE DAMS IN AUSTRIA

CONTENTS

CONCRETE DAMS IN AUSTRIA

By R. Widmann*

DEVELOPMENTS AND EXPERIENCES

1 INTRODUCTION

Austria's large dams – a term referring to dams more than 15 m high according to ICOLD recommendations – were almost exclusively concrete structures until well into the sixties. This was due to the fact that, during the first decades of dam construction only those sites were feasible where foundation on bedrock was possible. Another prerequisite to the realization of concrete structures was met by the presence of a powerful concrete industry, whereas the science of soil mechanics was still in its infancy at that time.

Therefore, it is not surprising that by the end of the Second World War about 10 gravity dams up to 28 m high and one 39 m high arch dam, but only a single embankment dam, 17 m in height, had been constructed. Dam construction on a larger scale did not start until after the Second World War; the following is a report on the large dams constructed since that time.

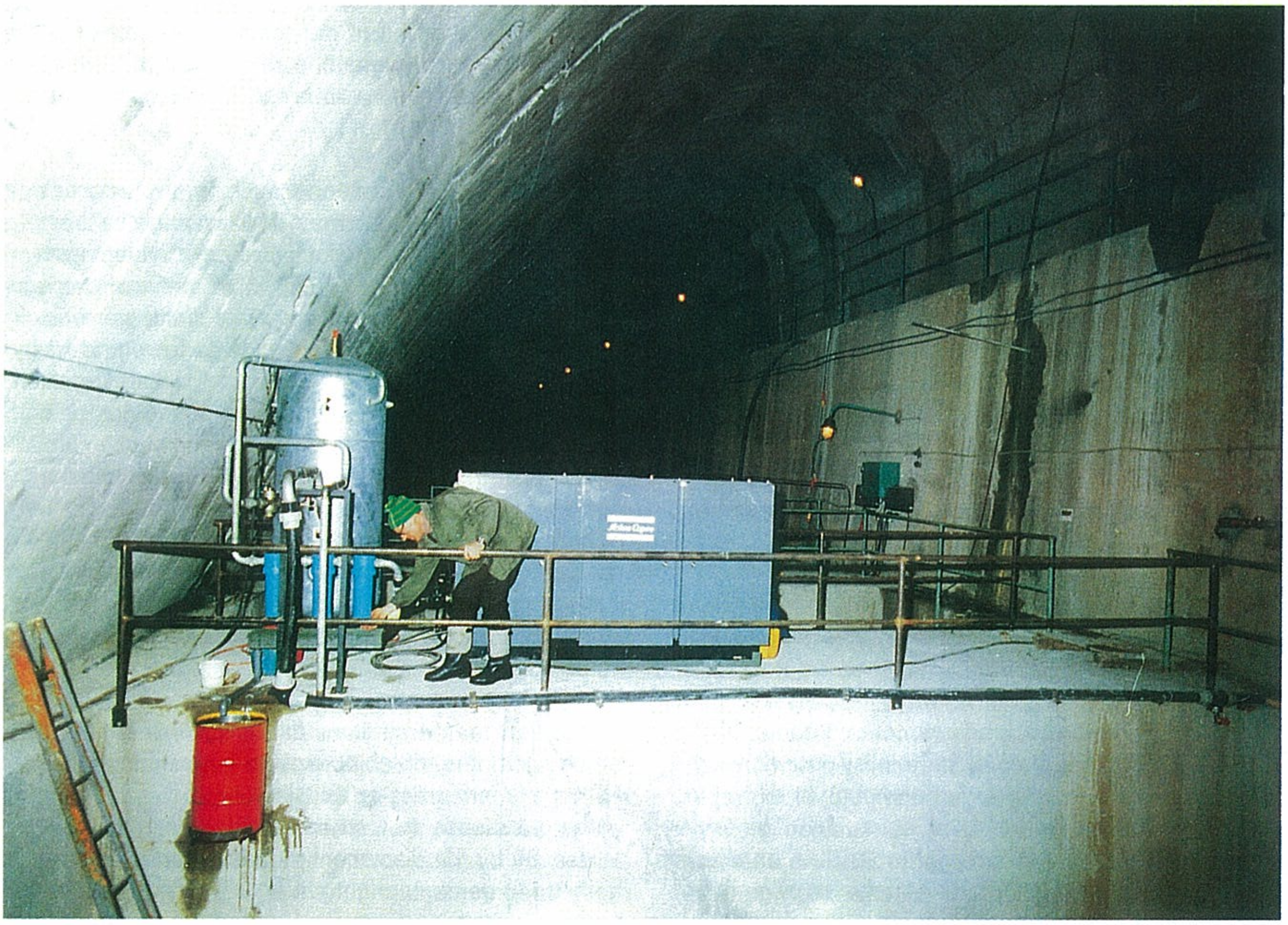
2 HIGHLIGHTS OF DAM HISTORY

2.1 Gravity dams

We all know that the principle underlying the design of gravity dams, employed already in antiquity, is founded on an apparently simple static system: In each individual concrete block, the weight of the concrete should be large enough to deflect horizontal water pressure downwards to a point where the shear and friction forces are transferred to the foundation to the extent of its bearing capacity.

A phenomenon which was not sufficiently understood at the time of the first gravity dams was, however, that seepage under the dam causes uplift pressure. This is why at some of the Austrian gravity dams dating from that period remedial measures have been required to reduce uplift pressure.

Figure 1 Large gallery on the rock surface at Grosser Mühldorfersee Dam



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Especially in gravity dams of major height, the horizontal component of the abutment forces, corresponding to the horizontal water pressure, may reach a magnitude that makes it necessary to satisfy high requirements in respect of dam base configuration and mechanical properties of the foundation rock. This implies that gravity dams, especially the high ones, call for substantial thicknesses, which may pose difficulties in respect of concrete technology. In addition, local zones of weakness in the foundation may severely affect the block concerned and, therefore, the whole structure.

Attempts to reduce uplift pressure and concrete volume, but otherwise retaining the same static system, have led to a great variety of complex dam types – the round-headed buttress dam is a typical example – where the increased formwork requirements, however, tended to neutralize the advantage of a reduced concrete volume. In the early fifties, a first attempt was made in Austria to reduce uplift pressures by providing a gallery of adequate size directly at the dam base with a cross section made to harmonize with the lines of force. This measure allowed the weight of the dam and, hence, concrete volume to be reduced for a constant safety against sliding (Steinböck, 1959). Another advantage was that this design reduced the problems of concrete technology by dividing the lower portion, where the dam has its greatest thickness, into two sections. The formwork, being the same over the whole length of the dam, is comparatively simple and has proved economical in several gravity dam projects (Grosser and Kleiner Mühldorfersee, Hochalmsee, New Tauernmoos dams, Fig. 1).

Another possibility of reducing concrete volume is to design a gravity dam curved in plan and to grout the vertical joints so as to enhance the sliding resistance of the individual block with the help of the arch action of the whole dam (Mooser, New Tauernmoos). Naturally, the adequateness of such a solution depends also on the topography of the dam site, but the additional expense from grouting the vertical joints can be much more than made up for by the savings in concrete volume for the same, or a higher, sliding resistance (Schüller, 1954).

As stresses are relatively low in massive gravity dams, a moderate concrete strength would be sufficient. However, the durability requirements, such as frost resistance and imperviousness to water, especially for the concrete surfaces, preclude low cement contents. The usual answer to this problem is to provide a rich facing concrete to a thickness just large enough to allow proper placing and compaction. In some Austrian gravity dams (Grosser and Kleiner Mühldorfer See) an attempt was made to use formwork of precast high-quality concrete slabs for the upstream dam face, which was then backfilled with the lean concrete needed for stabil-ity. This system guarantees a durable and smooth surface, but has posed problems especially with respect to the design of the joints between the precast slabs, probably due also to the extreme climatic conditions prevailing at an altitude of 2 300 m above sea-level. Repair became necessary after about 10 years (Magnet, 1970). Many years' experiences gathered in the application of this system can be applied to advantage in the use of rolled concrete.

The Spullersee North and South gravity dams, constructed from 1921 to 1925, were heightened by 4 m in the years 1963 to 1965 using corrosion-protected prestressed anchors. Subsequent checking, however, gave rise to doubts regarding the effectiveness of these anchors. Detailed investigations showed that it was possible to make up for the increased stresses resulting from the raised headwater level by reducing the uplift pressures to a minimum. This lasting reduction of uplift and joint water pressures was achieved by providing an adequate drainage and surveillance system (Flögl, 1991).

2.2 Arch dams

The first Austrian arch dam was constructed during the Second World War on the Gerlos river, a tributary to the Ziller river in Tyrol (Grengg, 1948). An artificial abutment was needed on the left side. Some 20 years later, weathering of the rock led to a shallow rock slide on the right-hand slope near the downstream face of the dam, which gave rise to a large-scale stabilization scheme in 1964 (Horninger, 1967). Recently, however, slow but steady swelling of the concrete, probably due to alkalinity in the cement, has reached a magnitude that will call for a general repair in the course of the next decade (Flögl, 1991).

By the end of the fifties, another eight arch dams had been constructed. The largest of them was the Limberg arch gravity dam of the Glockner-Kaprun development, which reached a height of 120 m. In all the arch dams dating from that time, the horizontal sections were circular arcs shaped as funicular curves to withstand hydrostatic loading. It was already for the Limberg arch-gravity dam that the circular arcs of the upper dam portion were lengthened by a straight section to account for the specific features of the valley configuration at the dam site. At three of the arch dams of medium height constructed in the fifties (Ranna, Bächental, Dobra), it was attempted to prestress the arcs by joint grouting to improve stress distribution in the dam structure. In applying this system it should be borne in mind, however,

- that vertical tensile stresses at the downstream dam toe should not reach objectionable magnitudes and
- that the prestressing is likely to decrease within a few years due to the creep of concrete, as evidenced by the displacement measurements taken at these three dams during the first years of operation.

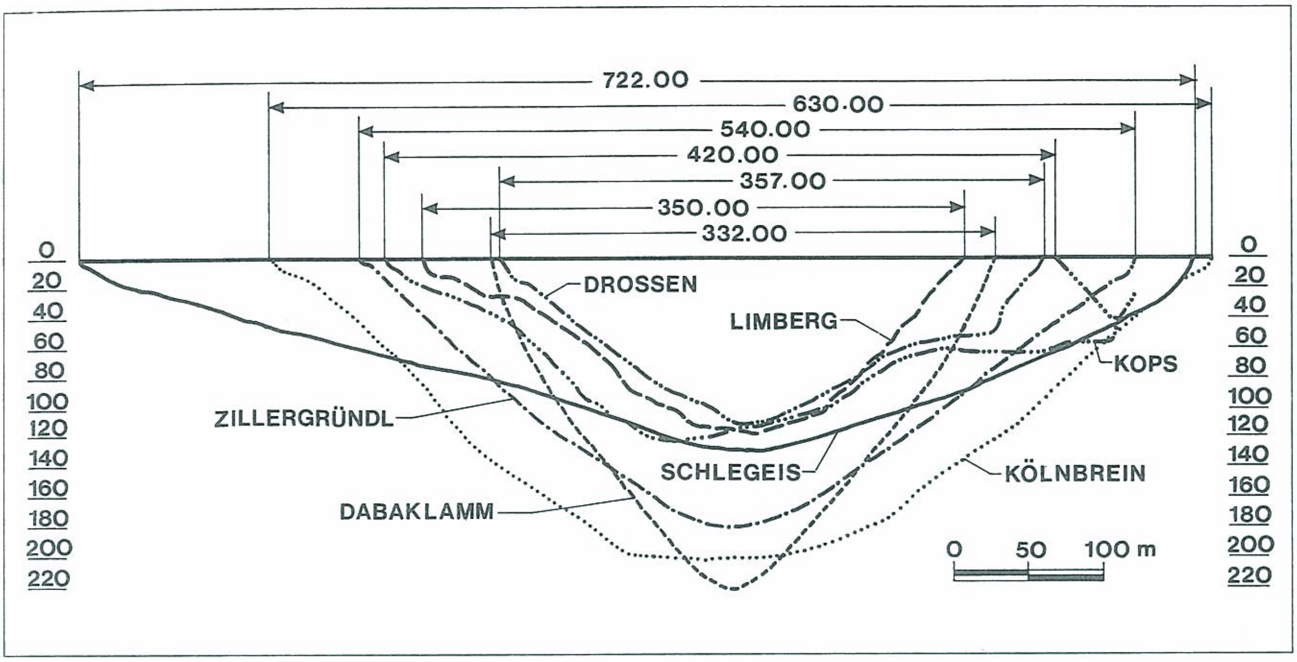
At Dobra, during maintenance work on the power conduit in 1953, a minimum flow of 5 m³/s had to be released through the bottom outlet, so that the whole gallery was subjected to the full hydrostatic load. Leaks in the lining led to the build-up of an increased joint water pressure and to a local rock slide immediately downstream of the arch dam (Petzny, 1956). Since an impervious tunnel lining was provided, there have been no further problems.

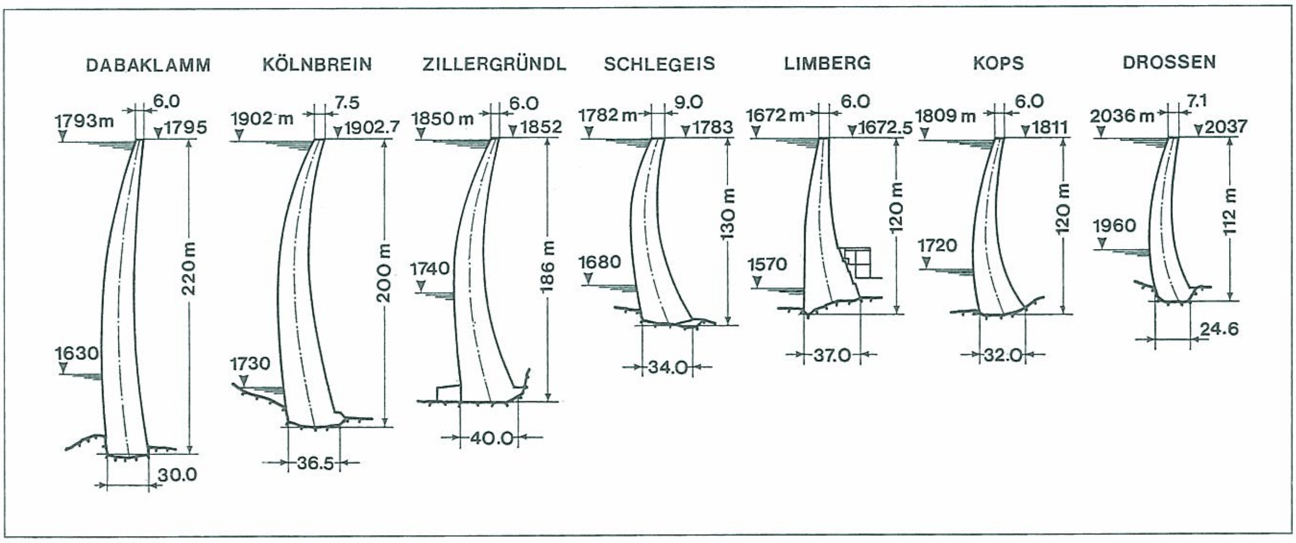
It was in particular for wide-span arch dams that the introduction of variable-curvature arcs for the horizontal sections appeared useful in order to achieve, by the larger curvature of the centre portion, an increased rigidity of the arches and also to obtain a second parameter for an improved adjustment to the statical and topographical requirements. For the Kops arch dam parabolas of the fourth degree were adapted for the horizontal arches.

The Schlegeis arch dam presented difficulties because of its extremely unfavourable crest-length to dam-height ratio of 5.6. Comprehensive design studies were required to find the optimal dam configuration. The best results were obtained by horizontal sections with a curvature decreasing from the crown towards the abutments in the upper dam portion and increasing from the crown towards the abutments in the lower dam portion. This design ensured both maximum possible rigidity for the arches in the uppermost dam portion and minimum abutment moments for the bottom portion. As in some places in the region of the highest dam blocks the rock mass yielded less than expected, a tension zone marked by the breaking-up of joints developed below the upstream dam toe (Heigerth, 1976). Extensometer measurements showed that this tension zone was limited to a layer of rock mass 2 m to 3 m in depth, where flat joints had opened. Repair consisted of providing an only 5 m deep elastic slurry trench cut off especially developed for this purpose. Reservoir operation was fully maintained during the repair (Stäuble, 1991).

Much in the same way as for gravity dams, the lowest inspection galleries of arch dams were placed directly on the rock surface. This ensured optimal reduction of uplift and ground water pressures, but risked to allow water to enter the base gallery in case the upstream base joint opened, as was the case at Schlegeis.

Figure 2 Cross sections and longitudinal sections across the valley of Austrian arch dams higher than 100 m





At the small arch dam at Klaus constructed only a few years later, provision of a base gallery immediately on the rock surface was again very successful. But at Kölnbrein, the largest arch dam in Austria constructed in the seventies, the lowest inspection gallery was no longer placed on the rock surface. Optimal dam design brought a substantial reduction of the bending moments, allowing a relatively thin structure to be realized. The horizontal water load that acted, parallel to the valley axis, on the upstream dam face attained more than 5.2 million t and led to high shear stresses at the dam base, where presumably in combination with the inevitable residual restraints from the construction period, oblique main tensile stresses built up which exceeded the tensile strength of the concrete. Water was forced under high pressure into the cracks that originated at the dam base, so that finally the upstream dam toe was separated from the rest of the dam structure at the valley bottom (Baustädter, 1985).

The resulting underseepage in this zone of the dam was substantially reduced in the years 1981–82 by relocating the grout curtain further upstream and by providing an apron between grout curtain and dam structure (with a rigid connection to the grout curtain and an elastic connection to the dam). In this way it was possible to eliminate almost completely the very high uplift pressures, which had extended to a point very near the downstream toe of the dam, and to reduce drainage flows to relatively low levels. In a second phase, the transverse forces within the dam body will be transmitted, via a bearing device, to a supporting structure placed against the downstream dam face, so as to ensure a large measure of load reduction for the existing dam structure (Ludescher, 1991).

Simultaneously with Kölnbrein arch dam, two smaller dams were constructed at Sölk and Paal in the Niedere Tauern mountains.

In the years 1982 to 1986, the 186 m high Zillergründl dam was constructed. It is at present the youngest of the high arch dams in Austria. Apart from placing an upstream apron over the zone where the dam is more than about 80 m high so as to allow the grout curtain to be located further upstream, a perimetral joint was provided at the valley bottom between the upstream dam face and the base gallery, which was again placed directly on the rock surface (Widmann, 1983). This perimetral joint was sealed on the upstream side and therefore was practically without pressure. When a hydrostatic head of 175 m was reached, the joint opened slightly and a parallel crack in one of the blocks developed, running from the upstream face to the base gallery, although normal loadings could not have caused any major concrete stresses in a vertical direction in this area. After a 40 m drawdown the crack closed almost completely and was grouted with Rodur so as to ensure the transfer of tensile forces. In order to accomplish a lasting improvement of the stress condition in this area, a water pressure corresponding to about half the full hydrostatic head was introduced in the perimetral joint. Since this measure was taken, normal reservoir operation has not presented any further problems.

The Dabaklamm project provides for a 220 m high arch structure, which would be the highest dam in Austria. Due to the favourable valley shape, its concrete volume would remain under about 1 million m^3. Although drillings, galleries and rock mechanics testing both in the laboratory and in situ have established the adequacy of the rock mass at the dam site, an embankment dam has been studied as an alternative project to account for recent developments and has finally been selected for construction because it involves the smaller loadings on the foundation. Construction of this dam and power station has not yet been started because of resistance on the part of environmentalists.

Fig. 2 is a diagram comparing the geometrical characteristics of the Austrian arch dams exceeding 100 m in height. Differing valley shapes and the design principles in respect of dam configuration developed in the course of almost three decades may render such a comparison interesting.

3 APPURTENANT WORKS

Design flows being fairly small by international standards, reaching maximum values of
100 m^3/s for power intakes (Ottenstein arch dam),
400 m^3/s for bottom outlets (Klaus arch dam),
650 m^3/s for spillways (Ottenstein arch dam),
these structures have moderate dimensions, but – perhaps for this very reason – allow interesting engineering solutions.

Bottom outlets are used for occasional flushing of empty reservoirs and for emptying the reservoir in an emergency. Design flows for bottom outlets are usually selected so as not to exceed the 50-year flood recorded for the downstream stream or river bed prior to the construction of the reservoir. A buried stilling basin was developed for energy dissipation, which was constructed for the first time for the bottom outlets of the Schlegeis arch dam (Widmann, 1973). As this design was successful, the bottom outlets of the Limberg arch dam were reconstructed according to this system, and it was also this design that was adopted for the new bottom outlets of the Zillergründl arch dam. A feature of particular interest is vibration absorption by the underground location of the basin, limiting vibrations to a range which does not represent a poten-tial cause of structural damage even in the long term.

Flood relief works are usually ungated spillways (Simmler, 1979). An exception is the Mooser gravity dam, where crest gates were provided. The design is based on the assumption of a flood wave corresponding to a PMF with a probability of occurrence of 5 000 years meeting a full reservoir. The volume provided for retention is only the space available above top water level. Turbine or bottom outlet operation is usually not permitted to be allowed for in the design. Energy dissipation is accomplished in stilling basins of

Figure 3 ~~Paal~~ arch dam with flood discharge
Großsölk

special design, especially at those dams where occasional overtopping is expected. At relatively small arch dams as Sölk and Paal (Fig. 3), overflow is over the entire dam crest, the discharge being conveyed along both flanks along the downstream dam toe in a stepped channel, where the two jets meet and the rests of the energy contained in the dropping sheet of water are neutralized. Gates used in appurtenant structures are all subjected to routine maintenance and to an annual functional check.

4 ASPECTS OF CONCRETE DAM DESIGN

4.1 Remarks on the design of the dam structure

4.1.1 Stability aspects of gravity dams

The apparently simple stability analysis for gravity dams consisting of statically independent blocks is valid only on the condition that the uplift pressures underlying the design are not exceeded. Comparative analyses (Widmann, 1974) have shown that where the forces due to the uplift pressure underlying the design are exceeded by not more than 2% as a result of additional uplift pressures occurring in the new crack, the stability of the dam is already jeopardized.

4.1.2 Stability aspects of arch dams

Up to the last decade, the design of arch dams was generally based on the load distribution method. Already for the first large arch dams, load distribution was calculated by means of a system of equations (Jurecka, 1949) also for multi-cantilever grids. These first analyses made allowance for radial displacement only. Then, step by step, all the other deformations were included in the adjustment.

Abutment deformations were accounted for by spring constants adjusted to suit the geological conditions along the circumference of the dam. These may be introduced in the analysis so as to make approximate allowance for anisotropy in different directions. The influence of horizontal cracks of limited extent in an upstream portion of the foundation contact on the stability of arch dams proved to be small (Eiselmayer, 1975).

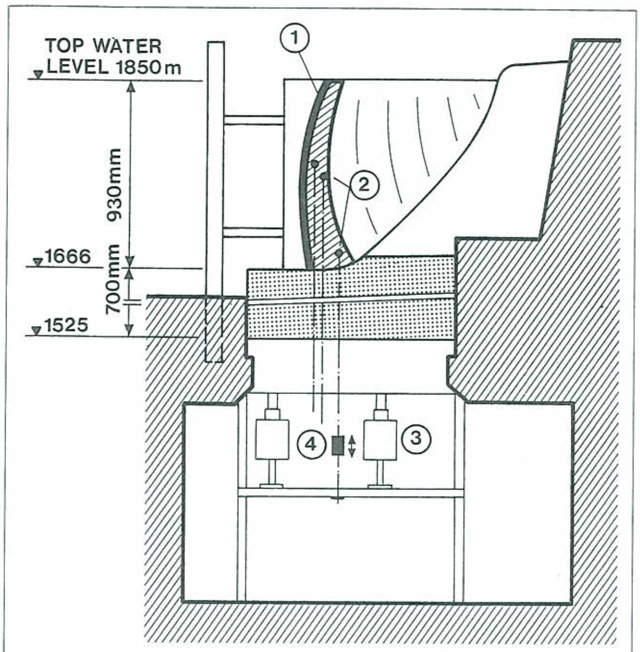
Over the last two decades, the finite element method has been used for check computations. At the same time, an attempt has been made to account for the effect of the geological structure of the foundation on its mechanical properties (Poisel, 1988). A very simple method of estimating without any major computation effort the stresses expected to occur in a dam is the determination of the loop stresses (Lauffer, 1960). Experience has shown that later, following optimal design and exact calculations, these calculated loop stresses are rarely exceeded by the main stresses.

4.1.3 Statical model tests

Due to their generally plane state of stress and deformation gravity dams generally require no model tests. It is only for establishing the effects of an enlarged base gallery on the pattern of stresses within the dam structure that photoelastic tests were carried out some time ago (Steinböck, 1959).

As to arch dams with their three-dimensional load-bearing behaviour, however, it is still necessary to perform model tests to check the stability analyses, although the use of computers has made it possible to make design studies approach reality very closely. In Tauern-

Figure 4 Model test installation for Zillergründl arch dam



kraftwerke's test laboratory at Kaprun, many model tests were carried out for all arch dams in Austria and some arch dams abroad since the fifties, using different materials and loading equipment. The statical tests only studied the effect of water pressure on the arch dam. In simulating the foundation, allowance was made for particularly yielding areas.

For the test of the Zillergründl arch dam, a special loading device was developed for applying the dead weight to the statically independent blocks, so that it has become possible since that time to simulate real conditions very closely; it is only after sealing of the vertical construction joints that water pressure is applied and increased until failure occurs (Fig. 4).

4.1.4 Aspects of dynamic dam analysis

Although the Austrian dams are almost all situated in regions where disastrous earthquakes are not expected to occur, research was undertaken in this field in the seventies which included the checking of the results by in situ testing.

If the design is based on a linearly elastic system – an assumption which can be considered as being fulfilled as long as the dynamic tensile strength even of the vertical joints is exceeded only within closely limited areas – the analysis may be reduced to a relatively simple modal analysis, which determines, following the determination of the natural vibrations by means of the earthquake response spectrum, the stresses present in the dam body. The basis for these studies is the well-known vibration equation, which essentially includes the mass matrix (of simple geometrical relationships) and the stiffness matrix. The stiffness matrix is also easily derived from the coefficients of the equation system for the load distribution method (Tremmel, 1961). Since the individual arch elements have already fulfilled a number of static conditions in the load distribution analysis, this method allows the sufficiently accurate analytical design of the dam body with a relatively small number of equations (Eiselmayer, 1982). The resonant mass of water was at first introduced using the approximations by WESTERGAARD, but in a second step was limited in proportion to the distance between the vibration nodes in the various modes of vibration. This resulted in a substantial improvement of agreement between analysis and test (Widmann, 1987). The relevance of this limitation increases along with increasing reservoir depth and is particularly pronounced for the higher vibration modes.

For prototype vibration testing, dams were made to vibrate in a radial direction by means of an excentric. The maximum force of 30 kN was first reached at 2 Hz. The vibrator unit was placed at those points on the crest where the results of the preceding computations had placed the maximum vibration amplitudes for this particular vibration mode.

Analyses and tests were carried out for four arch dams ranging between 60 m and 200 m in height, for low and high reservoir levels. Comparison of the results shows excellent agreement between tests and analyses, especially for the higher vibration modes.

4.2 Remarks on the consideration of the foundation in the analysis

As practised all over the world, foundation investigations for the early dams relied on qualitative rather than quantitative considerations. In order to arrive at a first general idea of foundation safety, Austrian dam designers introduced the term "spread angle" (Ausbreitungswinkel), which is understood to denote the smallest angle between the rock surface and the three-dimensional abutment resultant. For average rock conditions, this angle should be not less than about 25 to 30°. Comparative analyses have shown, however, that the direction of the horizontal component of the three-dimensional abutment resultant is mainly dependent on the valley shape and little influenced by the shape of the arch dam (Promper, 1987).

The foundation in 1951 of a Salzburg circle for rock mechanics was the manifestation of an effort to evaluate the behaviour of the rock mass also in quantitative terms and already during the stage of design.
In the early sixties, design studies for the Schlegeis arch dam included not only laboratory tests on speci-

mens and drill cores (deformation, triaxial and shear tests), but also in situ tests using the radial jack, originally developed for pressure shafts, to determine the modulus of deformation in different directions (Eiselmayer, 1970). At that time also an attempt was made to establish characteristics for the anisotropic unfaulted rock mass (Tremmel, 1970). Similar tests were carried out for the Kölnbrein, Zillergründl and Dabaklamm arch dams. For the latter project, the testing scheme was supplemented by a large-scale in situ shear test on a 20 m^3 rock in a steep cliff and several plate jack tests in exploratory galleries. The problem remains, however, that these tests cover a volume of rock mass that is infinitely smaller than the volume that will later be affected by the structure.

Neglecting for the time being analytical checking of foundation stability as is occasionally required, deformation behaviour of the foundation contact affects the stress pattern in the dam body, especially near the dam base, like the boundary condition of a shell does to a distance from the base approximately corresponding to the thickness of the dam. In the load distribution analysis spring constants, which are generally determined for unit loadings on the assumption of an elastic-isotropic semi-infinite mass, or deformations are introduced at the points of support. For the determination of these deformations, the body of rock mass can be looked upon as a continuum provided it is attempted to simulate a behaviour equivalent to the actual discontinuum on the assumption of "smeared joints". For the purposes of foundation safety considerations, this will be an adequate approach only as long as in zones of structural weakness their strength, particularly against tension and shear, is not exceeded. If this strength is exceeded, major deformations result locally which may severely affect the behaviour of the overall body of rock mass and, hence, the stability of the foundation. In such cases, more detailed studies allowing for these discontinuities will be required.

4.3 Aspects of static design criteria

Much consideration has already been given to the problem of vertical tension stresses at the upstream dam toe, as these are considered to be one of the causes of the difficulties encountered fairly commonly in this area. A parameter study has shown that particularly where the modulus of deformation of the foundation becomes greater than that of the concrete, these vertical tension stresses may increase rapidly (Promper, 1987).

Even when taking the two-dimensional approach to this problem, it will be seen right away that at least upstream of the dam base, horizontal tension stresses must also be present. These are due to the horizontal component of the abutment resultant causing displacements of the dam base towards the downstream. The rock mass immediately upstream of the dam can participate in these displacements only if the corresponding tension stresses can be transmitted (Lauffer, 1967). Whereas the vertical tension stresses can be influenced by the shape and thickness of the dam, horizontal tension stresses resulting from horizontal forces are essentially determined by the height of the dam. It is only the distribution of the horizontal forces along the perimeter of the dam base that can be influenced by an adequate dam shape, unlike gravity dams with statically independent blocks where this is largely predetermined. These horizontal tension stresses may result in near-vertical cracking in continuation of the upstream dam face into the foundation. Their exact shape and location is dependent on the local structure of the rock mass. The cracks not only risk to break up the grout curtain, but may also reduce the loadbearing body of rock mass, thus causing an increase in horizontal displacement for a constant modulus of deformation of the rock mass.

The problems encountered in Austria's three largest arch dams have led to a critical review of design criteria. The results are outlined below:

- Upstream and downstream rotation of the dam base, mainly a function of the stiffness of the dam body, should be compatible with the vertical displacements of the foundation. The usually practised superposition of the stresses resulting from dead load and water load makes no allowance for the permanent part of the settlements resulting from the dam's dead weight. This may result in near-horizontal cracks in the concrete or in the opening of near-horizontal joints in the rock mass, allowing water to enter under high pressure so as to increase the cracking tendency.
- The shear stresses at the base of high and thin dams may cause main tensile stresses in the interior of the dam cross section that are higher than those at the dam surface (Widmann, 1989). This trend may be intensified by the unavoidable residual restraints from concrete cooling after placement and by water pressure within these cracks (Widmann 1991).
- The entrance of water into an otherwise stress-free concrete under a pressure approaching in magnitude of the concrete tensile strength may cause spreading of the cracks, as is easily established by methods of fracture mechanics (Linsbauer 1985, Widmann, 1989). Where tensile stresses are already present, the spreading of the cracks may be prompted already by lower water pressures.

5 DAM CONCRETE

5.1 The cement

In terms of concrete technology, the requirements to be met by a concrete intended for massive structures differ from those to be met by a concrete for slim structures in one essential respect: Particular attention must be given to heat build up and the resulting temperature stresses whereas all the other concrete properties can be appraised according to the usual rules of concrete construction. Up to the mid-sixties, maximum concrete temperature in relation to the temperature of the rock foundation was the only criterion considered whereas the time-dependent stress-strain relationships of the young concrete and the corresponding strengths, in particular tensile strengths, were neglected.

In the years after the Second World War, normal Portland cement had a heat of hydration of 260 Joule at 7 days, which approximately answers the requirements of what is even by present standards considered a low-heat cement. During the decades that followed, the strength, particularly the early strength, of concrete was increased to meet the growing requirements of building and bridge construction, which brought the heat of hydration to more than 340 Joule. The only possibility of reducing heat build-up in the cementing agent is the use of hydraulic additives, which at the same time slow down the hardening process of the cementing agent. Consequently, as the cementing agent consists of components of different chemical reactivity, the adiabatic rise of concrete temperature gives values more relevant for appraising heat build-up than the heat of hydration does.

The first criteria for avoiding temperature cracks were only founded on the maximum temperature increase in the concrete, thus dictating small percentages of a low-heat cementing agent. Later criteria were founded on the time-dependent relationship between tensile strength of concrete and the simultaneous restraints resulting from temperature change.

By considering the crack resistance as the distance from ultimate strain, however, it is possible to avoid the modulus of deformation – which is hard to determine especially for the young concrete – for the conversion to stresses (Widmann, 1985). Determination of this ultimate strain, just like the determination of tensile strength, should make at least approximate allowance for the velocity of strain changes within the structure. Incidentally, investigations have also shown that the use of a cementing agent with minimum heat build-up does not necessarily guarantee a concrete with maximum crack resistance (Fig. 5).

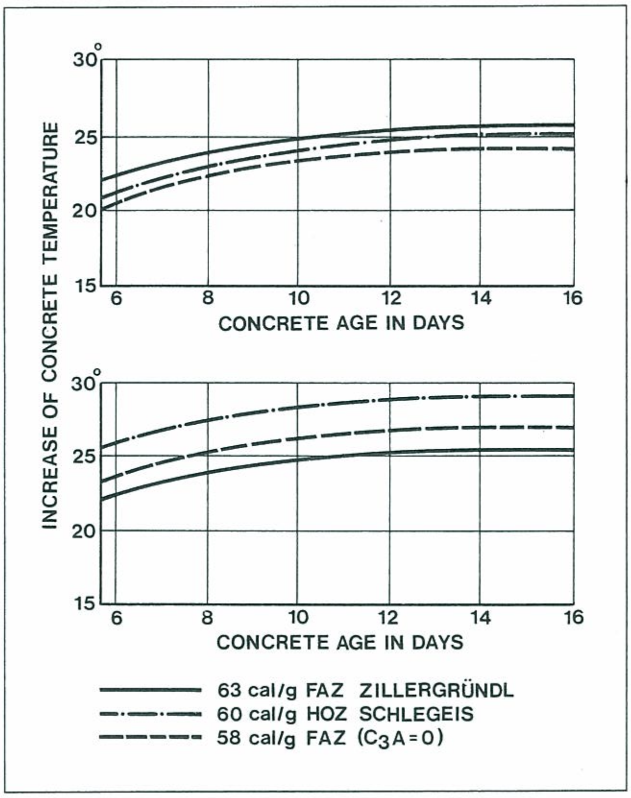
Up to the construction of the Schlegeis arch dam, Portland cements with an admixture of blast-furnace slag (55% slag in the cementing agent for the Schlegeis arch dam) were used. Variations in blast-furnace slag composition resulting from the manufacturing process made the use of flyash for reducing heat build-up appear desirable. Large-scale development work led to a process consisting of grinding separately the flyash and the cement clinker and then homogenising them. Thus, the flyash contents of mass concrete are 30% for the Kölnbrein arch dam, 33% for the Zillergründl arch dam, and 40% for the supporting structure of the Kölnbrein arch dam.

5.2 The aggregate

It was already for the construction of the Glockner-Kaprun development around 1950 that the great importance of the finest grain for the properties of the concrete was understood. Dedusting facilitates the introduction of air voids to improve frost resistance, and a better grading of the fine sand allows savings in the total water content and, hence, in cementing agent for otherwise the same fresh-concrete properties (Fritsch, 1960).

By using a vertical-process settling tank as developed by the mining industry, it became possible to introduce the separation of 0 to 3 mm fine sand at about 1 mm into two fractions. Aggregate preparation for the Kölnbrein dam was accomplished by mechanical-hydraulic separation of sand. The separation at 1 mm was obtained by means of inclined stationary screens. Superfluous fines smaller than 0.06 mm were eliminated in a horizontal settling plant. For the preparation of the raw sand for the Zillergründl dam, a sand classification method using an automatic horizontal settling tank was applied. In this way it was possible to produce a sand with a well controlled grading curve. With maximum variations of +/- 3% for the 0.25 mm grain size and of +/- 4.5% for the 1.0 mm grain size, the desired grading curve was obtained with optimal consistency. The coarse fractions were separated by means of rubber-plated flexible-drive screens.

Figure 5 Significant adiabatic concrete temperature rise



The concrete for the three highest arch dams in Austria had to be produced with an aggregate consisting of gneiss relatively rich in mica. A very detailed investigation program led to the conclusion that water requirements increase along with increasing mica content, especially in the fine sand fractions, whereas strength and modulus of deformation decrease (Huber, 1971).

5.3 Concrete fabrication

Until well into the sixties the concrete was mixed in the mixing tower in predetermined proportions without any further checks, which led to substantial variations in the effective water content as a result of the varying moisture contents of the aggregates. In order to achieve optimal uniformity for the concrete, a computer-controlled water proportioning plant was used for the first time at Kölnbrein. In this plant, the moisture content of the aggregate is measured by neutron probes and the additional water requirements are determined and batched by a computer. Provision of vibratory chutes in the weighing hoppers and checking of actual values against desired values allow a high degree of batching precision. A permanent record was kept of all mix compositions. In this way it was possible to reduce the variation coefficient as a measure of concrete uniformity to less than 8%.

In order to minimize the pouring temperature and hence the maximum temperature during the setting of the concrete for the Zillergründl arch dam, part of the total water requirements was added in the form of flake ice. Ice addition in an amount of 10 kg per m³ of concrete corresponded to an average temperature reduction of 2 °C in the fresh concrete. As the water requirements limited the total ice addition to 50 kg per m³, the reduction in fresh-concrete temperature reached a maximum of 10 °C. Due to these measures, the temperature of the freshly mixed concrete ranged between 5 °C and 8 °C, which was a maximum 5 °C above the annual temperature mean, or a maximum 10 °C below the average air temperature during concrete placement.

5.4 Concrete placement

Concrete is placed by tower cranes for the smaller dams and by cable-cranes for the large dams. Table 1 is a list of characteristic data of site equipment for the Austrian concrete dams exceeding 100 m in height. In terms of concrete technology, the mean rising velocity of concrete placement is also a determining factor. This is listed for some arch dams of major height in Fig. 6.

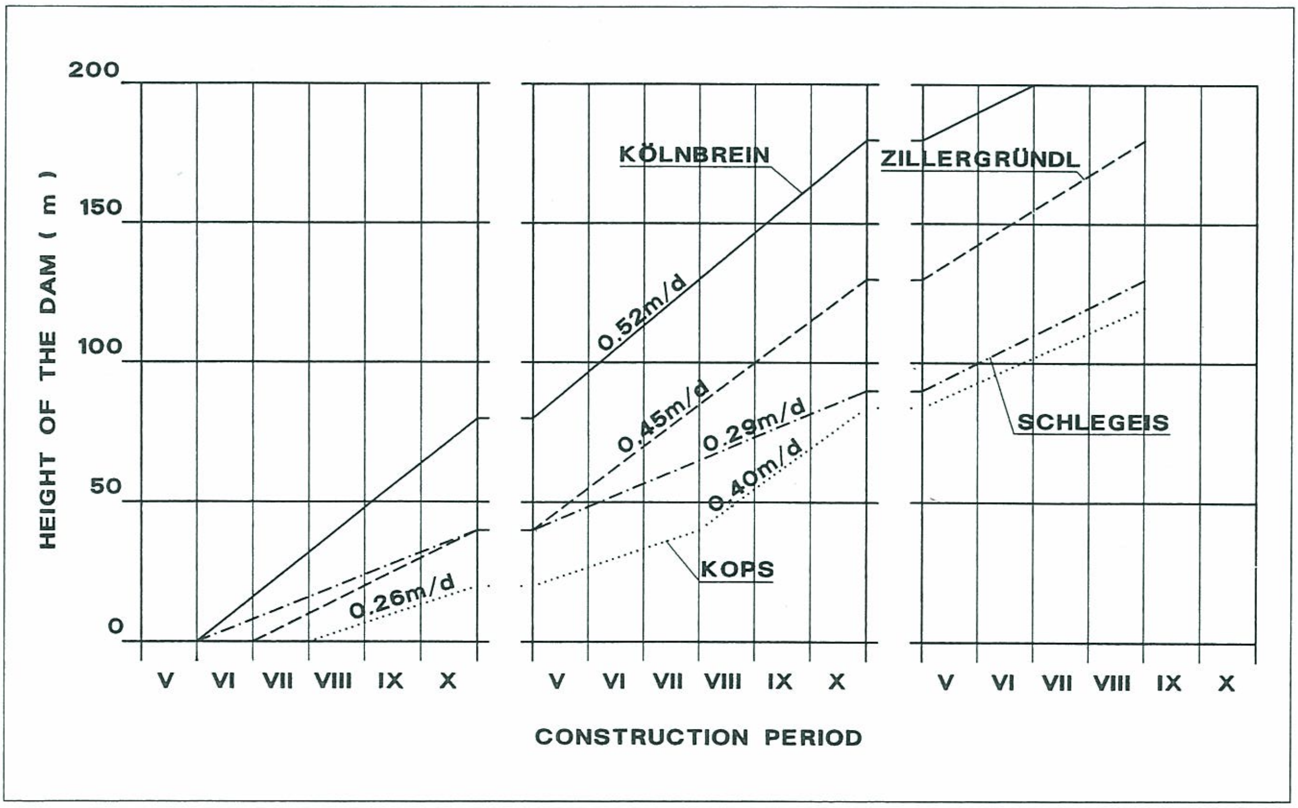
The efficiency of the cable-crane equipment determines not only the length of the construction period but also the dimensioning of all the other equipment needed for concrete production and is thus a determining factor for the cost of site equipment. As manual determination of the optimal design of cable-crane equipment is time-consuming due to the large amount of parameters needed for different alternatives, a computer program was developed for the deterministic simulation of concrete placement (Jurecka, 1973). This program was used for determining the optimal sequence of concrete placement operations for the Schlegeis arch dam and then also for a comparison of the different cable-crane systems proposed during the tendering stage for the Kölnbrein and Zillergründl arch dams. The decisions taken on the basis of results obtained from these preparatory studies were subsequently borne out by the reality of construction operations.

6 ASPECTS OF DAM SURVEILLANCE

6.1 Principles

It is a well-known fact that dam safety is not only a matter of optimal adjustment of the design to the specific features of the site and to the possibilities of construction practice; a requirement of equal importance is the early recognition of potential alterations during operation. For a check to be kept on the state of the dam

Figure 6 Mean rising velocity during concrete placement for several Austrian dams



structure, it is necessary to provide a surveillance system designed to suit the special features of the structure and its foundations and included already in the design. In Austria a record is kept of all the large dams, which includes main design data (e.g. assumptions underlying the design and results of design studies), construction data (e.g. results of concrete quality testing, geological data and methods of joint and foundation grouting) as well as all readings and observations.

Surveillance comprises three systems:

- appurtenant works, the good working order of which must permanently be guaranteed by careful maintenance and occasional checking,
- visual surveillance of the dam structure itself, of the reservoir slopes (for potential instability) and of the terrain immediately downstream of the dam (for instabilities and springs) by periodic inspection, and
- monitoring to check design parameters during the first years of operation and to establish long-term alterations of behaviour at specific points regarded as being characteristic, during the whole period of operation.

Undoubtedly, the required safety check on a large dam would already be accomplished by establishing the near-elastic behaviour of dam and foundation and the sufficient and constant imperviousness of dam and foundation. However, where indications of potential alterations are to be analysed and their causes found, it is useful to install a much larger and more intricate monitoring system.

6.2 Remarks on the present state of concrete dam monitoring

6.2.1 Modern monitoring systems

The following paragraphs will report on several unusual features of dam monitoring especially developed for Austrian dams (Monitoring of Austrian Dams, 1987).

New Tauernmoos gravity dam

Where gravity dams have open joints, each of the statically independent monoliths ought to be included in the monitoring system. At the Tauernmoos dam, an instrument measuring relative displacement of the blocks in three directions was installed in the joints, at crest level. The blocks situated between the blocks equipped with plumb lines are electrically connected with one another by a chain of limit value sensing devices in-stalled in the vertical joints. Response of the tightly adjusted limit value transmitters releases a signal at the signal board in the central control station. This newly developed system guarantees continuous monitoring of the behaviour of each independent block of a gravity dam.

Möll arch dam

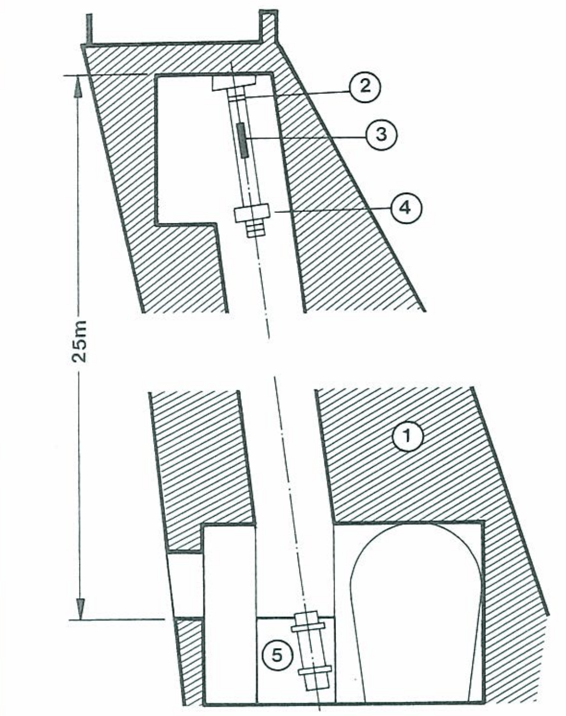
This thin 56 m high arch dam is not accessible during the winter months. Provision of plumb lines for measuring displacements was not possible because of the thinness and vertical curvature of the structure. Instead, a chain of clinometers was installed in recesses arranged along the downstream surface of the crown cantilever.

Table 1 Concrete pouring equipment data for Austrian dams exceeding 100 m in height

			Limberg	Drossen	Mooser	Kops	Schlegeis	Kölnbrein	Zillergründl
Construction period			1948/51	1952/55	1952/55	1961/65	1967/70	1974/77	1983/86
Concrete volume		1 000 m³	446	355	665	663	960	1 580	1 370
Aggregate fabrication capacity		t/h	300	400		450	500	900	800
Mixing tower capacity		m³/h	180	240		240	240	360	360
Blondin	Span	m	433	360	510	950	820	780	600
	Bucket capacity	m³	3×2.5	3×3.0	3×3.0	2×6.0	2×6.0	2×9.0	2×9.0
Number of concrete placing month		n	17	14	20	20	21	18	19
Maximum daily output		m³	2 000	4 100		3 550	4 400	7 100	7 100
Maximum monthly output		1 000 m³	40	40	43	65	85	148	146
Lift height		m	3.0	3.0	3.0	3.0	2.45	3.0	3.0
Average vertical rise of concrete		m/day	0.29	0.42	0.33	0.28	0.29	0.52	0.45
Average stand-by time		days	10.3	7.1	9.1	10.7	8.4	5.8	6.6

The measured changes in inclination correspond to the tangents of the bending line and thus allow the computation of radial crest deflections. The provision of these electrical instruments also involved the possiblity of teletransmitting the measured data to a computer at the control centre of the power station, where they are checked for plausibility and stored in a data base together with reservoir surface level and air temperature data.

Figure 7 Laser-pendulum at Zillergründl arch dam



① DAM BODY (CONCRETE)
② BEARING
③ LASER TUBE
④ MODULATING AND FOCUSING ELEMENT
⑤ RECEIVER

Schlegeis arch gravity dam

Schlegeis dam, 131 m high and 725 m in crest length, was equipped with inverted pendulums extending between 50 and 80 m into the rock foundations. Invar wire extensometers were installed both in these boreholes and in the pendulum shafts in the dam. This made it possible to measure dam displacement in three directions at right angles to one another at all intersections between pendulums and inspection galleries. Furthermore clinometers as well as horizontal and near vertical extensometers were installed at the dam base upstream and downstream in 11 measuring planes to measure deformation in this area. As this type of instrumentation has performed very well, it has also been used for the Kölnbrein and Zillergründl arch dams.

Kölnbrein arch dam

After the cracks had appeared near the dam base, sliding micrometers were installed in several blocks of the dam in order to allow metre-to-metre measurement of changes in length. Data obtained over a period of several years have conveyed valuable information on the deformation pattern within the dam body (Ludescher 1985).

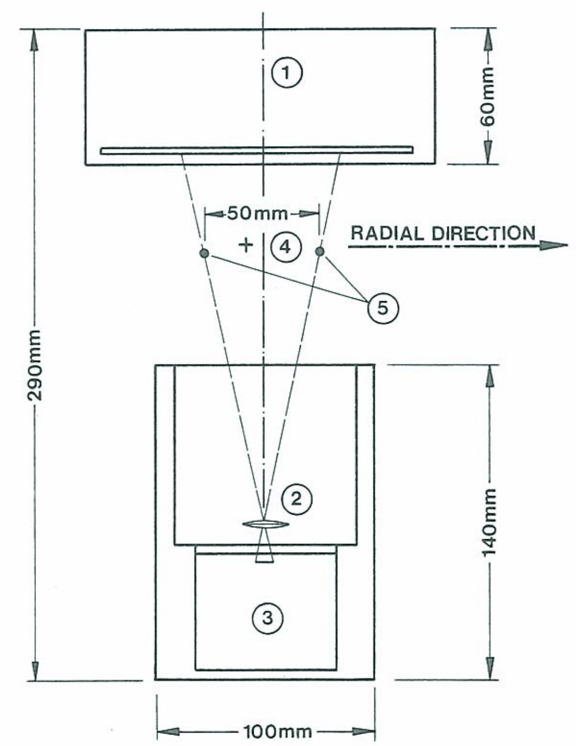
Zillergründl arch dam

Data of great relevance for the checking of dam stability include deflections of the dam crest relative to the dam base. Pendulums are provided for that purpose which require vertical shafts or boreholes. However, in thin arch dams with strongly curved vertical sections, provision of a pendulum shaft is often not possible. In the past, this problem was solved by providing pillars built against the dam face or by installing clinometer chains instead of a pendulum system. Now, by using a directed monochromatic light beam, it has become possible to take direct displacement measurements also in inclined shafts. Teletransmission of the measuring signal is possible without provision of additional equipment in the pendulum system. Transmitters and receivers are watertight; with a measuring range of +/-40 mm, measuring accuracy is +/-0.2 mm (Fig. 7).

For the geodetic surveillance of the Zillergründl arch dam instrumentation was provided for the observation of deformation by measuring

- reservoir slope convergence near the dam by means of newly developed geodetic distance measuring instruments,

Figure 8 Touchless pendulum readings system

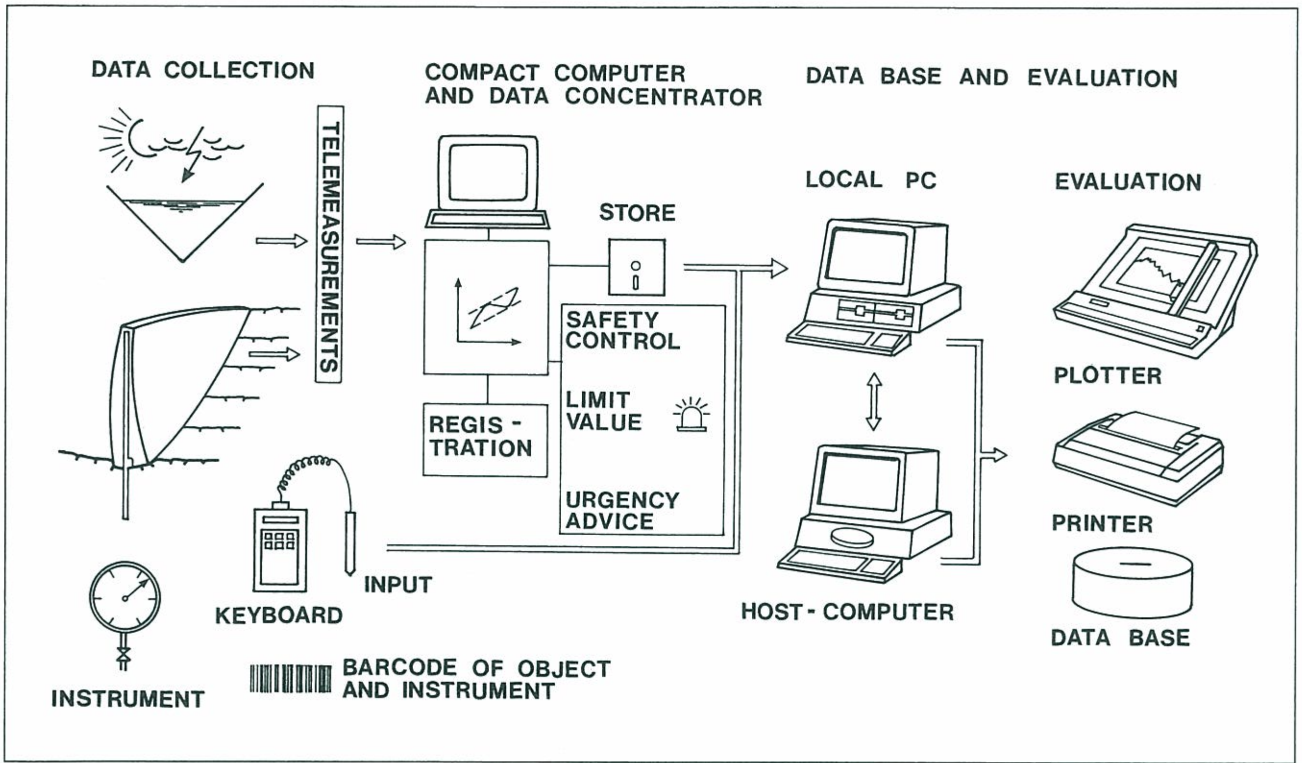


① SOURCE OF LIGHT
② OPTIC
③ CAMERA
④ PLUMB LINE
⑤ REFERENTIAL PLUMB LINE

- absolute displacements of the upmost gallery by means of geodetic sighting lines from measuring windows in this gallery to an observation grid downstream of the dam, to derive displacements of pendulum anchorage points,
- foundation settlement at the centre of the gorge by continuous levelling loops passing from downstream bench marks through the lowest inspection galleries of the dam.

In addition, for measuring rotation about the vertical axis, additional horizontal tangential extensometers were installed in the abutments, near the upstream and downstream dam faces. Twisting can be determined from differences in length variations.

Figure 9 Automatic telemonitoring system



Hierzmann arch dam

For determining pendulum movements, an optical line scanning camera has been used since 1988 with a resolving capacity that allows an accuracy of 1‰ of the measuring range. The range of movement of the pendulum wire is limited by two fixed reference pendulums arranged in the same plane. The specific pendulum value is computed in the camera from the given distances between reference pendulums and objective and between image plane and objective, and from the angle formed by reference pendulum and dam pendulum. This technique guarantees the continuity of measurements even when the camera is being slewed or readjusted during replacement of the pendulum wire (Fig.8).

Rationalization of manual acquisition and evaluation of instrument data

A data base (Fig.9) should collect not only automatically measured and teletransmitted data but also direct instrument readings. By introducing mobile and intelligent data terminals, it is possible to rationalize, simplify and accelerate the collection and evaluation of data and to reduce potential sources of error. The main advantages of this new data acquisition system are

- advance determination of the sequence of measurements to be taken and advance programming of checks and their observance,
- identification of the instrument code by means of a bar code reading device,
- keyboard entry of measured value,
- immediate checking of the new raw value by comparison with the corresponding preceding measurement,
- possibility of earlier assessment of measured values,
- absence of need to copy manual records several times in various lists and tables due to direct transmission of raw values stored in the portable terminal to the memory of the computer.

6.2.2 Telemonitoring

The need for telemetering resulted from the necessity of continuous full-year monitoring at many Austrian dams at remote locations that are not accessible during the winter months. The use of communications and computer techniques developed over the last decades has not only led to intensified continuous monitoring, but has in most cases allowed short-term amortisation of equipment due to the reduced staff expenses involved.

The general aspects of monitoring at Austrian dams have been treated in several publications (e.g. Monitoring of Austrian Dams, 1989). In general, instrument readings are recorded at a central station. In some cases, the reading is compared with a limit value already at the dam and it is only the exceeding of limits that is indicated at the central control station. As to limit value monitoring it should be noted that in most cases a constant limit value adjustment – with periodic re-adjustment where necessary – is considered sufficient; sliding limit value monitoring is carried out only at a few major dams. When the measured data exceed given limits, there is an optical or acoustic signal at the central control station.

By way of summary, it can be stated, that

- for almost all the old dams, limit value monitoring has subsequently been installed for at least one instrument value considered of relevance for the safety of the structure and
- for the dams of more recent construction, more instrument readings are being included in the telemonitoring and limit value monitoring system and
- sliding limit value monitoring, which has become possible by the development of computers, has been introduced by steps at the larger ones of the Austrian dams.

6.3 Aspects of instrument data evaluation

At the large Austrian dams, raw values are converted into the data needed for assessment after a system and reasonableness check, and are then stored in a data base. Where no extraordinary instrument readings or observations exist, stored data are evaluated on an annual basis.

Various statistical techniques based on a graphical or analytical system are available for the evaluation. For gravity dams and small dams of minor thickness the usual practice consists in separating by a simple graphical technique the temperature influence as a function of the concrete temperature at a characteristic point of the dam from the influence of the reservoir level. More information is obtained from a multiple linear regression analysis (Widmann, 1967), in which certain proportions of the measured quantities are allocated to the three main influences acting on a dam: water pressure, air temperature and time. This technique was supplemented and refined by means of a statistical trend analysis (Pürer, 1986).

These techniques allow the introduction of sliding limit value monitoring, because a reference variable can be computed from the data available at any time – date, air temperature and hydrostatic head – and can be compared with the actual measured quantity. If the measured value exceeds the allowable range (usually +/- 2 mm), an optical or acoustic signal is released at the control centre and the operators then order the inspection of the dam (Breitenstein, 1985).

7 SUMMARY

By this review an attempt has been made to give an outline of some developments and experiences in the design, construction and operation of Austrian dams. In the following paragraphs, some of these dams will be described in greater detail.

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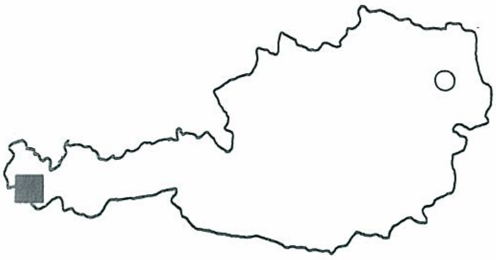
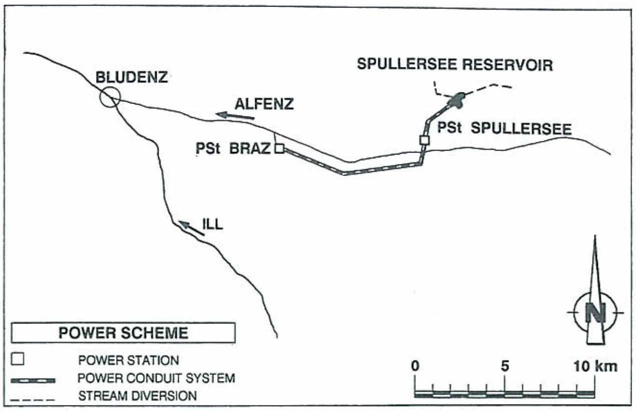
SPULLERSEE GRAVITY DAMS

Vorarlberg; Alfenz, Rhine
Nearest town: Bludenz





MAIN TECHNICAL DATA, Chapter K, 6 (1/7, 1/8)

General

Development	Klostertal	
Power Station	Spullersee	Braz
Construction Period	1922–25	1950–53
Gross Head	795 m	304 m
Installed Capacity	36 MW	30 MW
Mean Annual Generation	38 GWh	100 GWh

Reservoir

Catchment Area: Natural	11 km²
Inflow	14 hm³
Diversions	7 km²
Inflow	9 hm³
Normal Top Water Level (a.s.l.)	1 829.6 m
Minimum Operating Level (a.s.l.)	1 790.0 m
Gross Capacity	16.9 hm³
Live Storage	15.7 hm³
Area flooded by full Reservoir	0.57 km²

Dam

	North	South
Maximum Height above Foundation	28 m	39 m
Crest Length	200 m	294 m
Thickness at the Crest	4 m	4 m
Maximum Thickness at the Base	23 m	31 m
Volume: Excavation		
(overburden, rock)	10 000 m³	14 000 m³
Concrete	27 000 m³	66 000 m³

Appurtenant Works

Spillway	
2 ungated overflow spillways, l=20+102 m	
Capacity	20+17 = 37 m³/s
Bottom Outlet	
Capacity	15 m³/s
Power Intake	
Capacity	6.3 m³/s

1 GENERAL

Two gravity dams 39 m and 28 m in height, respectively, were constructed between 1922 and 1925 to raise the water level of Spullersee, a lake situated about 800 m above the Spreubach valley. The Spullersee winter storage scheme so created worked in interconnection with the Schönberg run-of-river station on the Ruetzbach stream to allow the electrification of the railway lines between Innsbruck and Bregenz. By the completion of a downstream power station at Braz in 1953, an independent Klostertal group of stations was created to ensure the adequate supply of 50/3 Hertz railway power to the Tyrol and Vorarlberg region.

In 1965, the two gravity dams were heightened by 4.6 m, which enabled another 2.6 million m^3 of summer runoff to be stored for utilization during the winter season. The natural catchment of 11 km^2 (with an annual volume of water yield of 14 million m^3) to the west of the watershed between Alfenz (Rhine) and Lech (Danube) was increased in 1943 by 3.1 km^2 by the construction of the Zürser See and Goldenberg diversions (4 million m^3 of annual water yield) and in 1965 by another two diversions from adjacent catchments with a total area of 4.0 km^2 yielding 5 million m^3 p.a. Having a storage of 15.7 million m^3 with an average annual inflow of 24 million m^3, the Spullersee reservoir allows fairly constant energy generation throughout the year.

2 GEOLOGY

The south dam was constructed on a narrow rock barrier composed of tough and impervious Liassic marls. Bedrock is covered by a thin overburden of debris and bog soil. An old fault passes beneath the left dam abutment. The north dam rests on a low ridge of intensely deformed Liassic marl with a steep dip to the upstream. Intercalations of Thiton limestones were found to be present in the eastern part of the foundation. The overburden of debris and boggy ground is thin.

The stability analysis for the two dams was based on the assumption of a horizontal/vertical acceleration of 0.04/0.02 g. Plumb lines have so far recorded several earthquakes, including those from distant places as e.g. Armenia.

3 DAM

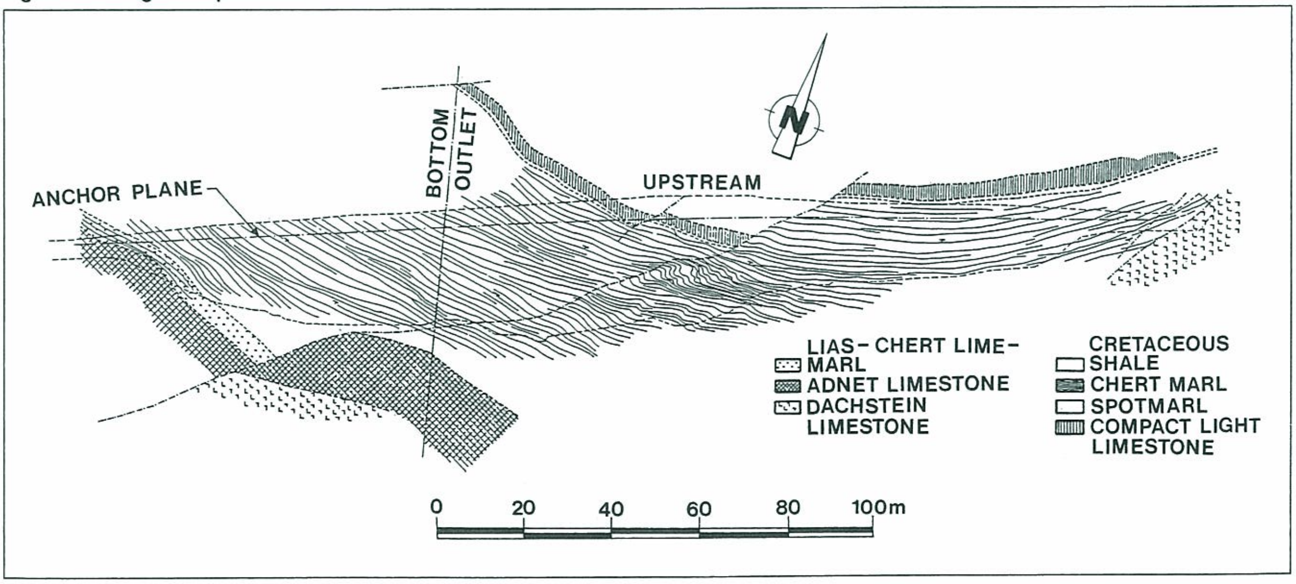
Spullersee was the first reservoir in Austria to call for two dams. Another novelty consisted in the dams being constructed in concrete instead of the traditional rubble masonry. The 39 m high south dam, although slightly curved in plan to suit the topography, was designed as a gravity dam with open joints.

The 28 m high north dam has a radius of curvature of 400 m. It was designed as a gravity dam with the same proportions as the south dam, but the vertical joints were filled. Use of prestressed anchors in the place of an additional concrete volume represented another technological novelty.

The low-level outlet is a pressure tunnel passing under the south dam. It is equipped with a shut-off valve and a ball cock located in the valve chamber for flow control (0 to 13 m^3/s).

For flood relief, a spillway section was provided in the western abutment block of the south dam. Spillway capacity is 20 m^3/s with a clear width of 20 m and a 0.6 m surcharge. As an additional precaution, the crest of the central dam portion (102 m in clear width) was designed as an emergency overflow spillway 40 cm above normal top water level with a capacity of 17 m^3/s for a 0.6 m surcharge.

Figure 1 Geological map



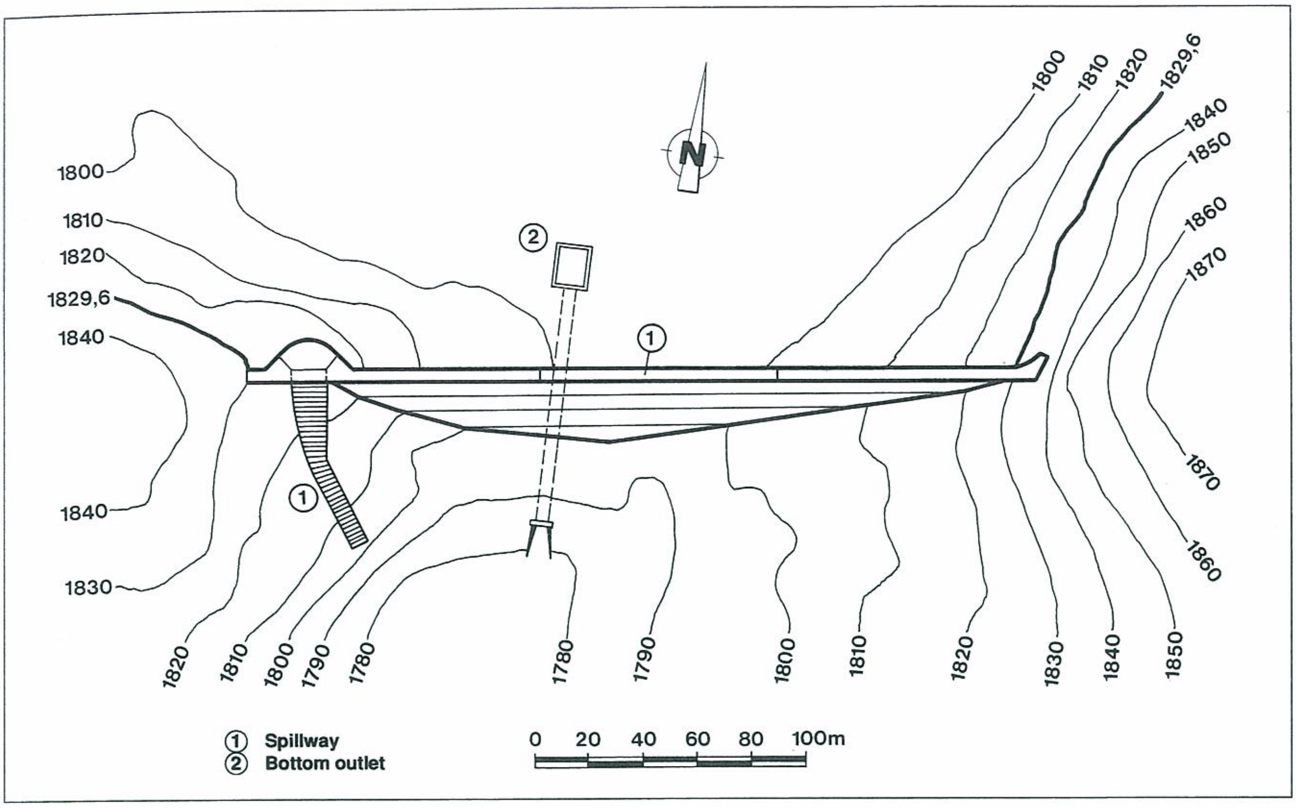
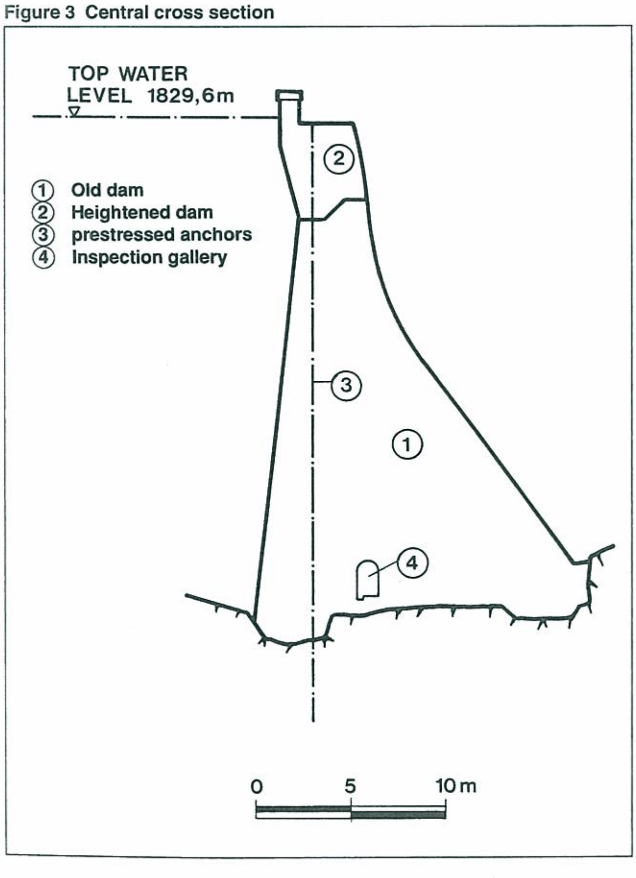


Figure 2 Plan

3.1 Material properties

Original dam: Aggregate for the original dam was obtained from a gneiss quarry, mixed by volume in 4 fractions with a maximum size of 60 mm. Cementing agent consisting of 80% Portland cement and 20% Bavarian trass was added to the three concrete types at the rates of 350/290/205 kg/m³. The respective water-cement ratios averaged 0.47/0.57/0.80. Compressive strengths reached 25/20.6/16.9 N/mm² at 28 days.

Figure 3 Central cross section



The tamped concrete was mixed in 500 l and 750 l gravity mixers, hauled by rail, distributed by scaffolding bridges to blocks between 20 and 22 m in length, and placed in 1.4 m lifts in the lower portions and in 1.9 m lifts in the upper portions.

Joints were protected with angle iron at the edges and sealed upstream with tarred hemp rope, lead wool and z-shaped copper sheet in the facing concrete. Downstream, joints were sealed with caulked lead wool.

Heightening of the dams: a trapezoidal concrete body projecting toward the upstream was placed on top of the old structure. Prestressed anchors extending into bedrock were provided to transfer the resulting additional overturning moment into the foundations. The required anchoring force was 400 kN and 150 kN, respectively, per linear metre of the north and south dams, or 1 320 kN and 630 kN per anchor. In terms of safety, however, the use of prestressed anchors corresponds to no more than a 0.3 m raising of the top water level. Rotary drilled holes were provided in the dam body to accommodate the anchors. In 40% of the holes seepage losses were found to amount to more than 1.5 l/s. The affected holes were filled with colloidal cement and then grouted and redrilled. 2 320 m and 1 090 m, respectively, of anchors were installed. The anchor ends were grouted with cement mortar composed of PZ 275 Portland cement and 25%

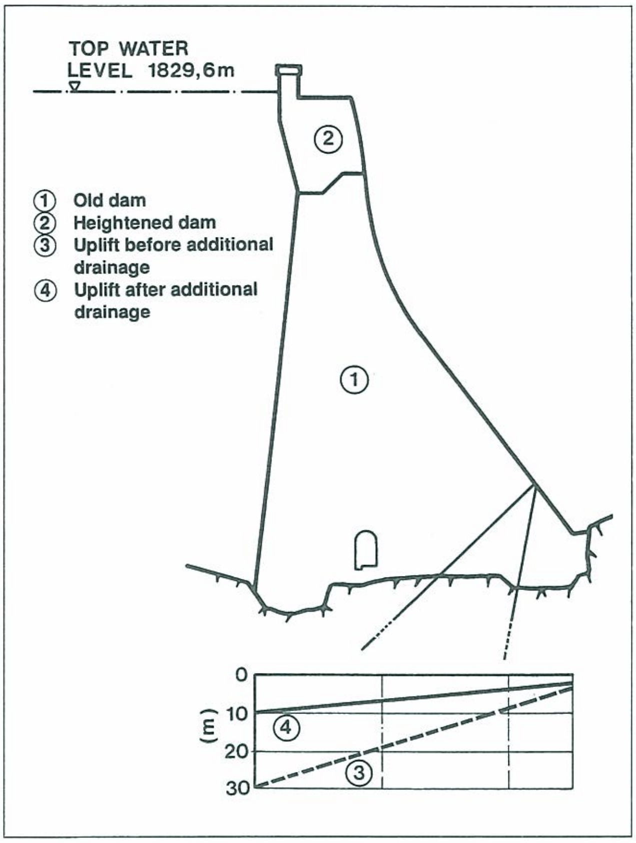
flyash and 0.6% intrusion aid. The cube crushing strength of the grout was 52 N/mm^2 at 28 days.

The trapezoidal body was poured in concrete consisting of 90% Portland cement and 10% Styrian trass and plastiziser addition to obtain an air content of 4.5%.

The new dam crest is at top water level. Freeboard is ensured by a 1.30 m high parapet wall.

The upstream dam toe was sealed with a clay apron. A shallow grout curtain was provided beneath the upstream cut-off wall. Cement grout was pumped 2.5 m deep in pipes laid in the rock at 5 m intervals. Mean grout take under a pressure of 5 or 6 bar was only 10 to 15 l per pipe. Additional imperviousness is ensured in the grouted anchor zones.

Figure 4 Uplift before and after drainage



The contractor for both the construction and the heightening of the dams was the Innsbruck-based firm Innerebner & Mayer. Insond, Salzburg, was responsible for the drilling and installation of the anchors. Construction supervision was carried out by Austrian Railways staff.

4 EXPERIENCES

4.1 Dam surveillance

Instrumentation: Only one inspection gallery was provided in the original dam. It was with the heightening of the dam that a crest alignment (only south dam), a plumb line, a temperature measuring station, a seepage measuring station and 3 piezometers were installed in each dam.

As there is no sufficient assurance of a lasting anchor effect, especially near the dam base (anchors are grouted over their entire length), a drainage system was subsequently provided to reduce uplift. The effect of pressure reduction is checked by teletransmitted pressure gauge measurements and manometers with limit contacts for telemonitoring (limits independent of measured value system). There are 16 piezometers in the south dam and 12 in the north dam.

In addition, 6 extensometers were installed in the south dam and 3 in the north dam with 4 and 2 limit sensors and joint meters (three-dimensional) in the inspection gallery and at the dam crest (in the later case with limit sensors) as well as drainage flow limit sensors.

These measurements are supported by surveying methods. Precision levelling is provided at each dam, in the inspection gallery and at the crest with reference points located in the terrain downstream of the dam for height measurements and small-scale triangulations for the dam crest.

Visual inspections of the Spullersee dams are also carried out (observation of rock and concrete condition, reservoir area, terrain downstream of the dam, etc.).

4.2 Experiences with instrumentation

Monitoring of pressure relief near the dam base combined with appropriate maintenance of instruments is an efficient means of meeting the safety requirements for the Spullersee dams.

4.3 Experiences with ageing of concrete

The dam concrete, about 65 years old, is obviously in a good condition. Samples have been taken and tested at irregular intervals. The test results have shown that concrete strength is about 40% higher than at the time of pouring and the modulus of deformation has increased by almost 40% over the last 30 years.

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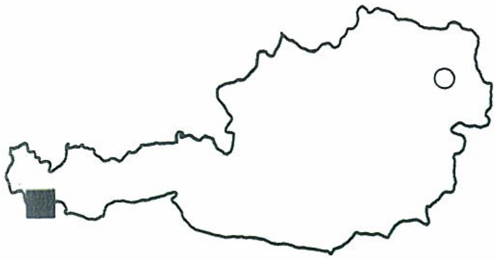
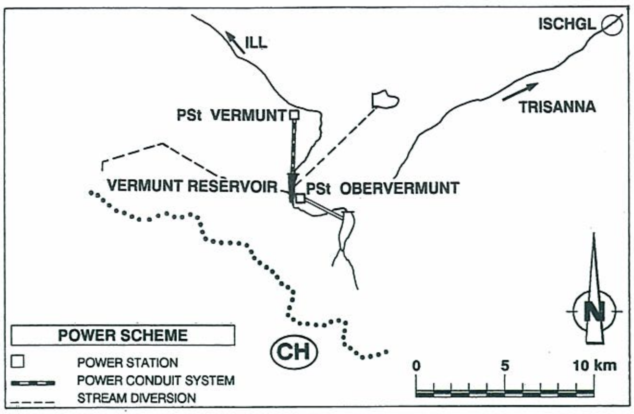
VERMUNT GRAVITY DAM

Vorarlberg; Jll, Rhine
Nearest town: Schruns





MAIN TECHNICAL DATA, Chapter K, 9 (1/12)

General

Development	Obere Jll-Lünersee
Power Station	Vermuntwerk-Partenen
Construction Period	1928 – 1931
Gross Head	714 m
Installed Capacity	156 MW
Mean Annual Generation	260 GWh
of which in winter	97 GWh

Reservoir

Catchment Area Natural	57 km²
Inflow	103 hm³
Diversions	50 km²
Inflow	70 hm³
Normal Top Water Level (a.s.l.)	1 743 m
Minimum Operating Level (a.s.l.)	1 719 m
Gross Capacity	5.7 hm³
Live Storage	5.3 hm³
Area flooded by full Reservoir	0.36 km²

Dam

Maximum Height above Foundation	53 m
Crest Length (22 blocks)	386 m
Thickness at the Crest	3.5 m
Maximum Thickness at the Base	38 m
Volume: Excavation (overburden, rock)	105 000 m³
Concrete	144 000 m³

Appurtenant Works

Spillway, 2 free overflow	
Capacity	32.5 + 65 = 97.5 m³/s
Bottom Outlet, 2 butterfly valves	
Capacity	37 + 14 = 51 m³/s
Power Intake	
Capacity	26 m³/s

1 GENERAL

The Vermunt reservoir was constructed in 1929/30 and is part of the Vermuntwerk, which was the first facility of the development scheme of the Vorarlberger Jllwerke AG for the generation of peak energy. Meanwhile the Vermunt reservoir has been incorporated into a continuous chain of power stations.

In the course of time the reservoir has had to assume different functions. Though in the beginning the facility was designed for weekly storage, it now serves only as an intermediate reservoir. Since completion of the far larger Silvretta reservoir in 1949, which is situated at a higher altitude than Vermunt, storage level variations have been limited to the uppermost part of the reservoir. Tributaries of the JII and Vallüla are conveyed to Vermunt reservoir. At adequate storage levels, water can be exchanged between the Vermunt and Kops reservoirs via a 6 km connecting tunnel.

2 GEOLOGY

The dam is founded on the sill of a glacial basin, with a polygonal axis to permit tie-in to the bedrock on the left-hand bank.

a large earthquake, but it has been proved that in the past there were minor movements due to earthquakes that had their origins in neighbouring Switzerland, especially in the area of the Lower Engadine and Chur.

From the results of seismic evaluations the Zentralanstalt für Meteorologie und Geodynamik determined a design earthquake of the intensity of 6.5° MSK and a destructive earthquake of the intensity of 7.3° MSK. Peak acceleration is of the order of 0.04 g and 0.10 g, with vertical acceleration one third lower.

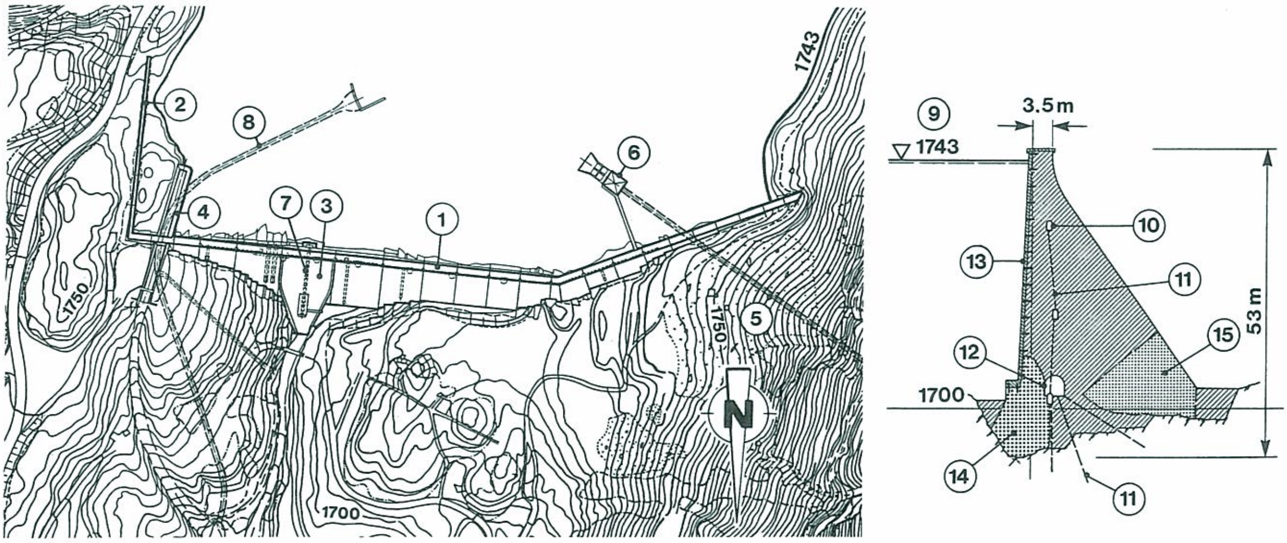
3 DAM

3.1 Design features

The gravity dam has an average downstream slope of 1 to 0.68 and an upstream slope of 1 to 0.05. In order to achieve maximum stability with the above mentioned design specifications, it is necessary to keep uplift pressure as low as possible in the dam foundation area and in the dam body itself.

The dam consists of 22 blocks with a length of 14 to 31 m each.

Figure 1 Plan and cross section



(1) MAIN DAM
(2) LATERAL DAM
(3) SPILLWAY
(4) ADDITIONAL SPILLWAY
(5) PRESSURE TUNNEL
(6) INTAKE TOWER
(7) BOTTOM OUTLET
(8) DIVERSION TUNNEL
(9) TOP STORAGE LEVEL
(10) INSPECTION GALLERY
(11) RELIEF BORINGS
(12) GROUTING CHAMBER
(13) NEW FACING CONCRETE
(14) SEAL GROUTING 1988
(15) GROUTING 1987

The foundation area consists of solid biotite gneisses of the Silvretta nappe, often interbedded with augen and nodular gneisses, with intercalation of fine-grained light-coloured gneisses rich in quartz. The foundation rock is free from major faults. The foundation of the dam is merely crossed diagonally by some small fractured strips.

So far the site of the dam has never been the epicentre of

The keyed contraction joints are sealed with copper sheet and a trapezoidal reinforced concrete bar. The facing and the core of the dam are of tamped concrete. Aggregates were obtained from deposits from the Silvretta reservoir (crystalline rock) and mixed by volume in three fractions with a maximum particle size of 80 mm. For the 2 m thick upstream facing concrete a ratio of 300 kg/m³ of cement was used, for the downstream facing 230 kg/m³ and for the hearting concrete 150 kg/m^3. The concrete was mixed in three 1 500-litre gravity mixers, hauled by belt conveyors and placed by means of distribution towers. Compaction was performed with pneumatic tampers.

The whole of the upstream face is sealed with a 5 cm layer of mortar.

According to the state of the art of that time a 2.5 m deep toe cut-off wall was incorporated on the upstream side. Due to the smoothing of the rock cut in the foundation area an inclined surface rising from the heel to the toe has been formed.

Sealing the dam foundation was performed at the immediate dam base with 4 to 6 m deep contact grouting along the axis of the dam.

3.2 Relief works and intake structure

The dam is equipped with a bottom outlet in the dam as well as a further bottom outlet in the diversion tunnel situated beneath the right-hand flank. Both bottom outlets are equipped with a trash rack as well as with a control valve and a shut-off valve.

Flood relief was initially provided only by a spillway at maximum dam height above foundation with a discharge capacity of 32.5 m^3/s.

In 1969 an additional lateral overflow section was installed at the right-hand flank of the dam with a discharge capacity of 65 m^3/s.

The intake structure for the headrace tunnel is situated in an intake tower 40 m in height equipped with a coarse and a fine rack, an intake sluice and a further valve at the beginning of the tunnel.

In 1987 – 1990 the dam was subjected to extensive remedial measures (see 4.2).

4 EXPERIENCES

4.1 Dam monitoring

The dam is equipped with a pendulum installed in the highest sector of the dam, a full range of uplift pressure monitoring equipment in the lowest inspection gallery, plus seepage water recording equipment. The results from the pendulum and total seepage losses are transmitted to the control room near the dam. The pendulum and the seepage water recording equipment are connected to an automatic alarm system in the control room near the dam as well as in the control rooms of the Obervermunt and Vermunt power stations.

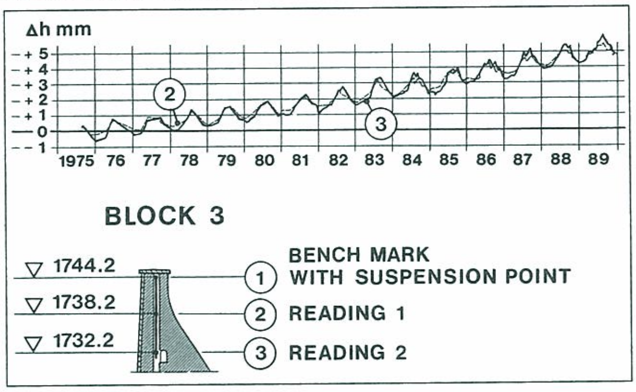
Since construction of the Silvretta reservoir at a higher altitude than the Vermunt reservoir, the storage level of the latter has been maintained in the zone of the top storage level. Therefore movements in the dam body are mainly due to temperature fluctuations.

Thanks to the good foundation conditions and the large number of relief borings, uplift pressure is very small, far less than 50% of hydrostatic pressure.

In addition to the above mentioned continuous monitoring, the dam is subjected every year to geodetic surveys, e.g. precision levelling of dam crest and toe and alignment at dam crest. Trigonometrical observation of targets on the dam is performed every 10 years. Continuous heaving in the dam crest has been observed in the last few decades and as a consequence additional vertical invar tapes have been installed. It has since been proved that the heaving in the crest was due to increases in volume of the immediate crest area (Fig. 2).

With a statistical model designed for the Kops dam and since adapted for gravity dams, a daily comparison is made between measured and predicted crest displacements.

Figure 2 Damcrest heaving



4.2 Events

The Vermunt dam was subjected to a complete range of remedial measures in 1987 – 1990. These measures had become necessary because the dam body was absorbing storage water that is aggressive to concrete, which in the long term would have led to a loss of strength in the concrete. Furthermore the zone of the dam crest had been seriously affected by the alternation of frost and thaw, with consequent heaving (see 4.1).

The seepage in the dam was caused by the fact that the upstream sealing mortar had become pervious due to the alternation of frost and thaw and to the mechanical effects of the ice in the area of the crest and due to the perviousness of the crest in the area of the vertical joints. The seepage concentrated in large nests which developed during the construction period in the generally high quality concrete used for the dam. A comparison between the results of the extensive analyses, especially the water pressure tests for 1960/62 and 1988, showed a marked deterioration and remedial measures had to be taken.

These remedial measures were as follows:

- renewal of the upstream sealing layer by placing a 60 cm thick concrete facing on the dam body and grouting for the upstream dam toe
- new relief boreholes for the dam foundation and the dam body
- renewal of the dam crest and
- grouting for the downstream dam toe in order to improve the strength of the dam.

These measures were completed in 1990. Total seepage loss at top storage level is 0.2 l/s and is entirely due to the relief boreholes in the area of the dam foundation.

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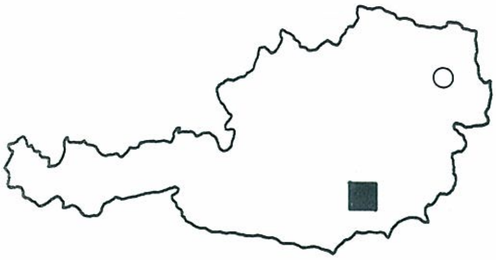
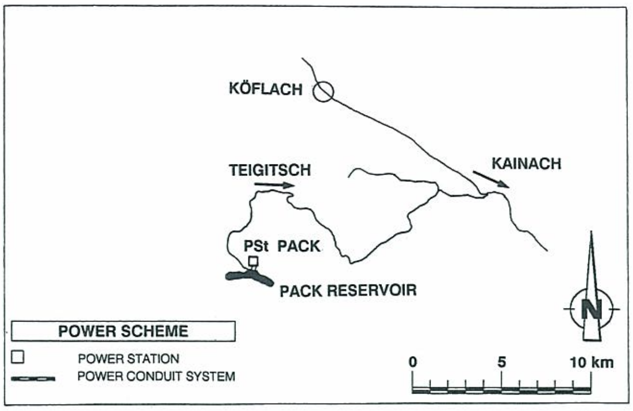
PACK GRAVITY DAM

Styria; Teigitsch, Mur,
Nearest town: Köflach





MAIN TECHNICAL DATA, Chapter K, 10 (1/11)

General

Development		Teigitsch	
Power Station	Pack	St.Martin	Arnstein
Construction Period		1929 – 1930	
Gross Head	28 m	74 m	227 m
Installed Capacity	0.6 MW	11 MW	30 MW
Mean Annual Generation	1.8 GWh	14 GWh	50 GWh

Reservoir

Catchment Area: Natural	63 km²
Inflow	43 hm³
Normal Top Water Level (a.s.l.)	866.3 m
Minimum Operating Level (a.s.l.)	843.6 m
Gross Capacity	5.6 hm³
Live Storage	5.4 hm³
Area flooded by full Reservoir	0.6 km²

Dam

Maximum Height above Foundation	33.4 m
Crest Length	183 m
Thickness at the Crest	4 m
Maximum Thickness at the Base	25 m
Volume: Excavation (overburden, rock)	20 000 m³
Concrete	39 000 m³

Appurtenant Works

Spillway	
ungated overflow along the dam crest	
Capacity	250 m³/s
Bottom Outlet	
through the dam, 0.9/0.7 m dia., 2 valves	
Capacity	7 m³/s
Power Intake	
Capacity	3 m³/s

1 GENERAL

The Pack dam was constructed in 1925 to impound a reservoir for long-distance water supply to the Arnstein power station with the Langman reservoir. Between 1947 and 1950, the Hierzmann dam and reservoir were constructed between the Pack dam and the Arnstein power station, which brought the winter proportion of power generation at Arnstein to 52%. In 1965 St.Martin power station was placed in operation. The head of approximately 150 m between Pack and Hierzmann has so far remained undeveloped. The group of reservoirs is situated in the south-eastern foothills of the Alps. The annual rainfall depth amounts to less than 1 100 mm.

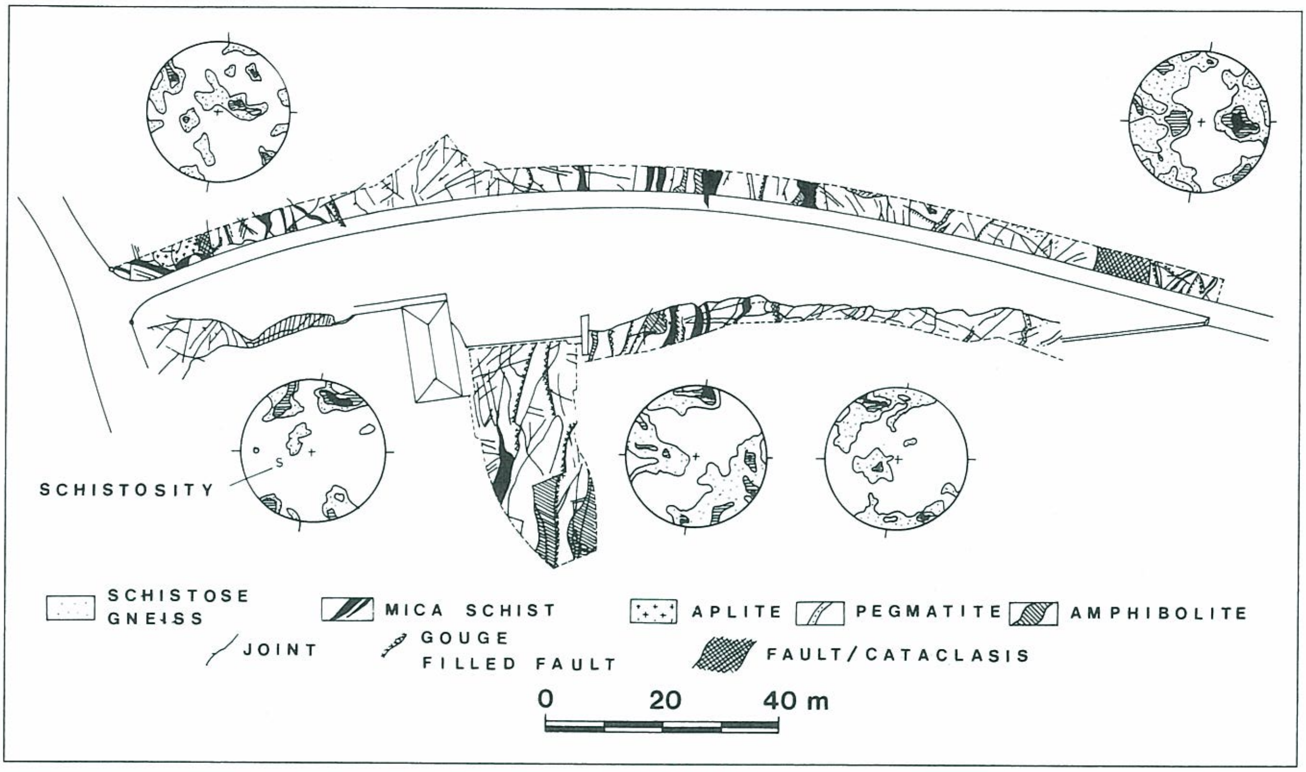
Economic necessities led the owner to decide in favour of a material rich in feldspar and mica available at the site. Good-quality aggregate would have been found in the Graz basin at a distance of about 60 km. Water quality was another concern. It was well known from the outset that with 1.4 degrees of German hardness and a pH of about 6.2, the water would corrode the dam concrete. But these risks were accepted.

It was realized already during the first years of operation that the dam concrete was neither watertight nor frost-resistant. Also, there was soon evidence of lime being leached from the concrete. Remedial measures taken in 1935 included the sealing of the upstream dam face with a reinforced shotcrete layer. In 1952–53 the downstream dam surface was cut to a depth of 20 cm and a new about 80 cm thick facing of frost-resistant concrete was applied. Approximately 100 t of grout was injected into the dam body and the contact between dam base and grout curtain was re-injected with about 55 t of grout.

The decreasing efficiency of the old drainage system led to a great number of piezometer and relief holes being sunk in an attempt to fight against growing uplift pressures and to investigate its distribution over the entire dam base. In the first place, however, it was the large amount of moisture absorption by the dam which called for large-scale remedial action. Thus, between 1982 and 1985, the general repair of the dam was undertaken. This had the following main purposes:

- Sealing of the joint between upstream dam toe and rock surface and of the foundation in order to reduce uplift pressures and moisture in the dam body;
- Sealing of the entire upstream dam face by means of a frost-resistant concrete wall as another measure to reduce moisture in the old dam body and in order to be able to include the cross section of new concrete as a loadbearing element in the stability analysis;
- Raising of the design flood from 190 m³/s to 250 m³/s.

Figure 1 Geological map



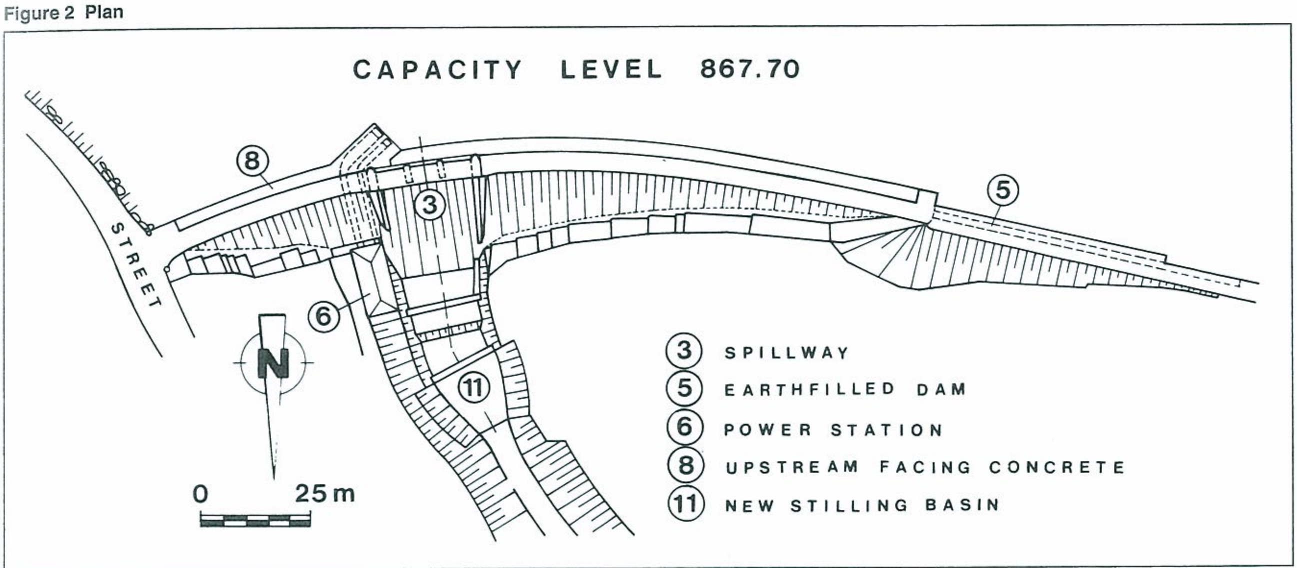
2 GEOLOGY

The wide V-shaped valley lies in an "old crystalline" arch where mainly gneissic mica schists are exposed. Depth to bedrock in the valley bottom is small. The basal rock mass is cut up by a closely knit system of joints, the greater part of which are gouge-filled. Passing on the left slope is a fault some 20 m in width and mainly consisting of fairly weathered sericite schist. The right-hand slope is more severely jointed, but less weathered. Still, the dam foundation is relatively impervious, although the feldspar components are weathered and have been exposed to chemical action.

3 DAM

Pack dam is a gravity structure with a vertical upstream face and with the downstream face inclined at 1 to 0.75. The dam is arched in plan, consisting of a two-centered arch with radii R = 200 m in the main portion and R = 450 m on the left-hand side. Stability analyses included an uplift pressure decreasing as a straight line from 100% at the upstream face to the zero at downstream face, without allowance being made for the arch action of the whole structure. On the left side, the dam continues as a low embankment with a core wall as an impervious element tied both into the rock foundation and into the concrete dam. Foundation treatment beneath the concrete dam at that time consisted of a single-line grout curtain, which was regrouted in the region of the foundation contact as part of the 1952–53 repair. The overflow spillway, arranged in the center portion of the dam, is equipped with three hand-operated gates. Water release is normally through a turbine with a rated discharge of 3 m^3/s and, when necessary, also through the bottom outlet. These facilities are accommodated in the powerhouse at the right-hand downstream dam toe.

Figure 2 Plan



Work for the general repair of the dam commenced in 1982 at the upstream face. Along the upstream dam toe the ground was cut to bedrock over a width of at least 4 m, and a 7 m high massive concrete block was poured from at the original foundation level of the old dam. By means of untensioned mortar anchors 3 to 4 m in length, it was possible to provide a very stable connection with the old dam body. The vertical joints were spaced 15 to 26 m apart so as to correspond to the joints in the old dam. By arranging a drainage gallery in the new concrete block, an attempt was made to provide for sufficient low-level pressure relief at the foundation contact. The rising facing wall is 120 cm and then 80 cm thick and consists, like the concrete foundation block, of reinforced B 300 concrete. It was also poured against the old concrete and connected to it by means of grout anchors. The anchors have a horizontal spacing of 3 m and a vertical spacing of 1.5 m to 3 m. Vertical split centre piping arranged at intervals of 4 m was to prevent potential pressure build-up in the joint resulting from shrinkage and together with the stripped surface of the old concrete in the drainage gallery, was to enhance drainage in the old dam body. All construction and vertical main joints of the new facing concrete were provided with external and internal water stops.

A single line of grout holes were sunk through the concrete block and to a depth of 35 m below the dam base. Borehole spacing was 3.5 m, grouting pressures reached 25 bar. Contact grouting was carried out from the drainage gallery to 8 m into the rock under pressures of 5 bar. About 90 t of grout was injected in total. A slurry trench cut-off wall was provided upstream of the old core wall to ensure imperviousness, and this was tied both into bedrock and into the new facing concrete.

For pressure relief in the foundation contact, 51 boreholes were drilled from the drainage gallery and supplemented at four selected instrument planes by an additional two boreholes each. The above described work at the upstream face was completed in 1983.

To 1985, structural work necessitated by the raised design flood requirement (250 m^3/s) was carried out. In order to ensure safety from erosion at the downstream dam toe in such an extreme case, an about 4 to 5 m wide concrete channel descending in cascade on both flanks along the dam toe was provided. The concrete of the old stilling basin had to be renewed and a second stilling basin had to be arranged below to ensure appropriate energy dissipation. Heavy stabilization of the streambed had been designed on the basis of model test results.

4 EXPERIENCES

4.1 Dam surveillance

Dam and slopes are visited and inspected daily by a dam attendant. Leakage measuring devices in the drainage gallery are read and serviced at regular intervals. Two limit values of cumulative leakage measurements in the drainage gallery are telemonitored from the Arnstein power station. The yields of the individual drainage holes

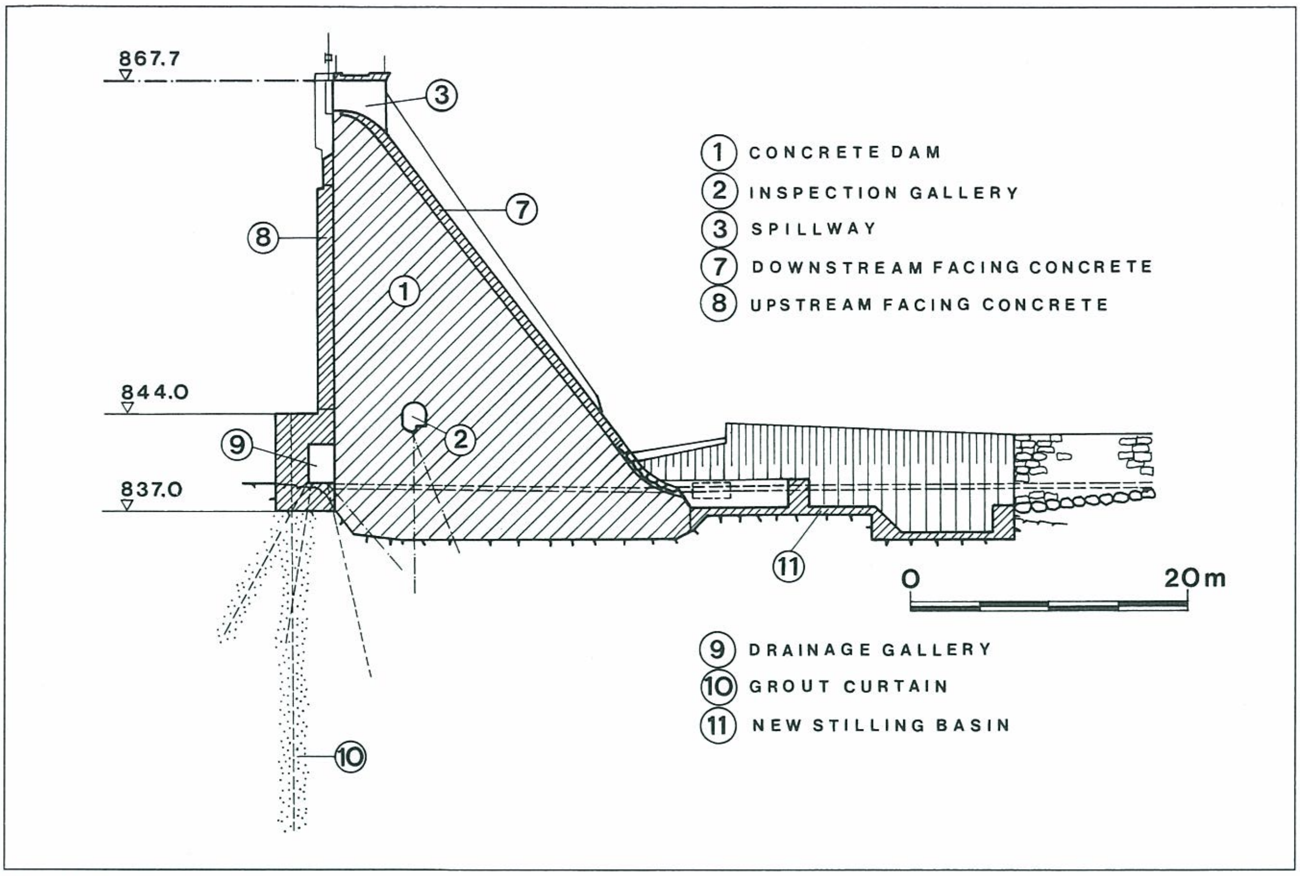


Figure 3 Central cross section

as well as the uplift pressures are measured at regular intervals and distance measurements are taken between dam body and facing concrete (at 3 points for radial, tangential and vertical directions, at 2 points for radial direction).

During floods, the dam is manned and flow control is performed by the dam attendant at the dam in accordance with the operating instructions.

Long-term observation has shown the regular annual pattern of movements mainly to be determined by the seasonal temperature cycle.

4.2 Special occurrences

By the remedial measures moisture absorption was stopped and uplift pressures as well as leakages were reduced considerably. Drainage holes still intact in the inspection gallery are either dry or yield no measurable leakage flow.

The resulting stability reserve was high enough to allow overtopping for the discharge of the maximum design flood to be assumed in the design.

The yield of a spring that appeared some decades ago in the left slope downstream of the dam has remained unchanged. It shows no clear dependence on the reservoir surface level. Two horizontal drainage holes between the slurry-trench cut-off and the old core wall also show only dripping water.

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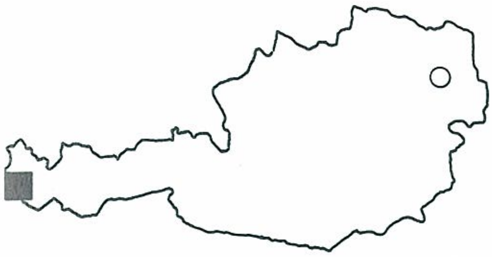
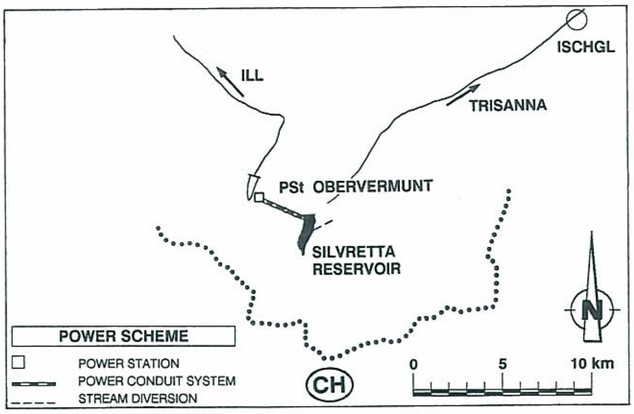
SILVRETTA GRAVITY DAMS

Vorarlberg; Jll, Rhine
Nearest town: Schruns





MAIN TECHNICAL DATA, Chapter K, 13a (1/22)

General

Depelopment	Obere Jll-Lünersee	
Power Station	Obervermuntwerk	Vermuntwerk
Construction Period	1938 – 1948	1928 – 1931
Gross Head	291 m	714 m
Installed Capacity	29 MW	156 MW
Mean Annual Generation	45 GWh	260 GWh
of which in winter	26 GWh	97 GWh

Reservoir

Catchment Area: Natural	35 km²
Inflow	63 hm³
Diversions	10 km²
Inflow	17 hm³
Normal Top Water Level (a.s.l.)	2 030 m
Minimum Operating Level (a.s.l.)	1 986 m
Gross Capacity	39.1 hm³
Live Storage	38.6 hm³
Area flooded by full Reservoir	1.31 km²

Dam

	Main Dam	Lateral Dam
Maximum Height above Foundation	80 m	31 m
Crest Length	432 m	140 m
Thickness at the Crest	3.5 m	3.5 m
Maximum Thickness at the Base	58 m	23 m
Volume: Excavation (overburden, rock)	250 000 m³	in total
Concrete	407 000 m³	18 000 m³

Appurtenant Works

Spillway, 2 free overflow	
Capacity	36 + 68 = 104 m³/s
Bottom Outlet, 2 butterfly valves	
Capacity	26 + 28 = 54 m³/s
Power Intake	
Capacity	14 m³/s

1 GENERAL

The Silvretta reservoir, constructed in 1938–1948, was the second reservoir of the Obere Jll - Lünersee scheme and is now the highest-altitude annual storage reservoir of the Vorarlberger Jllwerke AG. With the completion of the Silvretta reservoir, the winter percentage of total generation from the power scheme was improved from 15% to 37%.

The power stations of Obervermunt, Vermunt, Rodund and Walgau benefit from this capacity transfer. First partial filling was performed in 1943, and first complete filling in 1951.

The reservoir is formed by a 80 m high gravity dam, Austria's highest, a small lateral dam in the area of a fault zone and the 25 m high Bieler dam.

In 1969 an additional spillway was built on the left slope of the gravity dam.

The Bielbach, belonging to the Inn catchment area, is diverted into the reservoir.

2 GEOLOGY

At the right flank of the valley the foundation rock consists of heavily folded and thinly foliated mica gneisses changing into augen gneisses towards the valley floor. The left flank of the valley is composed of amphibolite.

In the centre of the valley, overburden composed of moraine material and river sediments with a maximum depth of 21 m had to be stripped.

The dam is situated in a seismically inactive area. From the results of seismic evaluations the Zentralanstalt für Meteorologie und Geodynamik determined a design earthquake of the intensity of 6.5° MSK and a destructive earthquake of the intensity of 7.3° MSK. Peak acceleration is of the order of 0.04 g and 0.10 g, with vertical acceleration one third lower.

3 DAM

3.1 Design features

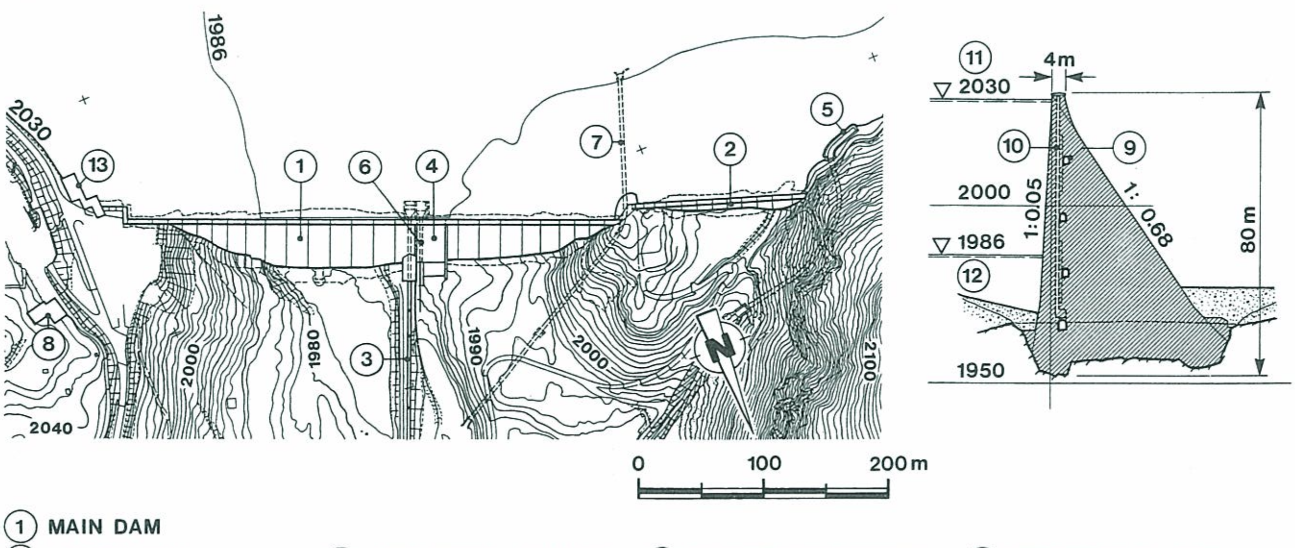
The gravity dam has an average downstream slope of 1 to 0.68 and an upstream slope of 1 to 0.05. In the static analysis, two different uplift pressure assumptions were considered. In one loading case, uplift was considered to be 0.5 of hydrostatic pressure at the upstream face, decreasing to zero at the downstream face. In the other loading case, uplift was calculated at full hydrostatic pressure at the upstream face, and decreasing to zero at the downstream face, but acting in only 60% of the dam foundation area.

In order to assure optimum stability by modern standards it is therefore necessary to continuously observe uplifts and to keep them low.

The dam was constructed with vibrated concrete with a Portland cement content of 150 kg/m^3 for the hearting concrete and of 300 kg/m^3 for the facing concrete. Water/cement ratios were 1.21 and 0.65. The aggregates were obtained from the sediments found in the reservoir area and separated into 4 fractions with a maximum grain size of 100 mm.

The dam was constructed in blocks 11–17 m in length, and the lifts were 1.70 m. The average compressive strengths obtained after 90 days were 17.1 N/mm^2 in the hearting concrete and 28.8 N/mm^2 in the facing concrete. The keyed contraction joints were sealed with a copper layer and a cone-shaped reinforced concrete bar. All the block joints are equipped with inspection shafts.

Figure 1 Plan and cross section



(1) MAIN DAM
(2) LATERAL DAM
(3) PENSTOCK
(4) SPILLWAY
(5) ADDITIONAL SPILLWAY
(6) BOTTOM OUTLET
(7) DIVERSION TUNNEL
(8) MONITORING CENTER
(9) INSPECTION GALLERY
(10) INSPECTION SHAFT
(11) TOP STORAGE LEVEL
(12) DRAW DOWN LEVEL
(13) RESTAURANT

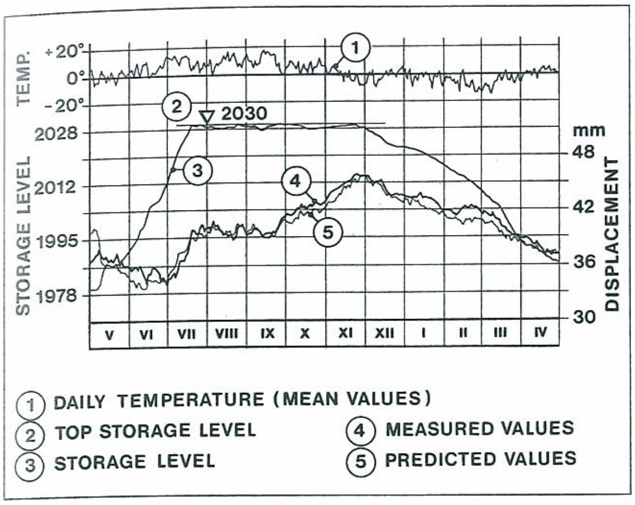


Figure 2 Crest displacements 1988/89

The foundation area was sealed by means of a grout curtain up to 35 m in depth and 5 to 6 m deep contact grouting from the lowest inspection gallery. After first partial filling of the dam, however, uplift was so great that additional grouting and relief borehole were required.

3.2 Relief works and intake structure

The dam has a bottom outlet in the dam body and a bottom outlet in the diversion tunnel situated beneath the left flank.

Flood relief was initially only provided by a spillway in the highest sector of the dam, with a discharge capacity of 36 m^3/s.

In 1969 an additional lateral spillway was installed on the left flank of the valley with an ungated crest and an additional discharge capacity of 68 m^3/s. The intake structure of the headrace is located in the middle of the dam. At the downstream toe of the dam, at the beginning of the penstock, there is a valve chamber equipped with a rapid-closing valve.

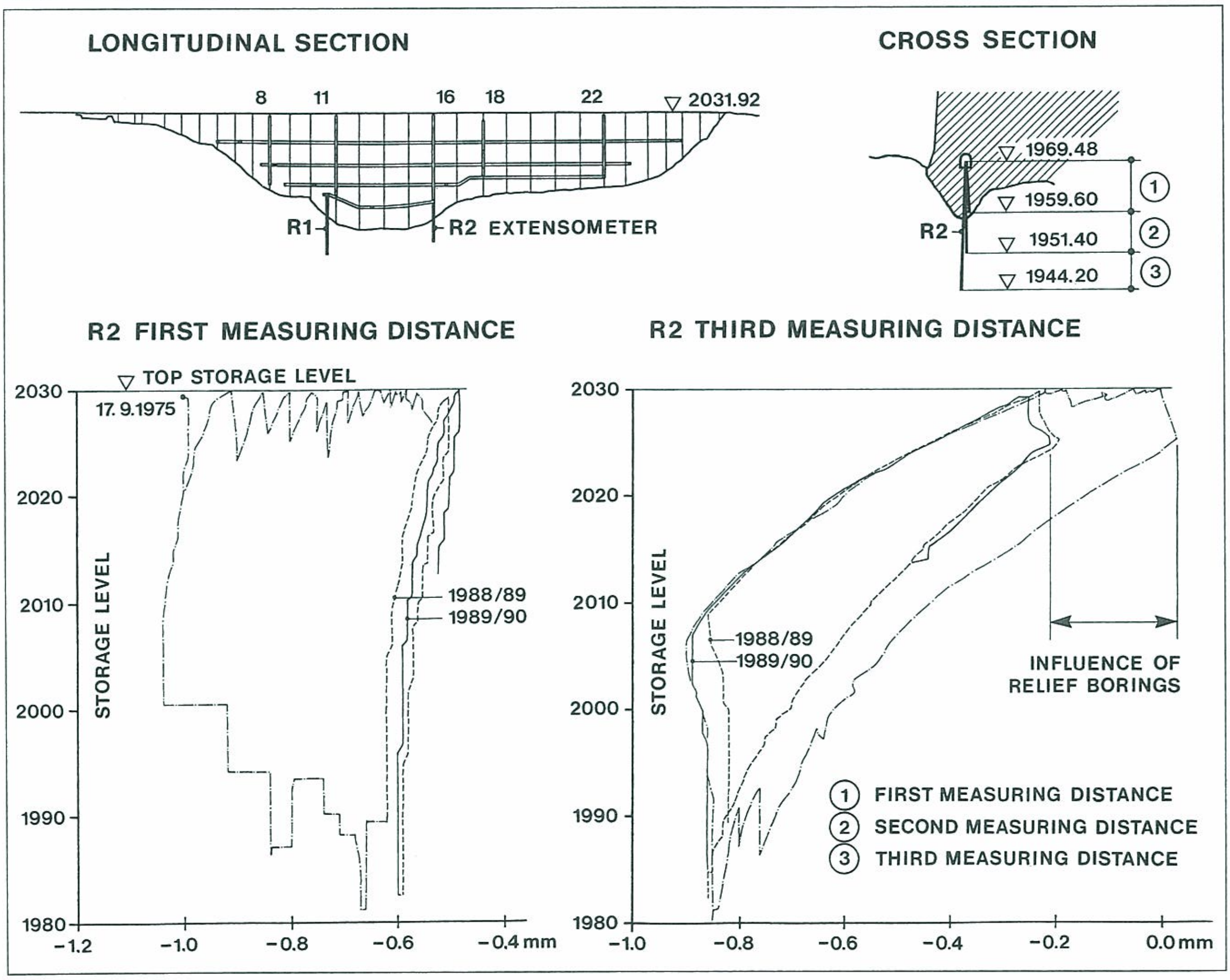
4 EXPERIENCES

4.1 Dam monitoring

Since the beginning, the dam has been equipped with 5 pendulums in the dam body and a large number of uplift pressure gauges. The seepage water is collected in several places and evacuated through the access tunnels to the inspection galleries.

In 1975 several clinometers were fitted and 2 vertical triple extensometers installed in the zone of the highest dam blocks. In 1984 the pendulum of the highest dam block was equipped with an automatic data transmission system linked to a warning device in the control room. In 1986 the upstream toe of the dam was equipped with

Figure 3 Rock deformations



2 inclined extensometers. In 1988 the system for measuring total seepage was changed and seepage flows chanelled to an automatic seepage measurement device situated in the lowest inspection gallery of the dam.
Once a year the downstream toe and the crest of the dam are checked by geodetic levelling, and the alignment of the crest is also measured.

In order to forecast movement in the dam crest, a statistical prognostic model has been elaborated. A comparison between the measured and predicted values for 1988/89 is shown in Fig. 2.

The extensometer readings have produced some remarkable results. First, uplift pressure has been reduced by drilling a number of relief boreholes in the dam foundation area, and this is clearly reflected in the results from the extensometer readings. Second, the shortest extensometer section length located completely within the dam concrete reveals heaving which, although slight, has been increasing continuously since monitoring began at a rate of 2.10^{-6} per year. The results of the extensometer readings are given in Fig. 3.

4.2 Events

In 1969 seepage losses were detected at the downstream end of the lateral bottom outlet. Therefore the pressure tunnel, which up to that date was equipped with a concrete lining, was sealed at the downstream end up to an adequate depth of rock overburden with a steel lining. The part without the steel lining was additionally grouted in 1989.

In 1989 the butterfly valve of the bottom outlet became inoperative due to a breach of the valve body and was replaced by a sluice gate in stainless steel.

In order to keep uplift pressure low, several new relief borehols have had to be drilled at intervals of 10 to 15 years.

Water tends to incrust the relief boreholes, which indicates erosion of the impervious blanket. Therefore it is possible that parts of the impervious blanket will have to be renewed in the future.

The sealing mortar on the upstream face has repeatedly been destroyed in places by the alternation of frost and thaw and by the mechanical effects of the ice. Therefore the sealing mortar has been renewed several times and has been replaced by shotcrete reinforced with steel wire mesh. Furthermore the upstream face of the dam has been treated by additional grouting.

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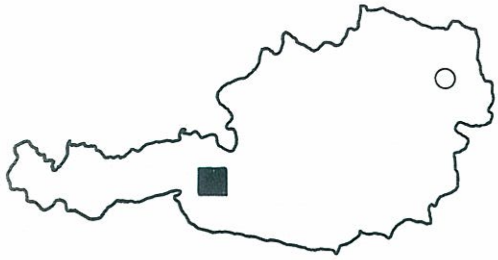
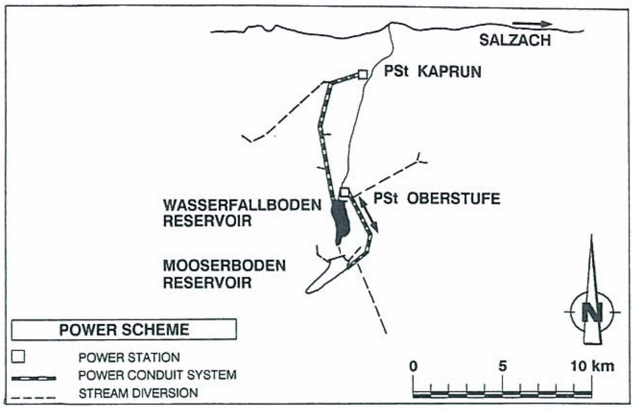
LIMBERG ARCH DAM

Salzburg; Kapruner Ache, Salzach
Nearest town: Zell am See





MAIN TECHNICAL DATA, Chapter K, 19 (2/6)

General

Development	Glockner-Kaprun
Power Station	Kaprun
Construction Period	1948–1951
Gross Head	860.7 m
Installed Capacity	220 MW
Mean Annual Generation	486 GWh
of which in winter	387 GWh

Dam

Maximum Height above Foundation	120 m
Crest Length (23 blocks)	357 m
Thickness at the Crest	6 m
Maximum Thickness at the Base	37 m
Volume: Excavation (overburden, rock)	287 000 m³
Concrete	446 000 m³

Reservoir

Catchment Area: Natural	15 km²
Inflow	21 hm³
Diversions	128 km²
Inflow	226 hm³
Normal Top Water Level (a.s.l.)	1 672 m
Minimum Operating Level (a.s.l.)	1 590 m
Gross Capacity	83.0 hm³
Live Storage	81.2 hm³
Area flooded by full Reservoir	1.5 km²

Appurtenant Works

Spillway	
Capacity	26 m³/s
Bottom Outlet	
Capacity	93 m³/s
Power Intake	
Capacity	36.5 m³/s

1 GENERAL

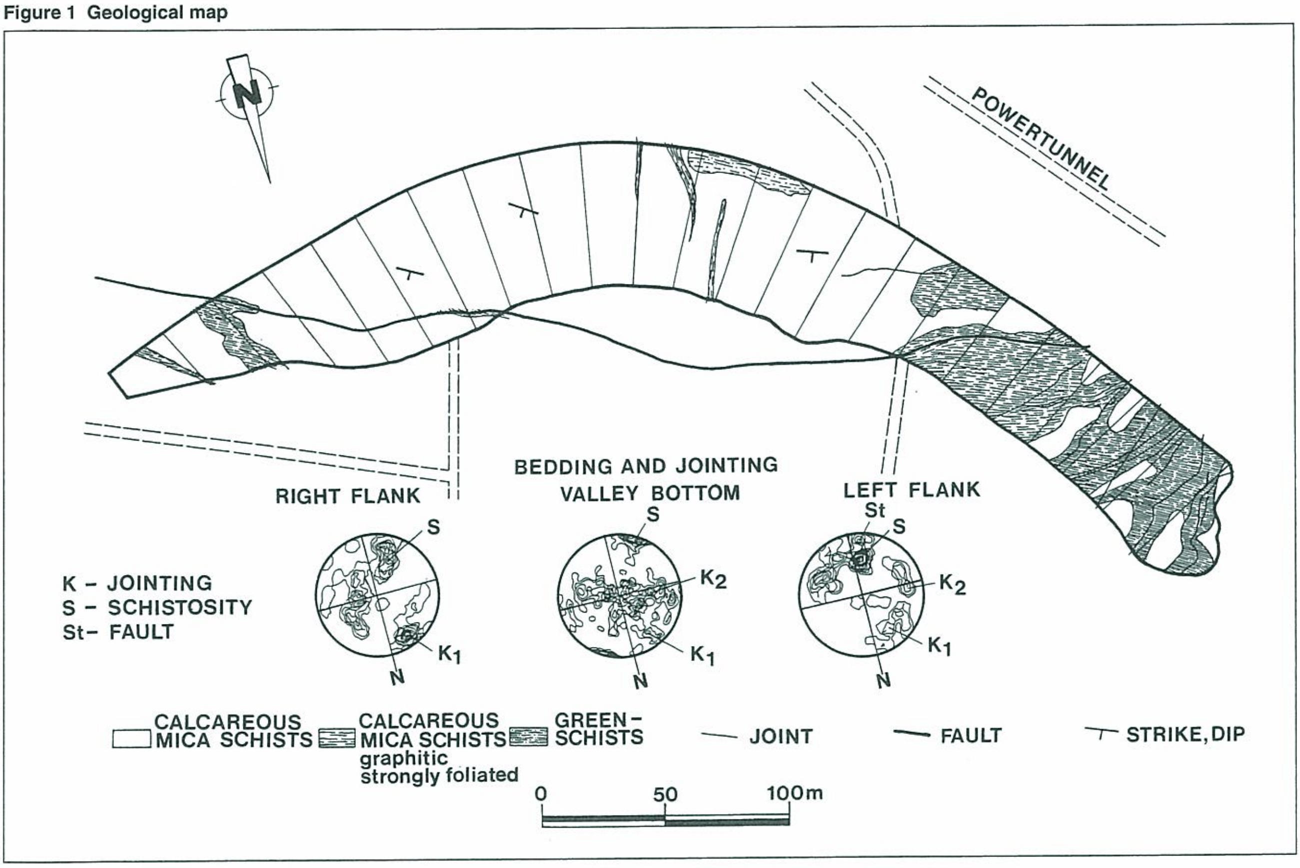
The Limberg dam of the Glockner-Kaprun scheme is built on the large natural basin at Wasserfallboden. This is a wide and flat glacial trough which narrows into a gorge at a rock sill before it descends to the valley terraces of the Kaprun Ache stream, thus offering ideal conditions for the construction of a dam. The first plan drawn up in 1938 was for a solid gravity dam of the conventional type, shored downstream against the rising rock, with a concrete volume of approximately 750 000 m³. Further studies, conducted with a view to reducing costs and especially to saving time and material, resulted in approval of a project incorporating a buttress dam slightly curved in plan. Excavation started in 1939 and continued until 1945. After the war there seemed no reason not to continue the project, which had in the meantime been redesigned by Professor Stucky as a simplified straight buttress dam with a volume of approximately 450 000 m³. But then, primarily because of the fews entertained by Swiss experts with regard to stability, the buttress type dam was abandoned and serious studies were undertaken on an alternative arched design. On the basis of preliminary plans drawn up by Professor Stucky, Professor Grengg, Dr. Lauffer and the Tauernkraftwerke the TKW 5 arch dam, a slightly arched double-curvature unsymmetrical arch-gravity dam present in spring 1948 was constructed. Development of the Wasserfallboden basin as a long-term storage reservoir for the Glockner-Kaprun scheme was the economic purpose of the first major storage power scheme on the Austrian grid. Construction of the Wasserfallboden reservoir was another prerequisite for the construction of the upper head pumped storage station. Due to the shift in the inflow pattern from summer to winter, the distribution of runoff to the Salzach river was improved to such an extent that construction of run-of-river stations became economically. In addition to the runoff from the catchment of the Kaprun Ache stream above the dam site, two left-hand tributary streams (Zeferetbach, Grubbach) were diverted into the power tunnel. Further diversions include the Möll river with the Käfer streams (1953) and the Hirzbach from the Fuscher valley to the east (1973).

2 GEOLOGY

The dam site is situated in the northern slopes of the Austrian Central Alps. In this area the Central Alps from a syncline of the Pennine Tauern window which is shaped like a dome over large areas. The Tauern "Schieferhülle" formation overlying this dome is bordered by the Granatspitz Zentralgneis core in the west and the Hochalm-Ankogel Zentralgneis core in the east. The southern Pennine rock formation of the Glockner nappe consists of calcareous mica schists with varying mica content, graphitic schists and some green schists, with a thin mantle of weathered material. A natural ridge is found between the shallow Wasserfallboden basin and the steeply descending Kaprun valley. The receding upper third of the western flank is covered by moraine material. A fault crosses the valley and the surface of the dam foundation. Joint sets and discontinuities running more or less parallel to the valley and traversing it with branches in places are partly sealed with calcite. Local karst phenomena along the joints are encountered only rarely.

The power scheme is situated between the seismically active regions of the Inn valley in the NW and the Mur-Mürz furrow to Villach in the SE, in an area of very low seismic intensity.

Figure 1 Geological map



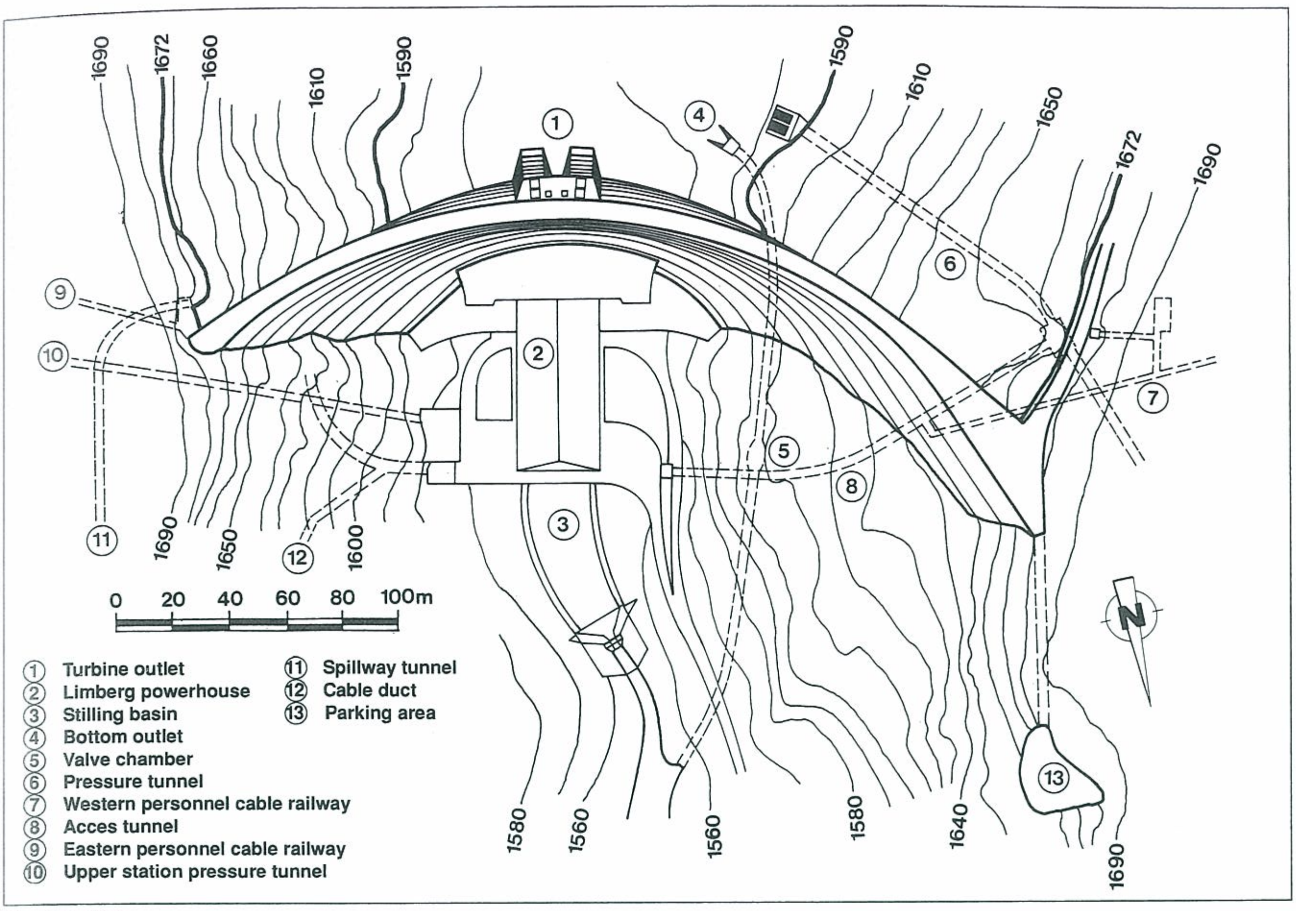


Figure 2 Plan

3 DAM

3.1 Design fundamentals

After preliminary single-section stability analysis, final analysis for the "autumn main load case" (maximum water level with summer temperature rise) was carried out for the actual dam shape and valley topography. This analysis was performed using the load distribution method with several sections and taking account of the criteria commonly observed for arch dams, such as homogeneous and isotropic material, and Hooke's law and Navier's hypothesis.

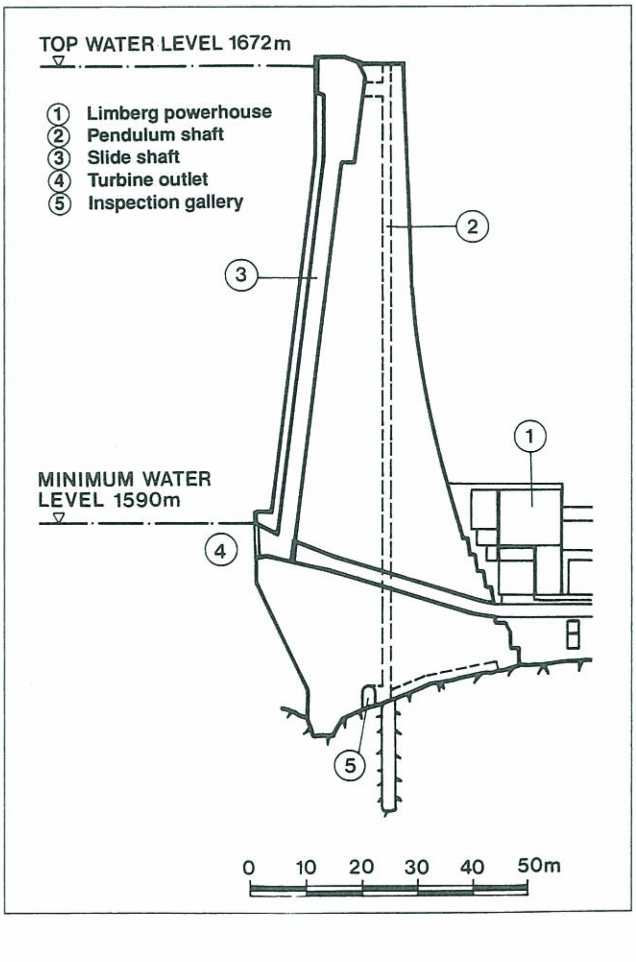
The rock-concrete elastic moduli ratio was assumed to be 0.75. Uplift pressure was based on the assumption of 0.25 of full hydrostatic pressure upstream decreasing linearly to zero at the downstream face.

3.2 Construction

The following concrete mix was adopted: 250–260 kg Portland cement PZ 225/m^3, water/cement ratio: 0.55–0.57, addition of Plastiment N at 1% of the cement weight. The mean concrete strengths at 90 days were 4.7 N/mm^2 bending tension and 31.1 N/mm^2 compression.

Concreting was performed in blocks with a maximum length of 15 m. To avoid cracking and to obtain uniform cooling, cooling slots were provided between the blocks (1.20 m wide) and closed in the following year. At a dam thickness of over 20 m the radial blocks were divided by longitudinal joints. First reservoir filling was in 1952 after three periods of impounding, one in 1949 (33% of filling level), one in 1950 (75% of filling level) and the third in 1951 (96% of filling level).

Figure 3 Cross section



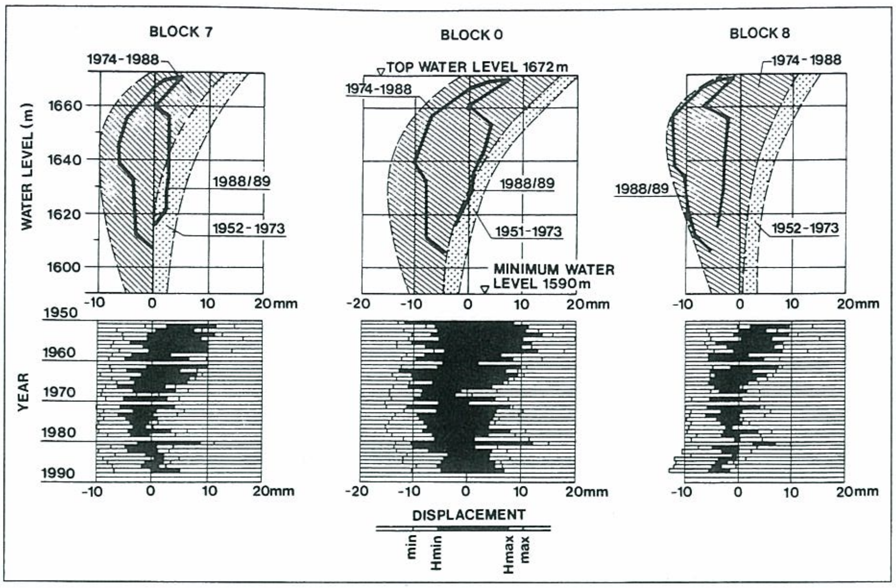


Figure 4 Crest displacements

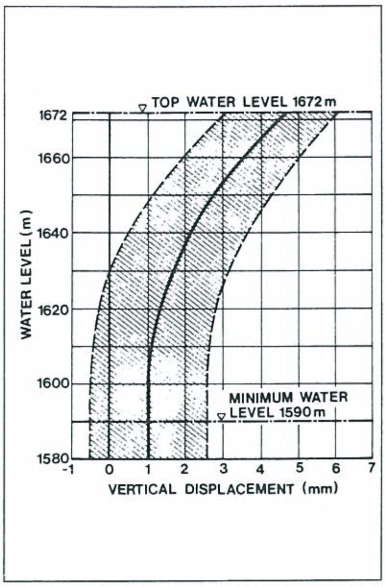


Figure 5 Vertical displacement

4 EXPERIENCES

4.1 Dam monitoring

Upon completion of the dam instrumentation mainly consisted of three pendulums to measure horizontal displacements in the dam crest relative to the dam foundation, ganging weirs to record the seepages emerging in the bottom inspection gallery which is directly located on the surface of the foundation rock, uplift gauges to measure uplift pressures, and numerous thermometers and deformeters embedded in concrete, which failed for the most part. In addition, geodetic measurements are carried out twice a year, such as crest levelling and trigonometric observation of horizontal displacement using 35 targets at the downstream dam face and 7 survey points on the rock near the downstream dam abutment. In 1983 pole socket extensometers (to measure rock deformation) and piezometers (to measure pressure on rock and uplift pressure) were additionally emplaced at each dam flank.

4.2 Measuring results

Since the suspension locations of the plumbs were not completed before the last year of concreting in 1951, monitoring started in the central block at about half the reservoir level during the first year of full filling, and in the lateral blocks in spring of the subsequent year.

Synoptic presentation of annual extreme displacements offers only an approximate guide to long-term tendencies, since variations in the level curves and ambient temperatures impede assessment. Multiple linear regression analysis furnishes more satisfactory results. Such analysis reveals permanent crest displacement towards upstream of about 3 mm in the central sector and some 4 mm in the lateral sectors over the first 15 years. Over the last 5 years this tendency has been reversed, now indicating a downstream displacement of about 2.0 mm and 1.0 mm, respectively. Crest displacements are about 30 mm in the central sectors and 19 mm in the two lateral sectors. This amplitude has remained constant over the complete monitoring period, so that there can have been no significant change in the modulus of deformation of the concrete. This amplitude is composed of one proportion relating to filling level (24 mm and 12 mm, respectively) and another relating to the season and temperature (± 9 mm and ± 8 mm, respectively).

The vertical displacements revealed by crest levelling disclosed a similar tendency. Superimposed on annual elastic displacements of ± 12 mm is a continuous heave in the crest of the order of about 0.18 mm annually, relative to the arch abutments. Assuming swelling of the concrete in the order of 0.0015 mm/m per year, the two tendencies are in good accordance. Restraints are undoubtedly eliminated by stress relaxation, so that this phenomenon is of no importance for dam stability, even in the long term.

Uplift pressures upstream of the bottom inspection gallery are close to 100% of hydrostatic pressure, while downstream of the bottom inspection gallery they are less than 10% of hydrostatic pressure. As an additional measure, ground water pressure was monitored in the downstream vicinity of the dam and did not reveal any influence of reservoir filling on the pressure regimen.

Seepages which emerging in the bottom inspection gallery amounted to about 0.8 l/s during the first year with full reservoir before dropping rapidly on selfsealing. Today the figure is less than 0.05 l/s. Annual seepage volumes have decreased from about 9 000 m^3 during the first year of full operation to 1 000 m^3 today.

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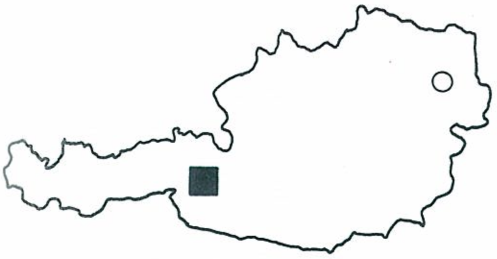
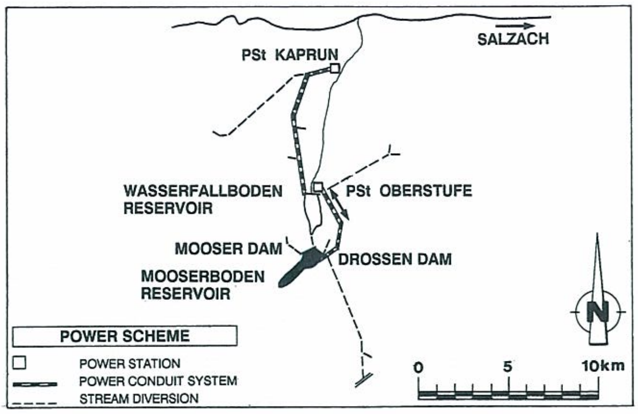
MOOSER GRAVITY DAM

Salzburg; **Kapruner Ache, Salzach**
Nearest town: **Zell am See**





MAIN TECHNICAL DATA, Chapter K, 26a (2/16)

General

Development	Glockner-Kaprun	
Power Station	Limberg	Kaprun
Construction Period	1950 – 1955	1948 – 1951
Gross Head	365.1 m	860.7 m
Installed Capacity: Turbine	112 MW	220 MW
Pump:	130 MW	–
Mean Annual Generation	152 GWh	486 GWh
of which in winter	86 GWh	386 GWh

Dam

Maximum Height above Foundation	107 m
Crest Length (25 blocks)	494 m
Thickness at the Crest	7 m
Maximum Thickness at the Base	70 m
Volume: Excavation (overburden, rock)	193 000 m³
Concrete	665 000 m³

Reservoir

Catchment Area: Natural	27 km²
Inflow	62 hm³
Diversions	72 km²
Inflow	130 hm³
Normal Top Water Level (a.s.l.)	2 036 m
Minimum Operating Level (a.s.l.)	1 960 m
Gross Capacity	87 hm³
Live Storage	85 hm³
Area flooded by full Reservoir	1.7 km²

Appurtenant Works

Spillway, controlled overflow spillway	
Capacity	100 m³/s
Bottom Outlet, 2 bypasses	
Capacity	62 m³/s
Power Intake	
Capacity	36 m³/s

1 GENERAL

Together with the Drossen dam the Mooser dam retains the Mooserboden reservoir, which forms part of the upper stage of the Kaprun scheme owned by Tauernkraftwerke AG. The overall scheme is a longterm storage scheme and exclusively serves generation of electrical energy.

In the twenties a number of projects were prepared for the utilization of the hydro potential of this area of the Hohe Tauern. The Kaprun valley is characterized by the presence of two valley basins that lend themselves to reservoir construction, with high heads and low lengths of the intake systems. Construction of the Kaprun main stage started in 1939. The Mooser dam was constructed between 1952 and 1955 together with the Drossen dam.

The catchment of the Kaprun Ache stream was greatly increased by the simultaneous construction of the Margaritze reservoir and the Möll diversion tunnel plus pumping station. The head of the reservoir is utilized at two power stations with a total capacity of 332 MW. The upper station is a pumped storage station with a pumping capacity of 130 MW.

2 GEOLOGY

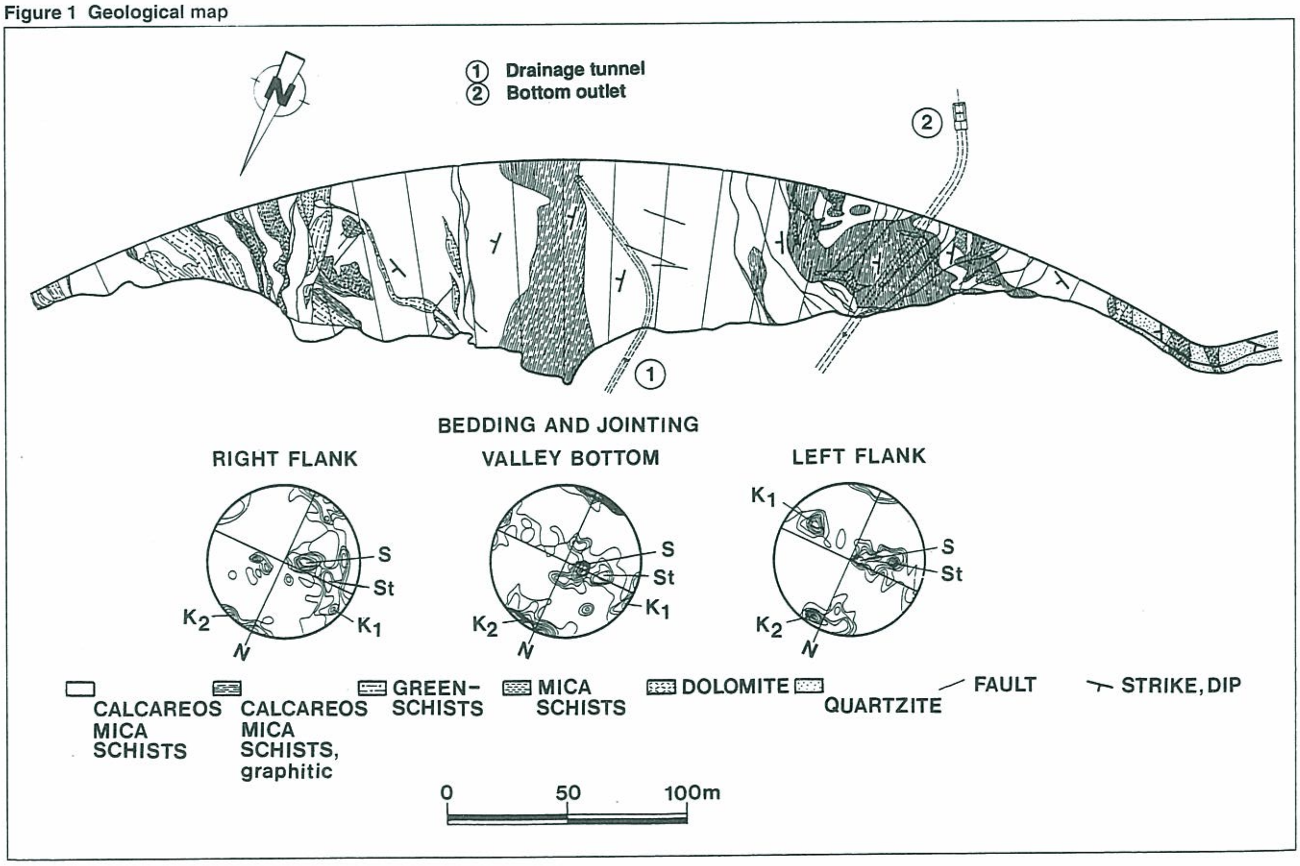
The dam site is located in the central area of the northern slopes of the Austrian Central Alps in the Pennine Tauern window, which is shaped like a dome over large areas. The Tauern "Schieferhülle" formation overlying this dome is bordered by the Granatspitz Zentralgneis core in the west and by the Hochalm-Ankogel Zentralgneis core in the east. The southern Pennine rock formation of the Glockner nappe is bedded with a shallow dip towards NE and consists of partly karstic calcareous mica-schists of varying mica content, as well as prasinite, quartz schist, mica-schist, dolomite and gneiss in places. A rock saddle exposed to weathering and karstphenomena is incised by the erosion of the Kaprun Ache stream at the downstream end of the shallow Mooserboden basin, which was formed by glacial erosion above the escarpment which steeply descends some 300 m down to Wasserfallboden. A NS striking fault, several metres wide, and joints striking NS and EW are present in the upper portion of the right flank and required deep grouting.

Seismicity: the power scheme is situated between the seismically active regions of the Inn valley in the NW and the Mur-Mürz furrow to Villach in the SE, in an area of very low seismic intensity.

3 DAM

The dam is a constant radius arch dam with an arch radius of 425 m. The upstream face is vertical, the downstream face slopes 1 to 0.64 down to El. 1 980, and 1 to 0.685 below that. The absence of tensile stresses at the upstream face resulted from an uplift pressure assumption of a linear decrease from 50% of full hydrostatic pressure at the upstream face to zero at the downstream toe. Even if uplift pressure rises to a theoretical maximum of 85%, bending tensile stress is not more than 0.5 N/mm². Maximum principal compressive stress under full reservoir load is about 3.5 N/mm².

Figure 1 Geological map



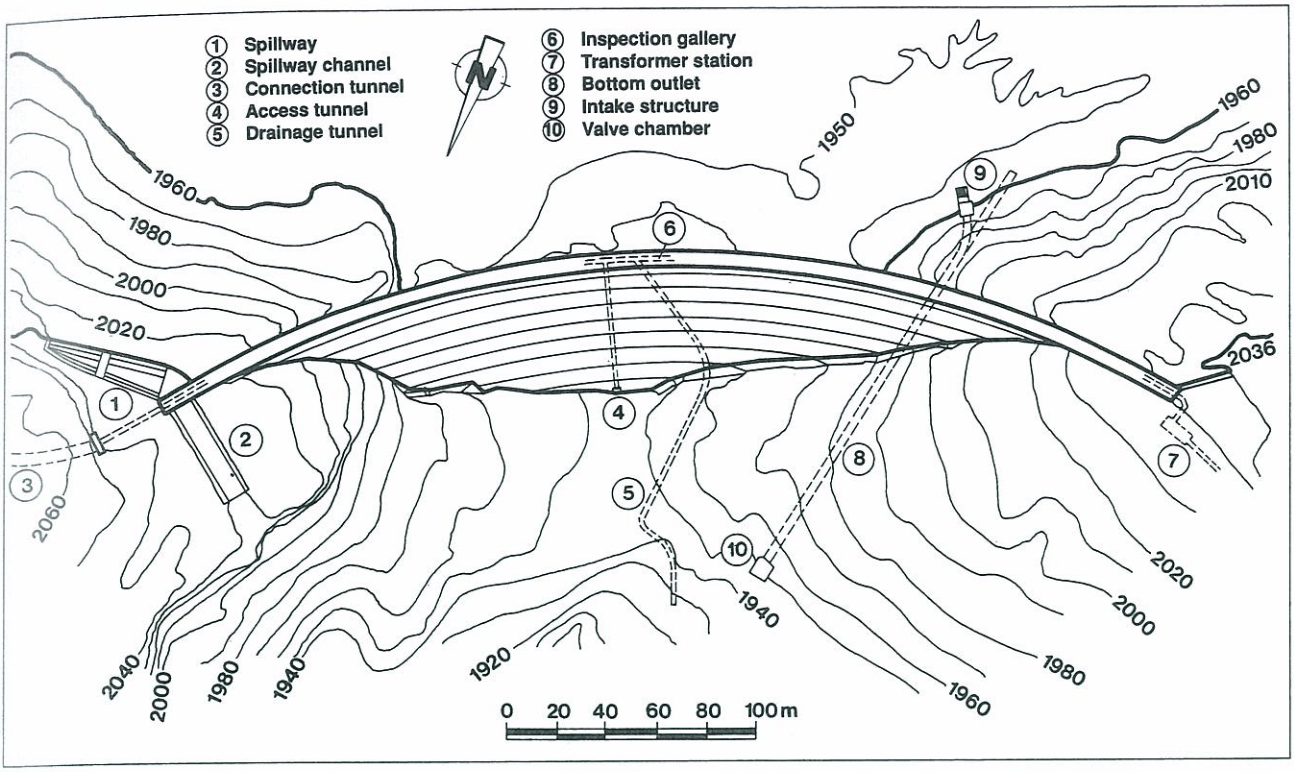
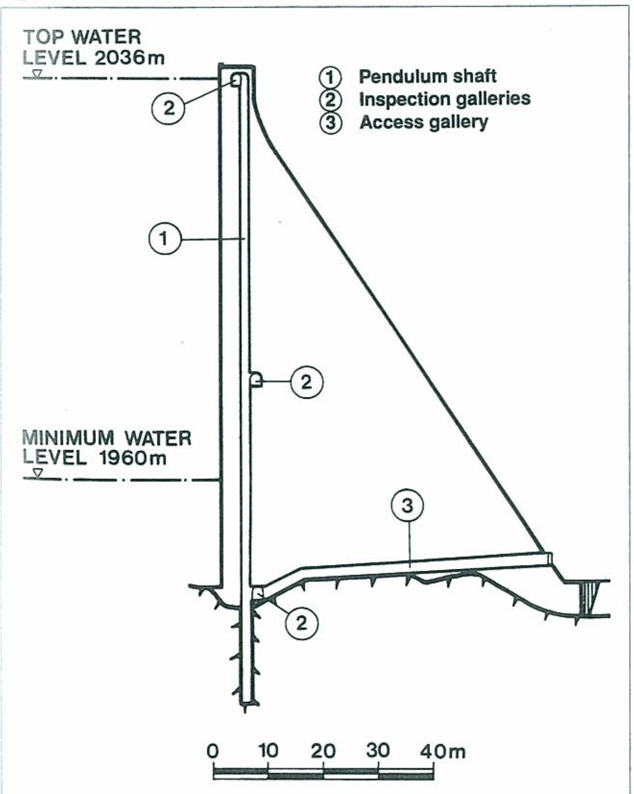


Figure 2 Plan

The dam consists of 25 blocks, each 20 m wide. The concrete was placed without special cooling measures in blocks no bigger than 15 by 20 m. The resulting longitudinal joints are staggered block-wise and provided with vertical shear keyways. The radial joints are keyed and sealed with sheet strips upstream and downstream. Both the longitudinal joints and the radial joints were grouted at the same time.

Figure 3 Central cross section



The hearting concrete contains 135 to 150 kg Portland cement per m^3, and the facing concrete 250 kg Portland cement per m^3. The mean strengths of the facing concrete after 90 days were 35.2 N/mm^2 compression and 4.6 N/mm^2 bending tension.

The dam is provided with an inspection gallery along the entire length of the rock foundation, which constitutes an efficient bottom drainage. In addition, a central inspection gallery at El. 1 977 and a crest gallery are provided.

In addition to a sufficiently deep rock tie-in, a vertical cement grout curtain was sunk in the cut-off wall to a depth of 120 m and supplemented by an inclined secondary curtain 60 m deep. Contact grouting, 15 m deep on average over the whole foundation area, served to consolidate the foundation and prevent seepages from emerging in the bottom joint.

The main instruments installed are three plumb lines. The crest displacements measured in these locations are remotely transmitted. In addition, about 50 uplift gauges were emplaced.

Mooserboden reservoir has a spillway at the orographically right flank of the Mooser dam, consisting of two automatic gates, each 25 m long, and discharging 100 m^3/s. Discharge is through a short tailrace downstream into the bed of the Kaprun Ache stream.

The bottom outlet for the Mooser dam is at the western, orographically left flank. Downstream of the dam, 50 m of a total tunnel length of 200 m are armoured, the remaining length being sealed with a reinforced concrete ring. Two jet valves with a discharge capacity of 31 m^3/s each are provided at the downstream end.

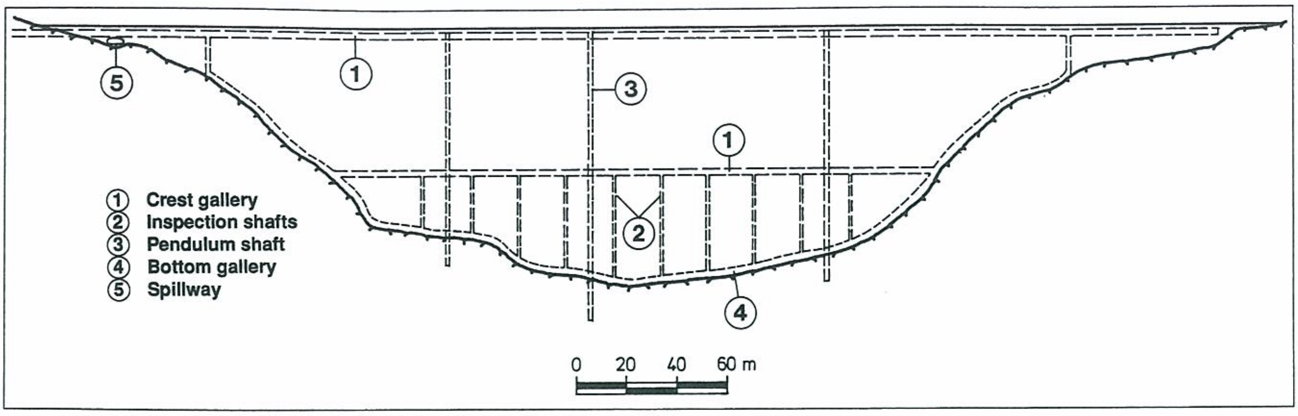
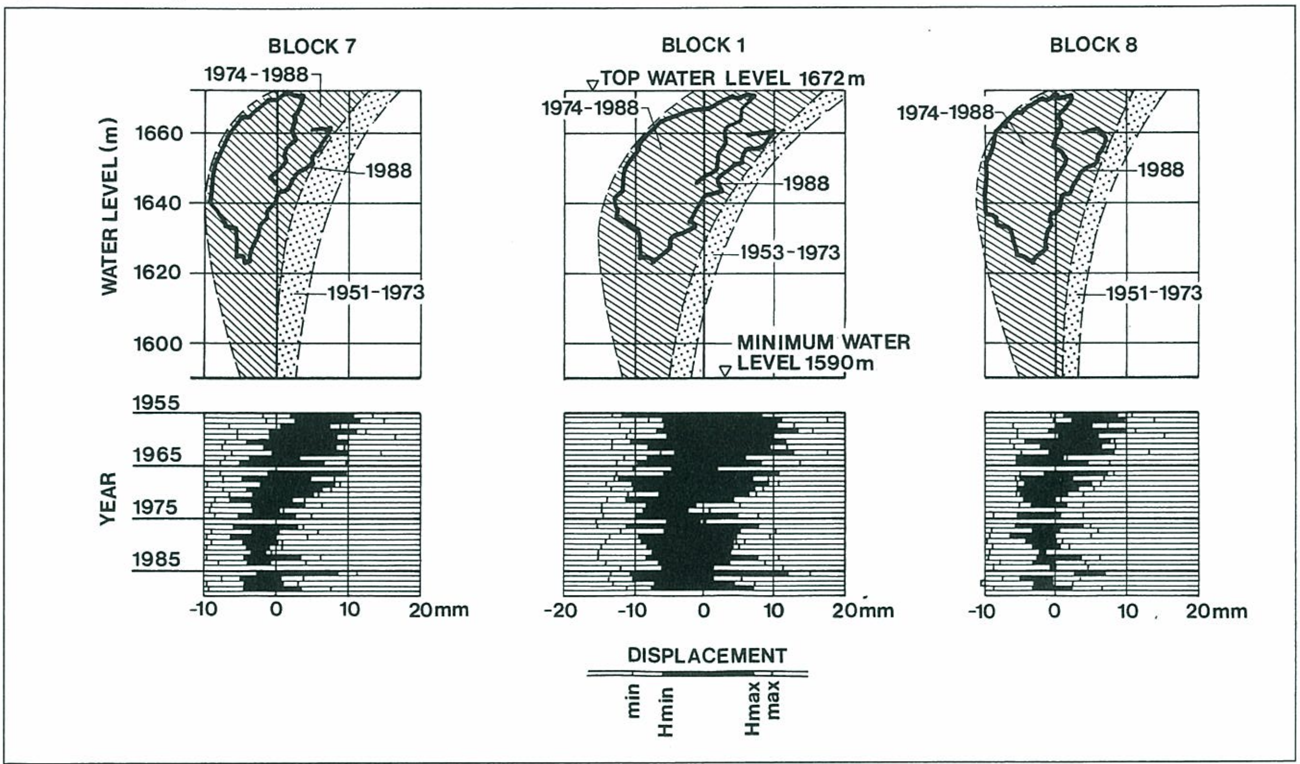


Figure 4 Longitudinal section

Figure 5 Radial displacements at crest elevation



4 EXPERIENCES

Dam displacements during the first years were ± 16 mm a year and have decreased to ± 12 mm after almost 40 years of sound dam operation. The seepage emerging in the bottom inspection gallery is extremely low.

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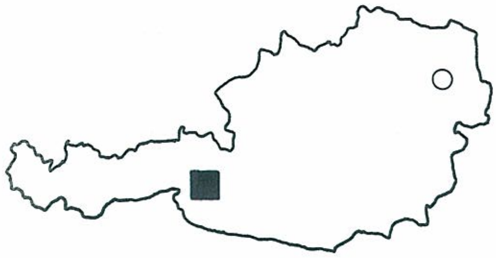
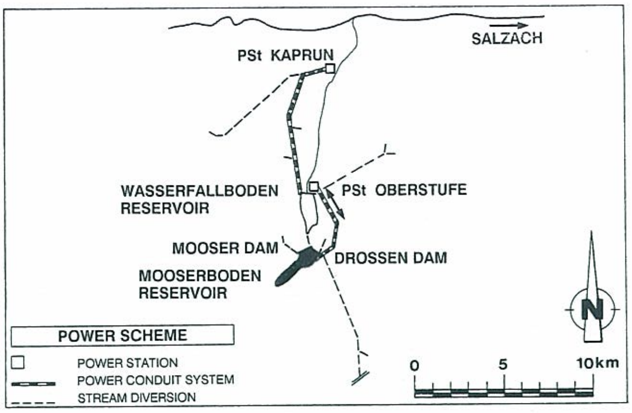
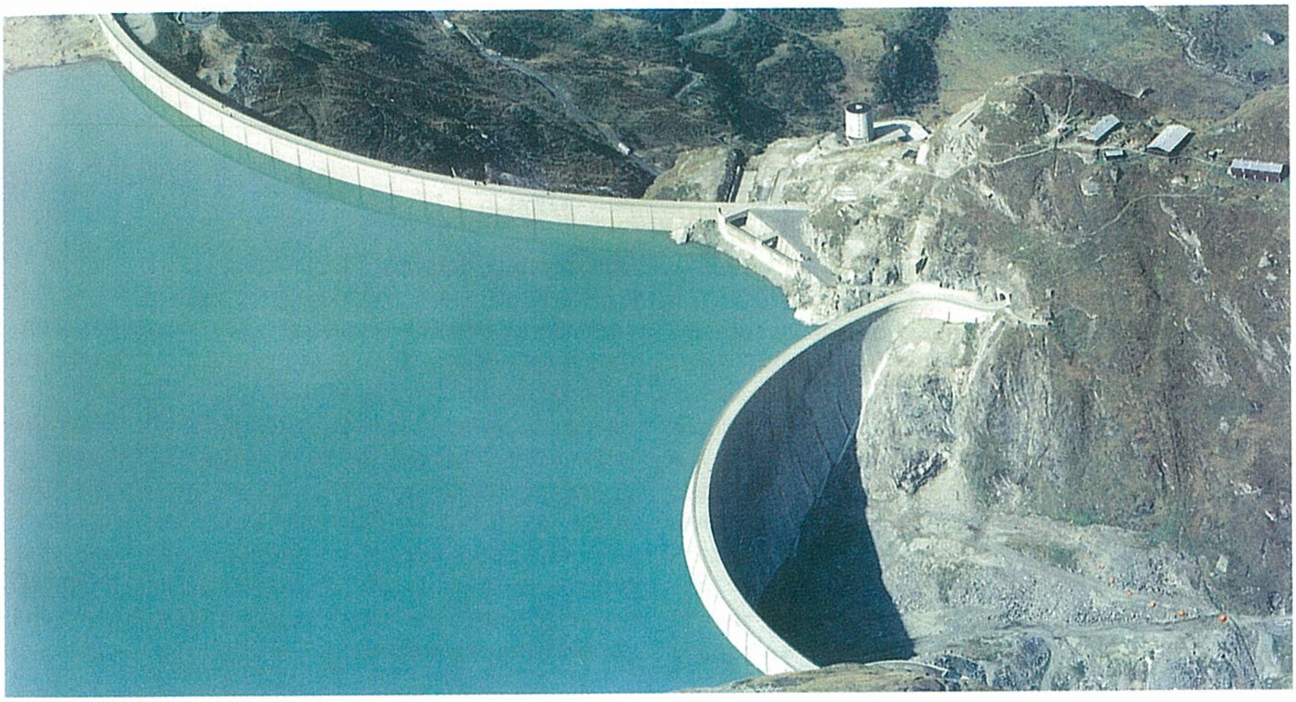
DROSSEN ARCH DAM

Salzburg; Kapruner Ache, Salzach
Nearest town: Zell am See







MAIN TECHNICAL DATA, Chapter K, 26b (2/17)

General

Development	Glockner-Kaprun	
Power Station	Limberg	Kaprun
Construction Period	1950 – 1955	1948 – 1951
Gross Head	365.1 m	860.7 m
Installed Capacity: Turbine	112 MW	220 MW
Pump:	130 MW	–
Mean Annual Generation	152 GWh	486 GWh
of which in winter	86 GWh	386 GWh

Reservoir

Catchment Area: Natural	27 km²
Inflow	62 hm³
Diversions	72 km²
Inflow	130 hm³
Normal Top Water Level (a.s.l.)	2 036 m
Minimum Operating Level (a.s.l.)	1 960 m
Gross Capacity	87 hm³
Live Storage	85 hm³
Area flooded by full Reservoir	1.7 km²

Dam

Maximum Height above Foundation	112 m
Crest Length (26 blocks)	357 m
Thickness at the Crest	7 m
Maximum Thickness at the Base	25 m
Volume: Excavation (overburden, rock)	315 000 m³
Concrete	355 000 m³

Appurtenant Works

Spillway	
Capacity	100 m³/s
Bottom Outlet	
Capacity	62 m³/s
Power Intake	
Capacity	36 m³/s

1 GENERAL

To allow full utilization of the natural basins of the Kaprun valley for annual storage, an upper stage had to be added to Wasserfallboden reservoir. Since the natural runoff of the Kaprun Ache stream is not adequate to fill both reservoirs, the scheme required expansion by the addition of the Möll river via a transbasin diversion. With two longterm storage reservoirs in line, pumped storage has been installed in the upper stage, thus creating ideal conditions for further efficient pumped storage projects. The geological and topographical conditions would have permitted construction of any dam type. For this reason detailed studies were carried out on an earthfill dam, a buttress dam, an arch-gravity dam and an arch dam. The arch dam, viz. a double-curvature constant-angle arch dam, proved to be the most economical structure for the same degree of stability. The natural catchment of the Kaprun Ache stream is augmented by diversions from the Möll catchment (southern slopes of the Tauern range) and the neighbouring Käfer valley. The whole catchment is 99 km² yielding a total runoff of 192 million m³.

2 GEOLOGY

The dam site is located in the central area of the northern slopes of the Austrian Central Alps in the Pennine Tauern window, which is shaped like a dome over large areas. The Tauern "Schieferhülle" formation overlying this dome is bordered by the Granatspitz Zentralgneis core in the west and by the Ankogel Zentralgneis core in the east. The southern Pennine rock formation of the Glockner nappe is bedded with a shallow dip towards NE and consists of partly karstified calcareous mica-schists with varying mica content, with prasinite, quartz schist, micaschist dolomite and gneiss in places. A rock saddle exposed to weathering and karstification is incised by the erosion of the Kaprun Ache stream at the downstream end of the shallow Mooserboden basin, which was polished by glacial erosion, above the escarpment which steeply descends some 300 m down to Wasserfallboden. A NS striking fault with a 4 m wide crush zone is found in the upper portion of the right flank, as are joints striking NS and EW. The insitu rock is overlain to a depth of up to 13 m by moraine, stream-deposited and talus materials. Bedding and jointing is unfavourable with regard to force transfer to the dam foundation (gently sloping bedding, joints and a minor fault striking NS). This caused difficulties with the left dam abutmenttie-in.

Seismicity

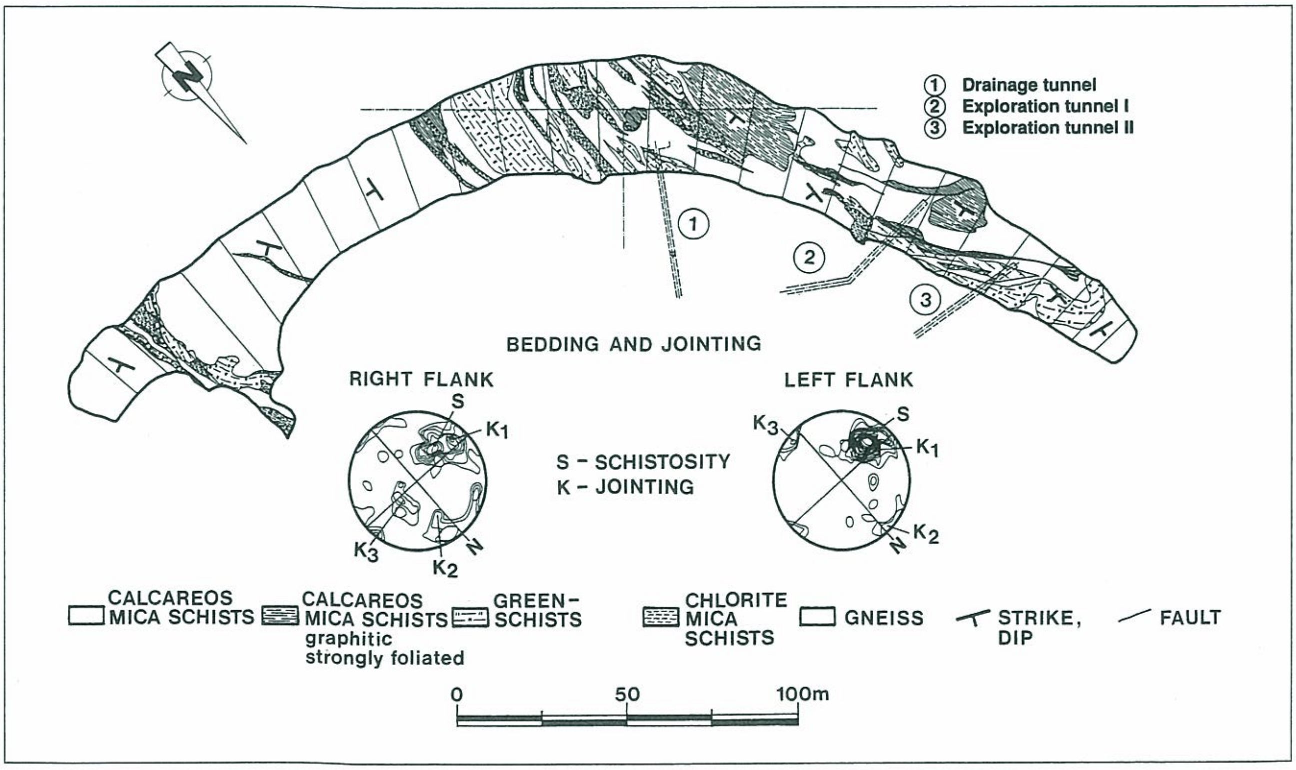
The power scheme is situated between the seismically active regions of the Inn valley in the NW and the Mur-Mürz furrow to Villach in the SE, in an area of very low seismic intensity.

3 DAM

3.1 Design features

A double-curvature constant-angle arch dam with constant-thickness arch elements was constructed. Increasing the arch radii resulted in a favourable angle between arches and rock faces (radius at crest 199.65 m, subtended angle 102°). This new dam type proved the effectiveness of artifical deepening of the foundation base. It yielded stress conditions, in areas with critical tensile stresses upstream, which were no worse than those obtained with arch dams with increasing thickness towards the abutments (up to 3.5 times the thickness at the crown) commonly constructed at that time.

Figure 1 Geological map



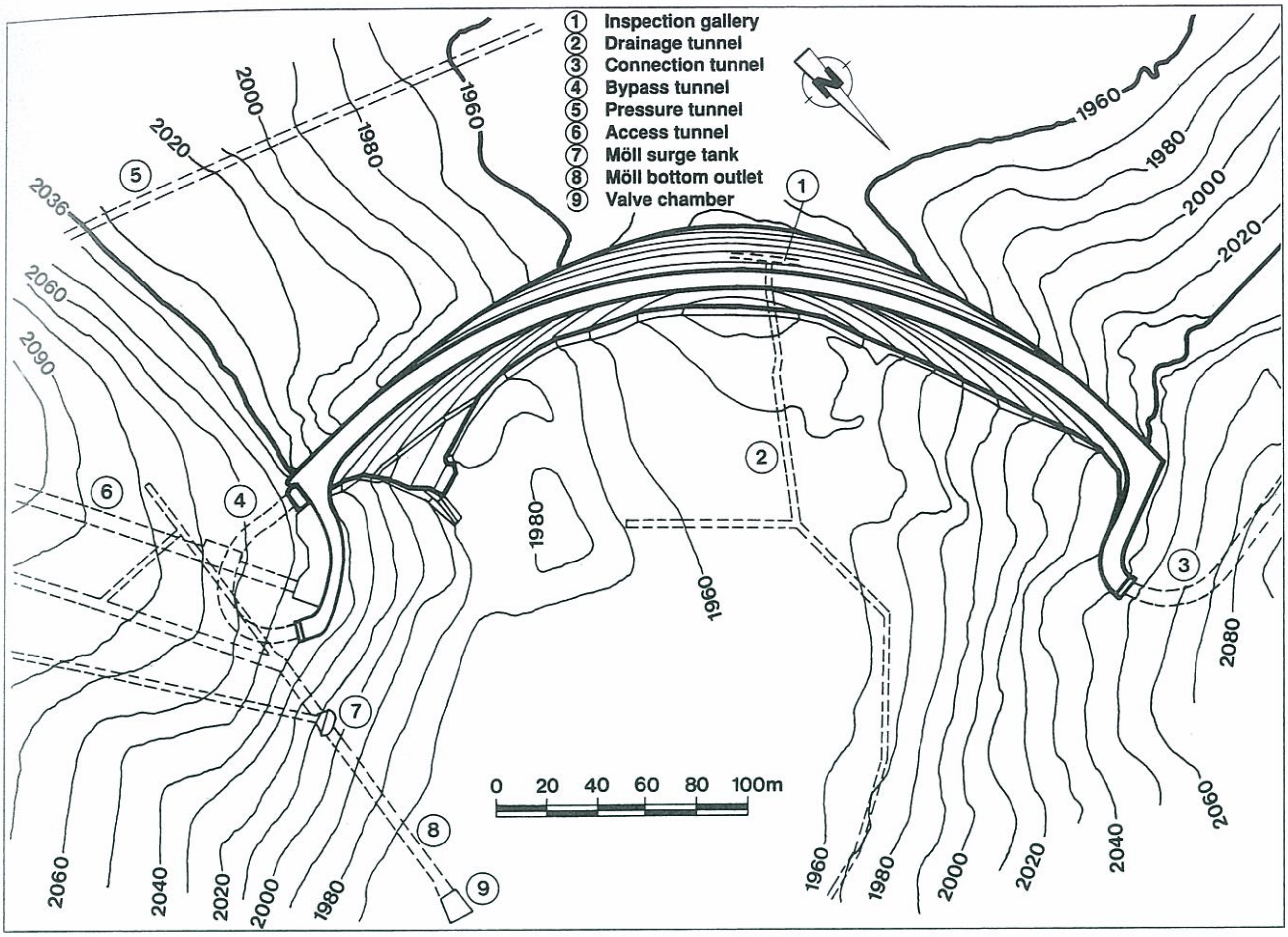
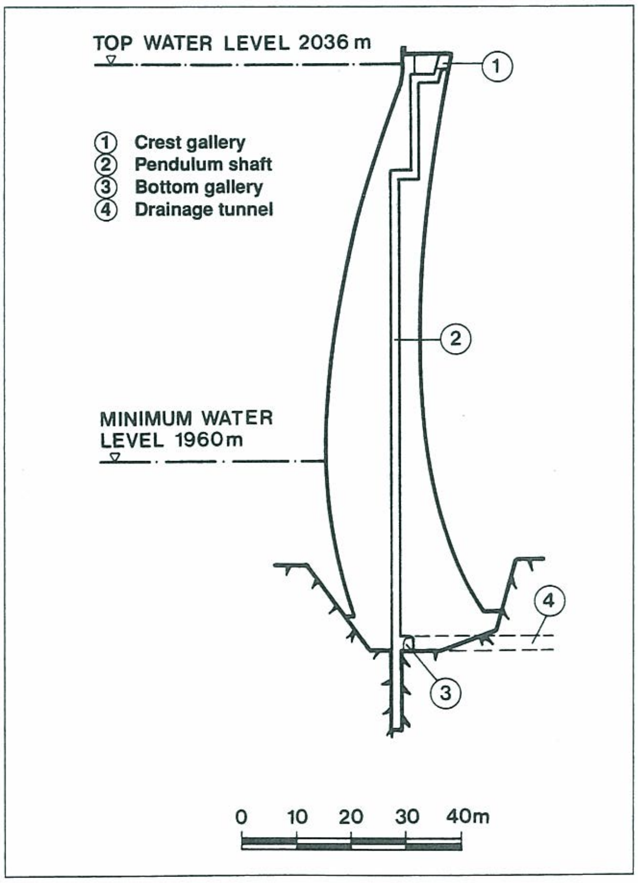


Figure 2 Plan

Figure 3 Cross section



3.2 Material properties

For the concrete used the mix was established as follows: aggregate with a maximum grain size of 120 mm, Portland cement 250 kg/m^3, Frioplast added at 0.5% of cement weight, water/cement ratio 0.48. Average concrete strengths after 90 days were 4.8 N/mm^2 bending tension and 36 N/mm^2 compression.

3.3 Construction

Excavation (rock and overburden stripping) lasted from October 1950 to July 1953. Concreting of Drossen dam started in June 1953 and was completed in August 1955. To avoid major cracking due to shrinkage and temperature variations, the dam was subdivided into blocks 15 m long, and cooling slots were provided in places.

4 EXPERIENCES

4.1 Dam monitoring

The instrumentation installed at Drossen dam comprises a plumb line system in the central block divided into two sections due to the curvature of the dam, clinometers, measuring lines for length gauging, uplift pressure cells and seepage flow monitoring, plus geodetic systems, such as crest levelling and trigonometric monitoring with 24 targets downstream and several monuments on the rock of the eastern dam flank.

4.2 Monitoring results

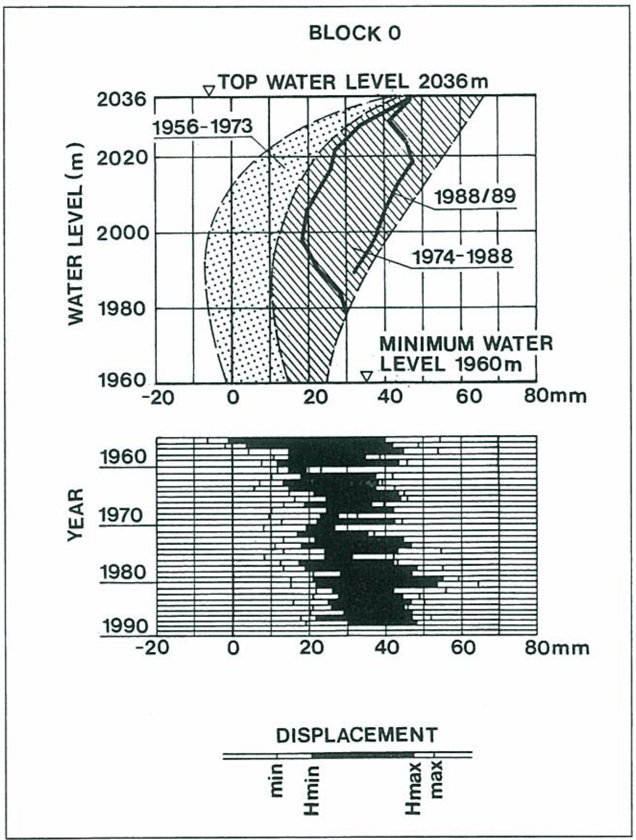
Crest displacements

Monitoring for horizontal displacements in the crest arch indicates permanent displacement of the order of about 20 mm, with 4.0 mm having occurred during the past 10 years. Superimposed on this value is an annual elastic displacement with an amplitude of 40 mm, including ± 13 mm attributable to seasonally fluctuating ambient and concrete temperatures. The vertical displacements obtained from crest levelling do not suggest any significant longterm tendency and indicate load fluctuations in the order of 5.5 mm depending upon the season and the filling level, with a 1.25 mm range of variation. ± 0.5 mm is tolerance in the levelling system and ± 0.75 mm is due to deviations in the actual filling and temperature conditions from the long-term annual mean value.

Hydrostatic pressures

As at all dams equipped with an inspection gallery located directly on the surface of the foundation rock, the uplift cells installed upstream of this gallery show a pressure of up to 100% of hydrostatic pressure, whereas those cells which were emplaced downstream of the bottom gallery indicate 15% of hydrostatic pressure as a maximum.

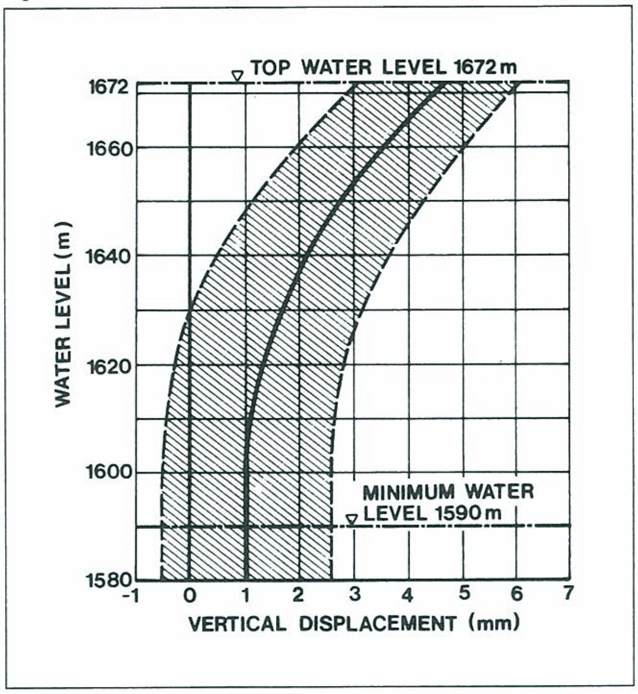
Figure 4 Horizontal crest displacement



Seepages

Those seepages which emerge in the bottom inspection gallery have dropped from about 7 l/s during the first year of operation with full reservoir to 0.2 l/s at the moment. Analogously, the annual seepage volume has decreased from about 57 000 m³ to 6 000 m³ today. Another location for seepage monitoring was installed in a dummy tunnel in the eastern flank at El. 1 934 m in an extension of the bottom inspection gallery. At drawdown level seepage flows are about 0.9 l/s, reaching some 1.9 l/s at topwater level. The annual seepage volume has remained more or less constant at 42 000 m³ over the whole period of observation. It is assumed that these waters do not stem from permeabilities in the foundation but from draining ground water, the level of which depends on the reservoir filling level over large areas.

Figure 5 Vertical crest displacement



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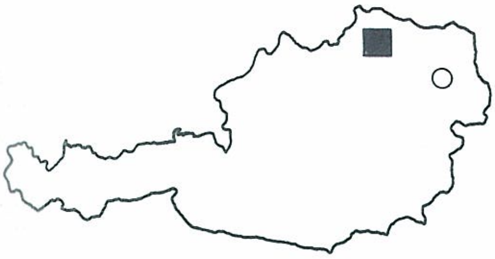
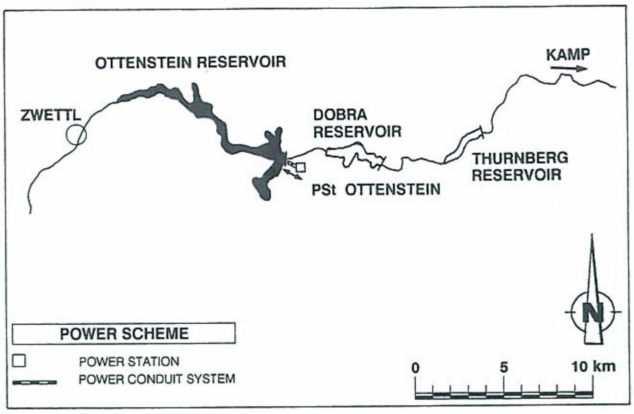
OTTENSTEIN ARCH DAM

Lower Austria; Kamp, Danube
Nearest town: Zwettl





MAIN TECHNICAL DATA, Chapter K, 27 (2/19)

General

Development		Kamp	
Power Station	Ottenstein	Dobra	Thurnberg
Construction Period	1954–56	1951–53	1950–52
Gross Head	58 m	65 m	20 m
Installed Capacity	47 MW	16 MW	3 MW
Mean Annual Generation	33 GWh	40 GWh	12 GWh

Dam

Maximum Height above Foundation	69 m
Crest Length	240 m
Thickness at the Crest	7 m
Maximum Thickness at the Base	24 m
Volume: Excavation (overburden, rock)	27 800 m^3
Concrete	124 000 m^3

Reservoir

Catchment Area: Natural	889 km^2
Inflow	221 hm^3
Normal Top Water Level (a.s.l.)	495.0 m
Minimum Operating Level (a.s.l.)	476.5 m
Gross Capacity	73 hm^3
Live Storage	51 hm^3
Area flooded by full Reservoir	4.3 km^2

Appurtenant Works

Spillway, gated overflow spillway, 2 crest gates, l = 2 x 27 m, h = 2.6 m	
Capacity (height of overflow 0.6 m)	574 m^3/s
Bottom Outlet, through the right-hand dam abutment, 1.2 m dia., 2 valves	
Capacity	16 m^3/s
Power Intake	
Capacity	100 m^3/s

1 GENERAL

The Ottenstein arch dam was constructed between 1954 and 1956 to impound the pilot reservoir for a three-stage development on the river Kamp in Lower Austria. Ottenstein is a pumped-storage scheme (20 MW pump capacity) utilizing flow pumped from the reservoir of the downstream Dobra-Krumau power scheme with the Dobra arch dam. The operation of the Kamp group of power stations is based on a uniform concept. Together with the Dobra reservoir (51 million m³ live storage), seasonal storage raises the winter proportion of annual generation from 42% (corresponding to the natural inflow) to 52% and substantially reduces flood risks to the populated downstream areas. As the reservoir has become a popular recreation area, the water level is kept near top water level during the summer months.

The reservoir has a live storage of 51 million m³ and is filled by the runoff from a catchment of 889 km², reaching a long-term average of 221 million m³ p.a. Due to the provision of a spillway crest equipped with automatic gates, the normal operating level is not exceeded during floods. As the presence of the town of Zwettl near the upstream end of the reservoir imposed a limit on the maximum water level, it was possible by means of this spillway arrangement to accomplish a maximum operating level 2 m higher than would have been obtained in the case of an ungated crest. Routine maintenance guarantees the good functioning of the gates at any time.

The Ottenstein power scheme consists of an arch dam with a powerhouse built against the dam on the right-hand downstream side. The switchyard is located on the powerhouse roof. Following partial filling at El. 486 m in 1956, top water level at the El. 495 m was first reached in 1957.

By the end of 1992, power station operation is planned to be converted from local manual control at Ottenstein to central telecontrol and telemonitoring from the EVN load control centre at Maria Enzersdorf am Gebirge near Vienna and from a regional grid control centre at Stratzdorf.

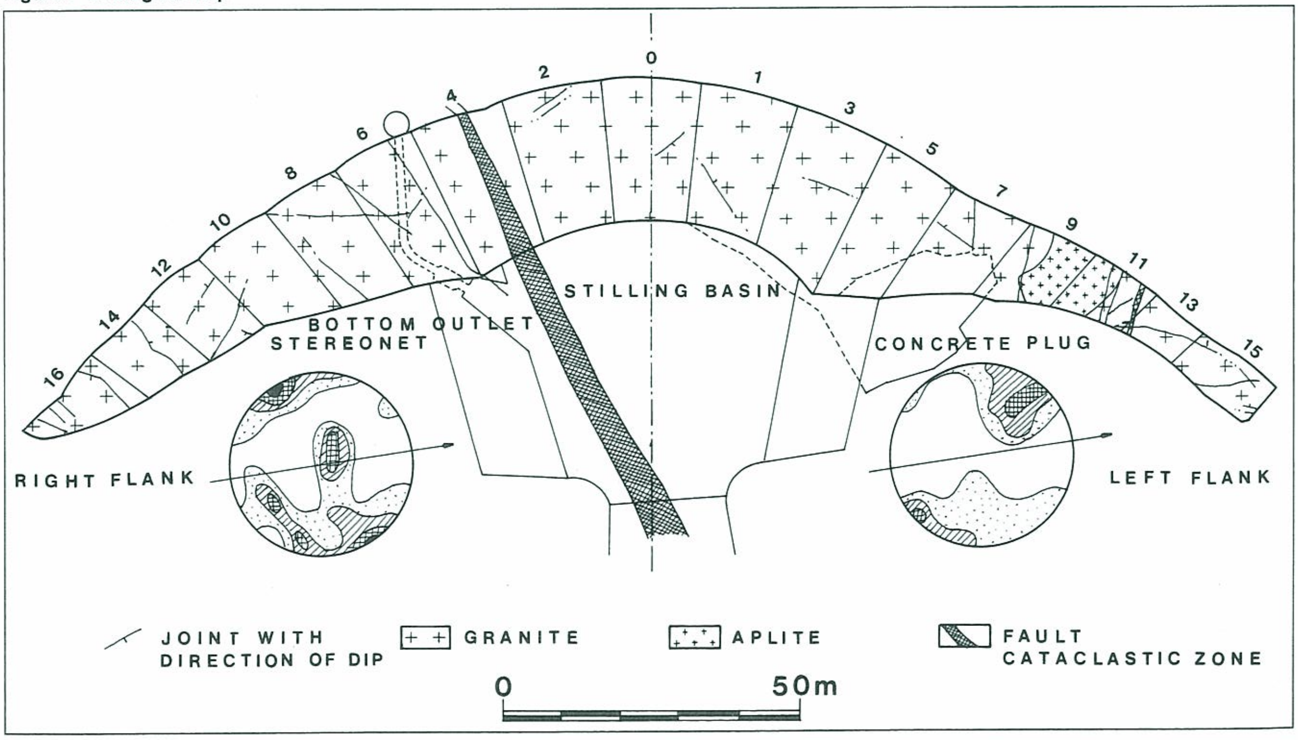
2 GEOLOGY

The Ottenstein arch dam was constructed at a trapezoidal valley section with a wide valley bottom and moderately steep slopes. Bedrock is composed of porphyritic granite covered with boulders and talus material in the valley bottom. An aplite layer some 15 m in thickness passes through the left rock abutment. On the right-hand slope, solid granite is encountered beneath a thin overburden. A system of steep joints with relatively large spacing prevails. Only in the valley bottom a completely inactive 4 m wide failure zone is present. The whole region is considered seismically inactive.

3 DAM

The dam is a double-curvature arch structure with arch thickness increasing from the crown towards the abutments. Only the upper arches are of constant thickness. Following preliminary static analyses of several alternatives of a symmetrical substitute dam by a single-section load distribution and a main static analysis of the idealized dam shape by the load distribution method with radial adjustment, a subsequent check analysis with a triple adjustment and allowance for dissymmetry was finally carried out. This gave maximum main stresses of 4.6 N/mm² compression and 2.5 N/mm² tension, a maximum tensile stress parallel to the surface of 1.4 N/mm² and maximum foundation stresses normal to the slope of 2.8 N/mm² compression and 0.1 N/mm² tension.

Figure 1 Geological map



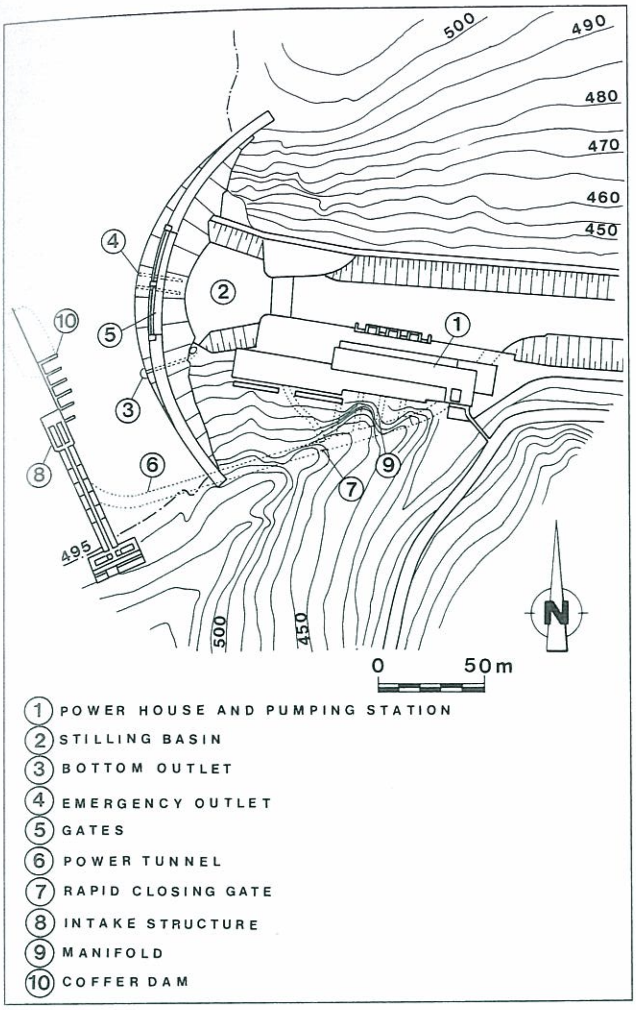


Figure 2 Plan

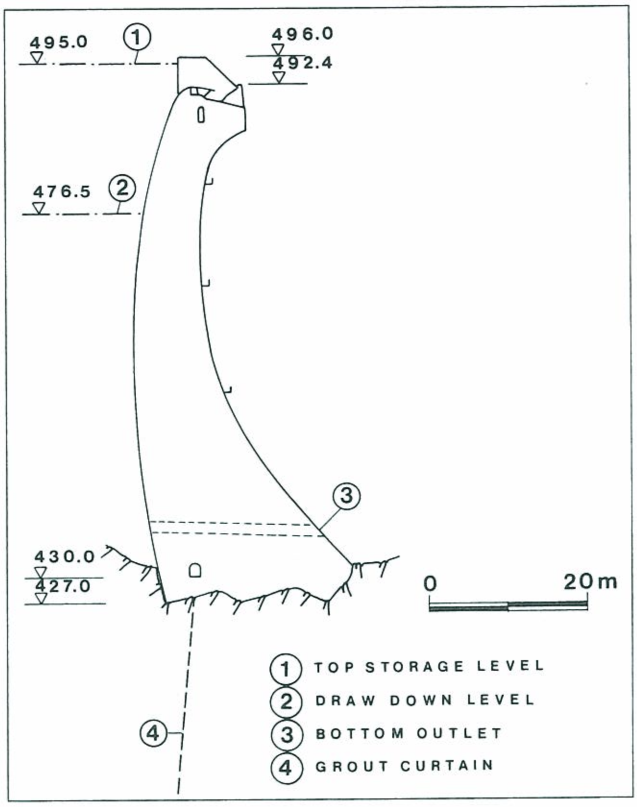
The vibrated concrete was produced of 240 kg of iron Portland cement or 216 kg of iron Portland cement and 24 kg of trass, per m³, with 0.5% plasticizer. The water-cement ratio was 0.53. Local gneiss was used as aggregate. This was separated into 5 fractions up to 120 mm particle size. 20% quartz sand of 0 to 3 mm size was added. Aggregate was heated during the winter months. Average concrete strength at 90 days was 31 N/mm² compression and 4.9 N/mm² bending tension.

The concrete was mixed in an automatic mixing plant with three gravity mixers of 1 500 l capacity each and was placed by a blondin in 2.0 m lifts in blocks with a maximum length of 16 m. In spite of the trass admixture, about half the concrete volume was placed using artificial cooling. Joints were sealed with 1 mm copper sheet at the upstream face. Downstream, main joints and construction joints were sealed with galvanished steel sheet and were grouted.

A grout curtain 15 000 m² in total area was provided under the dam and in the rock abutments. The grout curtain is inclined to the upstream and extends to a depth of 30 m with several bore holes reaching a depth of 50 m. The right-hand slope was grouted to a distance of 100 m upstream of the dam and to a depth of 60 m, the left-hand slope was grouted over an upstream distance of 70 m and to a depth of 40 m. Grout, consisting of Portland cement with trass cement, was pumped in under a pressure of 10 to 25 bar prior to concrete placement, occasionally under higher pressures of up to 35 bar. In addition, contact grouting was carried out in the foundation contact.

The overflow spillway with the two crest gates was developed on a scale model. Spillway capacity with gates open – without surcharge – is 420 m³/s (design flood). The safe discharge of the HQ 5000 in the order of 600 to 650 m³/s is possible. The power intake is located on the right-hand slope 50 m above the dam. A 5.80/5.50 m diameter power tunnel, which is steel-lined downstream of the grout curtain, supplies the powerhouse with a turbine discharge of 100 m³/s. Two 1.50 m diameter emergency outlets equipped with fuse plugs and capable of 53 m³/s each are located in the centre blocks at El. 436 m. Low-level discharge is provided through the right-hand dam abutment at El. 439 m. The outlet is 1.20 m in diameter and is equipped with a 1.20 m butterfly valve and a 1.0 m diameter hollow-jet valve. It discharges into the stilling basin with a capacity of 16 m³/s.

Figue 3 Cross section



4 EXPERIENCES

4.1 Dam surveillance

Dam instrumentation includes:

for measuring dam deformations:
- 4 pendulums (radial and tangential),
- joint gauges in the inspection gallery and at the dam crest,
- 1 radial clinometer each in the inspection gallery and at the dam crest;

for measuring deformations of the rock foundations:
- 1 pendulum and 1 inverted pendulum extending 12 m into the rock,

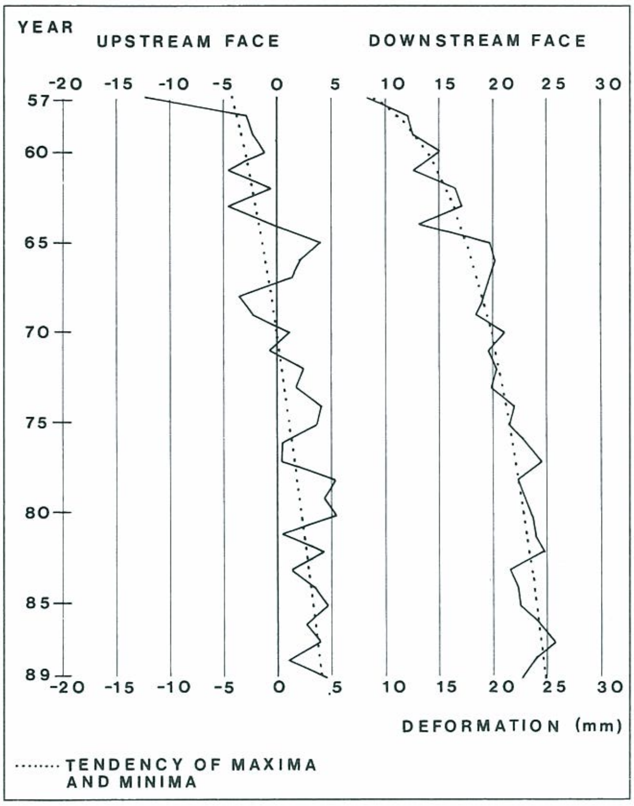
- 1 extensometer for vertical displacement in the middle of the valley,
- 2 telerockmeters on the left slope;

for measuring uplift pressures:
- 52 instrument stations;

for measuring seepage:
- 15 drainage holes in both slopes,
- 1 seepage measuring station for each slope in the base gallery,
- 4 piezometers in the left slope;

for measuring concrete temperatures:
- 30 thermometers.

Figure 4 Maximum and minimum radial deformation



Limit value contacts are installed at the pendulum in the main instrument plane and at two seepage measuring stations in the inspection gallery. Unusual pendulum deviations and seepage measurements are indicated by optical and acoustic signals in the Ottenstein control centre. Elastic radial deformations of the dam crest vary between 16 and 24 mm. Seepage flow measured in the inspection gallery varies between 0.1 l/s and 0.5 l/s. Maximum vertical movements between dam toe and rock foundations amount to 2.4 mm. The pattern of movements is in good agreement with the pattern of uplift pressures.

Manual data acquisition is accomplished by means of an electronic hand terminal. Identification of the instrument station by means of a bar code reading device is followed by input of the measured value, which is automatically checked for preset extreme values (plausibility check).

The principal instruments of the dam are equipped with automatic measured-data acquisition. Apart from direct instrument readings, data is checked for limit value exceedance on a local data logger and stored temporarily, and is teletransmitted to a control centre manned round the clock. Dam surveillance and teletransmission facilities are automatic.

4.2 Special occurrences

The original stilling basin floor was unlined except for a fault zone. During spillway tests in 1957 and 1959, the impact of the water-air mixture caused erosion about 20 to 25 m from the dam toe. Lasting safety was accomplished by filling the scour with concrete and providing a concrete slab anchored in the rock.

The 5 m to 7 m routine drawdowns every spring have not revealed any evidence of reservoir silting in spring, neither at the upstream end nor on the banks, and there is no silting risk to the power tunnel and the inlet of the bottom discharge.

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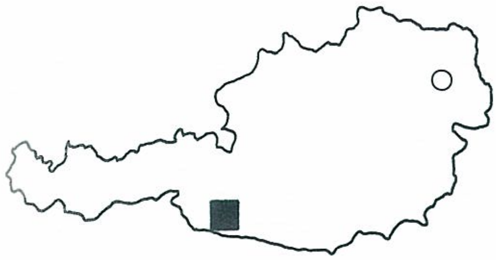
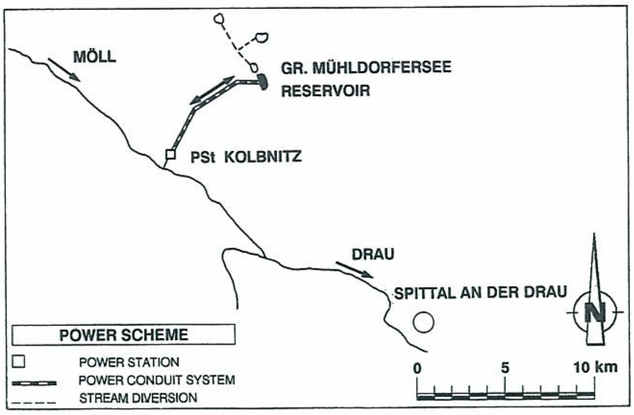
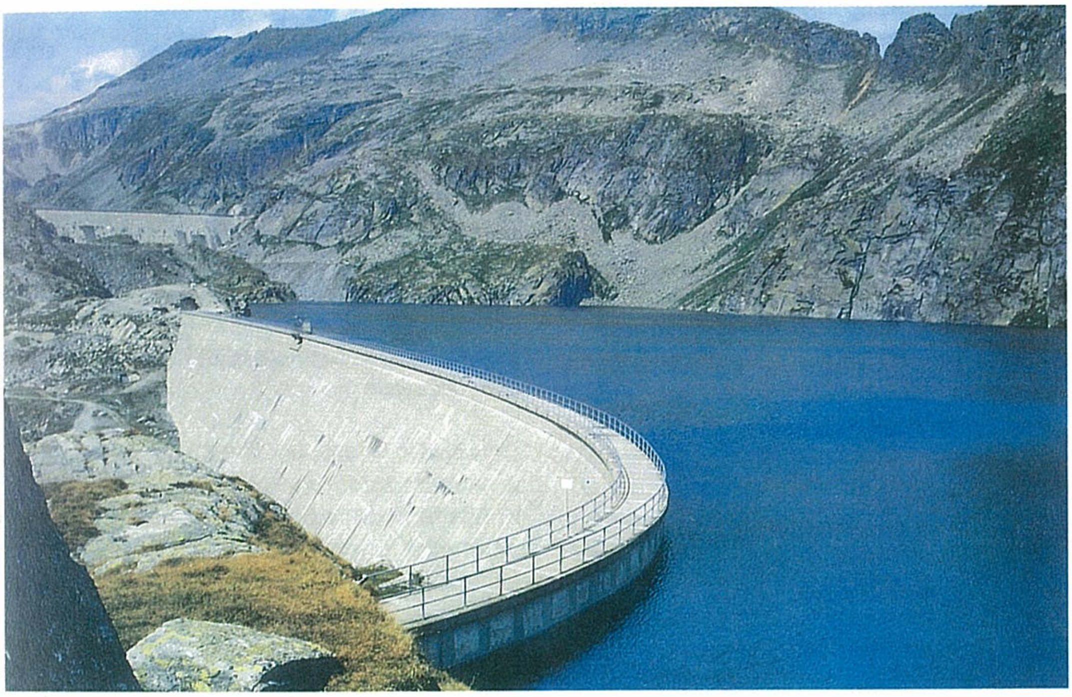
GROSSER MÜHLDORFERSEE GRAVITY DAM

Carinthia; Möll, Drau
Nearest town: Spittal/Drau







MAIN TECHNICAL DATA, Chapter K, 29 (2/21)

General

Development	Reisseck-Kreuzeck
Power Station	Kolbnitz
Construction Period	1955–1957
Gross Head	1 772 m
Installed Capacity: Turbine	139.0 MW
Pump	18.6 MW
Mean Annual Generation	307.4 GWh
of which in Winter (44.5%)	136.9 GWh

Reservoir

Catchment Area: Natural	2.7 km²
Inflow	4 hm³
Diversions	5 km²
Inflow	14 hm³
Normal Top Water Level (a.s.l.)	2 319 m
Minimum Operating Level (a.s.l.)	2 255 m
Gross Capacity	7.9 hm³
Live Storage	7.8 hm³
Area flooded by full Reservoir	0.24 km²

Dam

Maximum Height above Foundation	46.5 m
Crest Length	432.7 m
Thickness at the Crest	2.0 m
Maximum Thickness at the Base	31.3 m
Volume: Excavation (overburden, rock)	47 000 m³
Concrete	152 900 m³

Appurtenant Works

Spillway, overflow spillway at the highest block, l=10 m, with ski-jump	
Capacity	17.0 m³/s
Bottom Outlet, through the dam, 0.9 m dia., 2 valves	
Capacity	4.2 m³/s
Power Intake	
Capacity	4.5 m³/s

1 GENERAL

Utilization of the runoff from the Reisseck mountains was first considered before the First World War. A power station was constructed at Mühldorf between 1922 and 1924. In 1945 the projects were passed on to Kärntner Elektrizitäts AG and development was continued. In 1948, Österr. Draukraftwerke AG took over the project under construction and completed the construction of the group of power schemes in 1961.

The Reisseck-Kreuzeck group of power schemes utilizes the hydro potential of a catchment of 164 km² in the northwest of the province of Carinthia, on both sides of the river Möll. The main feature of the three-stage development is the Reisseck seasonal storage reservoir, which develops the hydro potential of a group of lakes situated in the Reisseck mountains between El. 2 319 and El. 2 399 for power generation during the winter months. At four of these high-level cirque lakes, dams were constructed to increase their storage from an original 5.37 to a total 17.5 million m³. Water from these reservoirs is collected in a special hydraulic system and conveyed to the Kolbnitz power station in the Möll valley. Water is utilized in a single stage with a maximum head of 1 772.5 m, which is still the highest in the world. The facilities provided for the construction and operation of the scheme have been adapted and then extended for tourist trade and now attract great numbers of visitors every year.

The 46.5 m high Grosser Mühldorfersee gravity dam, 433 m long at the crest, raises the water level of a natural lake by about 35 m, so as to create a live storage of 7.8 million m³ which, due to the substantial head, corresponds to a stored energy of 31 GWh.

2 GEOLOGY

Geologically, the Reisseck massif forms part of the Central Alps. It consists of a core of Zentralgneis ("Central Gneiss") that daylights in a lake plateau flanked by an inner and outer Schieferhülle (slate mantle) series. The Kreuzeck mountains are formed by an Alt Kristallin ("old crystalline") block.

The dam site proper of Grosser Mühldorfersee is a ridge of orthogneiss with feldspar eyes that has impounded the natural lake. Schistosity planes show a downstream dip of fairly uniform steepness; occasionally transverse faults have manifested themselves as notches, the largest of which formed the original outlet of the lake and has been closed by the dam.

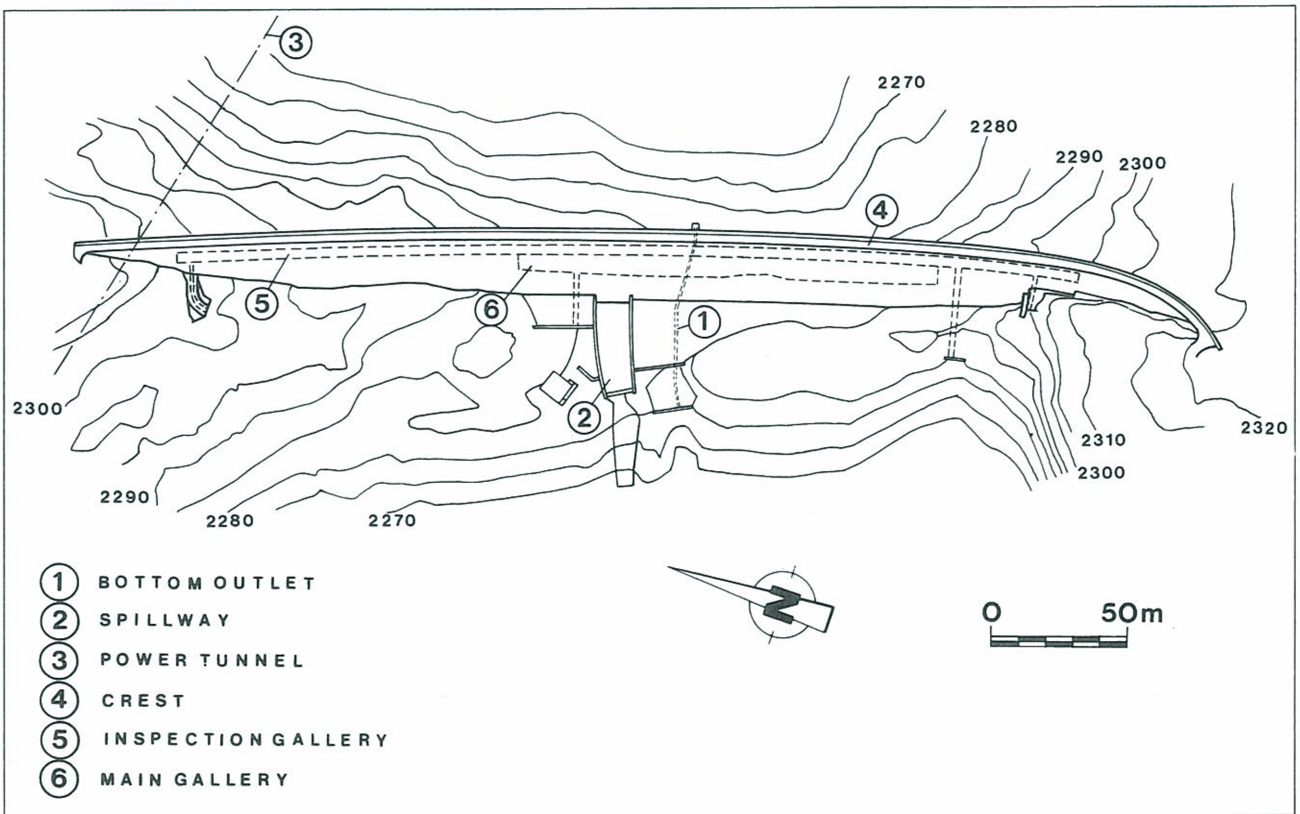
There have been practically no problems resulting from the geological conditions.

3 DAM

The gravity dam is slightly curved in plan to follow the crest of the natural ridge and ends to the left in a compound-curved abutment. A striking feature of the dam cross-section is the presence of two large galleries. The lower gallery at the base was provided to restrict uplift pressures at the upstream dam toe and also as concrete-saving measure.

Compressive stresses in the rock foundations along the downstream dam toe were calculated to be approximately 5 N/mm² with a full reservoir. Photoelastic testing performed to check the results of the static analysis confirmed the absence of tensile stresses in the gallery

Figure 1 Plan



wall. Concrete aggregates had to be hauled over a distance of 65 km from a limestone gravel pit on the bank of the river Drau near Villach, as no material of adequate quality and quantity was found in the vicinity of the dam site. Aggregates were prepared in 5 fractions with a maximum size of 130 mm. Concrete was mixed in four 1 000-l gravity mixers and placed by means of two blondins in blocks 11 to 14 m wide in 1.5 m lifts. A feature of particular interest is the fact that, instead of the conventional timber formwork, vacuum concrete slabs were used as upstream and downstream boundaries of the dam body. The slabs are 2.0 m long by 1.5 m wide and 6 to 10 cm in thickness and were precast at the site.

Table 1 Concrete

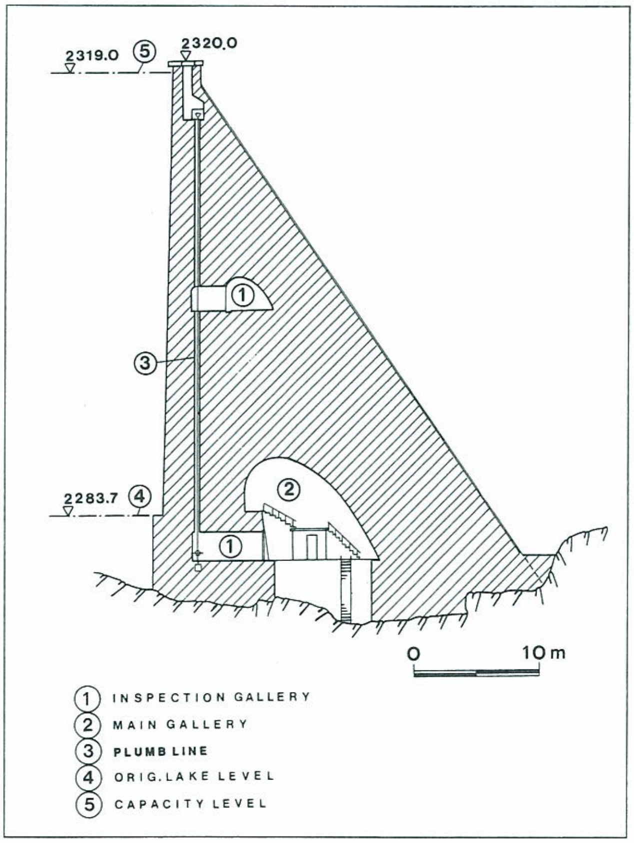
Part of structure		Foundation block	Hearting concrete	Above lower gallery	Upstream concrete slabs
Type of cement		PZ 225 + 8 % slag			PZ 485
Dosage of cement kg/m W/C		225 0.52	180	145.00 0.62	350.00 0.40
Strength after 28 days, N/mm	Compression Bending tension	24.5 3.9	20.3 3.7	17.5 3.0	32.0 8.2

The grout curtain was provided 1 m upstream of the dam foundation. Bore holes were sunk every 2 metres to depths alternating between 6 m and 12 m. A total amount of about 80 t of grout was pumped into bore holes totalling 2 000 m in length.

Appurtenant works include

- a power intake, arranged at the bottom of the lake 100 m upstream of the dam, continuing as a tunnel of 4.5 m³/s capacity driven through the ridge,
- an overflow spillway on the highest block, designed for a discharge of 14 m³/s at 74 cm surcharge, and
- a bottom outlet through the dam, with a capacity of 4.2 m³/s.

Figure 2 Cross section



The latter two facilities are capable of a total flood relief of 18 m³/s, which corresponds to 6.75 m³/s.km² of natural catchment (2.7 km²).

4 EXPERIENCES

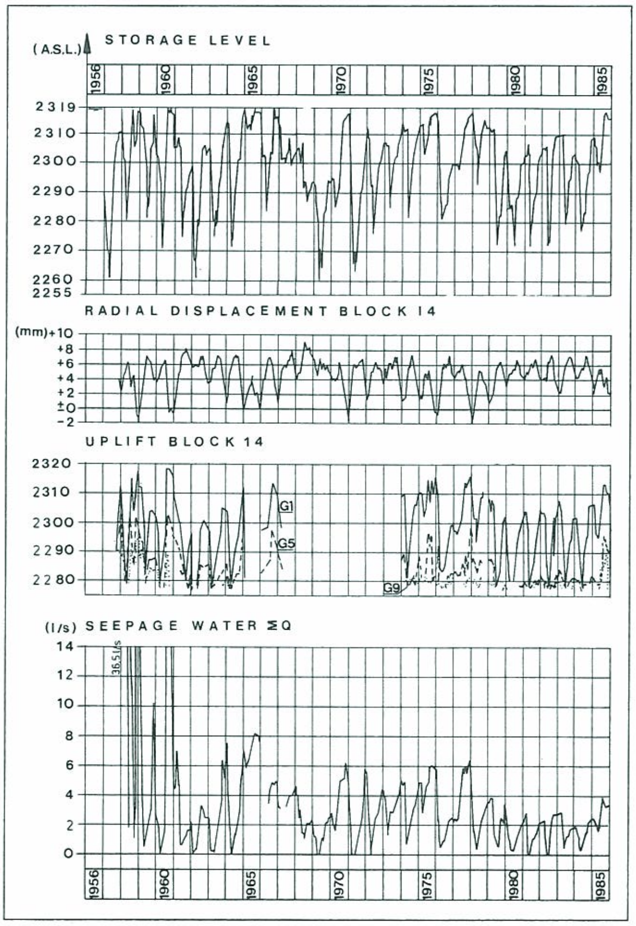
4.1 Dam surveillance

The dam surveillance system was completely renewed between 1986 and 1987. Dam behaviour is now monitored by an inverted plumb-line device, invar wire extensometers, a great numer of joint metres, uplift and joint water pressure measuring devices extending into the rock foundations and a central seepage measuring station. The plumb-line and seepage measuring devices are read automatically and their values teletransmitted to a central control station.

In addition, the dam and surrounding terrain are observed geodetically by precision triangulation.

4 dam attendants observe all the 160 instrument stations at the Grosser Mühldorfersee dam throughout the year, even in winter, in accordance with an inspection schedule imposed by the authorities.

Figure 3 Long term behaviour, results of measurements



Since 1957, continuous records have been kept of crest displacement, uplift and seepage measurements. In the full reservoir condition, the dam crest moves about 8 to 10 mm to the downstream and at the same time heaves by about 2 to 3 mm. Uplift at the upstream dam toe with a full reservoir reaches about 80% of the hydrostatic head. Seepage losses with a full reservoir have stabilized at about 6 l/s since 1965.

4.2 Special occurrences

Maintenance of the originally bitumen-filled joints between the prefabricated facing slabs, amounting to a total length of 11 600 m, turned out to be particularly burdensome. Following several unsuccessful attempts to repair defective seals, the entire joint system was repaired with a compound called Kemperol in the years 1979 to 1982. Upon careful cleaning of the concrete surface and application of a prime coat, elastomer-coated diolene fibre mat, 35 cm in width and 3 to 4 mm thick, was glued across the joints. Another coating using polyester resin and finally Kemperol synthetic resin sealing has so far ensured the watertightness of the joints.

These sealing measures have been supplemented by the installation of a compressed-air bubbling device to avoid ice formation in the vicinity of the dam and the resulting damage to the seals.

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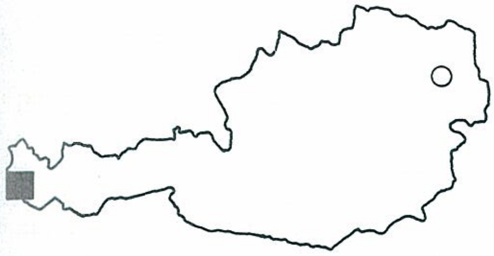
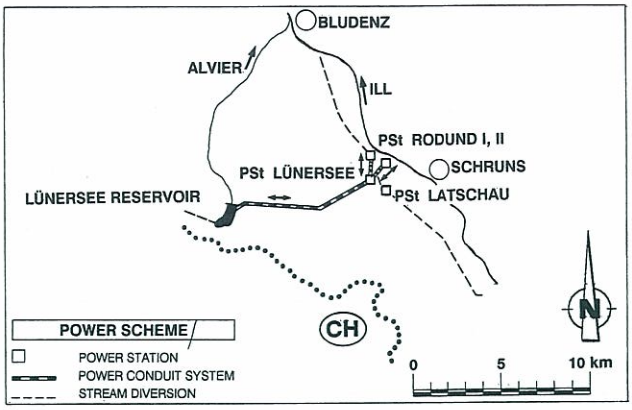
LÜNERSEE GRAVITY DAM

Vorarlberg; Alvier, Jll, Rhine
Nearest town: Bludenz





MAIN TECHNICAL DATA, Chapter K, 33 (3/2)

General

Development	Obere Jll-Lünersee
Power Station	Lünerseewerk
Construction Period	1954 – 1958
Gross Head	974 m
Installed Capacity: Turbine	232 MW
Pump	224 MW
Mean Annual Generation	371 GWh
of which in winter	291 GWh

Reservoir

Catchment Area: Natural	9 km²
Inflow	12 hm³
Diversions	3 km²
Inflow	3 hm³
Normal Top Water Level (a.s.l.)	1 970 m
Minimum Operating Level (a.s.l.)	1 897 m
Gross Capacity	94 hm³
Live Storage	78 hm³
Area flooded by full Reservoir	1.55 km²

Dam

Maximum Height above Foundation	30 m
Crest Length (30 blocks)	380 m
Thickness at the Crest	4 m
Maximum Thickness at the Base	20 m
Volume: Excavation (overburden, rock)	15 000 m³
Concrete	41 000 m³

Appurtenant Works

Spillway, overflow	
Capacity	58 m³/s
Bottom Outlet, 2 butterfly valves	
Capacity	15 + 5 = 20 m³/s
Power Intake	
Capacity	32 m³/s

1 GENERAL

The reservoir is situated in a glacially eroded lake 90 m deep confined by a mighty natural rock sill transversing the valley. Outflow from the natural high- mountain lake is via a karstic system in the natural rock barrier. This the largest reservoir of the Vorarlberger Jllwerke AG was thus created by sealing the rock barrier and by constructing a 30 m high gravity dam on the apex of the rock barrier. The reservoir is utilized in the Lünerseewerk pumped-storage plant. It is used as both a short-term and an annual storage reservoir and forms part of the Obere Jll - Lünersee power scheme.

The power plants downstream of Lünersee, i.e. Rodund I and II, and also the Walgau power plant all benefit from the shift of generation to the winter period.

The first plans for the exploitation of energy supplied by the Lünersee lake were drawn up at the beginning of the twenties.

In 1920 the Vorarlberg regional authority began the first preliminary works for a Lünersee power plant with the construction of a 190 m long outlet tunnel. The 90 m deep lake was to be drawn down 50 m to below water storage level, so that the natural rock barrier could be inspected and sealing works performed.

After this outlet tunnel had been driven to the immediate proximity of the lake, it was equipped in the middle with two wedge type valves 400 and 600 mm dia., operated through a 42 m deepshaft. For safety reasons a second pair of wedge type valves 400 mm dia. were installed at the downstream end of the tunnel.

An aquiferous layer encountered at a distance of 11 m before the lake made it necessary to divert the tunnel to the right. Then it was finally possible to continue the heading of the tunnel until the remaining distance to the lake was only 2.1 m.

These preliminary works conducted by the Vorarlberg regional authority were continued after 1925 by the Vorarlberger Jllwerke AG including preparations for the final blasting.

Depth gauging conducted from the frozen surface of the lake provided a detailled survey of the rock surfaces for the blasting and it was demonstrated that the rock was not covered by any deposits. Thus it was possible to head a few boreholes up to a distance of 40 cm from the lake. Before blasting, the face of the tunnel was sand tamped in order to improve intrinsic strength and absorb the water hammer.

The final blasting on August 26, 1925 was a full success, permitting controlled drawdown.

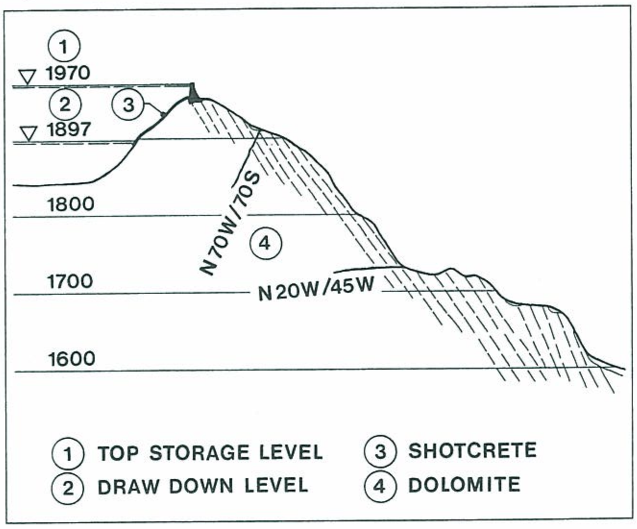
In 1926/27 and 1930/31 sealing measures were executed and seepage through the natural rock barrier reduced from an initial 350 l/s to 30 l/s.
It was not until construction of the Lünersee power plant and erection of the dam in 1955 – 1958, however, that utilization of the reservoir began. In this period the natural barrier was once more sealed with shotcrete and grouting. After partial filling to El. 1 960 in 1958, first full filling was performed in autumn 1959.

2 GEOLOGY

The dam foundation is a narrow natural rock barrier composed of dolomite, polished by glacial erosion and sloping away steeply to the Alvier valley. Strata strike across the valley and dip to the downstream. The dam foundation is located on the dolomite ridge in such a way that the downstream dam toe rests against on the crest of the barrier. The exposed location required that the structure be carefully fitted into the terrain, with an undulating axis and variable height.

The dam is situated in a seismically inactive area. The design of the dam was based on a horizontal acceleration of 0.04 g.

Figure 1 Cross section through the dam and the rock barrier



3 MAIN FEATURES

3.1 Dam

The gravity dam has an average downstream slope of 1 to 0.68 and an upstream slope of 1 to 0.05. The crest of the dam has a width of 4 m. The low height of the dam permitted a degree of freedom with regard to the theoretical triangle and is a source of significant reserves concerning the stability of the dam. The exposed location of the dam on the natural rock barrier and the karstic character of the foundation rock explain the favourable uplift conditions.

The dam body is constructed of concrete with coarse with aggregates and low cement content viz. 179 kg of type 225 Portland cement per m^3 and a water/cement ratio of 0.56. The aggregate was obtained from a quarry and an alluvial deposit and fractioned into 3 grain sizes of 0 to 30 mm and stones of 150 to 300 mm. The stones were added using a tower crane and vibrated into the fine-aggregate concrete. The ratio fine-aggregate concrete/stones is about 1:1. The dam was constructed in 30 blocks, the

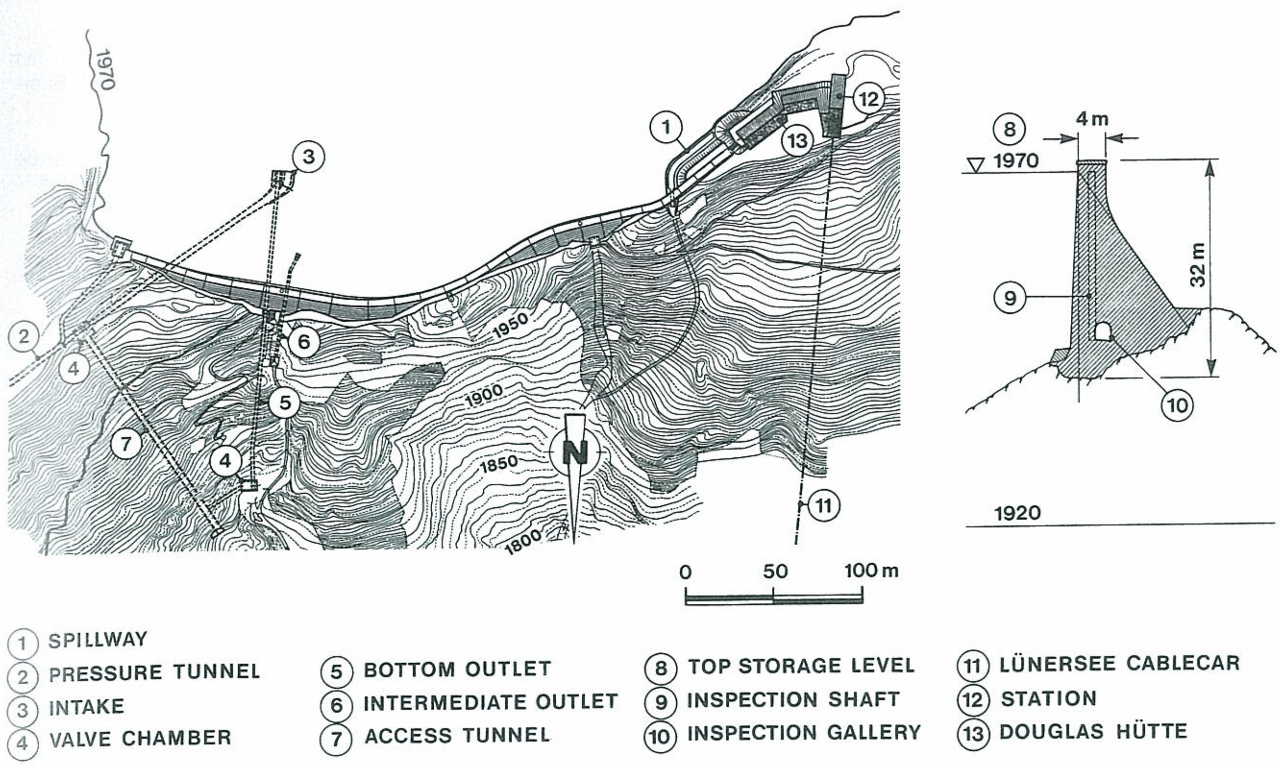


Figure 2 Plan and cross section

lengths of the blocks varying from 9 to 21 m. The lifts were 1.5 m. The average compressive strength of the fine-aggregate concrete was 45.3 N/mm².

The contraction joints were sealed with z-shaped copper sheet. Then the cylindrical cavity was sealed with bitumen. Behind the copper sheet there is always a vertical inspection shaft.

The foundation area, i. e. the natural rock barrier, required extensive sealing measures. The fissures, crevasses and faults in the natural rock barrier had to be cleared and washed out, then sealed and closed with concrete plugs. Smaller fissures and crevasses had to be grouted with cement. Afterwards most of the upstream surface of the rock barrier was sealed with shotcrete. Contact grouting of the dam base was performed from the inspection gallery.

3.2 Relief works and intake structure

The former outlet tunnel constructed in 1925 in order to draw down the storage level of the natural lake has been equipped with a butterfly valve and a annular valve and serves as a bottom outlet. An intermediate outlet was constructed in a tunnel situated between the bottom outlet and top storage level. This tunnel is equipped with a shut-off unit and a discharge valve. Flood relief is provided in the form of a spillway in the left dam abutment with a fixed overflow crest which empties through an opening in the dam body.

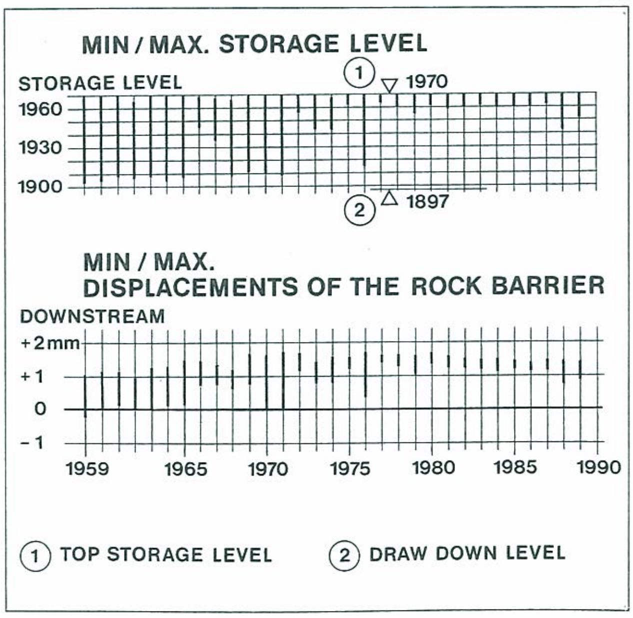
The intake structure is situated immediately above the bottom outlet, the two forming one combined structure. It is equipped with coarse and fine racks. After 100 m there is a valve chamber in the rock with 2 butterfly valves. With its total head of 950 m and a length of 9.8 km the power tunnel was a further engineering achievement at what was then Europe's largest annual storage system.

4 EXPERIENCES

4.1 Dam monitoring

The two highest blocks of the dam are equipped with a pendulum each. In addition the dam foundation in the area of the deepest cut in the natural rock barrier and at the highest point of the barrier is surveyed by a pendulum and by an invert pendulum respectively. The movements of the latter are recorded automatically and teletransmitted. The long-term behavior of the highest point in the dam base is shown in Fig. 3. The dam also incorporates

Figure 3 Long term displacements of the rock barrier



joint gauges and two seepage measuring devices.

Once a year the dam is submitted to geodetic checks. Levelling is monitored from different points in the inspection gallery on the downstream dam toe and on the dam crest. The crest alignment is also monitored.

On the downstream side of the rock barrier a large number of small springs are intercepted in a water measuring structure.

4.2 Events

After the first complete filling new springs were detected, especially in the larger western part of the rock barrier (anticlinal axis). A sophisticated investigation programme, - including the use of colouring agents, enabled the exact points of water ingress to be located. In 1960 these points were sealed with shotcrete. In the area of the western anticlinal axis a deep grout curtain was provided.

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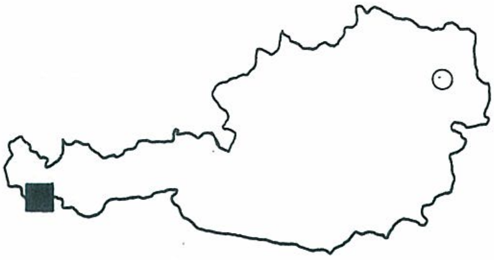
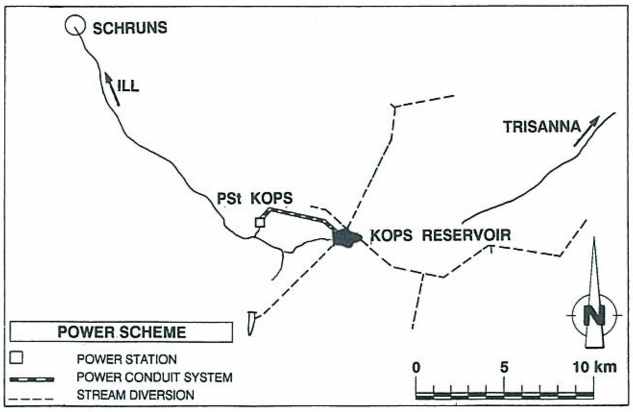
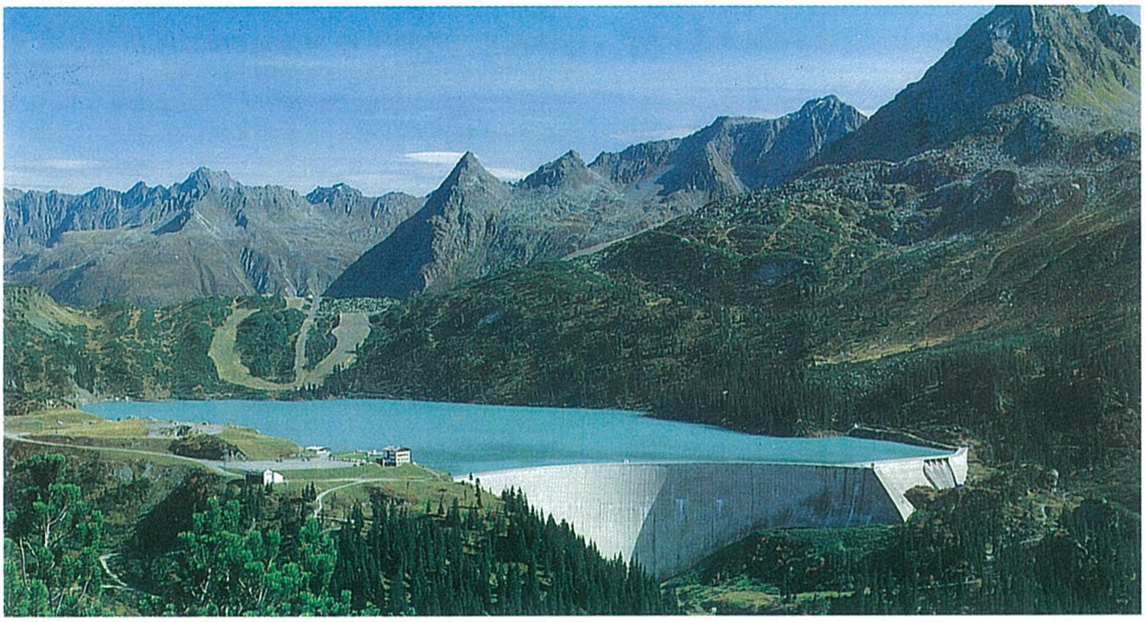
KOPS ARCH AND GRAVITY DAM

Vorarlberg; Vallülabach, Jll, Rhine
Nearest town: Schruns







MAIN TECHNICAL DATA, Chapter K, 38 (3/16)

General

Development	Obere Jll – Lünersee
Power Station	Kopswerk - Partenen
Construction Period	1962 – 1969
Gross Head	780 m
Installed Capacity	247 MW
Mean Annual Generation	392 GWh
of which in winter	134 GWh

Dam

	Main Dam	Lateral Dam + Art. Abutment
Maximum Height above Foundation	122 m	43 m
Crest Length	400 m	214 m
Thickness at the Crest	6 m	6 m
Maximum Thickness at the Base	30 m	33 m
Volume: Excavation (overburden, rock)	219 000 m³	109 000 m³
Concrete	485 000 m³	178 000 m³

Reservoir

Catchment Area: Natural	7 km²
Inflow	10 hm³
Diversions	163 km²
Inflow	223 hm³
Normal Top Water Level (a.s.l.)	1 809 m
Minimum Operating Level (a.s.l.)	1 730 m
Gross Capacity	45 hm³
Live Storage	44 hm³
Area flooded by full Reservoir	1.0 km²

Appurtenant Works

Spillway, ungated, l = 30 m	
Capacity	41 m³/s
Bottom Outlet, 2 sluice gates	
Capacity	21 + 41 = 62 m³/s
Power Intake	
Capacity	37.5 m³/s

1 GENERAL

The 122 m high Kops arch dam was constructed to form the power scheme's third annual storage reservoir for the Vorarlberger Jllwerke AG. With a relatively small natural catchment area, water from the diversions from the Tyrol – from the catchment areas of the Inn and Danube to the catchment area of the Rhine – is stored and exploited in the Kops power station, which was constructed at the same time as the reservoir. The power station forms part of the Obere Jll - Lünersee power scheme. The stored water is also utilized in the two Rodund power stations and in the Walgau power plant.

The dam extends over the main valley and a lateral valley. After thorough investigations into various alternatives, an arch dam with an artificial concrete abutment on the rock and an adjoining gravity dam in the shallow secondary valley were constructed.

At the time of construction, the dam was the highest arch dam in Austria. The reservoir created helped to raise the winter proportion of annual energy produced by the Vorarlberger Jllwerke from 37% to 46%.

The dam was constructed in 1961–1965. After partial fillings in 1965 to El. 1 765 and in 1966 to El. 1 800, the first complete filling was reached in November 1967.

2 GEOLOGY

The dam site is embedded in the crystalline formations of the Silvretta nappe. The Kops basin, polished by glacial erosion, is closed by a natural barrier, which slopes away steeply about 200 m downstream of the dam site. The dam site is mainly composed of aplitic gneisses and amphibolites interbedded with minor thicknesses of quartzites, mica schists and foliated gneisses. In situ rock was exposed in many places, and only covered by a 10 to 20 m deep overburden at the valley floor and lower levels of the right-hand flank.

A zone of geological weakness in the valley floor required local widening of the dam foundation area, as well as special sealing measures at the upstream dam toe. Up to now the site of the dam has never been the epicenter of a severe earthquake, but it has been proved that in the past there have been minor movements due to earthquakes that had their origins in neighbouring Switzerland, especially in the area of the Lower Engadine and Chur.

From the results of seismic evaluations, the Zentralanstalt für Meteorologie und Geodynamik determined a design earthquake of the intensity of 6.5° MSK and a destructive earthquake of the intensity of 7.3° MSK. Peak acceleration is of the order of 0.04 g and 0.10 g, while vertical acceleration is of one third lower.

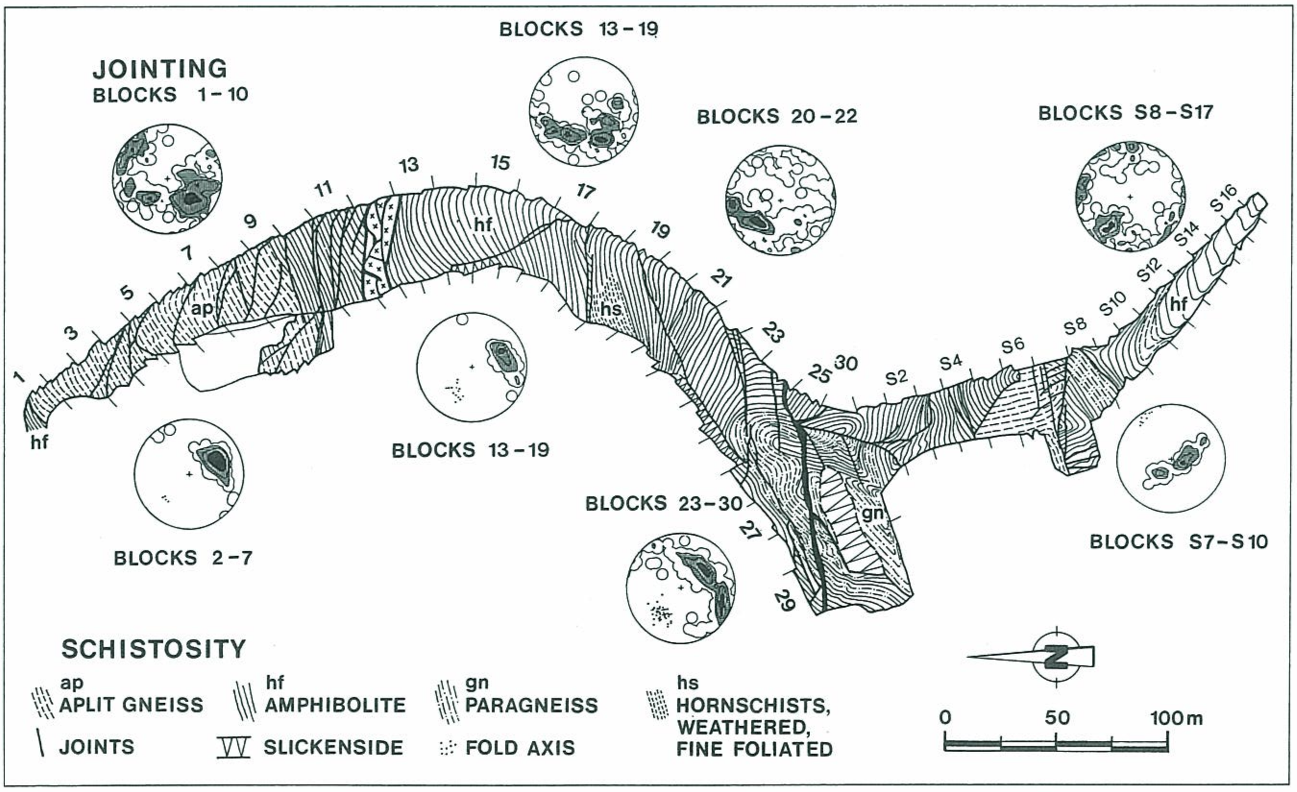
3 DAM

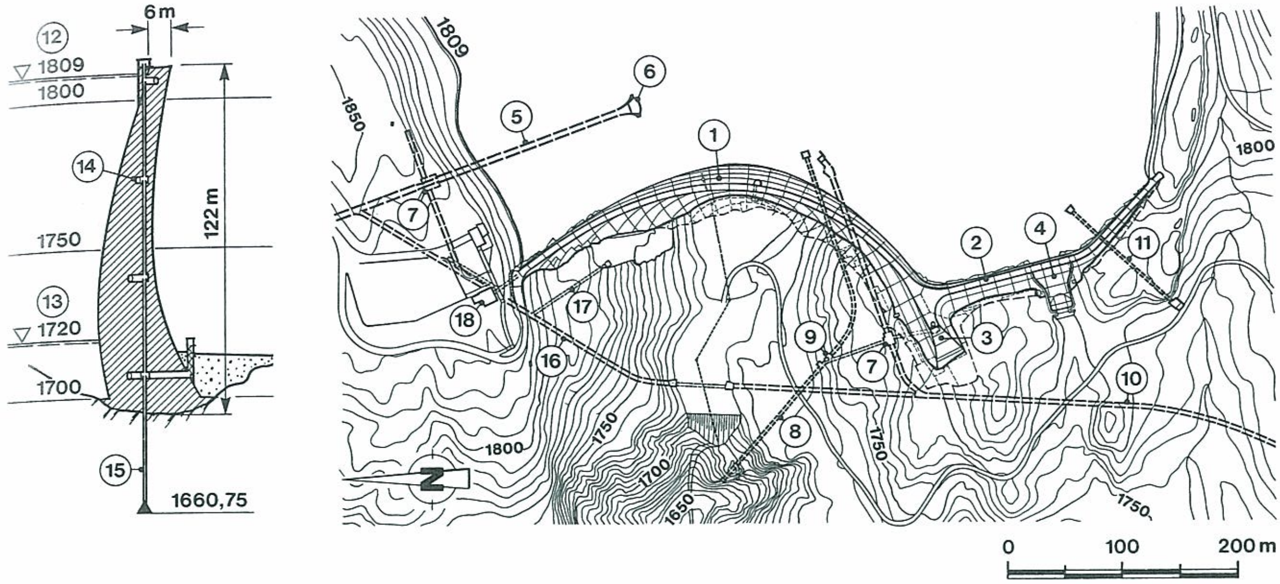
3.1 Main features

Stability analysis for the arch dam was performed using a load distribution method for 5 cantilevers and 8 arch elements. Four principal load assumptions were analysed and radial and tangential adjustments made. Later, a further analysis was made to determine the influence of a torsional adjustment. The maximum compressive stress was assumed to be 6.2 N/mm^2 and the maximum tensile stress 1.3 N/mm^2.

The results were checked by model tests at a scale of 1

Figure 1 Geological map





(1) MAIN DAM
(2) LATERAL DAM
(3) ARTIFICIAL ABUTMENT
(4) SPILLWAY
(5) PRESSURE TUNNEL
(6) INTAKE
(7) VALVE CHAMBER
(8) BOTTOM OUTLET
(9) BOTTOM VALVE
(10) POWER TUNNEL TO VERMUNT
(11) INTERMEDIATE OUTLET
(12) TOP STORAGE LEVEL
(13) DRAW DOWN LEVEL
(14) INSPECTION GALLERY
(15) PENDULUM
(16) ACCESS TUNNEL
(17) INSPECTION TUNNEL
(18) MONITORING CENTER

Figure 2 Plan and cross section

to 500 in the laboratory of the Tauernkraftwerke AG at Kaprun.

The gravity dam has an average downstream slope of 1 to 0.68 and an upstream slope of 1 to 0.05. In order to achieve maximum stability with the above mentioned design specifications, it is necessary to keep uplift pressure as low as possible.

Aggregate was obtained from alluvial deposits in the Kleinvermunt valley (crystalline rock), screened into 6 fractions with a maximum grain size of 150 mm. The aggregates then had to be hauled 8 km by road to the concrete plant at the dam site.

Concrete was produced in three different qualities using low hydration-heat type PZ 275 Portland cement and adding an airentraining agent:

- Facing concrete with 230 kg of PZ 275 Portland cement per m^3, water/cement ratio 0.49; average strengths after 90 days 36.6 N/mm^2 compression and 3.8 N/mm^2 bending tension.
- Hearting concrete for the arch dam with 180 kg of Portland cement per m^3, water/cement ratio 0.60; average strengths after 90 days 30.2 N/mm^2 compression and 3.4 N/mm^2 bending tension.
- Hearting concrete for the side dam and abutment with 150 kg of Portland cement per m^3, water/cement ratio 0.71; average strengths after 90 days 24.9 N/mm^2 compression and 3.2 N/mm^2 bending tension.

The concrete was placed in 16 m long blocks and 3 m lifts. The facing concrete is 1.5 m thick. Artificial cooling was achieved with a system of 22 mm cooling pipes spaced at 2 m intervals. The maximum concrete temperature ever reached was 32 °C.

The contraction joints of the arch dam were designed as helical surfaces with buckled keying. Grouting of the block joints was performed in sections at a joint closure temperature of 3 to 4 °C.

The foundation was treated with a single-line vertical grout curtain extending to a maximum depth of 60 m and covering a surface of 63 000 m^2. The grout consisted of colloidal cement emulsion and was injected under pressures of up to 40 atmospheres. Drilling and grouting were performed from the foundation area before the concrete was placed. Merely the upper 10 m were grouted together with the contact injections from the lower inspection gallery.

Beneath the gravity dam, additional grouting with acetate gel was necessary to plug seepage paths. The zones of geological weakness in the valley floor, where the main dam is placed, have been covered upstream of the dam by a layer of hydrated clay 1.5 m thick.

3.2 Relief works and intake structure

The dam has a bottom outlet and an intermediate outlet, both equipped with two gates. Both intake structures are equipped with a rack.

At the deepest point of the reservoir a 300 mm stainless steel outlet pipe for complete discharge has been installed with 2 gates.

The flood relief works consist of an overflow spillway located in the zone of the highest dam blocks of the side dam.

The power intake is situated in the area of the right-hand dam flank and is equipped with a coarse and a fine rack in precast concrete elements. A valve chamber situated within the rock is fitted two butterfly valves.

4 EXPERIENCES

4.1 Dam monitoring

The Kops dam was the first Austrian arch dam with separate monitoring of the dam and the foundation. Pendulums and invert pendulums have been installed at several points of the dam, in the rock foundation, in the right-hand rock abutment, in the area of the right dam crest, in the concrete abutment, and in the highest block of the gravity dam. In the right-hand dam abutment there are 3 triple extensometers plus one each in the left-hand dam abutment and beneath the concrete abutment. In addition, a large number of clinometers, thermometers and telepressmeters are integrated in the dam.

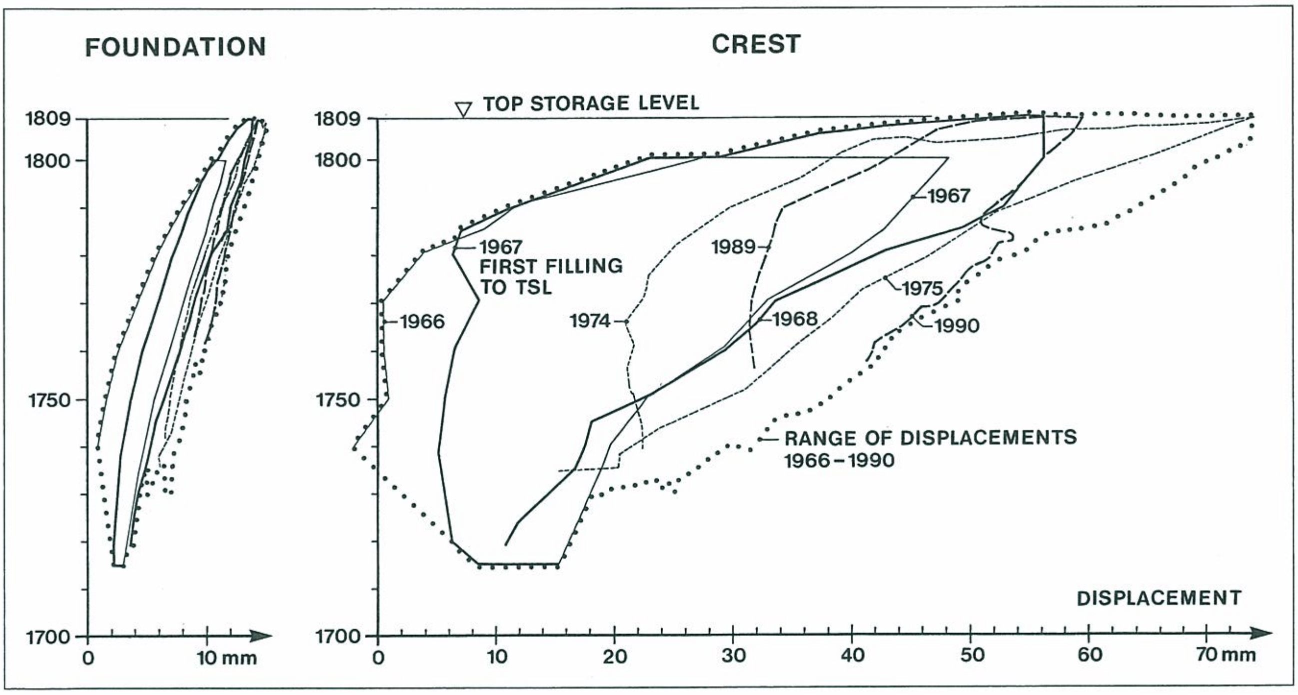
The readings from 3 pendulums, rock strain meters, thermometers and telepressmeters are teletransmitted to the control cabin. The pendulum in the highest dam block has an integrated safety contact pickup. If the prescribed limit values are exceeded, an alarm device in the monitoring centre of the Kops reservoir or the Vermunt power plant is actuated.

After 25 years of operation it can be said that dam foundation and dam body behaviour are regular and very constant relative to storage level and temperature and that plastic deformations have practically ceased. Deformations due to the storage levels, as indicated by the pendulum at measuring points near the foundation and the dam crest, are shown in Fig. 3.

In addition to continuous dam surveillance using the integrated monitoring systems, geodetic checks are also performed, once a year. Surveying is carried out of geodetic alignment of the dam crest and precision levelling of the dam crest, of the lowest gallery and of the downstream dam toe are surveyed.

For dam surveillance 2 statistical prognostic models have been elaborated with regression analysis and autoregression analysis in order to be able to forecast deformations of the dam and the foundation, and the values calculated are compared daily with measured values.

Figure 3 Crest displacements



4.2 Events

So far there have been no special incidents in the normal operation of the dam. But as only very few piezometers had initially been installed in the vicinity of the dam, additional piezometers were arranged in the right section of the dam foundation and in the zone of the highest dam blocks.

In 1990 the reservoir was completely emptied for the first time since the beginning of operation. This permitted a thorough inspection to be made of the intake structures and of the sealing of the bottom outlets and power intakes.

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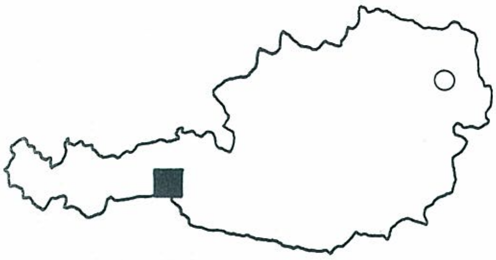
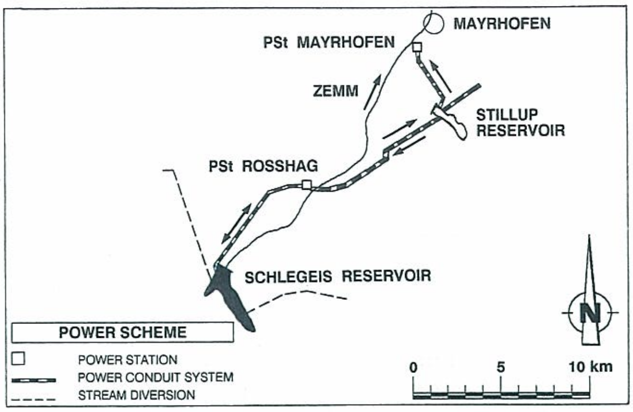
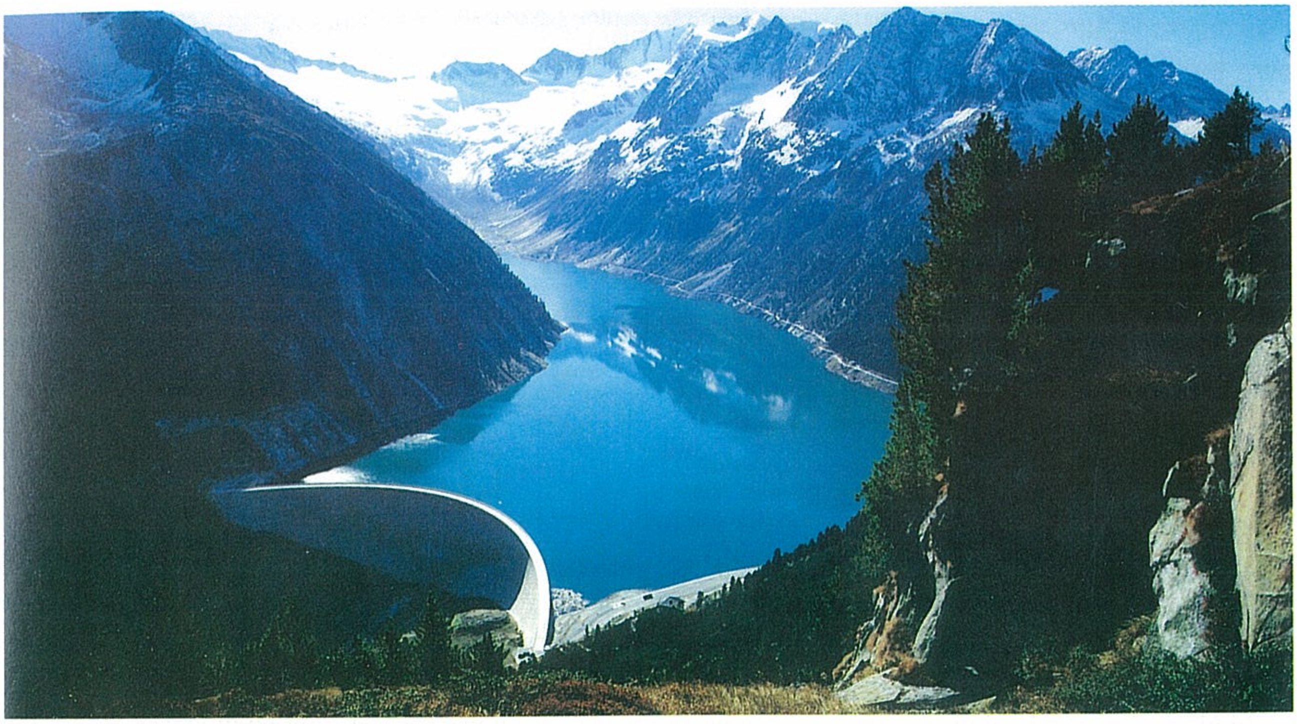
SCHLEGEIS ARCH DAM

Tyrol; Zemmbach, Ziller
Nearest town: Mayrhofen







MAIN TECHNICAL DATA, Chapter K, 43 (4/3)

General

Development	Zemm–Ziller	
Power Station	Mayrhofen	Rosshag
Construction Period	1965 – 1969	1967 – 1971
Gross Head	470 m	634 m
Installed Capacity: Turbine	345 MW	230 MW
Pump	–	230 MW
Mean Annual Generation	426 GWh	284 GWh
of which in winter	194 GWh	217 GWh

Reservoir

Catchment Area: Natural	58 km²
Inflow	97 hm³
Diversions	63 km²
Inflow	102 hm³
Normal Top Water Level (a.s.l.)	1 782 m
Minimum Operating Level (a.s.l.)	1 680 m
Gross Capacity	129 hm³
Live Storage	127 hm³
Area flooded by full Reservoir	2.2 km²

Dam

Maximum Height above Foundation	131 m
Crest Length (43 blocks)	725 m
Thickness at the Crest	9 m
Maximum Thickness at the Base	34 m
Volume: Excavation (overburden, rock)	285 000 m³
Concrete	960 000 m³

Appurtenant Works

Spillway, ungated	
Capacity	320 m³/s
Bottom Outlet	
Capacity	153 m³/s
Power Intake	
Capacity	52 m³/s

1 GENERAL

The reason for constructing the dam was to create the first big annual reservoir for Zemm-Ziller hydroelectric development.

The natural catchment area of the Schlegeisbach and Zamserbach at the dam site is increased by means of several diversions from 58 km² to a total area of approx. 121 km². In a first stage the storage water is utilized in the Rosshag power station (4 Francis turbines, 230 MW) and subsequently stored in the Stillupp (Eberlaste) weekly reservoir. From there the water is utilized in a second stage at the Mayrhofen power station (6 Pelton double turbines of 57.5 MW each). The entire power scheme has been designed exclusively for producing peak-load energy for grid regulation. A positive side effect is the improvement of flood protection throughout the entire Ziller Valley.

2 GEOLOGY

The dam is situated in the western part of the Austrian Central Alps, which in that area consist of the deepest central gneiss cores of the Pennine Tauern window arched in the shape of a dome. The dam is embedded in granitic mica gneiss of almost vertical schistosity and in parallel biotite-schist layers of varying thickness. The eastern flank of the dam runs almost parallel to the schistosity, whereas the central part and the western flank are transverse to the schistosity. The wide glacial valley is covered by a thin layer of talus material. Joints parallel to the slope surface are superficial. Because of these joints, an elastic cut-off was incorporated in the central portion of the dam.

Figure 1 Geological map

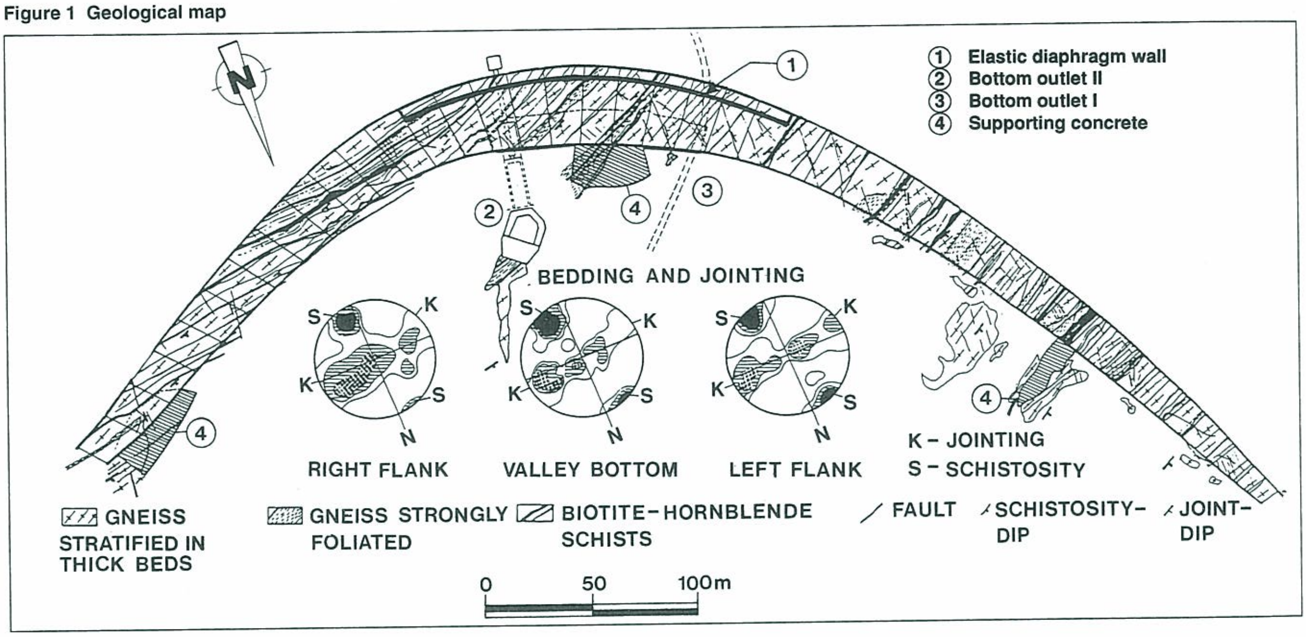
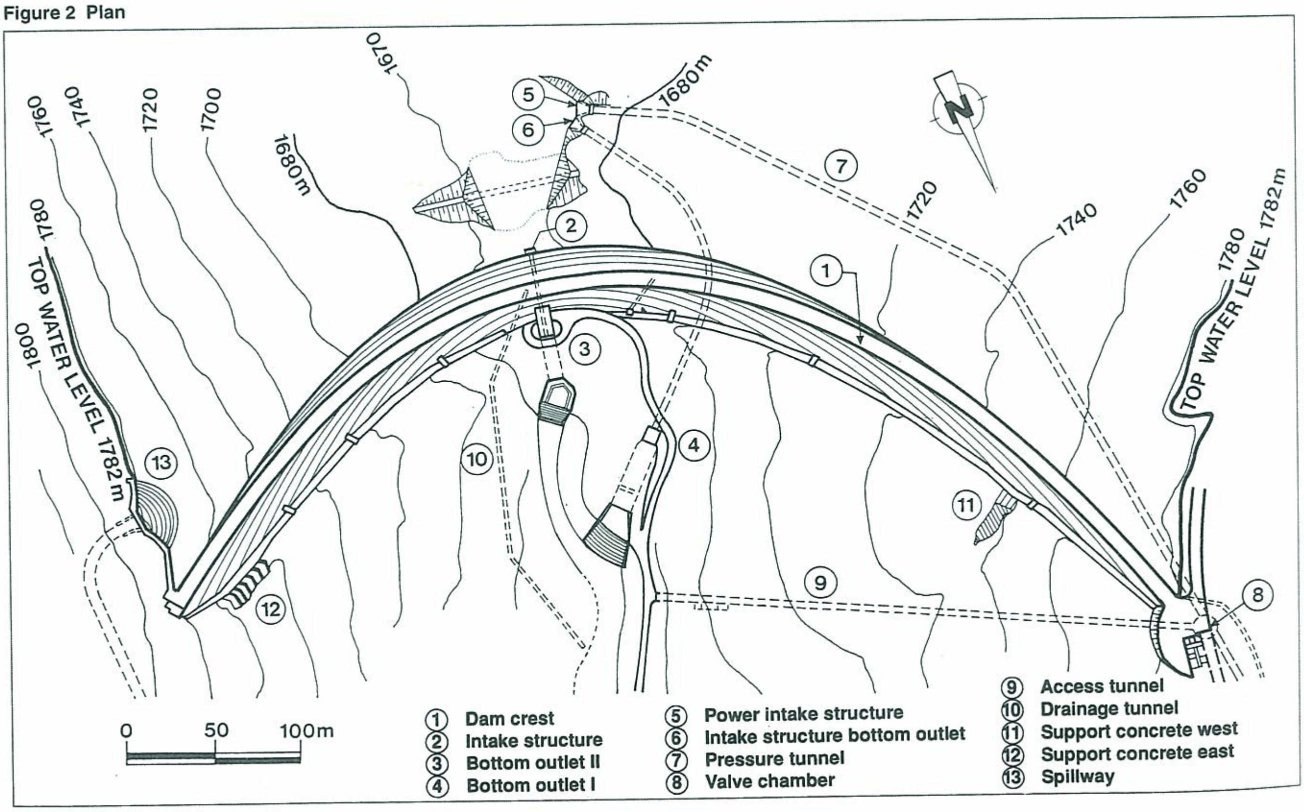


Figure 2 Plan



Seismic conditions: The dam lies at a linear distance of approx. 30 km SE of the seismically active Inn valley in an area of low seismic intensity with a slightly rising tendency.

3 DAM

The favourable site geology permitted the construction of a double-curvature arch-gravity dam with the unusual crest/height ratio of 5.5. The horizontal sections are elliptical, which permitted optimum fitting into the unsymmetrical valley configuration. The stresses the full reservoir loading condition were calculated at 5.5 N/mm² compression and 1.0 N/mm² tension at the foundation rock.

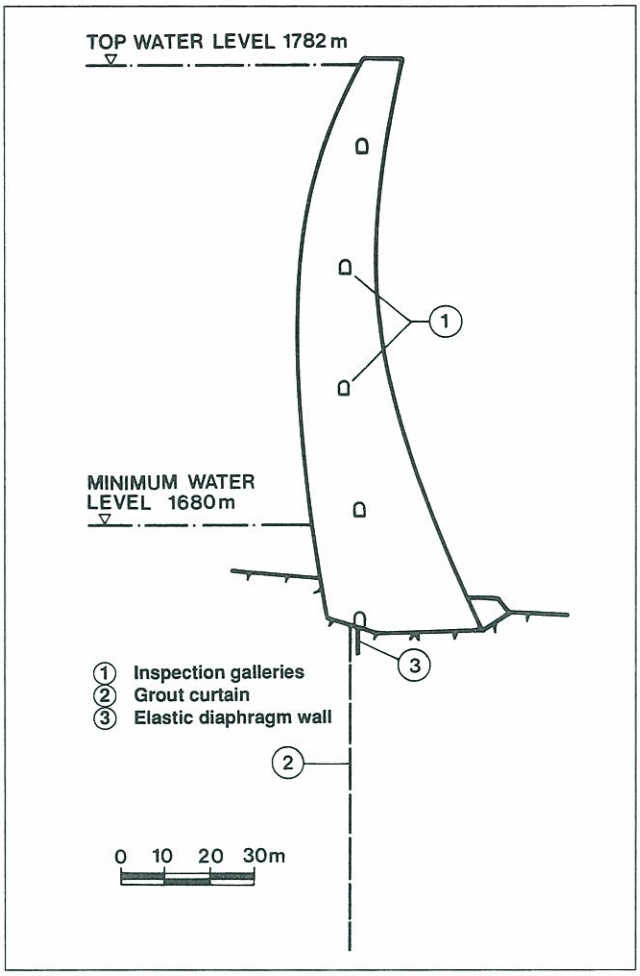
The dam consists of 43 blocks, each 17 m wide. The blocks have plain vertical joints. The dam is provided with 4 horizontal inspection galleries and a base gallery located on the foundation rock.

The dam has two bottom outlets. Bottom outlet I is located at the orographically left-hand flank and served as a diversion tunnel during construction. The steel-lined bottom outlet II runs through the centre block of the dam and serves to empty dead storage. The spillway is located on the orographically right-hand flank. It consists of an elliptical spillway with ungated crest at top storage level and an approx. 240 m long diversion tunnel.

The upstream grout curtain originally consisted of a vertical main curtain beneath the dam base extending to a depth of 50 m, and a shallower secondary grout curtain extending from the base gallery and inclined towards the upstream.

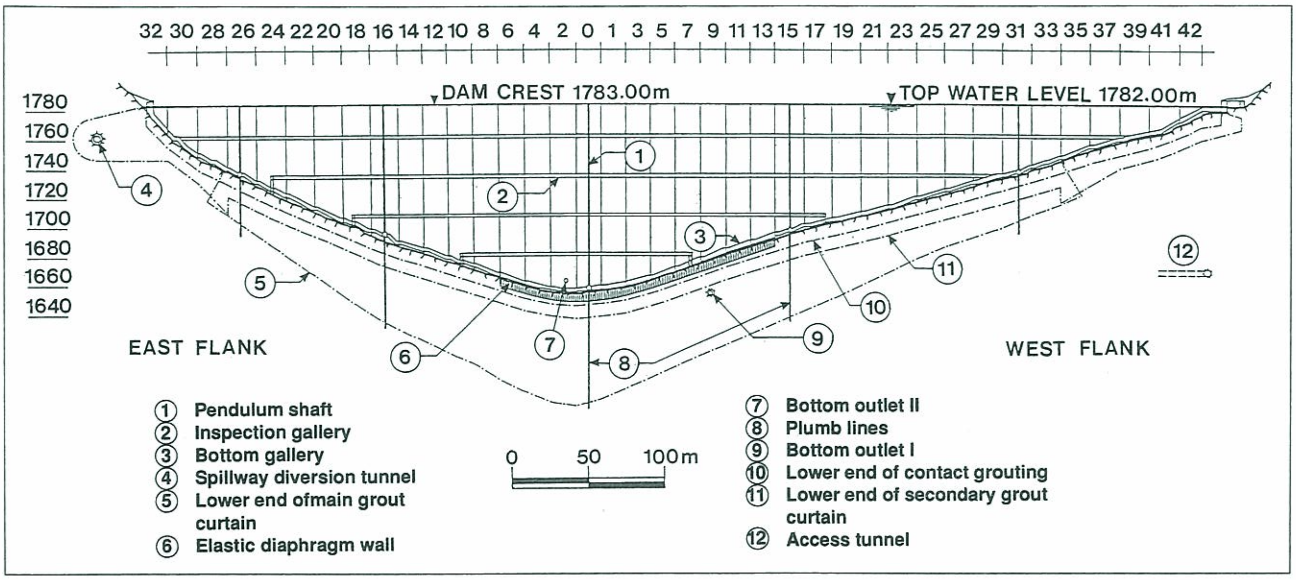
The hearting concrete has a maximum grain size of 120 mm and 160 to 175 kg of special cement per m³ at a water/cement ratio of 0.74 to 0.68. The facing concrete contains 230 to 250 kg of special cement per m³ and has a water/cement ratio of 0.51 and an air content of 3 to 4%. The binder is a special cement with 40 to 55% interground blast-furnace slag and develops a heat of hydration of approx. 60 cal/g. Mean concrete strengths at 180 days were 32.3 N/mm² compression and 5.6 N/mm² bending tension for the facing concrete, and 22.4 N/mm² compresssion and 4.7 N/mm² bending tension for the hearting concrete. The concrete was placed in 2.45 m lifts.

Figure 3 Central cross section



Because of the large span in relation to its height, the dam is monitored with a wide range instrumentation. The most important surveillance instruments are 5 shafts with direct and inverted pendulums extending to a depth of 80 m

Figure 4 Longitudinal section



below the dam base, more than 70 extensometers located at 7 vertical sections of the foundation, 50 uplift pressure gauges.

4 EXPERIENCES

When top storage level was first reached in 1973, seepage rates of 200 l/s were measured in the middle part of the base gallery. They are believed to have been due to cracking in the foundation rock caused by tensile stresses. To cut off this seepage water, extensive tests and experiments were carried out, until eventually an elastic diaphragm wall was constructed, imbedded in a base vault in the base gallery extending 5 m into the bedrock (Fig. 5). Between 1980 and 1983 the diaphragm wall was built in a total of 11 blocks. Since then the seepage rate at top storage level has not exceeded approx. 25 l/s.

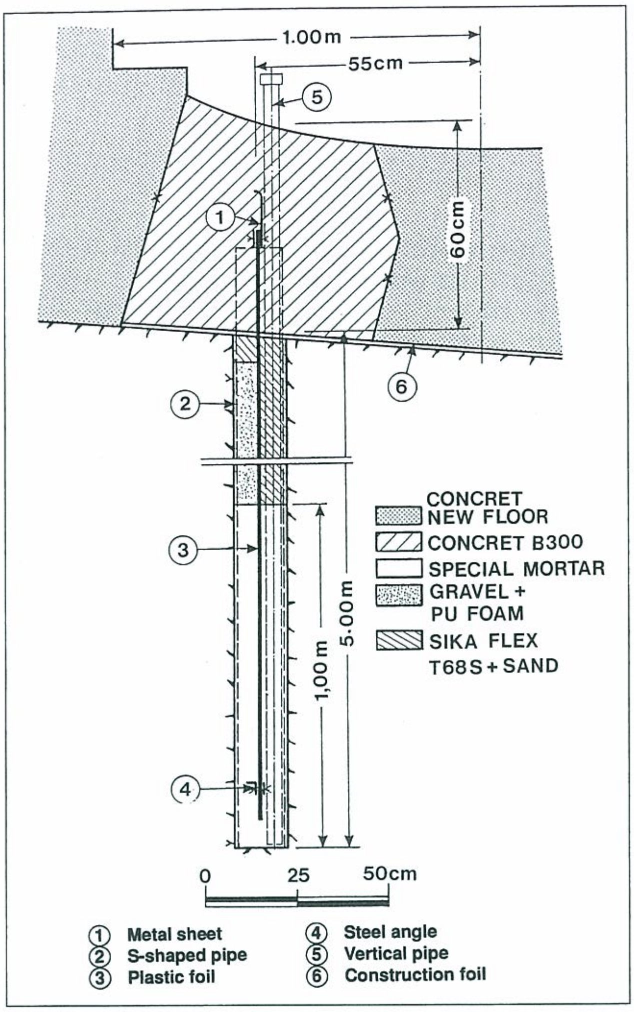


Figure 5 Foundation sealing

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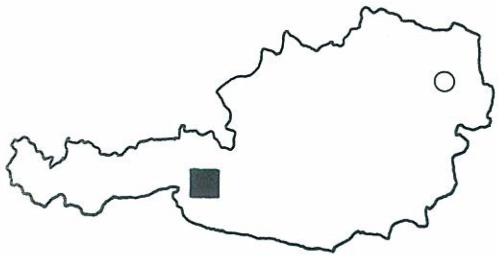
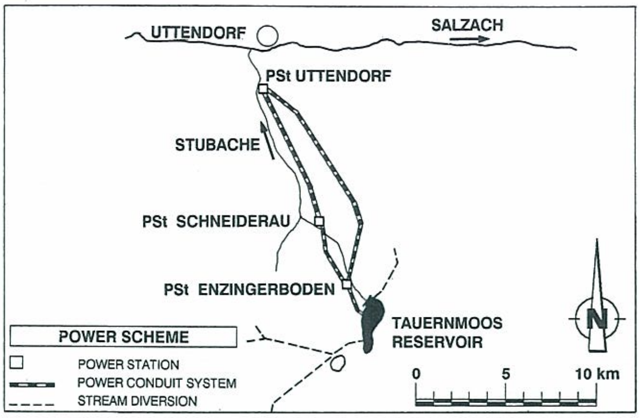
TAUERNMOOS GRAVITY DAM

Salzburg; Stubache, Salzach
Nearest town: Uttendorf





MAIN TECHNICAL DATA, Chapter K, 46 (4/10)

General

Development	Stubach			
Power Station	Enzinger-boden	Schnei-derau	Utten-dorf I	Utten-dorf II
Construction Period	1926–29	1937–40	1940–50	1988–90
Gross Head	540 m	421 m	238 m	659 m
Installed Capacity	81 MW	35 MW	27 MW	66 MW
Mean Annual Generation	120 GWh	53 GWh	41.5 GWh	111.5 GWh

Reservoir

Catchment Area: Natural	22	km²
Inflow	51	hm³
Diversions	28	km²
Inflow	61	hm³
Normal Top Water Level (a.s.l.)	2 023.0	m
Minimum Operating Level (a.s.l.)	1 984.5	m
Gross Capacity	57	hm³
Live Storage Capacity	55	hm³
Area flooded by full Reservoir	1.89	km²

Dam

Max. Height above Foundation	53	m
Crest Length (65 blocks)	1 100	m
Thickness at the Crest	4	m
Max. Thickness at the Base	37	m
Volume: Excavation (overburden, rock)	94 000	m³
Concrete	250 000	m³

Appurtenant Works

Spillway, ungated overflow spillway, l = 67 m		
Capacity	108	m³/s
Bottom Outlet, through the dam, 1.55/1.40 m dia., 2 valves		
Capacity	33	m³/s
Power Intake		
Capacity	18	m³/s

1 GENERAL

The Tauernmoos dam forms part of the Stubachtal development constructed in several stages from 1926 in order to supply the Austrian railway system with 50/3 Hertz power. Between 1926 and 1929 the Enzingerboden power station and the 28 m high Tauernmoos dam were constructed as a first stage. A second stage, comprising the Schneiderau power station and the 29 m high Enzingerboden gravity dam, was constructed immediately downstream and was finally followed by the construction of the Uttendorf I stage as the lowest station of the cascade. From 1950 to 1952, a 37 m high gravity dam was constructed to raise the water level of Weissee, a natural lake above Tauernmoos, with the aim of storing more of the summer runoff for utilization during the winter months. Then followed, between 1955 and 1958, the small Salzplattensee and Amersee gravity dams 16.5 m and 30 m high, respectively, for stream intakes.

To answer the Austrian railways' increasing demand for peak energy, a new Tauernmoos gravity dam 53 m in maximum height was finally constructed immediately below the old Tauernmoos dam in the years 1969 to 1973, so that power station operation was maintained during the whole construction period. Following partial filling to El. 2 021 m in 1973, top water level at El. 2 023 m was first reached in 1974. The Uttendorf II power station utilizes the head above Schneiderau and Uttendorf I in a single stage.

The Stubachtal scheme mainly utilizes the runoff from the northern slope of the Tauern mountains in the Pinzgau region in the province of Salzburg. In addition, there are three diversion systems to fill the reservoirs: a northern system with three intakes and 6 kilometres of tunnels entering the Weissee reservoir at El. 2 250 m; a southern system crossing the main ridge of the Tauern mountains at El. 2 050 m, with 6 water intakes and 10 kilometres of tunnels; and the upper Wurfbach diversion at El. 2 200 m, with 2 kilometres of tunnel. These diversions have increased the natural catchment of the Tauernmoos reservoir from 22 km^2 to 50 km^2 and the utilizable annual volume of water yield from 51 million m^3 to 112 million m^3; 78 million m^3 of the total runoff may be stored in all reservoirs during the water-rich summer months for utilization during the dry winter season.

2 GEOLOGY

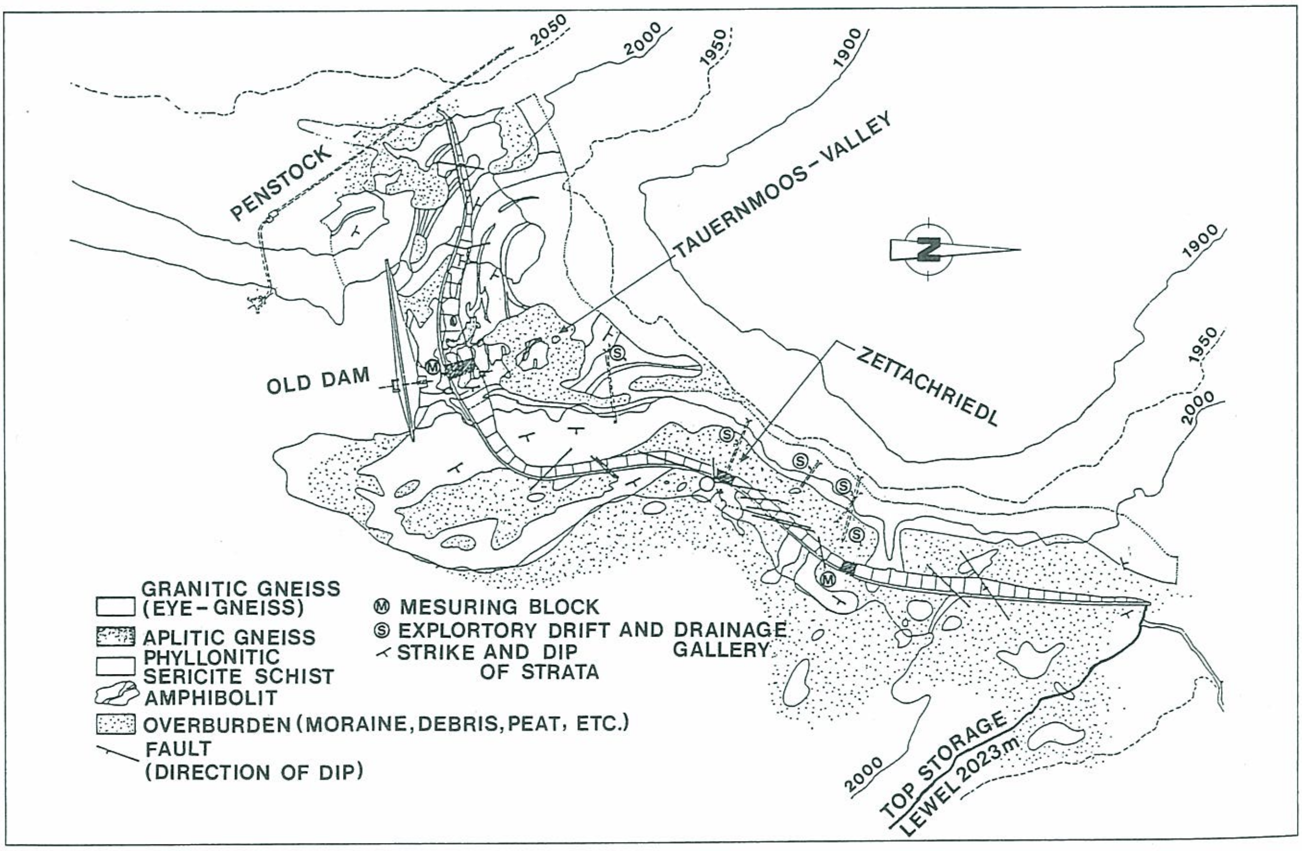
The dam site is located in the gneiss formations, which make up the long barrier closing the lake basin to the right, where they are covered by a shallow overburden of moraine material and debris. In the valley itself, gneiss strata (mainly eye-gneiss) alternate with plastic phyllonitic sericite schist (called Weiss-schiefer) which, although impervious, involve some risk of sliding. Slight arch action appeared desirable at this place to provide additional stability against sliding. The lower right-hand dam portion spans six open joints with chlorite-covered slickensides.

The earthquake-stability analysis was based on the assumption of a horizontal acceleration of 0.35 m/s (intensity VI/VII on the Mercalli-Sieberg scale). The instrumentation installed has not indicated any earthquakes so far.

3 DAM

In order to maintain power station operation through the

Figure 1 Plan and geological map

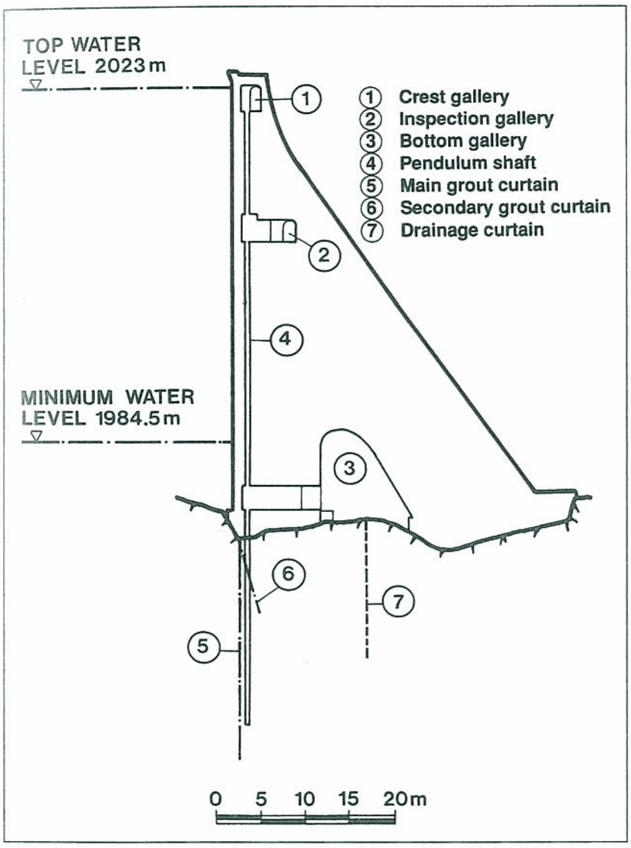


construction period, the new dam was built 25 m downstream of the old dam to enable the work to be carried out in a dry pit. A base gallery was provided for uplift pressure relief and to facilitate subsequent inspection and foundation treatment. Joints were grouted only in the main dam section spanning the valley to accomplish arch action. The right-hand dam section is undulating in alignment to follow the crest of the barrier across the outlet of the lake basin. The dam is located at a safe distance from the right-hand valley flank, where the ground surface slopes away steeply to the downstream.

The Tauernmoos dam was constructed by a joint venture between Stuag - Hamberger - Hofman & Maculan - Kunz & Co. The Elektrobauleitung Uttendorf department of Austrian Railways was responsible for the construction supervision.

The stability analysis was performed for a theoretical triangle with a vertical upstream face and with the downstream face sloping 1 to 0.70 over its upper portion and 1 to 0.74 over its lower portion. The uplift pressure assumption varied from 0.85 hydrostatic pressure at the upstream face to zero in the gallery. Maximum compressive stresses were calculated to be 1.2 N/mm²; the analysis gave no tensile stress at the foundation contact. A safety factor of 1.5 against sliding was obtained by providing a sloping foundation surface.

Figure 2 Central cross section



Aggregate was obtained from a gneiss quarry and separated into 6 fractions with 100 mm maximum size. Vibrated concrete was produced in a mixing tower with three 1 500-l capacity pugmill mixers, placed by means of two blondins with capacities of 6 t and 5 t, respectively, and two tower cranes, in blocks 12 to 8 m in length, and in 2.5 m lifts. The facing concrete varies between 2.5 and 2.0 m in thickness at the upstream face and is 1.5 m thick at the downstream face. Railway rails were used as reinforcement in the area above the slickenside joints in the rock barrier.

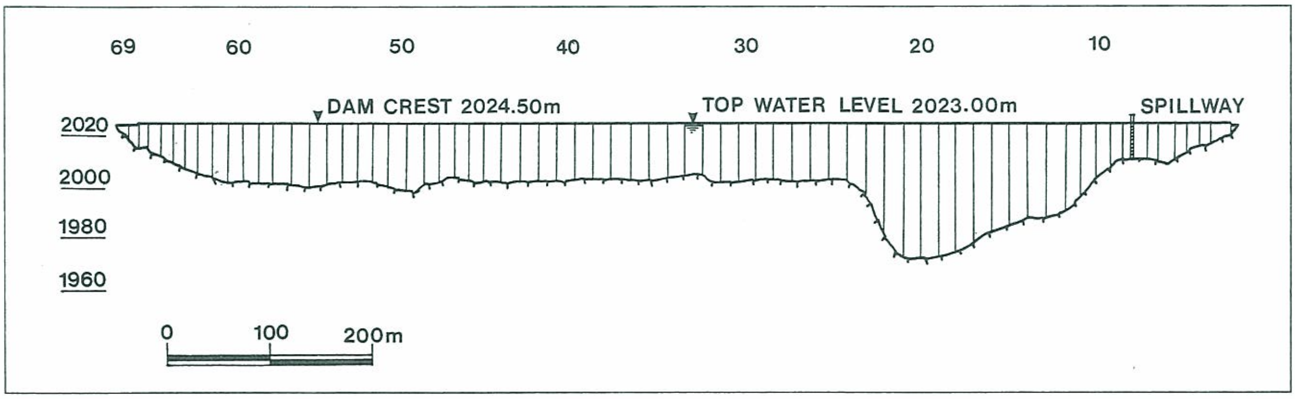
Contraction joint keying was obtained by means of buckled-plate formwork. Over the arched section of the dam, contraction joints were sealed with plastic waterstops to El. 2 005 m, upstream and downstream, and were grouted with type 275 Portland cement and 3% bentonite, under a maximum pressure of 5 bar.

Concrete composition and properties

Foundation treatment consisted of a single-line vertical grout curtain with a shallower secondary grout curtain further upstream extending 25 m deep beneath the main dam section and 115 m deep beneath the lateral section. The zone of slickenside joints was treated with a grout curtain 25 m deep and inclined at 30° to the upstream. Grout curtains cover a total area of 23 500 m² and comprise 7 700 m in total drill hole length. Grout was pumped in under pressures of up to 25 bar. Mean grout acceptance was 41 kg per linear metre of drill hole, or 14 kg per m² of grout curtain area. Mean grout acceptance in the slickenside joints was 32 kg/m², while remaining below 10 kg/m² in the other areas.

In order to ensure pore pressure relief, wells were sunk into the foundation rock from the base gallery, and into the rock barrier from lateral drainage galleries. Contact grouting was performed at the dam base.

Figure 3 Longitudinal section



The bottom outlet is located in the highest part of the dam (block 20). It is equipped with a shut-off valve located in the inspection gallery and a Howell-Bunger valve for flow control between 0 and 33 m^3/s.

Flood relief is afforded by an overflow spillway built into the west abutment of the dam. With a clear width of 67 m, it is capable of handling 108 m^3/s at 1 m surcharge and 0.5 freeboard to the dam crest.

Total discharge capacity is 140 m^3/s corresponding to 4.8 $m^3/s.km^2$ of natural catchment and areas directly connected by diversions (upper Wurfbach and part of the northern diversion). The original power intake, situated in the left valley flank above the dam, with the sill at El. 1 982 m, was retained. The intake is equipped with coarse and fine racks. A bent inlet tunnel continues as a 1.3 m dia. power tunnel capable of 18 m^3/s.

4 EXPERIENCES

4.1 Dam surveillance

Direct-reading instrumentation includes 3 inverted pendulums extending to a maximum depth of 35 m into the foundation rock, 30 tiltmeter stations, 114 joint metering stations, 11 extensometers, 53 thermometers, 15 teleformeters and 4 telepressmeters; 77 uplift pressure cells, and seepage flow measuring devices in the base gallery and in the drainage and exploration galleries.

Telemetering instrumentation includes 2 radial pendulum measurements, as well as water level and air temperature measurement.

Marginal checks include the movements of the vertical joints (three-dimensional at the crest), base gallery drainage, extensometers, pendulums (radial displacement only in the crest gallery).

Continuous monitoring of all the main joints at crest level allows for the first time in a gravity dam with open joints all the monoliths to be observed. Results show that the influence of water pressure is relatively small, while the influence of temperature changes is comparatively large.

Geodetic measurements consist of precision levelling over the entire length of the dam and over local sections.

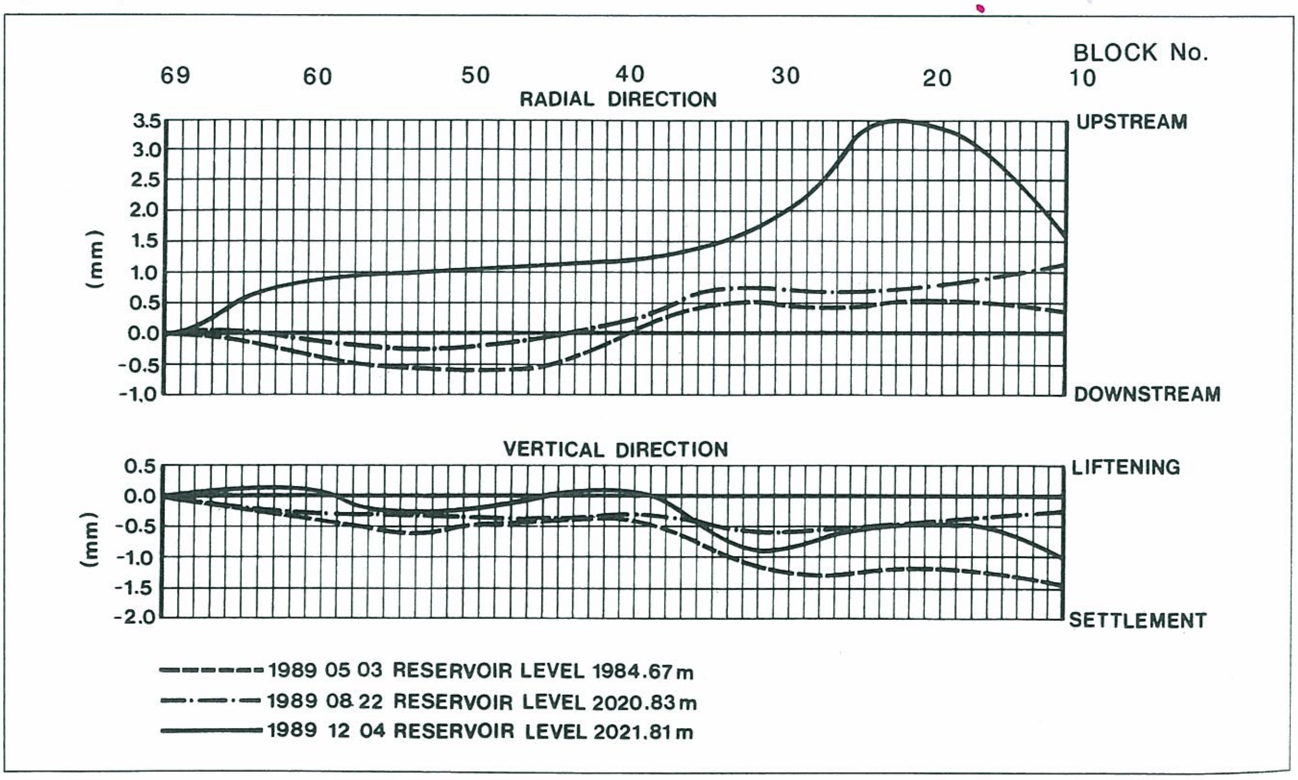
Periodic visual inspection is made of the state of rock and concrete, reservoir area, terrain downstream of the dam, etc.

Concrete properties were investigated 12 years after completion of the dam using comparable test cores. The results showed that the compressive strength of the facing concrete had increased by 73% and that of the hearting concrete by 141%, the tensile splitting strength of the facing concrete had increased by 91%, that of the hearting concrete by 53% as compared with the strength at 90 days; and the modulus of deformation of the facing concrete had increased by 38% and that of the hearting concrete by 54% as compared with the values at 180 days.

4.2 Experiences

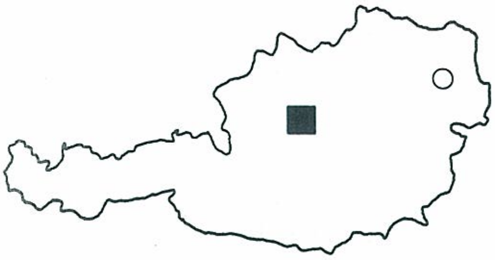
This type of dam surveillance, developed during the first years of the dam's existence, has proved to answer its intended purpose satisfactorily. Regular maintenance and performance checks of all instruments and appurtenant works ensures that technical safety requirements are met.

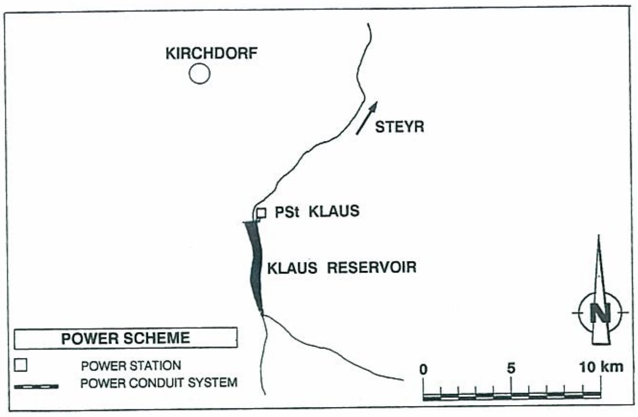
Figure 4 Displacements of the dam crest

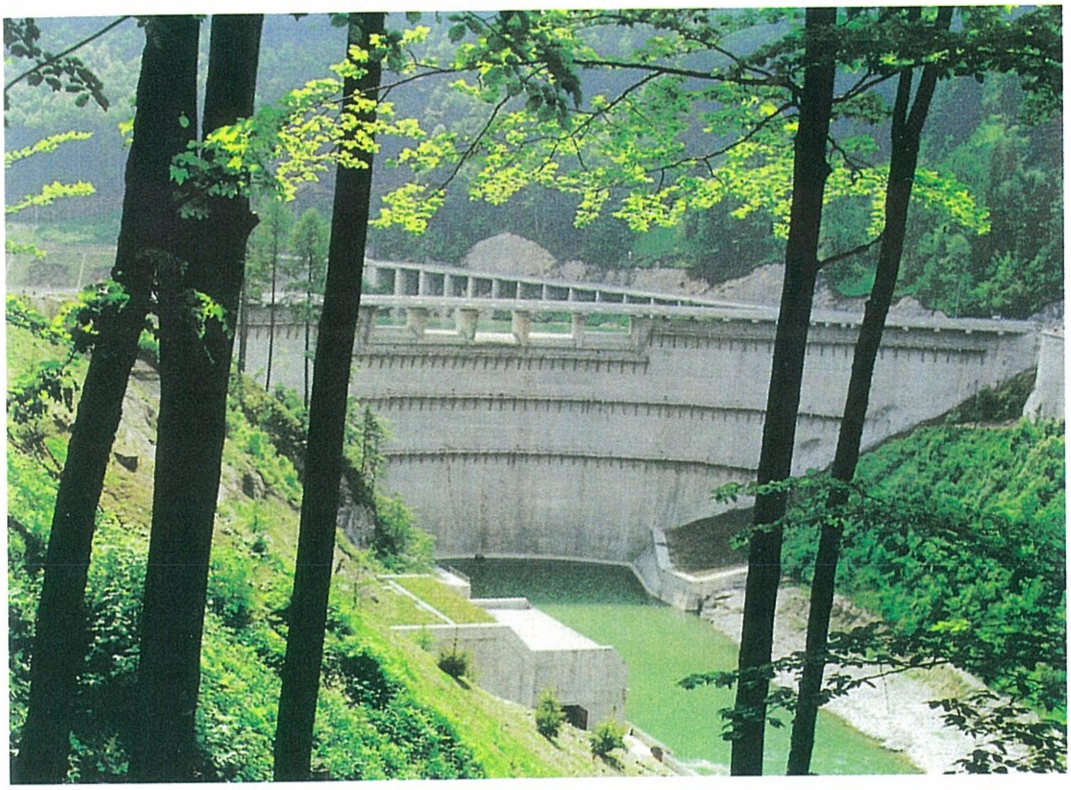


KLAUS ARCH DAM

Upper Austria; Steyr, Enns, Danube
Nearest town: Kirchdorf







MAIN TECHNICAL DATA, Chapter K, 50 (4/17)

General

Development	Molln
Power Station	Klaus
Construction Period	1973 – 1975
Gross Head	40 m
Installed Capacity	18 MW
Mean Annual Generation	72 GWh

Reservoir

Catchment Area: Natural	539 km²
Inflow	805 hm³
Normal Top Water Level (a.s.l.)	463 m
Minimum Operating Level (a.s.l.)	457 m
Gross Capacity	12.6 hm³
Flood Retention Capacity	7.8 hm³
Area flooded by full Reservoir	0.9 km²

Dam

Max. Height above Foundation	55 m
Crest Length	188 m
Thickness at the Crest	2.0 m
Max. Thickness at the Base	9.4 m
Volume: Excavation (overburden, rock)	14 500 m³
Concrete	39 000 m³

Appurtenant Works

Spillway, ungated overflow spillway	
Capacity	340 m³/s
Bottom Outlet	
a) Bypass, 6.0 m dia., tunnel l=260 m,	
Control gate 5.4 x 3.7 m	
Capacity	400 m³/s
b) Through the dam, 1.0 m dia. dam gate	
Capacity	18 m³/s
Power Intake	
Capacity	50 m³/s

1 GENERAL

The Klaus hydroelectric development was originally conceived as a first stage of the planned Molln project, which would have included a diversion of the river Steyr combined with a pumped storage facility in the river Enns and a large compensating basin. Realization of this multi-stage development was not possible for several reasons, but the Klaus unit of the development was designed to allow economical operation even as an isolated run-of-river station. Following a 2-year construction period, the power station was placed in operation in 1975.

Klaus was designed as a multi-purpose facility, its main purpose being the generation of energy as a run-of-river station. It is equipped with two power units of different capacities to allow for the substantial flow variations of the river Steyr. Had the multi-purpose project been realized, the smaller unit would have utilized the compensation flow. A 120 m long 4.0 m diameter power tunnel supplies a total flow of 50 m³/s to the power units.

2 GEOLOGY

The Klaus dam was constructed in a U-shaped glaciated valley with a fairly level bottom and medium-steep flanks. The structure is founded on dolomite (Hauptdolomit). The rock is thick-bedded especially in the right slope and shows a flat to medium-steep dip towards the reservoir basin. The bedding joints occasionally show shale fills varying between several centimetres and tens of centimetres in thickness. In the right-hand slope, which faces west, the shale fills have weathered to clay in near-surface zones. Several fractured zones, most of which were associated with steep joints, were not considered to have any major bearing on dam stability or on the mechanical properties of the rock. The valley does not follow a fault.

Exploratory drillings revealed a perched water table in the foundation. In order to preserve this, the depth of the grout curtain was limited to 15 m. In spite of the existence of a base gallery, leakage amounts to no more than an average of 0.2 l/s.

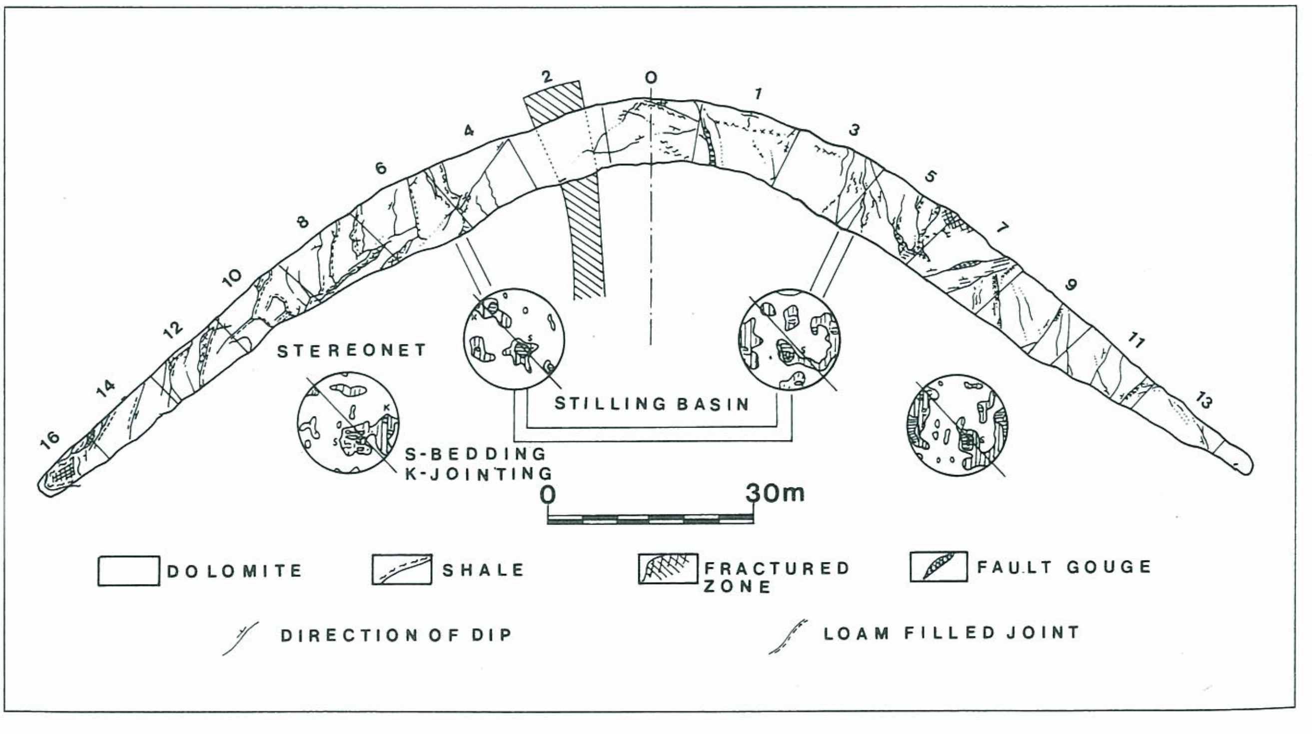
Seismicity in the Klaus area is below average even by the modest Austrian standards. To allow for earthquake effects on the dam, a horizontal acceleration assumption of 0.4 m/s², based on a report by ZAMAG, Vienna, was included in the design.

During first filling, a seismic station situated near the power scheme recorded an increase in micro seisms at an old well-known focus of "swarm" earthquakes. The earthquake richest in energy had a magnitude of 2.1, focal depths varied between 1 and 4 km. The impoundment had obviously released formerly latent earthquakes, but was not their actual cause.

3 DAM

At the selected valley cross section, compact bedrock showing no major faulting is exposed. This allowed the design of a thin double-curvature arch dam. Some gravel overburden had to be excavated on the right-hand slope.

Figure 1 Geological map



The dam abutments were embedded to a depth of no more than 2 or 3 m into the rock flanks. The lowest point of the dam foundation in the valley bottom is about 9 m below ground level. An about 5 m deep scour in the pre-glacial river bed had to be filled with a concrete plug.

The stability analysis was performed for 4 m surcharge level, using the load distribution method with 5 cantilever and 5 arch elements. Optimal conditions resulted for a dam combining three different conic-section arcs. Assumption of an uplift variation from 0.25 hydrostatic pressure to zero gave maximum concrete stresses of 3.9 N/mm² compression and 0.7 N/mm² bending tension.

The dam was poured in the period from July 1974 to April 1975 without interruption during the winter months. The vibrated concrete was made of 220 kg type 275 Portland cement per cubic metre, and with a mean water/cement ratio of 0.48. Cerinol AEAK was added as an air entraining agent. At 28 days, an average compressive strength of 35.5 N/mm² and a bending tensile strength of 6 N/mm were obtained. All samples showed good frost resistance.

Dolomite-aggregate was obtained from a borrow pit on the right-hand river bank and was separated into 5 fractions with 120 mm maximum size. Sand from external sources had to be added to the 0 to 1 mm fraction.

Concrete was produced in a punched-card controlled 1 500-l capacity mixer with a daily performance of 1 000 m², placed by means of 2 tower cranes, in 2.5 m lifts, and compacted by immersion vibrators. The contraction joints between the 12.5 m long blocks were sealed with waterstops upstream and downstream.

A single-line grout curtain with a total area of 3 200 m² was provided to 15 m below foundation surface against seepage suspected to occur in the dolomite. Mean grout acceptance was 15 kg per linear metre of drill hole, or 4 kg per square metre of grout curtain. Drainage holes beneath the dam base and in the rock downstream of the dam were provided to afford relief of potential uplift pressures downstream of the grout curtain.

Following joint grouting, first filling was commenced on April 24, 1975, and the reservoir full level was reached two months afterwards. Since that time, the reservoir surface has been maintained at this level, except for short-term variations for flood retention or partial drawdown for joint regrouting.

Flood control is effected by two facilities. Preliminary drawdown is through a 260 m long bottom outlet tunnel 6 m in diameter. A discharge structure directs the flow towards the natural river bed. Energy dissipation is by aeration and by a scour that has formed in the middle of the river channel. The 5-section ungated overflow spillway with a total length of 45 m ends in a stilling basin at the toe of the dam.

Figure 2 Plan

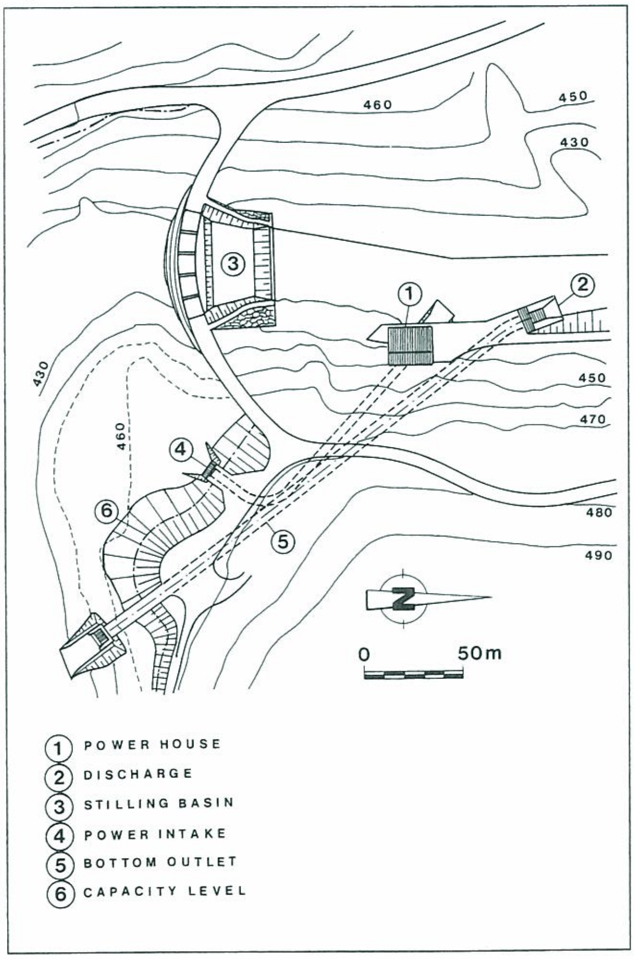
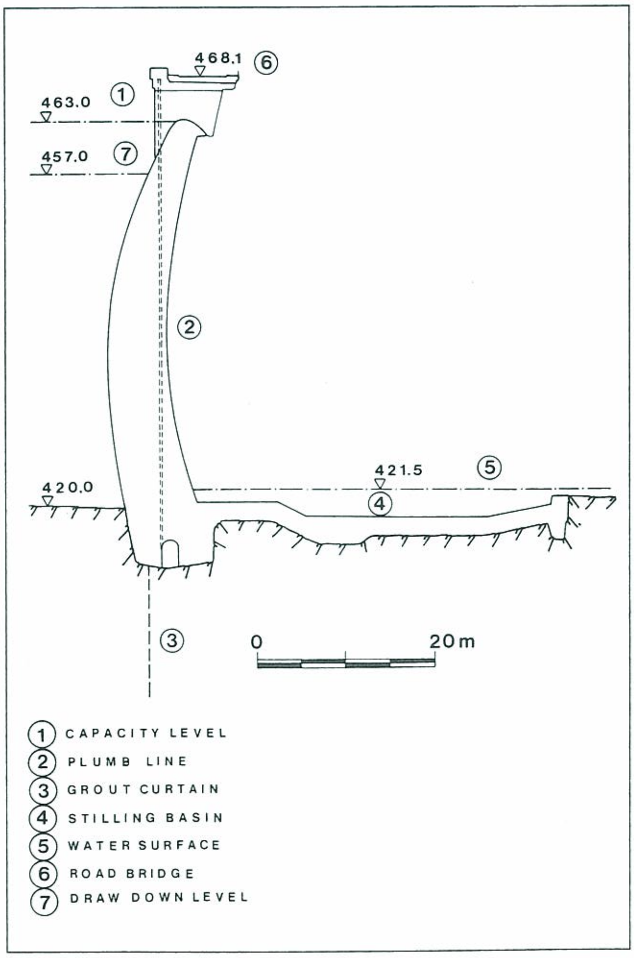


Figure 3 Central cross section



4 EXPERIENCES

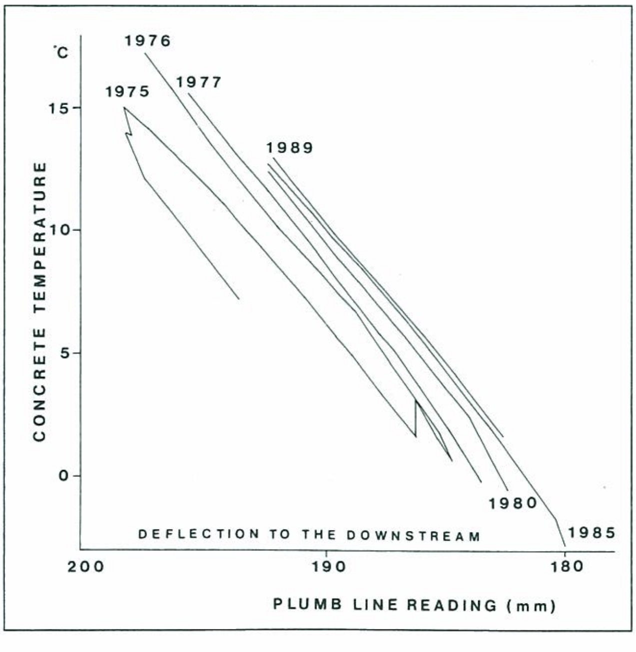
4.1 Dam monitoring, instrumentation

Abundant instrumentation was provided to monitor the dam structure. Due to the station being operated on a run-of-river basis with a constant headwater level, several effects on deformation behaviour can be established with great accuracy. By means of a great number of analyses, that instrument was selected among the great number of temperature gauges embedded in the dam concrete which responds with the same delay as the pendulum indication does. This gauge supplies a "representative concrete temperature" with which it is possible to correlate pendulum movements with an accuracy of less than 1 mm. A deviation of 1.5 mm already releases an alarm at the power station control centre. Deflection due to the water load is about 7 mm, movement resulting from temperature effects is about 17 mm p.a. An outstanding feature is the low coefficient of temperature expansion of the dam concrete, which has been calculated to be approximately 7 x 10^{-6}. Correlation between pendulum and concrete temperature with a constant reservoir level also reveals the run of irreversible deformation due to shrinkage and creep. Following an amount of 4 mm in the first year, this displacement has not exceeded 0.1 or 0.2 mm per year ever since. Total deformation so far amounts to 6 mm.

Leakage from the drainage holes amounts to 0.2 l/s with a full reservoir, measured in the inspection gallery at the base of the dam.

An instrument of particular interest is a pickup for vibration measurement installed at the dam crest to measure potential vibration caused by the spillway. The natural frequency of the dam was determined to be 6.5 Hz by testing during construction. Vibrations for an overflow depth of 2 m range around 10 µm double-amplitude, which is far below the limit of discernibleness. It is striking though that vibrations of the same magnitude occur when

Figure 4 Radial deflection of the crest



flood flow is being discharged through the bottom outlet. There is reason to assume that dam vibration is not caused by overflow at the crest but by energy dissipation in the stilling basin or at the bottom outlet.

4.2 Flood events

The second important purpose of the Klaus development is flood control for an about 40 km long reach downstream from the dam, ending at the town of Steyr, with its densely populated areas and a large number of structures and buildings located within the danger zone. A several-hour flood forecast is made on the basis of rainfall data from the catchment. Precautions are based on the respective 4-hour forecast. This allows the water level in the reservoir to be drawn down in time before the arrival of the flood wave. Abatement of the flood peak is ensured by the reservoir volume so obtained and an additional 3 m surcharge.

Forecast data are constantly determined by the power station computer and compared with inflows to the reservoir. The principal instrument data are directly transmitted from three rainfall stations and three inflow gauges. Designed to answer flood retention requirements, the overflow spillway and the bottom outlet are capable of handling 600 m^3/s and 400 m^3/s, respectively.

Flood peaks were reduced by 25 and 30 percent, respectively, during the two largest floods since the commissioning of the project, in 1975 and in 1977.

4.3 Tourist trade

The third feature of the multi-purpose project is the development of the reservoir lake and its banks of great scenic beauty as a recreation area for both the local population and holiday-makers. 7 km in length and holding 12,6 hm^3 of water, the reservoir lake is remarkable for its canyon-like character with its steep conglomerate shores. First landscape preservation studies were already conducted at the stage of design and the necessary structural measures were included in the project. For example, a gravel borrow pit was equipped as a marina and incorporated into a recreation centre, and a swimming pool and a camping site were created at the upstream end of the reservoir.

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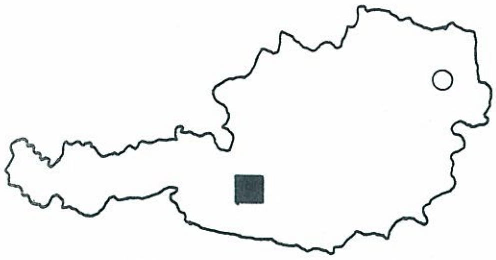
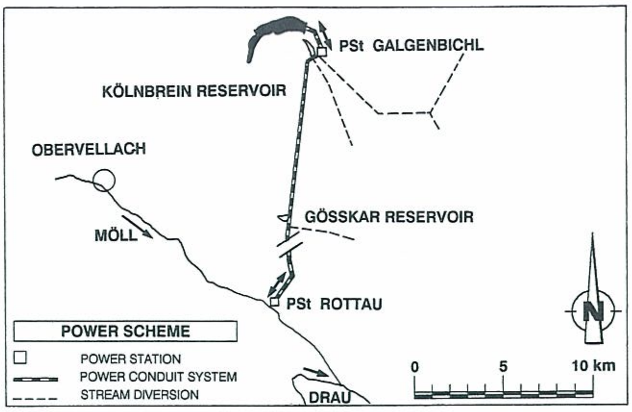
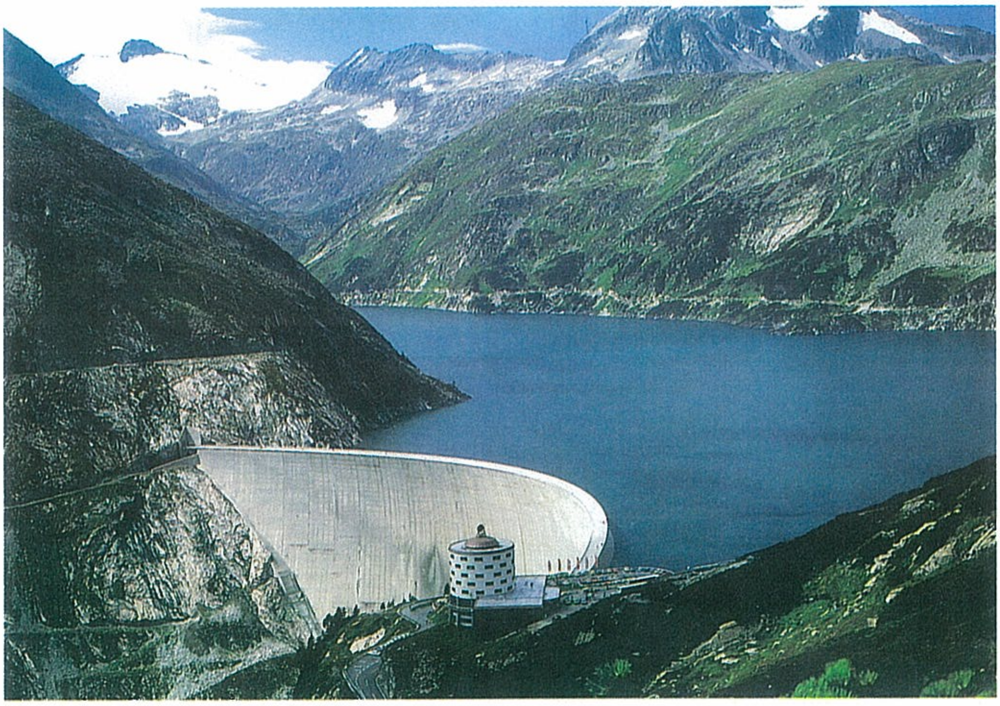
KÖLNBREIN ARCH DAM

Carinthia; Möll, Drau
Nearest town: Gmünd







MAIN TECHNICAL DATA, Chapter K, 54 /4/20

General

Power Station	Galgen-bichl	Rottau	Möll-brücke
Construction Period		1971 – 1979	
Gross Head	135 m	1 106 m	45 m
Installed Capacity			
Turbine	120 MW	730 MW	41 MW
Pump	116 MW	290 MW	–
Mean Annual Generation	76 GWh	715 GWh	114 GWh
of which in winter	71 GWh	590 GWh	52 GWh

Reservoir

Catchment Area:	Natural	51.3 km²	6.5 km²	1 081.3 km²
	Inflow	105.6 hm³	8.3 hm³	1 023.1 hm³
	Diversions	– km²	70.8 km²	– km²
	Inflow	116.0 hm³	121.0 hm³	– hm³
Normal Top Water Level (a.s.l.)				1 902 m
Minimum Operating Level (a.s.l.)				1 750 m
Gross Capacity				205 hm³
Live Storage				190 hm³
Area flooded by full Reservoir				3.5 km²

Dam

Maximum Height above Foundation	200 m
Crest Length	626 m
Thickness at the Crest	7.6 m
Maximum Thickness at the Base	41.0 m
Volume: Excavation (overburden, rock)	420 000 m³
Concrete	1 580 000 m³

Appurtenant Works

Spillway, uncontrolled overflow spillway at the right slope,	
Capacity	138 m³/s
Bottom Outlet, through the dam, 1.2 m dia., 2 valves	
Capacity	50 m³/s
Power Intake	
Capacity	80 m³/s
Flushing Outlet	
Capacity	28 m³/s

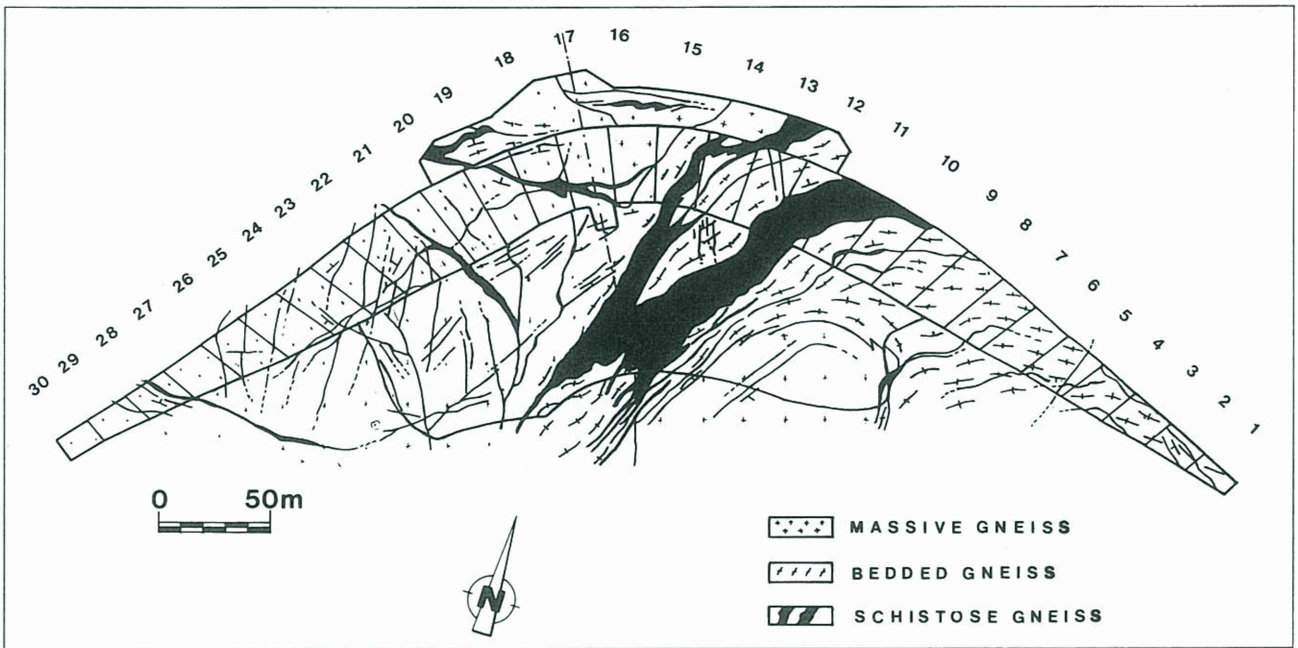
1 GENERAL

Development of the Malta stream for power generation was first envisaged in the thirties by Allgemeine Elektrizitätsgesellschaft (AEG) and later by Alpenelektrowerke AG. Construction was finally commenced in 1971. The development was completed in 1978.

The Malta power station develops the greater part of the hydro potential of the eastern Hohe Tauern mountain range in three stages. The so-called upper stage comprises the Kölnbrein seasonal-storage reservoir created by the 200 m high Kölnbrein arch dam and the Galgenbichl powerstation, utilizing a total head of 198 m. The main stage is formed by two weekly-storage reservoirs, Gösskar and Galgenbichl, and the Rottau powerstation working under a head of 1 106 m. This is immediately followed by a run-of-river station on the Möll river developing a head of 45 m.

The catchment of the upper and main stages has an area of 129 km² and a mean altitude of 2 400 m above sea-level. 11% of this area is glaciated. The average annual runoff corresponds to a height of 1 826 mm, the usable annual volume of water yield is about 235 million m³. In order to save summer runoff, which accounts for as much as 91% of the total annual runoff, for the winter months, it was necessary to provide for seasonal storage affording a net capacity of 190 million m³. As the most powerful group of power schemes in Austria, Malta generates high-quality peak energy, mainly in winter. The development is also used for peaking on the interconnected grid and serves as a reserve in the case of power station failure.

Figure 1 Geology



2 GEOLOGY

The Malta power development touches three large geological units occurring in the Hohe Tauern mountains, Zentralgneis ("central gneiss"), Schieferhülle ("slate mantle") and the eastern Alpine Altkristallin ("old crystalline"). The dam site and reservoir in the Malta valley are entirely located in the central gneiss formations. At the base of the Kölnbrein dam, three geological zones may be distinguished according to schistosity and jointing:

- Massive granitic gneiss predominates in the western flank and in the valley bottom.
- Bedded gneiss is exposed in the left, eastern valley flank, and
- at the toe of the left-hand slope these two rock types are separated by a zone of schistose gneiss.

The extensive rock mechanics investigations yielded the following properties for the three types of rock:
These values, combined with complementary investigations, were used as a basis for the 1986 repair.

3 DAM

Detailed study of alternative designs led to the selection of a double-curvature arch dam. The horizontal arches of the Kölnbrein dam correspond to cone sections, which permitted a continous transition from hyperbolas at the crest to circular arches in the lower dam portion.

The load distribution method gave the following max. stresses for the reservoir full condition:

Table 1 Rock mechanical characteristics

Geological formation		Deformation modules lower than 10 m	Deformation modules deeper than 10 m	Shear strength	Angle of friction
		kN/mm		N/mm	°
Massive gneiss		25	35	2.0	45
Bedded gneiss	II s	25	30	0.8	35
	⊥ s	18	20	2.0	45
Schistose gneiss	II s	15	15	0.3	35
	⊥ s	7	7	1.0	40

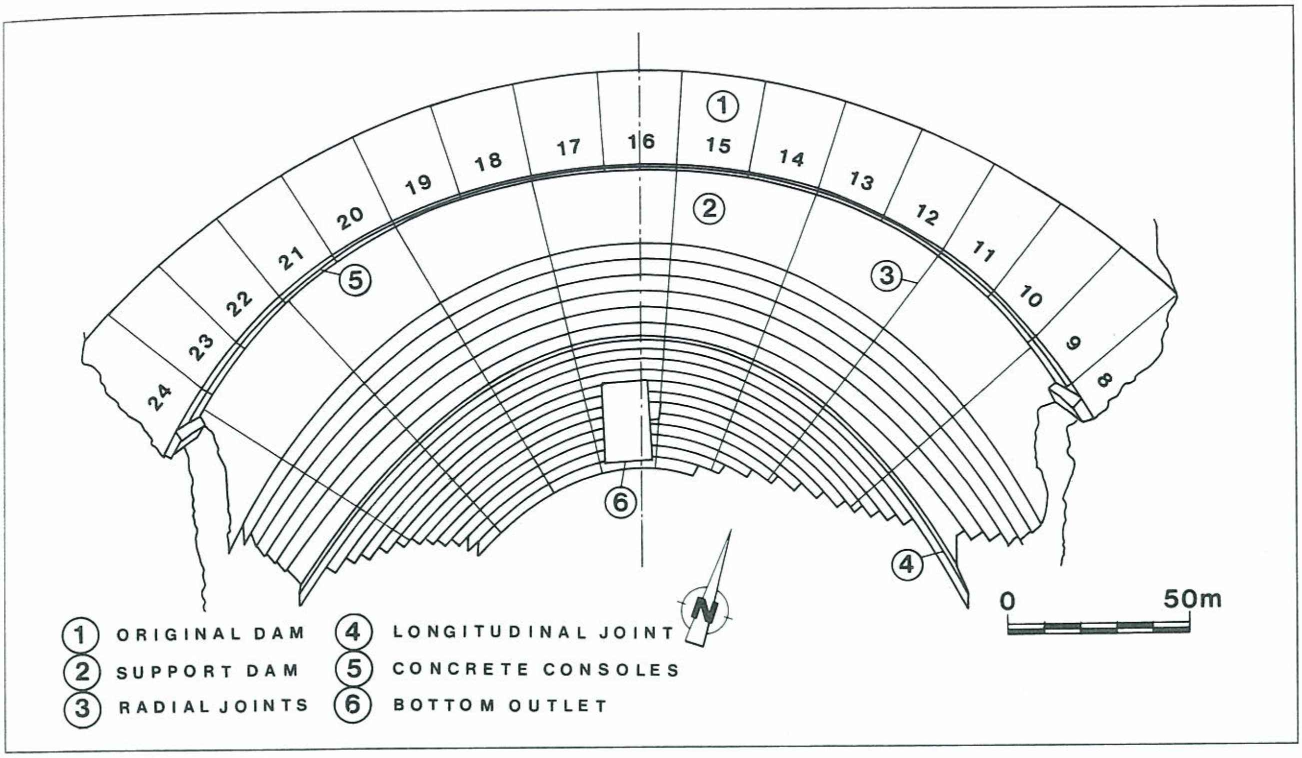


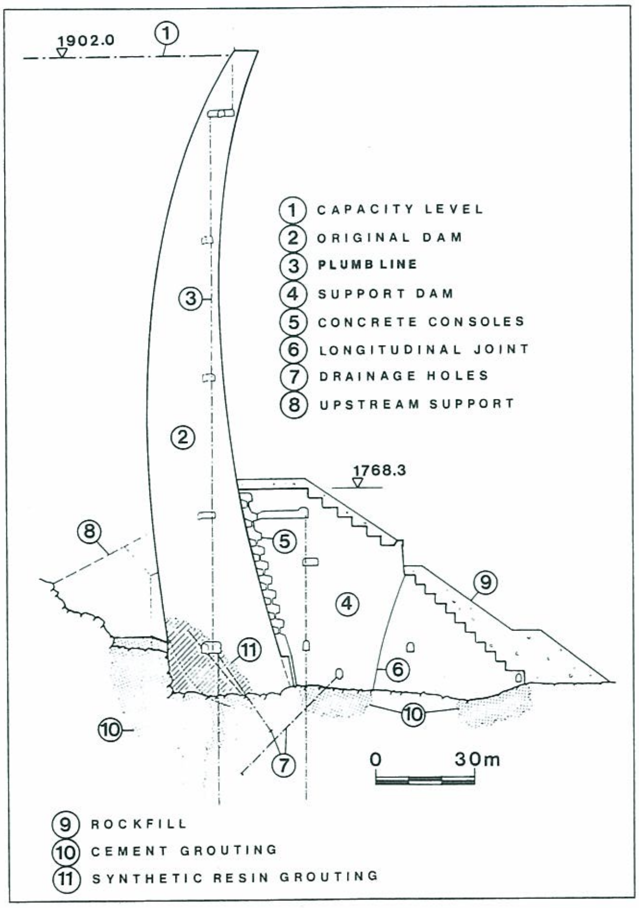
Figure 2 Plan

9.4 N/mm² in a horizontal direction at the crown
1.6 N/mm² tension at the upstream dam toe in a vertical direction and
9.0 N/mm² compression at the downstream dam toe, normal to the dam base

Abutment forces attain a maximum 140 MN/l.m.

Dam safety against failure was calculated from the quotient of compressive strength of concrete (31 N/mm²) and the calculated maximum compressive stress (9.4 N/mm²) to be s = 31 : 9.3 = 3.3. Model tests have confirmed these results.

Figure 3 Cross section



Concrete aggregates were obtained from a quarry in the vicinity of the site. A flyash cement consisting of 70% Portland cement 375 and 30% flyash was used. Maximum concrete temperature near the dam base was measured to be 29 °C.

During construction 20 000 m of drill holes were sunk and 480 t of cement was pumped into the foundation for consolidation and imperviousness. Vertical construction joints were grouted in four successive years, during the months of April and May. A total amount of 860 t of grout was injected under pressures of up to 15 bar.

Kölnbrein dam is equipped with the following appurtenant works:

- a power intake in the left slope, 60 m upstream of the dam, capable of a rated flow of 80 m³/s
- an uncontrolled overflow spillway in the right slope, designed to handle a flow of 138 m³/s under a 50 cm surcharge
- a bottom level outlet located in the middle block of the dam, capable of 50 m³/s, and
- a flushing outlet beneath the bottom outlet, capable of a maximum discharge of 28 m³/s.

Bottom level outlet and flushing outlet will be continued to the downstream face of the supporting arch as part of the large-scale repair scheme for the Kölnbrein dam.

Spillway and bottom outlet reach a total discharge capacity of 188 m³/s or 3.7 m³/s.km² corresponding to a HQ 1000 with retention afforded by the reservoir.

4 EXPERIENCES

4.1 Dam surveillance

The instrumentation of the dam body and adjacent foundation includes plumb lines, extensometers, inclinometers, piezometers, teleformeters and telepressmeters, a seismic station as well as an acoustic emission installation with more than 2 000 reading points. The 400 most important of them are equipped with an automated data acquisition unit with a check for exceeding of limits, and teletransmission systems.

4.2 Special occurrences

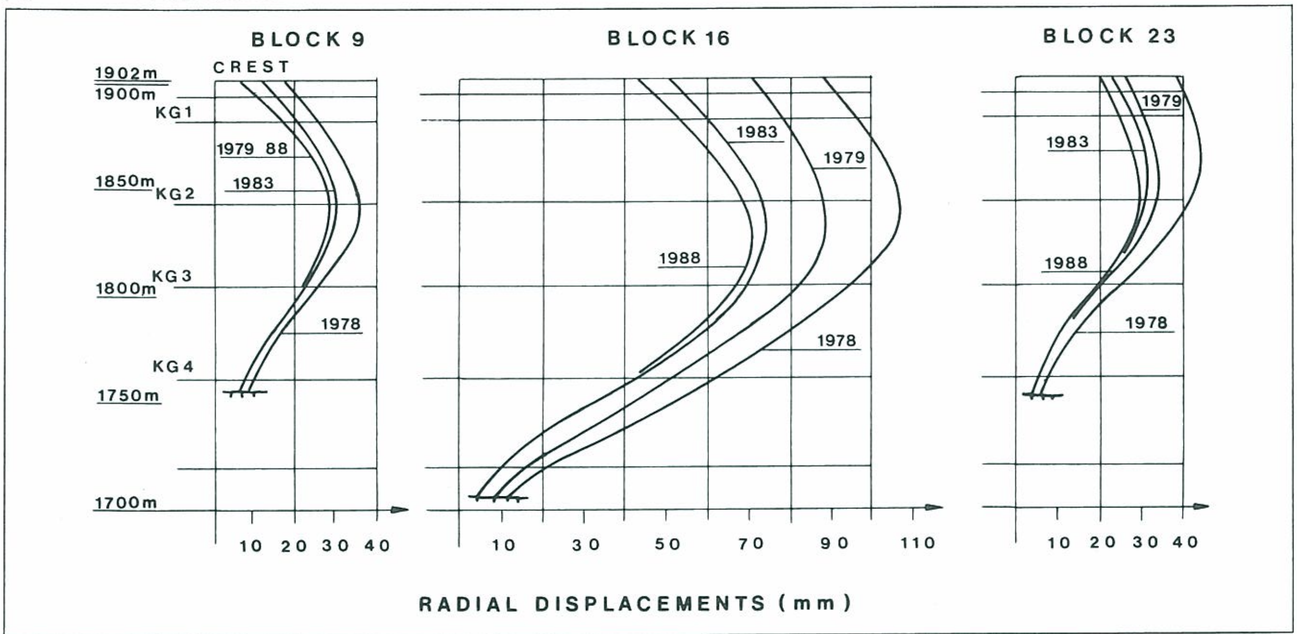
When the reservoir surface level had risen beyond El. 1 860 m above sea-level and 42 m below normal top water level in 1978, the following observations were made:

- The leakage flows from the drainage holes in the inspection galleries increased sharply, reaching values exceeding 200 l/s, with the reservoir surface 11 m below top water level, and
- uplift and joint water pressures along the downstream half of the bases of the highest dam blocks reached values corresponding to up to 100% of the hydrostatic head.

To reduce these uplift pressures and seepage losses at high reservoir surface levels, the following remedial measures were taken in the years 1979 to 1985:
- Foundation grouting,
- temporary sealing by providing a freeze courtain in the cracked zone in the valley bottom, which separated a wedge-shaped part at the upstream toe from the rest of the dam, and
- provision of an apron covering the cracked areas upstream of the highest dam blocks.

Although the intended purpose of reducing uplift pressures and seepage flow was reached by these measures, no convincing improvement in the overall loadbearing behaviour of the dam was accomplished. Therefore, the water authority imposed a limit on the filling of the reservoir to 22 m below top water level. In close cooperation with the Swiss Consultant Dr. Lombardi a general repair scheme was developed comprising the following structural measures:
- In order to reduce the high transverse stresses (the oblique main tensile stresses resulting from these transverse forces are considered to be the main cause of the oblique cracks) in the area of the highest dam blocks, the dam will be stabilized over its lowest one-third by providing a supporting arch with a special load transmission system.
- For stabilizing crack zones and for restoring the imperviousness of the foundation, cement and artificial resin grouting will be provided.
- To safeguard the stability of the dam with an empty reservoir, another grouting scheme will be carried out, which may have to be supplemented by upstream supports.

Figure 4 Radial displacements



The repair of the Kölnbrein dam was started in 1989. Work will probably be completed by 1993, and top water level is planned to be reached in 1994.

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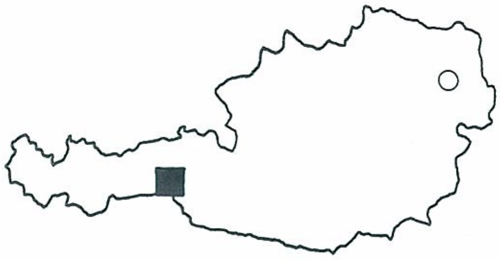
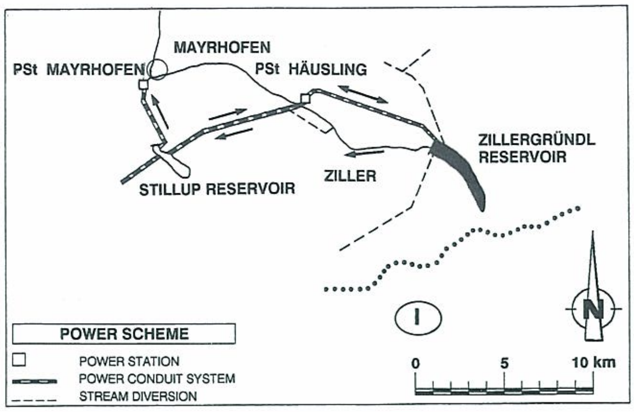
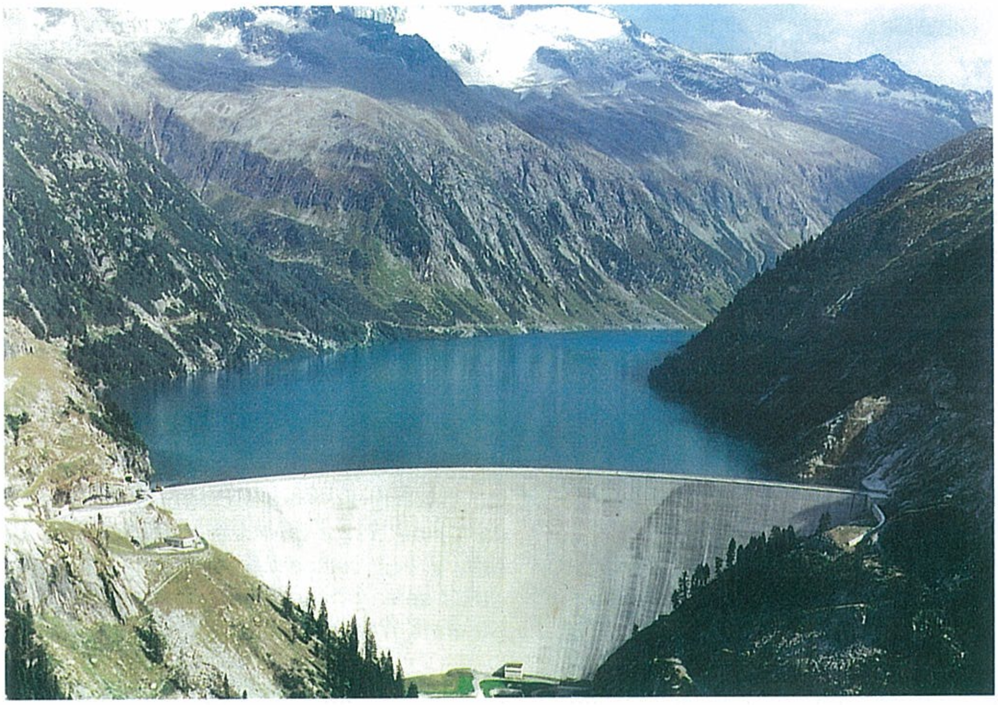
ZILLERGRÜNDL ARCH DAM

Tyrol; Ziller, Inn
Nearest town: Mayrhofen







MAIN TECHNICAL DATA, Chapter K, 60 (5/23)

General

Development	Zemm-Ziller	
Power Station	Häusling	Mayrhofen
Construction Period	1982 – 1986	1965 – 1969
Gross Head	694 m	470 m
Installed Capacity: Turbine	360 MW	345 MW
Pump	360 MW	–
Mean Annual Generation	176 GWh	426 GWh
of which in winter	160 GWh	194 GWh

Reservoir

Catchment Area: Natural	30 km²
Inflow	49 hm³
Diversions	38 km²
Inflow	61 hm³
Normal Top Water Level (a.s.l.)	1 850 m
Minimum Operating Level (a.s.l.)	1 740 m
Gross Capacity	89 hm³
Live Storage	87 hm³
Area flooded by full Reservoir	1.4 km²

Dam

Maximum Height above Foundation	186 m
Crest Length (26 blocks)	506 m
Thickness at the Crest	7 m
Maximum Thickness at the Base	42 m
Volume: Excavation (overburden, rock)	1 700 000 m³
Concrete	1 370 000 m³

Appurtenant Works

Spillway	
Capacity	165 m³/s
Bottom Outlet	
Capacity	45 m³/s
Power Intake	
Capacity	65 m³/s

1 GENERAL

The Zillergründl dam was intended to increase total reservoir capacity and to extend the Zemm scheme to a turbine capacity of 960 MW and 600 MW in the pumping mode, with an average annual energy of 1 140 GWh. Runoff from the natural catchment area of the Ziller stream above the dam site in the order of 30 km² is more than doubled by two diversion systems.

Zillergründl dam, Häusling power station, the intake system from Zillergründl to Häusling, and stream diversions were constructed between 1980 and 1987. From 1974 to 1977 the first stage of the Ziller scheme was constructed, viz. the diversion of the Ziller stream into the Stillupp weekly storage reservoir. The head from the reservoir level to the Ziller diversion into the Stillupp reservoir is utilized in the Häusling station (2 Francis turbines, 180 MW each), before being utilized a second time in the Mayrhofen station (6 Pelton turbines, 57.5 MW each).

The whole scheme exclusively serves the generation of electrical energy to meet peak load demands for grid control. Improved flood protection of the whole Ziller valley is a positive side-effect.

2 GEOLOGY

The central portion of the Austrian Central Alps in which the dam site is located consists of the lowest Zentralgneis proportions of the Pennine Tauern window, which is shaped like a dome. The dam foundation consists of granitic two-mica gneiss with transitions to tonalite intercalated with amphibolite. Bedding of the rock series is steep to vertical. Layers of sand in the deep overburden at the valley bottom called for a supporting structure consisting of bored piles to protect excavation for the dam foundation proper. Due to the presence of slightly opened joints with weathering phenomena and sand to clay fill running parallel to the slopes, an extremely deep excavation was necessary, especially at the right flank. A fault several metres wide with disintegrated and mylonitised rock runs along the downstream dam toe on the left flank before crossing the foundation surface almost at mid-valley. At the lower portion of the left flank a narrow secondary fault branches off the main fault, forming an acute angle with the foundation surface.

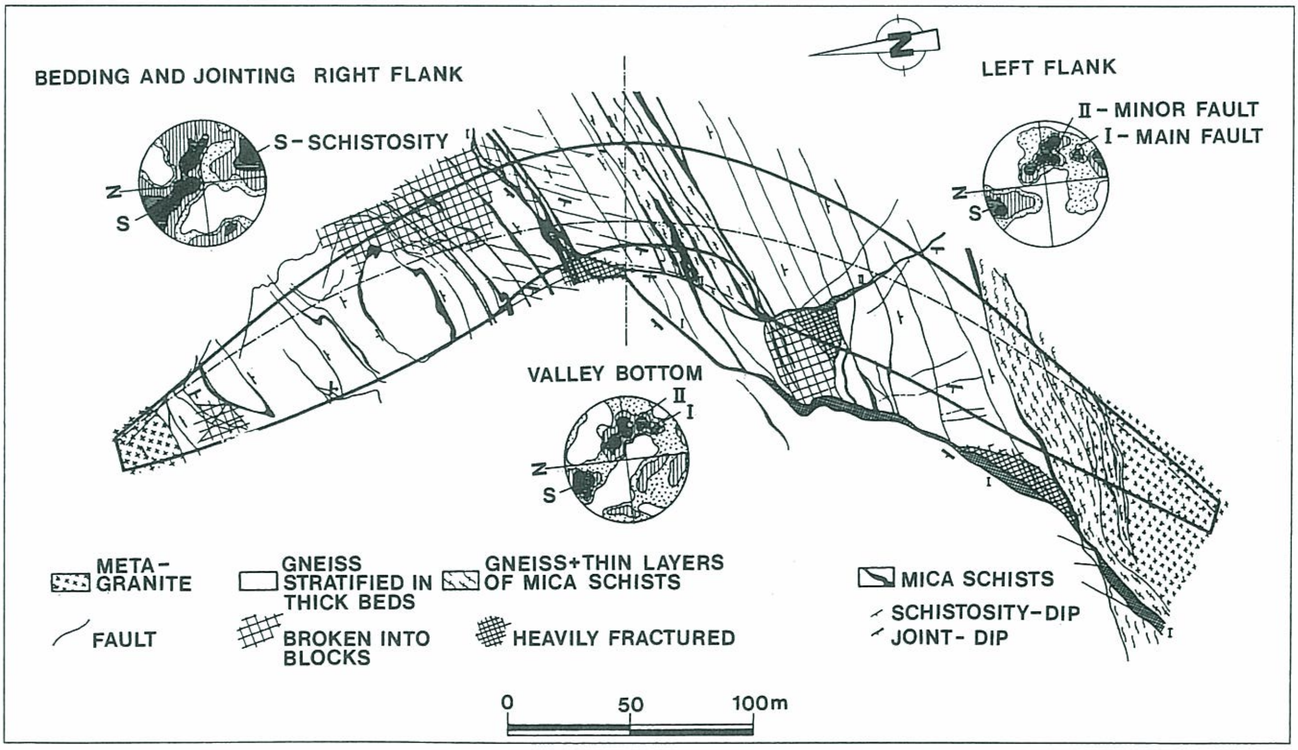
Seismicity: The dam is located some 35 km linear distance SE of the seismically active Inn valley in a region of low seismic intensity and slight continuous rising.

3 DAM

On the basis of the experiences gained with the Schlegeis and Kölnbrein dams constructed before, maximum stresses for Zillergründl were determined as follows: 7 N/mm² arch compression, 6 N/mm² compression perpendicular to the rock surface, no tensile stresses upstream or only minor ones under extreme assumptions. Parabolic at the crest, the elliptical arches of the almost symmetrical dam become circular in the lower dam part.

The dam exhibits a number of interesting structural features. Upstream a concrete slab was constructed, about 15 m in width and about 7 m in height, including two galleries. At its upstream end the grouting gallery is situated, from where the grout curtain was sunk. This means the grout curtain is situated more than 10 m upstream of the dam in an area unaffected by dam movements. On the dam side of the concrete slab there is an inspection gallery. This accommodates the main sealing element which seals off the movement joint between the static concrete slab and the dam moving under hydrostatic load. The inspection gallery is flooded during normal operation, whereas the grout gallery is accessible.

Figure 1 Geological map



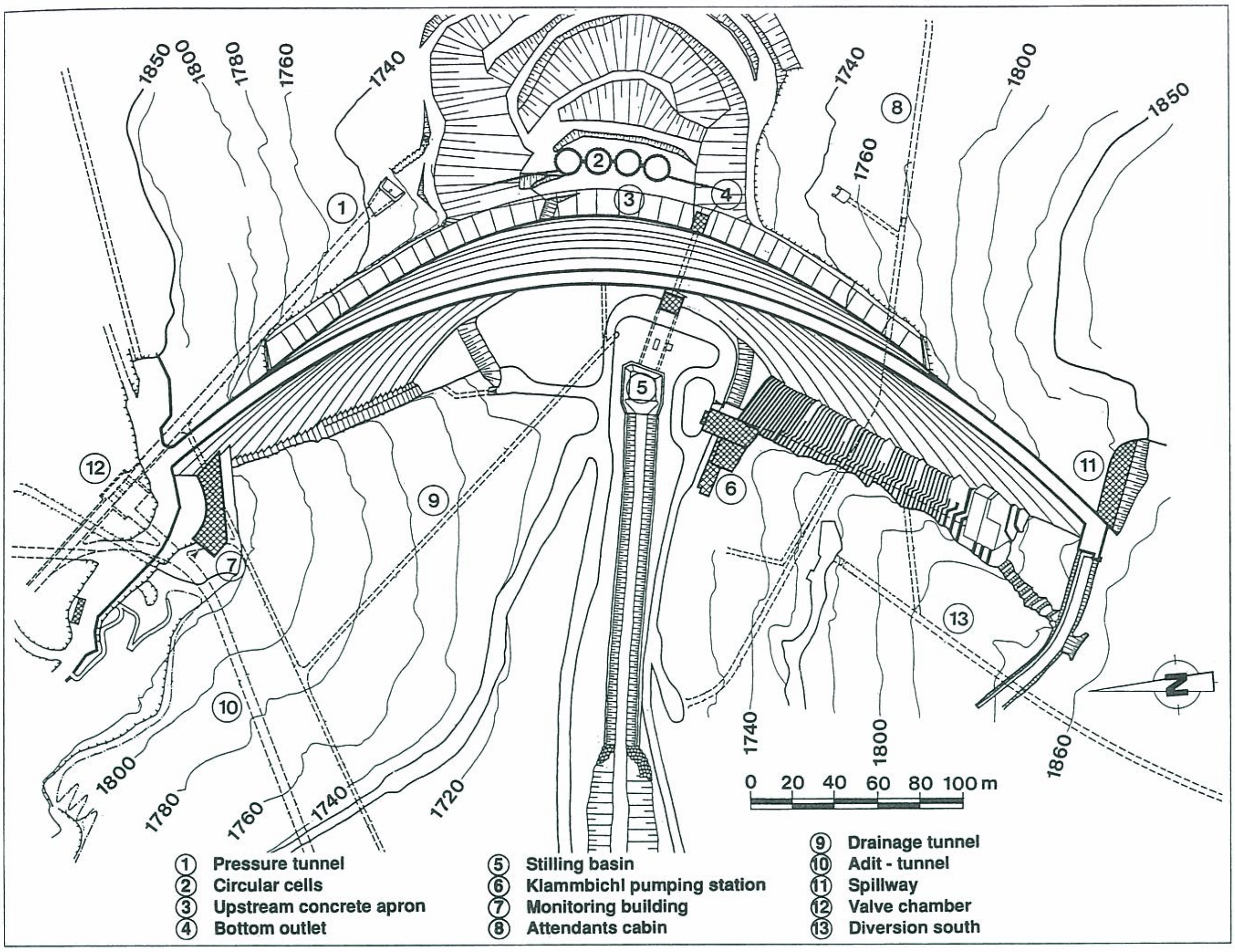
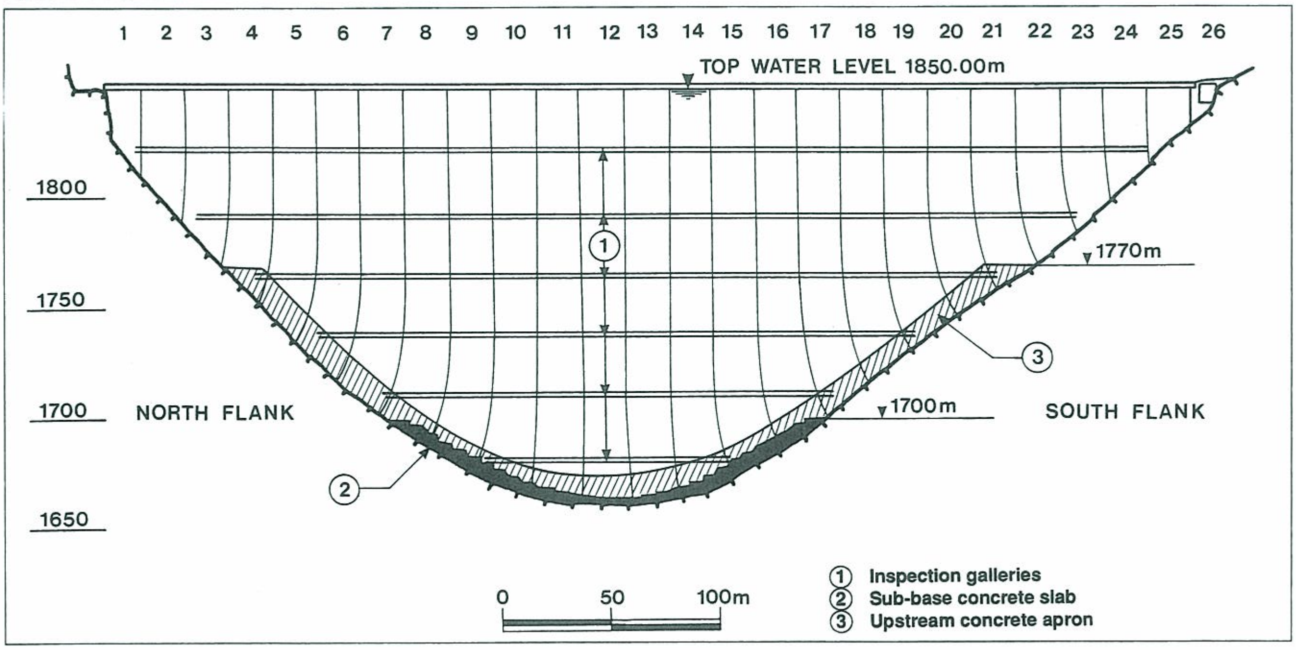


Figure 2 Plan

In the upstream dam foundation a horizontal movement joint was arranged allowing for the construction of the concrete slab supporting the dam. In this way structural prevention of vertical tensile stresses at the upstream dam heel is assured.

At the left-hand flank a massive concrete plug was provided to cover the rock fault zone paralleling the dam downstream and thus support the dam.

Figure 3 Longitudinal section



The dam consists of 26 blocks, each 20 m long at the crest. The block joints are somewhat twisted towards the foundation surface.

The reservoir is provided with a bottom outlet through the dam with two slide gates. Energy dissipation is provided by a surge chamber backfilled with earth and a stilling basin.

Flood relief works consist of an ungated spillway on the left portion of the dam, discharging into a rock channel at a safe distance of the dam. The dam comprises 6 horizontal inspection galleries, one bottom gallery a quarter into the dam base on the upstream side, which is directly located on the rock foundation and almost extends over the whole foundation sur-face, and a downstream inspection gallery.

The dam concrete has a maximum grain size of 120 mm and a binding agent proportion of 170 kg per m^3. The binding material is a mixture of two thirds PZ 375 and one third fly ash. Concrete was placed in 3 m lifts, and in 1.5 m lifts in the massive bottom zones. Almost all the joints were provided with a pipe cooling system. To reduce the heat of hydration the mixing water was partly replaced by flake ice. Dam concreting was performed between 1983 and 1985.

For dam monitoring, an extensive instrumentation system comprising more than 1 000 gauges was installed. The most important readings and data are transmitted to the control centre at Mayrhofen.

4 EXPERIENCES

4.1 Dam monitoring

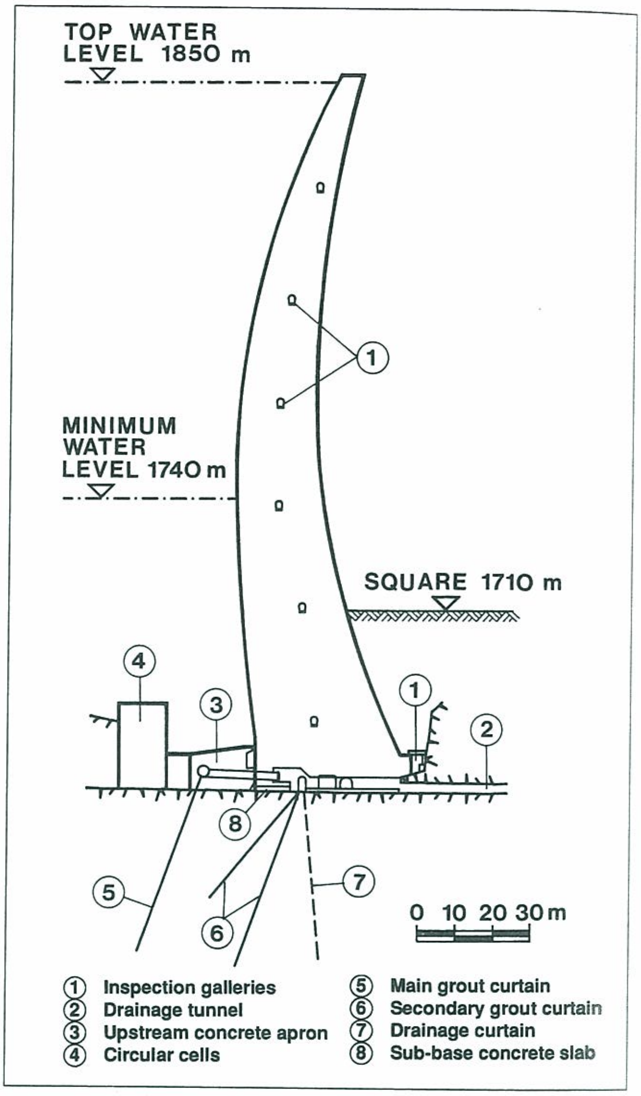
Initial filling of the reservoir upon completion of concreting was in autumn 1985. In 1986 the reservoir was filled up to El. 1 820 m, in 1987 and 1988 up to El. 1 840 m and in 1989 up to El. 1 845 m. Top water level was reached for the first time in 1990. The main data and readings relate to displacements of the crest and bottom of the dam and are obtained by means of plumb lines from three measuring diameters. These readings are automatically transmitted, as are the data for seepage flows, permanently polled by the dam computer, compared with limit values and stored hourly. Long-term results are not yet available.

4.2 Special events

In 1987 a crack appeared closely above the movement joint in an area, where the concrete could not have any vertical tensile stresses. The crack appeared in one of the 26 blocks, immediately before the envisaged filling level at El. 1 840 m was reached. This crack extended from upstream to the level of the bottom gallery. Leakage was about 150 l/s. Apart from that the overall dam behaviour was not affected to a measurable extent by this damage. Occurrence of such crack was due to lacking vertical compression in the upstream zone of the block. The unfavourable stress conditions resulted from the construction history and the sealing system applied. The crack was caused by water penetrating into a small initial crack with a pressure of 1.6 N/mm² corresponding to the depth of the crack below storage water level.

The crack was grouted with epoxy resin with a high adhesive strength in two stages (with partly filled reservoir and with empty reservoir). A watering system was installed, which introduces a water pressure of approximately half of the storage level at the highest vertical section of the dam into the movement joint. Peattern of compressive stresses a sufficient safety against similar cracks was achieved. No more such problems were not neither during the periods of partial filling in 1988 and 1989 nor in 1990 with full reservoir.

Figure 4 Central cross section



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Chapter G

EMBANKMENT DAMS IN AUSTRIA

CONTENTS

EMBANKMENT DAMS

By W. Schober/H. Schwab*

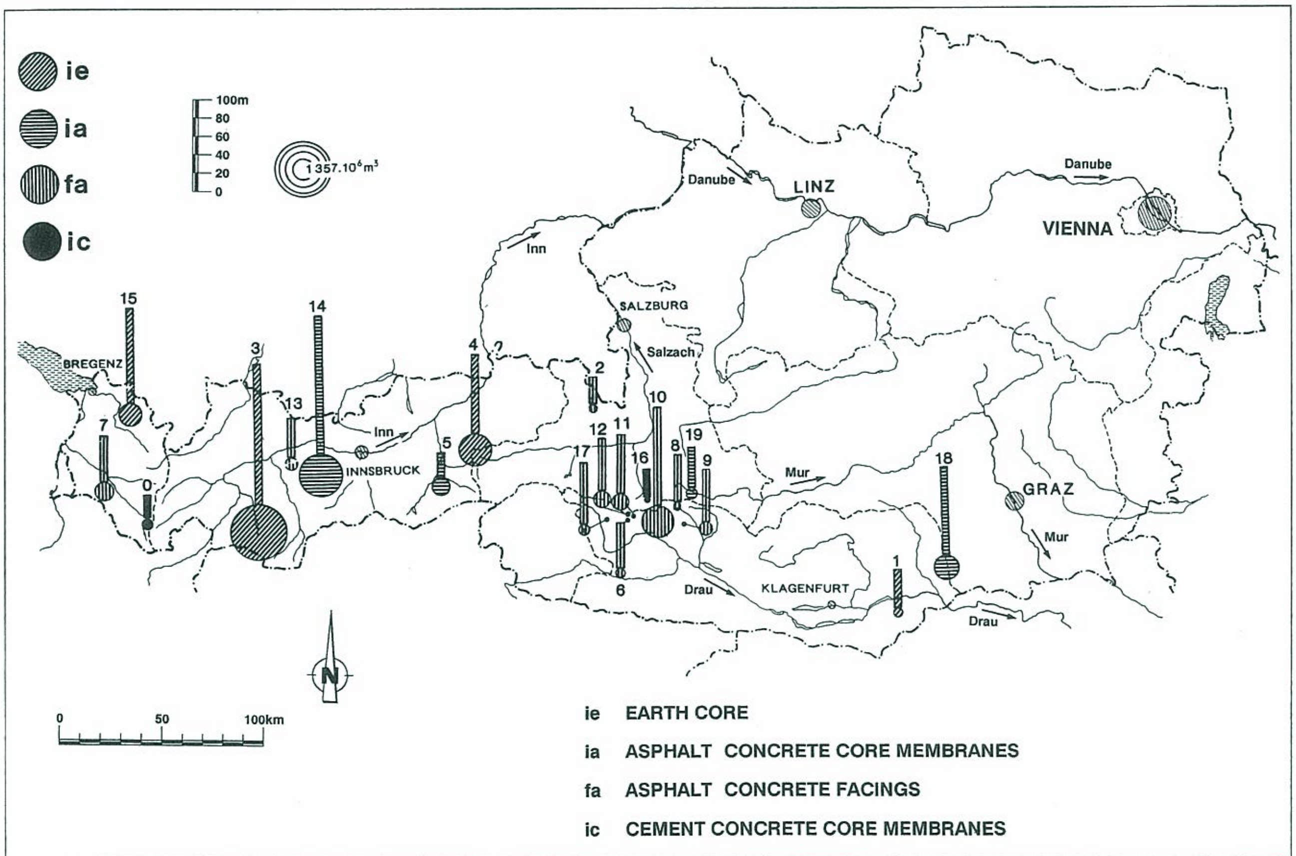
EMBANKMENT DAMS – GENERAL

1 INTRODUCTION

All Austrian embankment dams are hydropower installations. Prior to 1960 they were built for heights of up to 25 m, with concrete dams used for higher heads. With a growing shortage of sites for concrete dams that would be suitable in terms of both engineering and costs, however, greater attention was paid to embankment dam projects in Austria.

The dams were built with earth cores (ie), asphalt concrete core membranes (ia), asphalt concrete facings (fa), and cement concrete core membranes (ic). In Fig. 1, dam volume is represented by a circle and dam height as a superimposed bar. The data are summarised in Table 1, which also distinguishes between seal height above ground level (HDo) and seal depth underground (HDu). As can be seen from the table, dam heights vary between 25 m and 153 m, and dam volumes between 0.06 hm³ and 7.1 hm³. Of the four types

Figure 1 Location of embankment dams over 25 m high



As can be seen in Fig. 1, the twenty embankment dams built with heights of over 25 m are primarily located in the west and south of Austria.

of sealing works, type ie was employed four times, ia four times, fa ten times and ic twice. The full data for ten selected dams are given in the detailed descriptions at the end of this chapter.

No clear preference for a specific dam design can be derived from the list. The most frequently used impervious element (type fa – asphalt concrete facings) was obviously the most popular choice from 1970 to 1980, but it has not always proved effective in coping with the difficult conditions encountered in high mountain locations (rockfall and climatic influences).

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No.	Name	Power company	Hmax[1]) [m]	HDo[2]) [m]	HDu[3]) [m]	V[4]) [10^6 m³]	Type		Year of completion
0	Bielerdamm	VIW	25	25	5	0.375	TE	ic	1948
1	Freibach	KELAG	49	41	–	0.235	TE	ie	1958
2	Diessbach	SAFE	36	29	–	0.165	ER	fa	1963
3	Gepatsch	TIWAG	153	153	–	7.100	ER	ie	1965
4	Durlassboden	TKW	83	83	60	2.520	TE	ie	1966
5	Eberlaste (Stillupp)	TKW	28	28	53	0.790	TE	ia	1968
6	Wurten	KELAG	51	34	15	0.265	ER	fa	1971
7	Latschau II	VIW	50	18	–	0.865	TE	fa	1975
8	Galgenbichl	ÖDK	50	50	–	0.065	TE	fa	1974
9	Gösskar	ÖDK	55	55	–	0.540	TE	fa	1975
10	Oscheniksee	KELAG	116	61	–	2.250	ER	fa	1978
11	Hochwurten	KELAG	55	50	16	0.600	TE	fa	1979
12	Gross-See	KELAG	57	49	–	0.740	ER	fa	1978
13	Längental	TIWAG	45	32	12	0.400	TE	fa	1980
14	Finstertal	TIWAG	150	96	–	4.500	ER	ia	1980
15	Bolgenach	VKW	102	92	–	1.200	TE	ie	1978
16	Bockhartsee	SAFE	69	31	–	0.228	ER	ic	1982
17	Zirmsee	KELAG	44	37	–	0.525	ER	fa	1983
18	Feistritzbach (Koralpe)	KELAG	88	85	–	1.500	ER	ia	1990
19	Rotgüldensee	SAFE	45	45	–	0.350	ER	ia	1990

[1]) maximum dam height
[2]) height of seal in dam body
[3]) height of seal in dam foundation
[4]) dam volume

Table 1 Main data for embankment dams over 25 m high

2 DAM DESIGN AND CONSTRUCTION

2.1 Dam design

The four dams built with an earth core (type ie) all have central cores. Only in the case of Bolgenach fill dam (see detailed description) is the lower half of the core inclined to the upstream side. Core permeability varies between $k = 10^{-8}$ m/s and 10^{-10} m/s. The cores are made up of well graded talus and moraine material up to a maximum particle size of 100 mm. The cores are built on a foundation of bedrock, with the exception of Durlassboden fill dam (see detailed description), where the seal extends down through a 60 m alluvion layer.

The transitional zones mainly consist of well graded sediment deposits or talus. The fill for the shoulders is mostly quarry material, although sediment deposits and talus material are also used. Maximum particle size, which used to be 1 m, now seems to have been reduced to 75-50 cm. For the load case of rapid drawdown, the upstream shoulder should have both high shear strength and a permeability $> k = 10^{-4}$ m/s. Quarry materials are primarily used to meet these requirements. Upstream slope protection against wave erosion is normally provided in the form of riprap, while the downstream slope is finished either with a rock dressing layer or soil and vegetation cover.

The type ia asphaltic concrete core membrane incorporated in the Finstertal dam (see detailed description) in 1981 was a pioneer feat of dam engineering. With maximum nominal load at a height of 97 m, the core membrane is 77 cm thick in the upper third, 82 cm thick in the middle section and 94 cm in the lower section. Another innovative feature of the Finstertal dam was the use of mechanical placement techniques for a core with an upstream inclination of V:H = 1:0.4.

Another new development is to be seen in the vertical asphaltic concrete core built for Eberlaste dam (see detailed description), which connects with a 53 m deep clay concrete cut-off wall. At the contact with the valley flanks, and over a length of approx. 40 m, the two sealing walls now display up to 2 m vertical deformation from the original plane of the walls.

As already mentioned, type fa was the seal of choice in the Seventies. The advantage of this type is that it permits the seal to be constructed independent of the embankment and that impounding has little effect on dam behaviour. Asphaltic concrete facings are normally laid in a single seal layer. The type fa impervious element also offers other advantages, like the ease with which dam height can be increased and the possibility of using all shear-resistant materials for the embankment. Seepage water drains through a dewatering zone beneath the impervious blanket. The disadvantages of the design are the vulnerability of the facings, the difficulty of achieving a good connection with steep valley flanks, the difficulty of locating leaks, and exposure to climatic influences.

A new development of type ic is the 60 cm thick concrete core of Bockhartsee fill dam (see detailed description), which has a 4 mm bituminous upstream sliplayer to prevent excessive loading of the core caused by skin friction and to provide an additional seal.

In the case of a type ia core, the upstream transition zone is composed of fine-grain material as an additional impervious element, while the downstream transition zone is composed of gravel and serves as a drain for seepage flows. The shoulders and slope finishing are identical with type ie.

The first concrete core (type ic) in Austria was built forty years ago. The 25 m high "Bielerdamm" listed as number 0 in Table 1 is built on a foundation of moraine material and to that extent is exceptional for concrete cores.

2.2 Material quarrying, treatment and placement

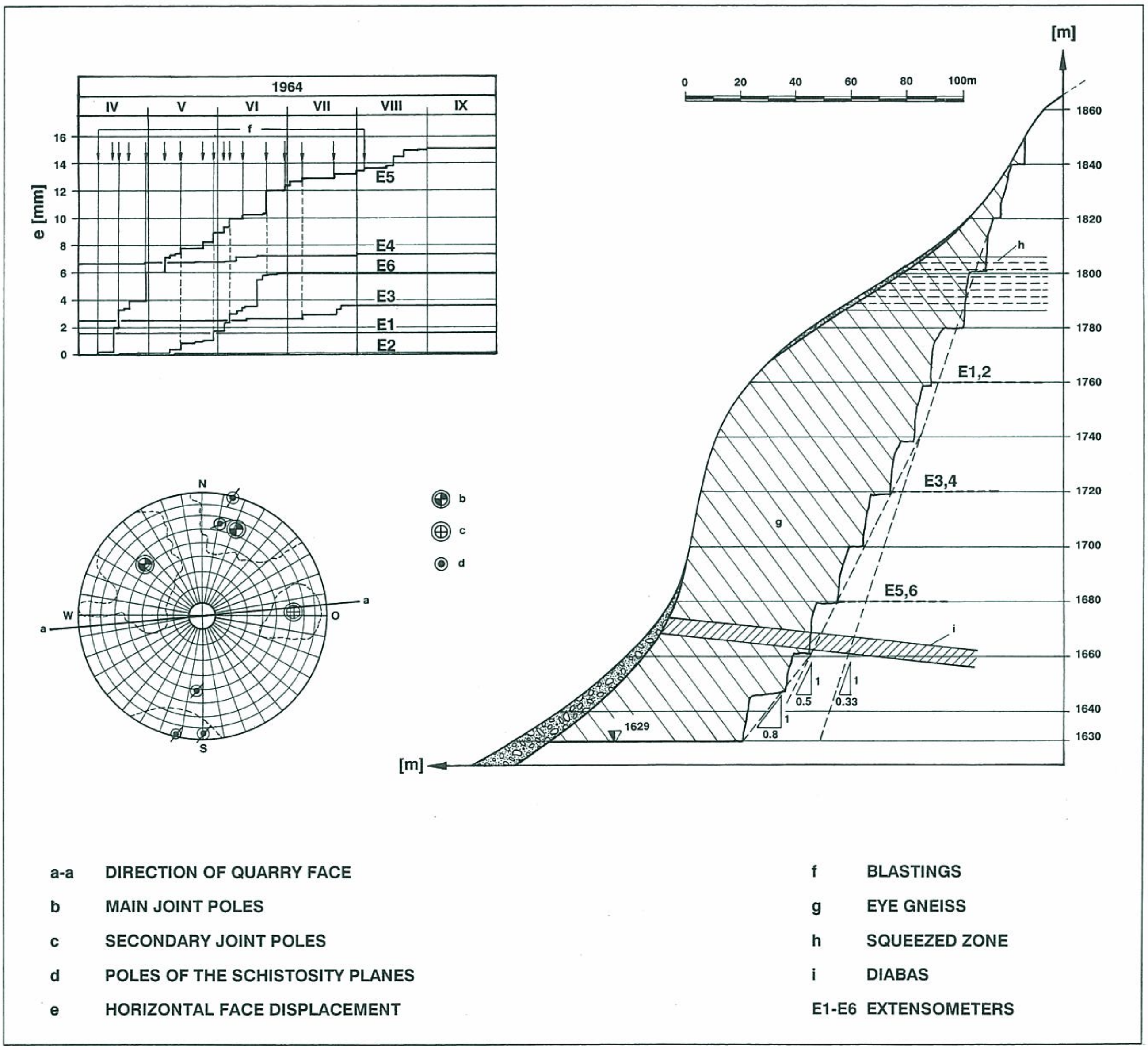
Even on steep slopes, quarrying the fill materials is rarely a problem as long as the face is always worked from top to bottom. Nevertheless the actual logistics of a quarry can be a demanding proposition. Fig. 2 presents a section through the quarry used for the Gepatsch fill dam (see detailed description), which had to be worked with a maximum face height of over 230 m.

Core material treatment usually involves screening for removal of oversize particles. In addition, wetting or drying is often required to achieve the desired water content. Fig. 3 shows the screening, drying and mixing plant for the Gepatsch dam, with a wobbler used to remove the grain fraction > 80 mm, four rotary tubular kilns for drying with a throughput of 100 t/h each, and a mixing plant for adding bentonite to part of the core material.

Construction of the Gepatsch dam in 1961 involved the first use of 8-ton vibratory roller trailers to compact the quarried rockfill material placed in 2 m lifts. This compaction proved inadequate, however, and settlement in the shoulders reached 5 m and more. This in turn caused longitudinal cracking along the dam crest, although no negative effects have been observed.

In the case of the Finstertal dam (see detailed description), compaction of the quarry-run rockfill material was significantly improved by placing the fill in lifts of 0.75 m and 1.0 m and by the use of 15-ton vibratory rollers. Maximum settlement in the shoulders was only 10% of the Gepatsch figure for roughly the same dam height but using harder quarry materials. Self-propelled vibratory rollers or trailers have now largely replaced the rubber-tire rollers originally used for core compaction.

Figure 2 Gepatsch dam: Versetz quarry



Extensive on-site and laboratory testing is performed to ensure that material specifications are met. Tests on the placements are normally conducted at the rate of one test for every 200-500 m^3 of earth core placement, every 2 000-5 000 m^3 of transition zone placement, and

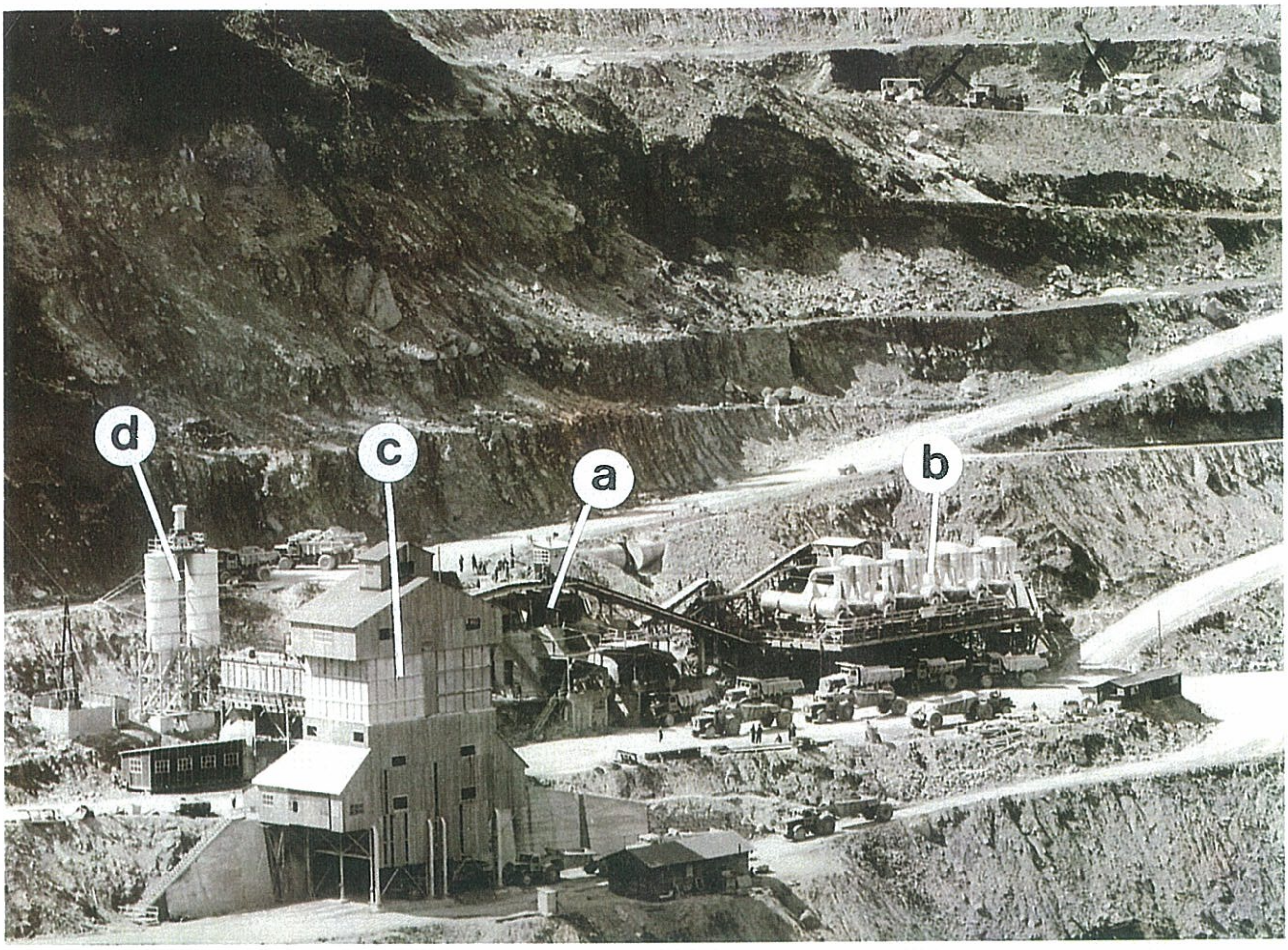


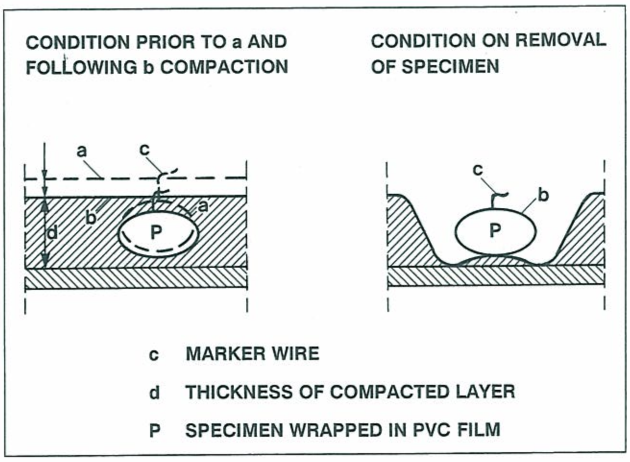
Figure 3 Gepatsch dam: screening, drying and mixing plant

every 20 000-200 000 m³ of shoulder. Laboratory tests are conducted to check grading curves, water content, Atterberg's limits, proctor density and sometimes shear strength and permeability.

For the Gepatsch dam a new test to establish the density of the core was developed, which has since been incorporated in Austrian standards as the "test body method". As shown in Fig. 4, up to 10 kg of the fill material is wrapped in film or filled in a bag made of film and buried with the fill prior to compaction. The location of the specimen must be marked, e.g. with a red wire. After compaction, the specimen is excavated and the film removed. The film serves to separate the test material from the surrounding fill and also to produce a smooth surface. The actual compaction test is then conducted by dipping and weighing in an oil bath. The advantage of the test body method over conventional substitution methods is that the result will always err on the side of caution, i.e. relaxation in the specimen ensures that the result can never be an incorrectly high density. Where a ditch has to be dug, as in the water substitution method of density-testing, the surrounding soils, having been prestressed through compaction, release stress forces into the ditch. The resulting reduction in the volume of the ditch can lead to an incorrect result, i.e. excessively high values for density. The test body method is only suitable for soils with at least minimum cohesion.

For coarse-grained materials (e.g. quarry-run material) a different compaction test method was developed for the construction of the Finstertal dam. The procedure is illustrated in Fig. 5. First, a layer of fill material is placed and a hollow created in the form of a pyramid base that is square on plan. The length L1 of the side of the square forming the bottom of the hollow should be at least twenty times maximum particle size. A geodetic survey of the hollow is then performed on a matrix basis, and one or two gauges are placed to measure

Figure 4 Density test using the test body method in accordance with ÖNORM B 4414, T2



settlement in the bottom of the hollow. The next step is to fill the hollow loosely with weighed material and to survey the surface of the fill using a matrix pattern. This dead level is used to calculate volume Va of the loose

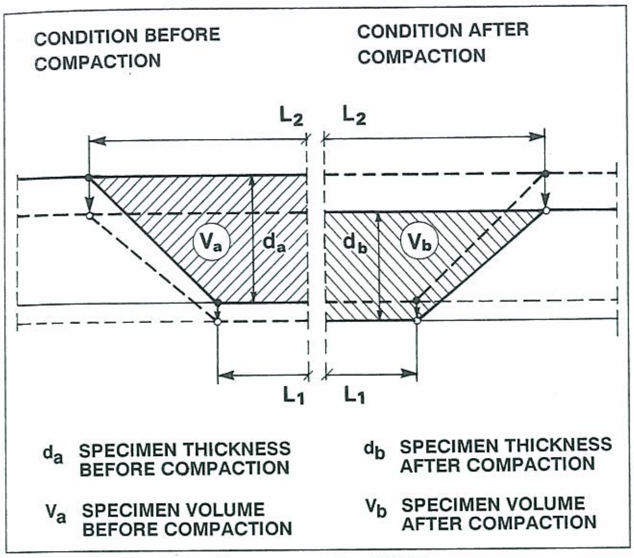


Figure 5 Density test with coarse grained fill [1]

fill. Then the fill is compacted with a vibratory roller, and the settlements measured at intervals of two passes to calculate volume Vb. The advantage of this method is that no excavation has first to be dug, and that the lengths L1 and L2 remain practically unchanged, so that the base and top surface of the fill body are not changed by the compaction process.

2.3 Core connection and foundation sealing works

The connection between the earth core and the prepared foundation rock took the form of an approx. 10 cm thick layer of puddle clay for the Gepatsch dam, while fine-grained core material was employed for the core contact of the Bolgenach dam. Man-made core materials are tied into the foundation rock by means of cut-off walls, which can also be designed as inspection galleries. In the case of earth cores, the bedrock is sealed by means of a blanket grout and a grout curtain. In either case grouting is normally performed from the grouting gallery with a part of the dam load already in place. Foundation treatment is similar for type ia and type ic cores. For both facings and core membranes, the cut-off wall is connected to the bedrock with tie bolts. For contact grouting it is then possible to work at a grouting pressure of at least 3 bar without the risk of displacing the cut-off wall.

The more unusual solutions adopted include the alluvion grout curtain used for the Durlassboden dam and a clay concrete diaphragm wall for Eberlaste dam. In both cases the sealing works had to be carried out from ground level before fill placement began. Difficult and time-consuming sealing works were also required for the left flank of the Freibach dam (see detailed description), namely a concrete cut-off wall and an alluvion grout curtain.

3 MONITORING EQUIPMENT

Table 2 shows the monitoring equipment installed in the embankment dams listed in Table 1.

Dam surveillance comprises monitoring internal and surface deflections at the various measuring points, pore water and earth pressures, seepage flows and the decrease of piezometric level in the foundation zone. Total seepage water losses are normally monitored and the results transmitted to the control room in the power station. Seepage monitoring is also linked to a warning system. One solution that has been particularly successful is the plumb-line shaft and monitoring system installed in the Finstertal dam. Fourteen out of the twenty dams also have inspection galleries or

Table 2 Main monitoring equipment in Austrian fill dams over 25 m high

No.	Name of dam	Number of measuring stations							
		Deformations				Stresses		Seepage	Piezometers
		Dam surface		Internal		Pore water pressure	Earth pressure		
		Settlements	Deformations	Settlements	Deformations				
0	Bielerdamm*)	24	7	3	3			44	
1	Freibach	5	5					8	43
2	Diessbach	3						2	
3	Gepatsch	58	58	285	285	32	51	3	20
4	Durlassboden	18	18	143	140	44	6	17	23
5	Eberlaste (Stillupp)	21	6					16	11
6	Wurten	11	6	32	32			5	
7	Latschau II	102	29	5		16		6	
8	Galgenbichl	37	37	6				5	7
9	Gösskar	64	64	8		6		8	6
10	Oscheniksee	62	62	47	47			4	
11	Hochwurten	31	31	34	34			4	11
12	Gross-See	25	25	22	22			4	
13	Längental	22	22					11	2
14	Finstertal*)	111	111	114	118	14	83	25	13
15	Bolgenach	28	28	20	29	28	10	5	3
16	Bockhartsee*)	23	23	36	104		21	8	2
17	Zirmsee	14	14	46	20			7	1
18	Feistritzbach (Koralpe)	39	39	95	95		62	7	28
19	Rotgüldensee	38	38	81	81	10	24	11	8

*) plumb-line shaft

shafts, which are primarily used for monitoring seepage water losses in the individual sections. They also serve as an access point for monitoring the decrease in piezometric level and for laying the lines from the pressure transducers. They are also used for regrouting work and as a drainage tunnel during the construction phase in some cases and also after completion of the dam. Given the information made available via such galleries, tunnels and shafts, the high cost of the construction work must be considered acceptable.

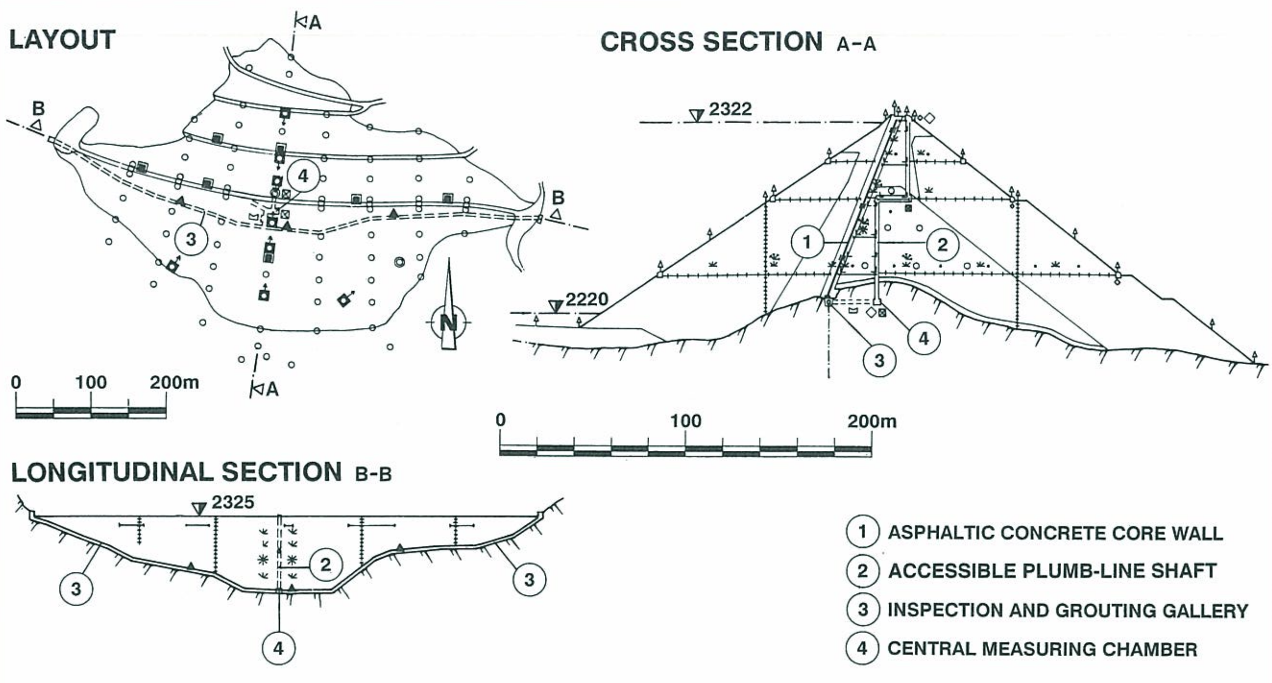
As a matter of principle, the greatest importance is attached to dam surveillance for all dams in Austria. The use of the latest equipment, including new developments like the horizontal plate gauges for the Gepatsch dam, has ensured a high standard in dam surveillance. The Finstertal embankment dam described below is an example of a sophisticated monitoring system [2].

Finstertal dam is remarkable in that it is currently the highest rockfill structure with an asphaltic concrete core and that its foundation area straddles a rock sill.

Accordingly, the Tiroler Wasserkraftwerke AG (TIWAG) installed a very full surveillance system, which is described in this report.

The overall monitoring system is designed so as to permit reliable appraisal of the condition of the dam at any time during construction and operation. The majority of the instruments are located at maximum cross-section and five secondary sections, and at three different levels, with concentrated monitoring of the zone immediately downstream of the inclined asphaltic concrete core. The number of instruments installed and the measuring techniques employed are listed in the table in Fig. 6, which also shows instrument locations for cross-section, longitudinal section and plan.

Figure 6 Finstertal dam – monitoring equipment



Symbol	Monitoring equipment	Number
	TWO DIMENSIONAL GROUP OF EARTH PRESSURE CELLS	92
	THREE DIMENSIONAL GROUP OF EARTH PRESSURE CELLS	18
○	PORE PRESSURE CELLS	12
●	RESISTANCE THERMOMETERS	9
◇	PEAK RECORDING ACCELEROGRAPHS	3
◇	STRONG MOTION ACCELEROGRAPHS (THREE DIMENSIONAL)	2
▲	PIEZOMETER IN BOREHOLE	13
	RECORDING OF SEEPAGE WATER LOSSES	3

Symbol	Monitoring equipment	Number
○	SURFACE MONUMENTS	110
	HORIZONTAL PLATE GAUGES	8
	VERTICAL PLATE GAUGES	7
	EXTENSOMETERS	13
	GROUPS OF 4 STRAIN METERS	4
	FLUID LEVEL SETTLEMENT DEVICES (PERMANENTLY INSTALLED)	9
⊠	PLUMB-LINES	2
	MEASURING DEVICES FOR DETERMINATION OF CORE-WALL THICKNESS VARIATIONS	3

3.1 Deformations

3.1.1 General

Much of the comprehensive dam surveillance system is designed to monitor deformations. Instrumentation is provided both for selected points on the slopes and at the dam crest and also to monitor movements in the interior of the dam, especially in the core area and foundation. The large number of parameters provided for in the monitoring system necessitated the use of a corresponding number of different measuring techniques.

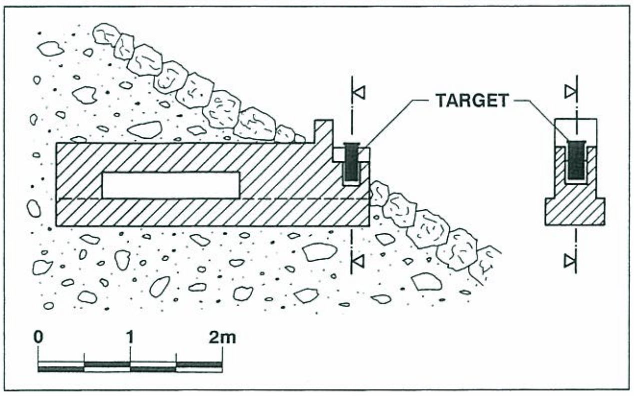
3.1.2 Surface monuments and benchmarks

Geodetic methods are employed to measure deflections using a total of 110 surface monuments and benchmarks arranged at eight profiles distributed over the whole dam including the downstream foreland.

Monitoring provides information on the deformation properties of the fill as well as on the effect of its S-shaped plan and its saddle-shaped foundation contact area, which – although mainly bedrock – also includes some moraine overburden. For surveying the dam slopes, 3.4 m long precast concrete surface monuments were designed and embedded approx. 80% in the fill (Fig. 7).

A recess at the protruding end of the surface monument houses a steel target (14 cm in diameter), which was fixed in place with sealing compound after placement of the surface monument. The target is surrounded on three sides by reinforced concrete to offer protection from rock fall, especially during construction. To reduce the weight of the surface monument, there are cavities in the embedded portion of the precast block.

Figure 7 Finstertal dam - monitoring equipment: surface monument



As it extends relatively far into the shoulder, this device provides meaningful deformation data and avoids the misleading results from tilting often produced by small monuments with only surface contact. The crest was fitted with benchmarks of conventional design. The surface monuments installed at the Finstertal dam produce measurements that are accurate to within +/-5 mm.

3.1.3 Vertical plate gauges

Tubular gauges with sliding metal plates for measuring dam deformations using the induction coil probe developed by Dr. Idel were first installed on a large scale in the Gepatsch dam. In view of the success of the basic design there, the system was used with certain modifications and improvements for the Finstertal dam, too.

Each tubular section was accurately positioned during installation, and no subsequent twisting was observed. In order to avoid pendulum motion in the sensors, all the gauges were installed at a slight inclination (89°). Out of a total of seven vertical gauges, two were placed in the main section and five in zones above steeply dipping foundation surfaces. Inclination is measured with the help of an inclinometer sensor which runs in grooves on the inside of the tubes.

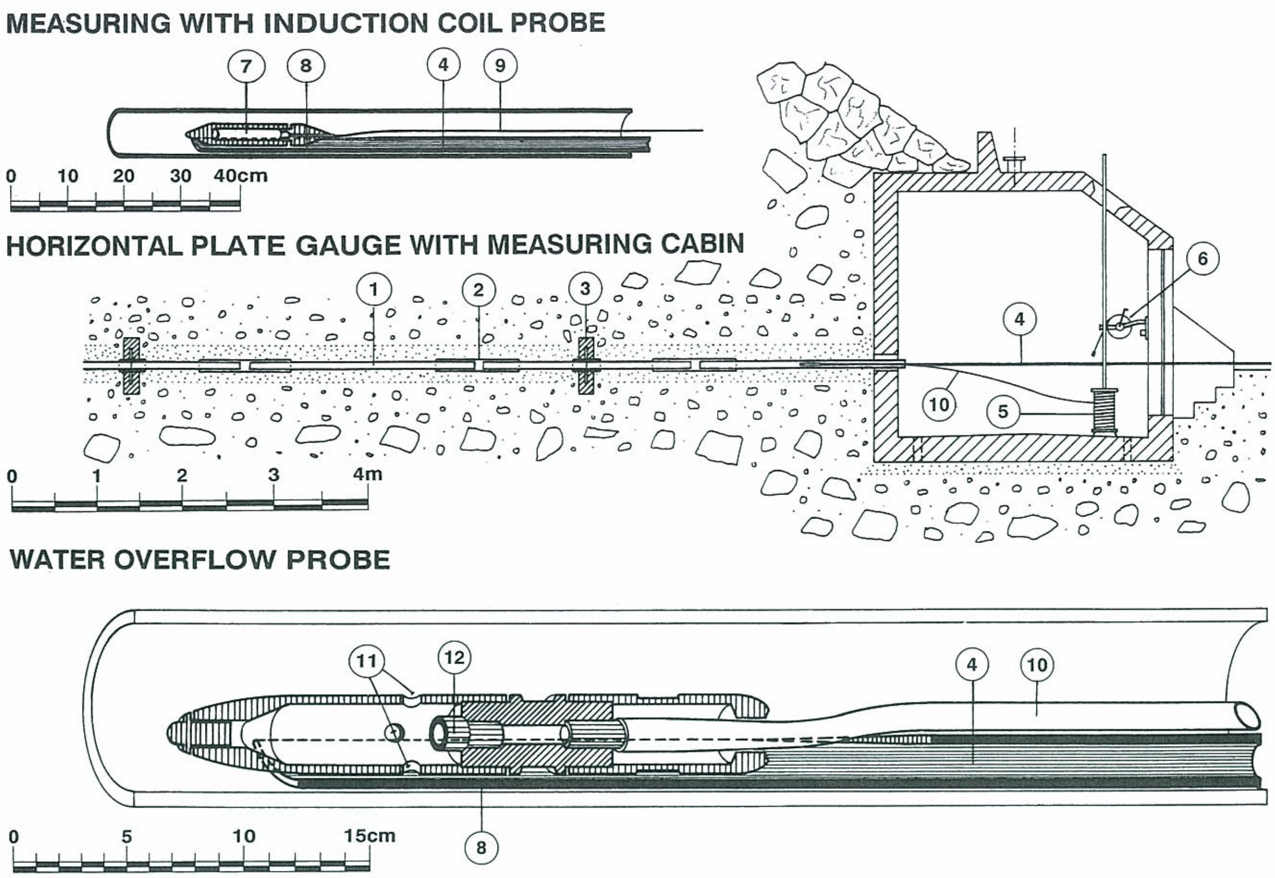
3.1.4 Horizontal plate gauges

At the Gepatsch dam, the induction coil probe and the fluid level were fed into the tube and positioned with the help of a rope, which passed through rings within the tube and around a return pulley at the end of the tube before being connected to the front of the gauge. This device worked very well at first, but in the course of time the ropes broke as a result of pipe constrictions, dirt accumulation and icing. For the Finstertal dam, a number of modifications were therefore made with regard to the tube material and installation, gauge feed and fluid level.

In order to avoid the ingress of dirt and the formation of water pockets, the tube sections were joined together with tightly fitting coupling sleeves, and the tube system laid to provide continuous drainage, i.e. at a slight permanent inclination towards the tube aperture, with allowance being made for expected settlements. The control ropes were replaced by thin PVC rods (25 mm external diameter, 3 mm wall thickness), which do not require guide rings along the tube wall and therefore do not restrict tube cross-section, so that the gauge does not block even when the tube is squashed somewhat. These graduated feed rods are composed of individual pipes, which are stored in the monitoring cabin when not in use. The first section of the feed rod is equipped with a carrier to hold the interchangeable probe. The rod is easily moved even in very long tubes (127 m) and has given excellent results in practice (Fig. 8).

The actual monitoring involves measuring displacements and settlements at each aluminium plate location before proceeding to the next.

With eight gauges installed in the Finstertal dam, i.e. six at maximum cross-section and one each to explore the overburden and fill on the steeply sloping foundation, measuring is very time-consuming. However, this disadvantage is more than offset by the reliability and consistency of the results as well as by the relatively low cost of the system.



(1) TUBE Ø INT = 76.6mm Ø EXT = 90.0mm
(2) COUPLING Ø INT = 93.6mm Ø EXT = 110.0mm
(3) PREFABRICATED CONCRETE MEMBER WITH ALUMINIUM MEASURING PLATE
(4) PLASTIC RODS WITH GRADUATION, SCREWED TOGETHER, FOR INTRODUCING THE MEASURING PROBES
(5) DRUM FOR WATER TUBE AND STAND PIPE
(6) DRUM FOR INDUCTION COIL PROBE
(7) INDUCTION COIL PROBE
(8) CARRYING SPOON FOR MEASURING PROBES
(9) CABLE
(10) WATER SUPPLY TUBE
(11) VENTILATION-HOLES
(12) INTERCHANGEABLE NOZZLE

Figure 8 Finstertal dam – monitoring equipment: horizontal plate gauge

3.1.5 Accessible plumb-line shaft

In order to monitor the inclined core membrane and the adjoining zone, an accessible shaft, with an internal diameter of 1.3 m, was incorporated in the dam in two sections to take account of core inclination. In addition to other instrumentation, the shaft houses two precision plumb-lines permitting continuous telemetering of crest deflections (Fig. 9).

The shaft also affords access to the central area of the dam over its full height, permits precise monitoring for movement at selected points by means of fluid level settlement devices and extensometers, and provides for central laying and the orderly arrangement of all cables and lines from the various instrumentation horizons to the central monitoring chamber.
The shaft is constructed of precast concrete rings initially spaced 10 cm apart by means of metal bolts. The bolts were torch cut as soon as the fill was 3 to 4 m above the ring, which was then kept suspended in the fill by its double-conical shape. The shaft is backfilled with moraine material to protect against seepage water. To prevent fill material from entering the spaces between the precast concrete rings, the gaps were covered with geotextile and wire mesh.

The lower plumb-line 1 measures displacement of the suspension point in the middle of the dam relative to the immovable foundation rock, whereas deflection of the crest point is composed of the readings from plumb-lines 1 and 2 plus differential movement in the connecting gallery. In addition, the upper suspension point can be measured by geodetical means as a check.

Since the plumb-lines available on the market are designed for measuring only small displacements as normally found in concrete dams, it was necessary to develop an additional device to meet the requirements of an embankment dam, i.e. displacements exceeding 100 mm. The system permits exact adjustment of the direct-reading device (co-ordiscope), the telemetering unit and the damping-oil tank through a system of two

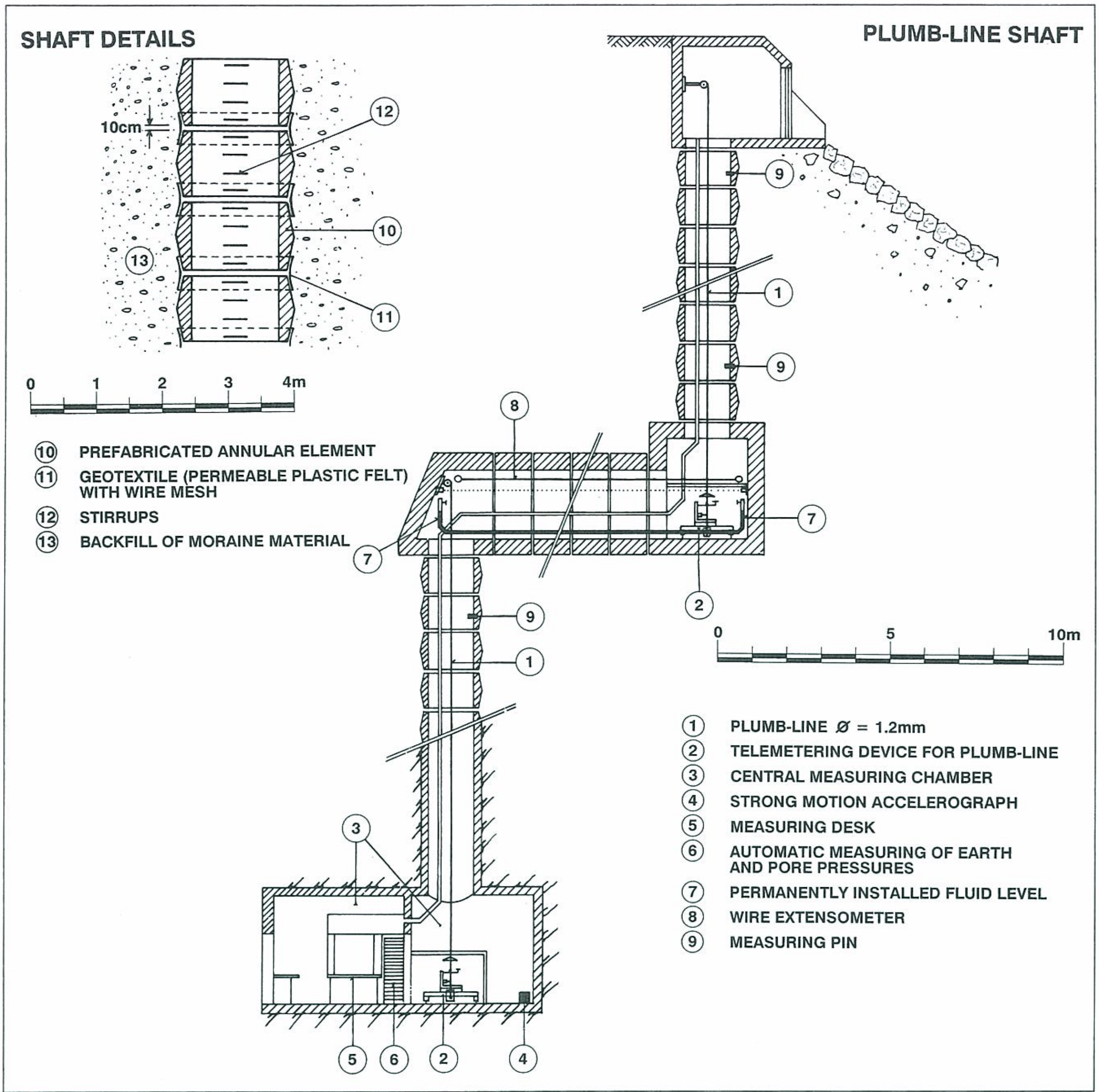


Figure 9 Finstertal dam — monitoring equipment: details of shaft and plumb lines

frames that can be moved in the horizontal plane for the two coordinate directions.

3.1.6 Extensometers

Strains developing between the plumb-line shaft and the asphaltic concrete core, and also absolute displacements are measured by means of two to five-fold extensometers installed at five levels distributed over the height of the dam. For intensified monitoring of crest deformations, extensometer chains were installed in a cross-sectional plane extending from the slopes to the core wall and thus permitting continuous measurement of transverse strain in the crest area.

Additional extensometer chains (one to six-fold) were placed lengthwise in the crest to measure longitudinal strain near the abutments and above pronounced changes in the longitudinal profile of the foundation surface. The strain gauges, designed on the vibrating-wire principle, are totally enclosed and impervious to water under pressure.

3.1.7 Asphaltic concrete core thickness gauge

Extensive investigations, including long-term triaxial tests, were carried out to ascertain and test the optimum mix for the asphaltic concrete core of the Finstertal dam. As there have been few dam surveillance results relating to such laboratory data to date, it was decided to verify the data in the case of the Finstertal dam by measuring for transverse strain actually occurring in the core wall, and to do so without having to pierce the core wall with the measuring device.

Accordingly, a new thickness testing device was developed by TIWAG in collaboration with Professor F. Brandstätter of Innsbruck University. The thickness tester has two strong permanent magnets attached to either side of the core wall, and the magnetic field they create permeates the asphaltic concrete (Fig. 10).

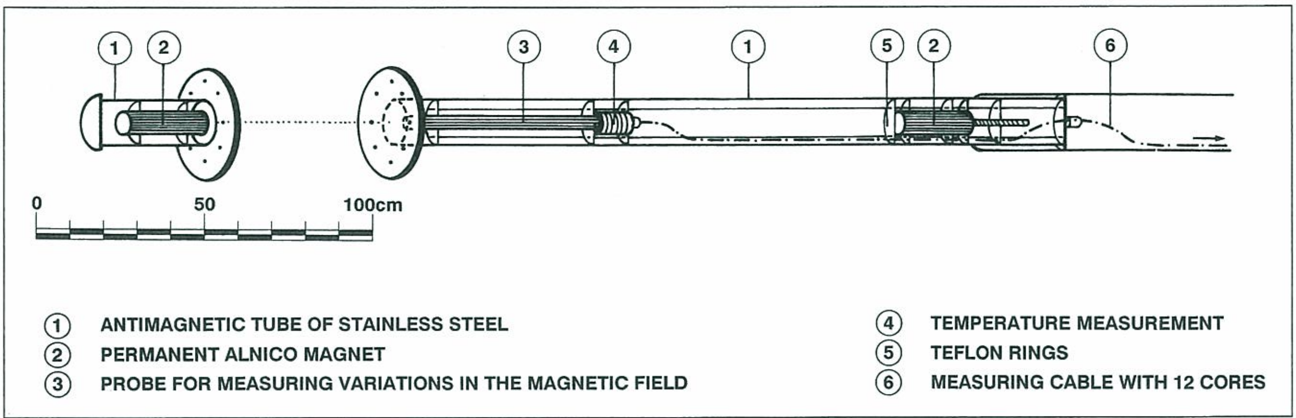
Any variation in the thickness of the core wall creates a corresponding variation in the magnetic field. This permits measurement to within a tenth of a millimetre with a sensor installed on the downstream side between the core wall and the magnet.

the asphaltic concrete core wall, some of them in groups. To permit monitoring of two-dimensional stress conditions, four cells were carefully positioned for measuring planes at 0°, 45°, 90° and 135°. The surplus cell is for mutual checking and as a standby in case of failure.

3.3 Thermometry

In view of the mean annual temperature of about 0 °C prevailing at an altitude of 2 300 m, nine Huggenberger

Figure 10 Finstertal dam – monitoring equipment: magnetic measuring device for core wall deformations



All the components of the gauge are accommodated in two water-tight non-magnetic tubes and are temperature-resistant up to 200 °C. 36 m^2 of copper netting was installed above the thickness tester to ensure uniform distribution of earth currents in case of lightning.

3.1.8 Strain gauge groups

This measuring device records the local deformation components affecting the fill material in the cross-sectional plane. It consists of one vertical, one horizontal and two 45° electrical strain gauges set in an anchor block, the latter for mutual checking and standby in case of a failure. The gauges have a range of 3 m each. Three out of the four groups are installed on the downstream side, while the fourth, with full protection from water under pressure, is located on the upstream side. In conjunction with the earth pressure cells installed at the same locations, they permit observation of the stress-strain relations in the fill material as they vary during construction and operation, and especially creep and saturation behaviour.

3.2 Earth pressure cells

Monitoring stress conditions in large embankments is now common practice, as it yields valuable information on the behaviour of the material and the stability of the dam.

Out of a total of 83 earth pressure cells, 56 are installed on the upstream and downstream sides of the lowest measuring horizon alone. The remaining 27 cells are distributed on the downstream side in the vicinity of

resistance thermometers were placed in the dam fill to permit investigation of temperatures within the fill and particularly in the vicinity of the core wall. Such temperature data is of particular interest because of the temperature-dependent deformation properties of asphaltic concrete.

3.4 Pore water pressure

Part of the downstream shell body of Finstertal dam and several zones in the dam foundation comprise moraine material. Conventional hydraulic pressure cells are used to monitor for excess pore water pressures.

3.5 Seismic activity

Although potential earthquake intensities for the project area are low (MSS VI) – hence the very steep design of the dam slopes – provision was nevertheless made for monitoring the dynamic behaviour of the dam and its foundations. For this purpose, two strong motion accelerographs were installed, one at the crest and one in the central monitoring chamber located in the foundation, to ensure complete automatic recording of seismic events in three planes. They were supplemented by peak-recording accelerographs installed at three locations on the downstream slope.

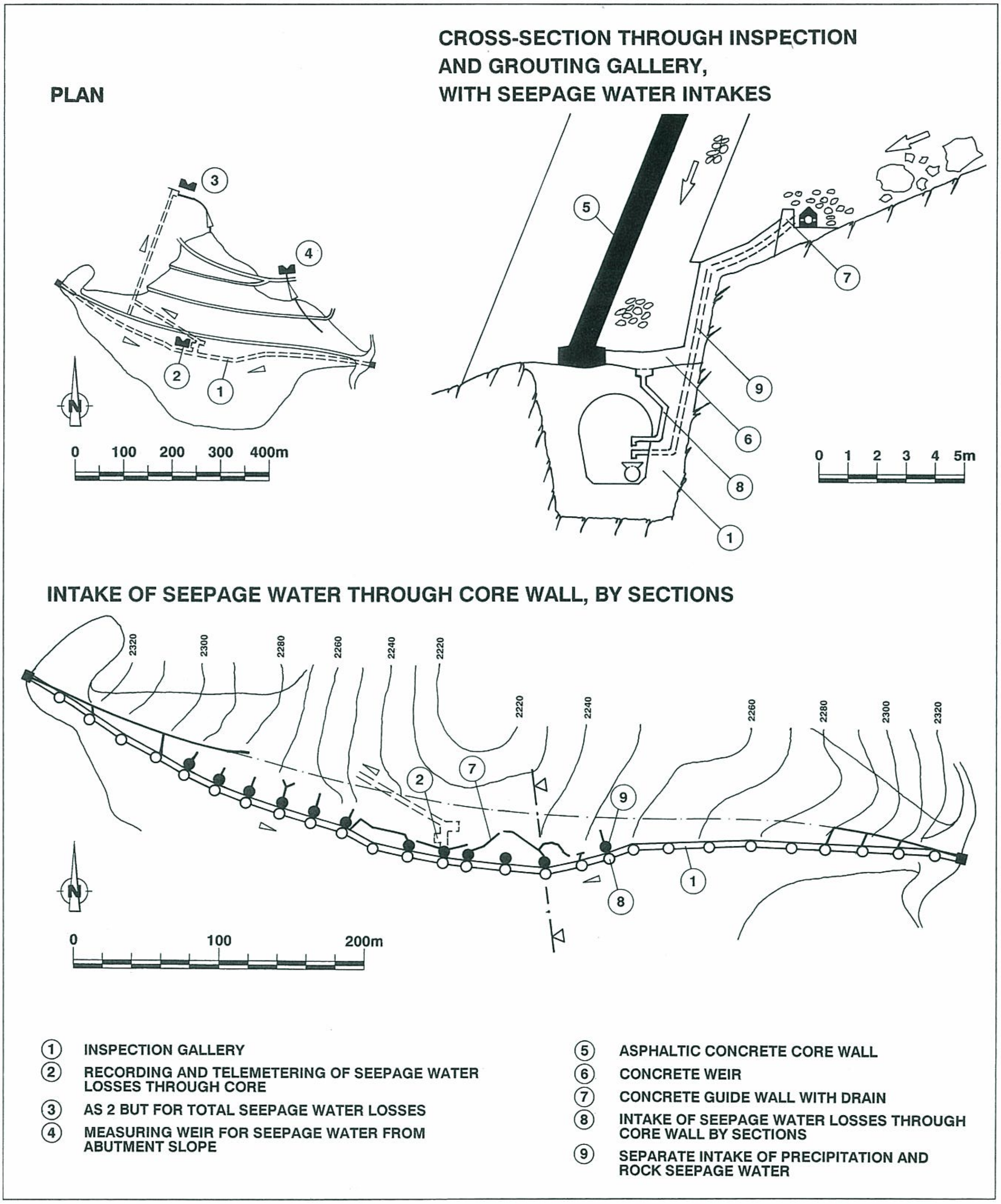
3.6 Seepage water losses

Accurate identification of seepage flows is essential for monitoring the asphaltic concrete core and its foundation contact for seepage water losses, and also – in the most unfavourable case – for locating leakages prior to performing remedial measures. Accordingly, small weirs were installed at intervals of 25 to 30 m on top of the inspection gallery under the drainage zone. They permit the separate intake, by sections of core wall length, of seepage water passing through the core and flowing down the inclined contact of the drainage zone with the impervious moraine zone. The seepage flows collected for the individual sections are measured as they discharge into the inspection gallery. Rock seepage and precipitation water that would flow to the gallery from the lie of the terrain are diverted by concrete walls, also intercepted separately with the help of drain pipes, and measured in the inspection gallery (Fig. 11).

3.7 Piezometers for measuring pore water pressures

The bedrock at the dam site consists of sound, competent and relatively impervious schistous gneiss with amphibolite inclusions. An inspection gallery, concreted

Figure 11 Finstertal dam – monitoring equipment: drainage system



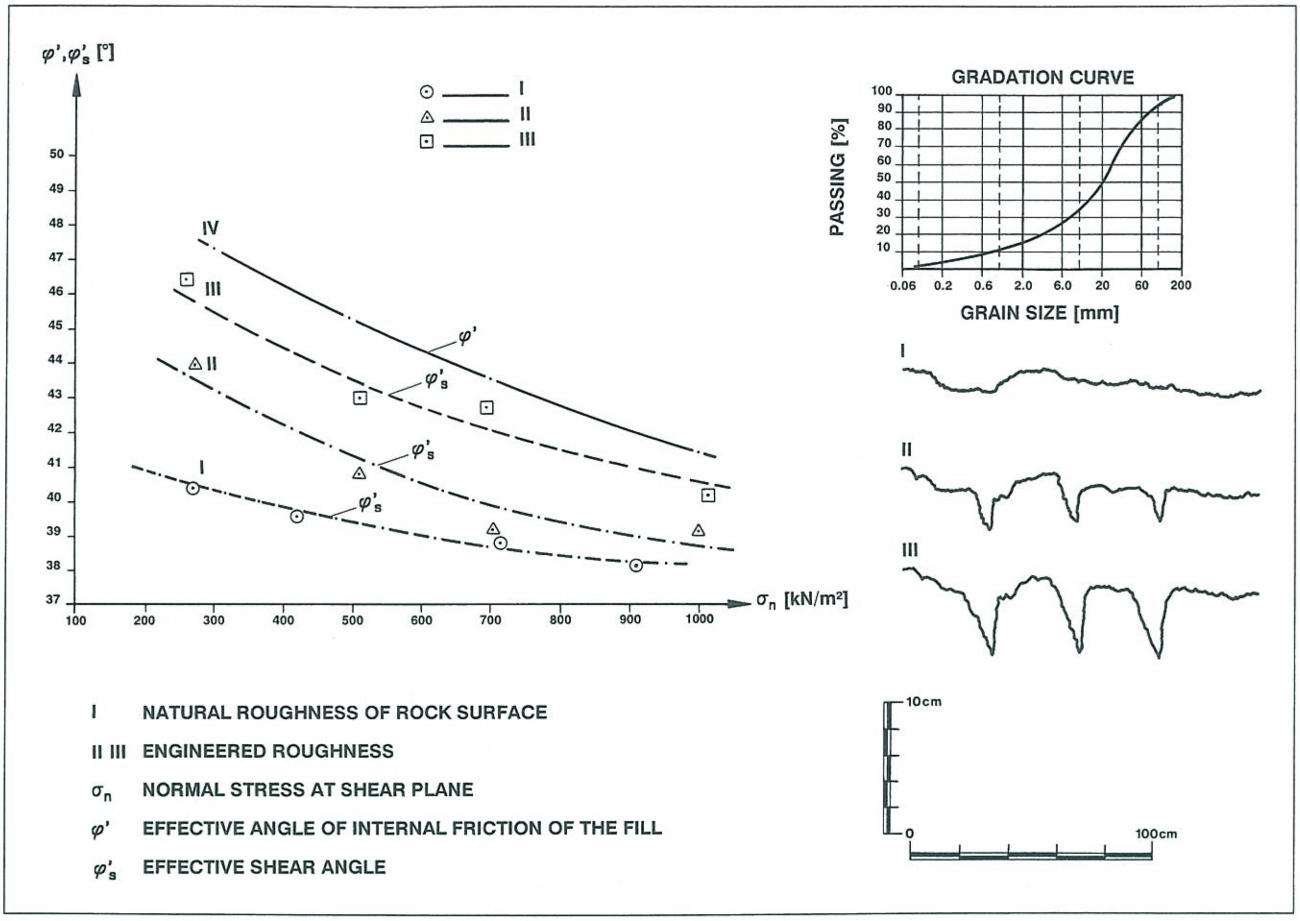
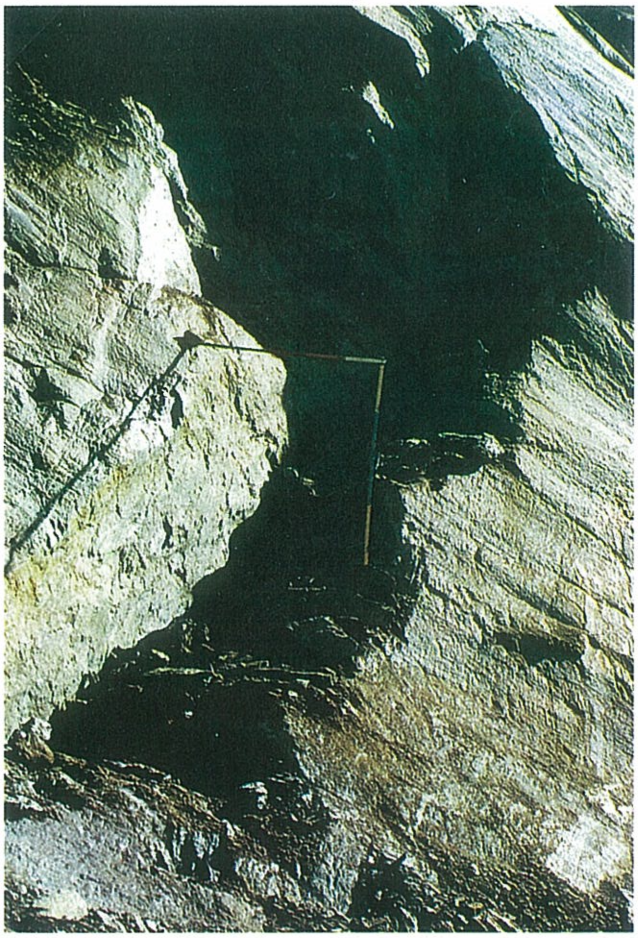


Figure 12 Finstertal dam – results of the large–scale shear tests

Figure 13 Finstertal dam – surface roughening for the bedrock



more or less flush with the dam foundation in a groove blasted into the rock, runs underneath the whole dam from one end of the crest to the other. The gallery also serves as a starting base for the asphaltic concrete core wall as well as for the vertical single-row grout curtain and the radially arranged contact grouting.

The adopted design implies very short seepage paths around the inspection gallery. Therefore, radially arranged piezometers installed at its invert and on the two sides are used to monitor the decrease in piezometric level in this zone.

4 RESEARCH AND DEVELOPMENT

Austria has a long tradition of research and development relating to the construction of big dams, making the results available to an international audience via the reports submitted to the various ICOLD congresses. In addition to the valuable work conducted by the individual hydroelectric power companies, they have also provided funding for a research programme into embankment dams that was established at the Institute of Soil Mechanics, Rock Mechanics and Foundation Engineering at Innsbruck University in 1973. To date the following subjects have been researched or are in progress:

- Shear resistance of loose rock on foundation rock [3]
- Filter criteria for geotextiles [4]
- Stress conditions in fill dams with impervious membranes [5]
- Bearing capacity calculations for fill dams taking

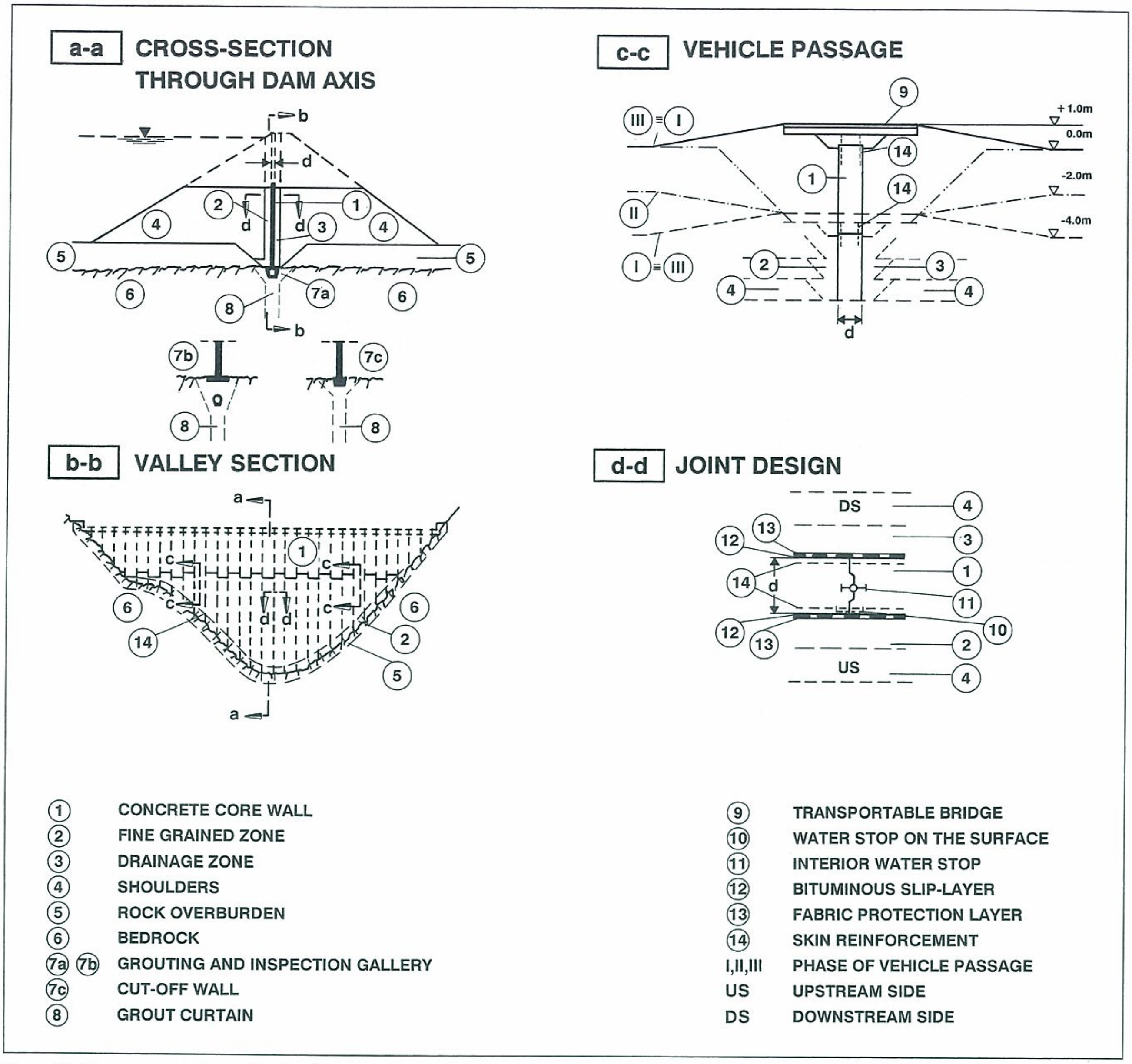


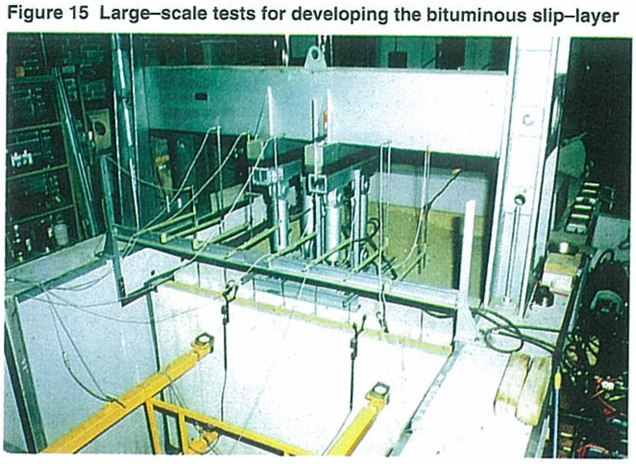
Figure 14 Concrete core dam with slip–layer

account of 3-D effects [6] [7]
- Concrete core membranes with slip layers [8] [9] [10]
- Load transfer in dams [11]
- Prestressed fill dams with impervious foils
- Loading on inspection galleries in embankments (in progress)
- Stability of slopes in reservoirs (in progress) [12]

All these subjects derive from actual questions arising during the planning or construction of fill dams in Austria. To that extent the results are directly relevant for the practice of dam construction. The following are typical examples of the direct utilisation of the research results.

Construction of the Finstertal dam involved fill placement on extensive inclined planes of smooth rock. Large-scale shear tests were conducted to study the influence of the surface roughness of the rock on interface shear resistance, and a coefficient of roughness was derived to express the loss of shear strength in any material on foundation rock. Fig. 12 shows the results for the Finstertal dam, and Fig. 13 the grooves that had to be blasted to ensure reliable load transfer to the foundation rock. A novel feature of the work was the fact that the laboratory tests were conducted with rock slabs with shear areas measuring approx. 2 m².

Figure 15 Large–scale tests for developing the bituminous slip–layer



The Bockhartsee embankment dam was the first example of the incorporation of a concrete core membrane with slip-layers. Fig. 14 illustrates a dam with a central concrete core and bituminous slip-layers incorporated to reduce the main sources of skin friction loading to an acceptable level. This permits such cores to be employed for very high dams. The construction of the Bockhartsee dam offered an opportunity to test the practical application of the relevant research. All data obtained have been carefully evaluated and are now available for further applications. Fig. 15 illustrates the test setup for developing the slip-layer.

The use of a physical model to demonstrate load transfer transverse to the axis of a narrow valley is shown in Figs. 16 and 17. Because of the roughness of the valley flanks, a share of dam load is transferred to the bedrock at a higher level, so that reduced stresses can be expected at the bottom of the valley. A comparative study using data from a physical model and FEM calculations provided more accurate quantification of the influence of surface roughness in the valley flanks. Fig. 17 shows the experimental setup with the 2-D model of a valley section

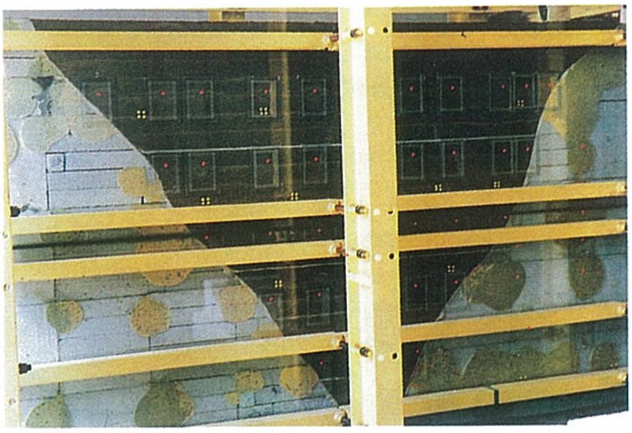
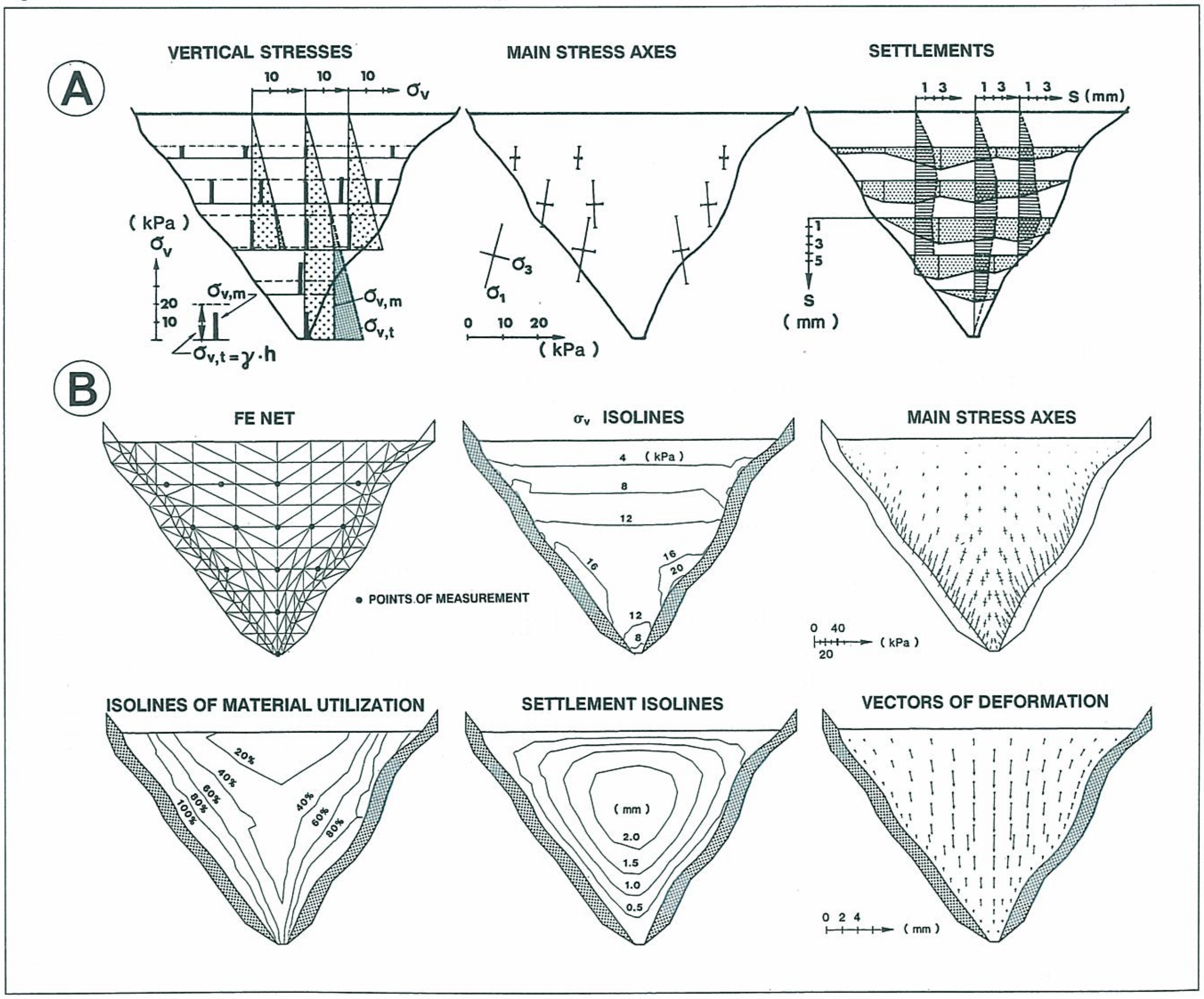


Figure 17 Experimental setup with the 2-D model of a valley section

The results obtained with FEM calculations for the bearing behaviour of embankment dams were checked by post-calculation of the stresses and deformations measured in completed dams. Extensive post-calculations were performed for the Finstertal and Bockhartsee dams, where monitoring has produced a particularly full picture of dam behaviour. Fig. 18 shows the results of FEM analysis based on stress path diagrams for selected points at maximum cross-section of a projected embankment dam. The continuous lines represent the load case for deadweight and the broken lines the load case for first filling up to maximum reservoir

Figure 16 A Load transfer transverse to the axis of the valley – model tests
Figure 16 B Load transfer transverse to the axis of the valley – FEM calculations of model behaviour



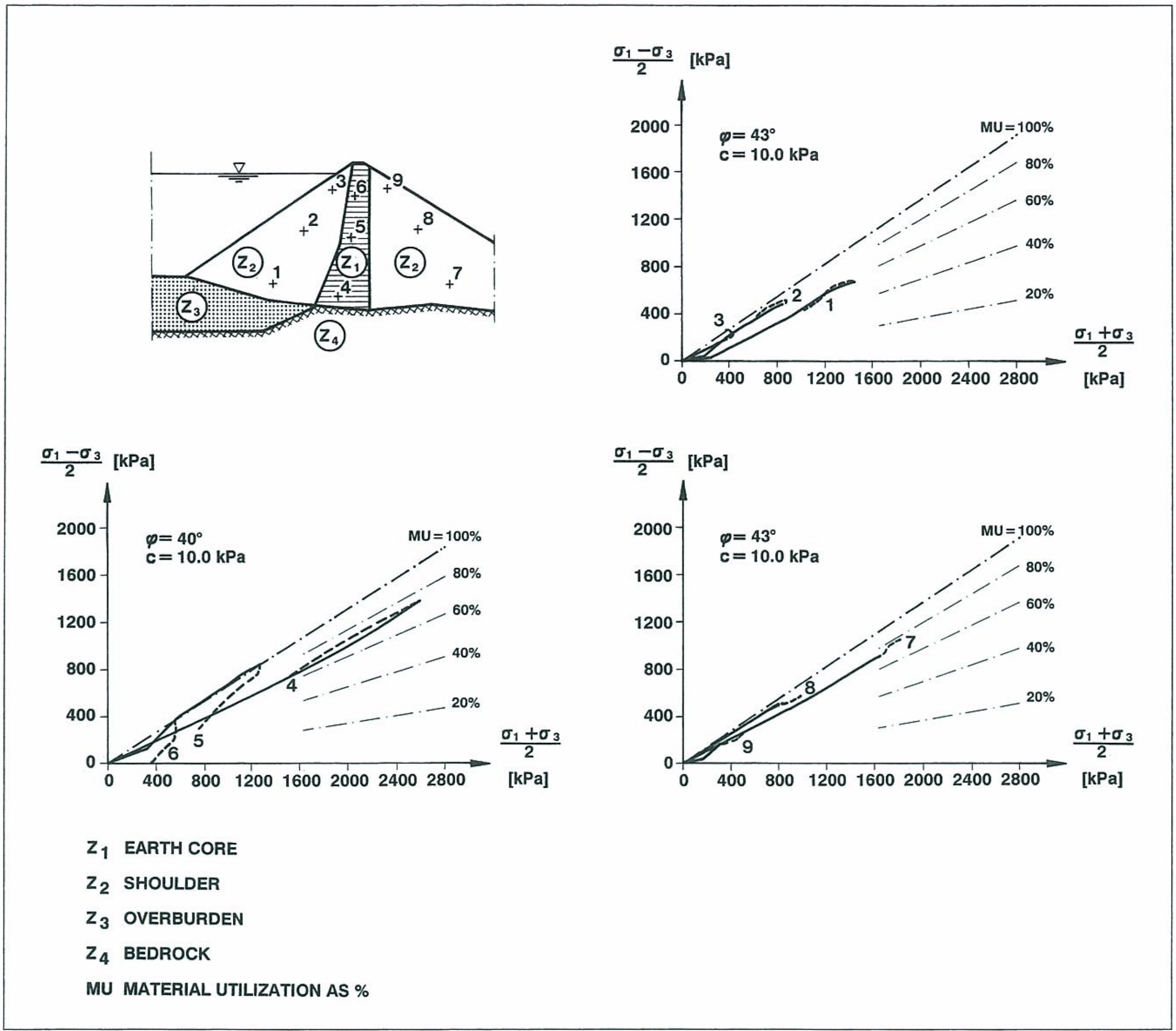


Figure 18 Stress path diagrams for selected points

level. The straight reference lines MU relate to material utilisation. 100% MU means that 100% use is made of the shear strength of the material. As can be seen, this occurs at points 5 and 6 for deadweight and at point 3 for MRL. Full material utilisation at point 3 corresponds to the case of active earth pressure and results from the downstream core deflection caused by water load. It is therefore irrelevant for the stability of the dam. At points 5 and 6, on the other hand, the load case for MRL involves states with very low material utilisation.

As the above examples show, research and development form an integral component of dam engineering in Austria, and the Austrian system of close collaboration between research and practice has produced extremely promising results for future work.

5 APPURTENANT STRUCTURES

In the case of embankment dams, it is essential to incorporate reliable structures to prevent overflow both during construction and on the completed structure. This presupposes accurate calculations for the expected inflows. The decades of run-off data now available in Austria permit the use of stochastic calculation techniques. The return period for a flood event is 5 000 years for the completed dam. In view of the narrow valley locations usually involved, diversion tunnels are normally bored for the construction site, and shafts and tunnels are the most common solution for flood discharge. The intake structures are normally designed as morning glory or uncontrolled spillways. Bolgenach dam (see detailed description) illustrates the less frequent case of gated spillways. In this case, however, the structures are designed to ensure that a 5 000-year flood would not overtop the dam even if the gate mechanisms were to fail.

Bottom outlets are provided for rapid drawdown. In Austria the usual practice is to provide just one deep outlet operated with vertical lift gates or Howell- Bunger valves. In some cases spherical valves are employed to control outflow into the tailwater (e.g. Finstertal dam). The size of the bottom outlet is calculated on the basis of the discharge capacity of the tailwaters. As a safety precaution against blockage of the bottom intake, additional higher level intakes are provided for drawdown and turbine water (e.g. Gepatsch dam) and connected to the same tunnel. Flood discharge and drawdown can also be provided via a common tunnel. As mentioned above, the main data for the ten selected dams are provided in the appendix.

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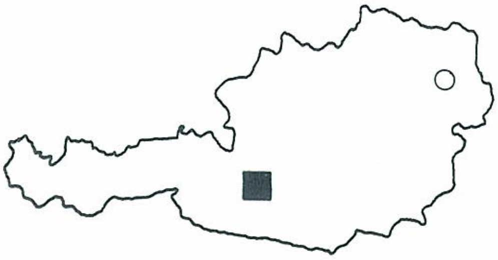
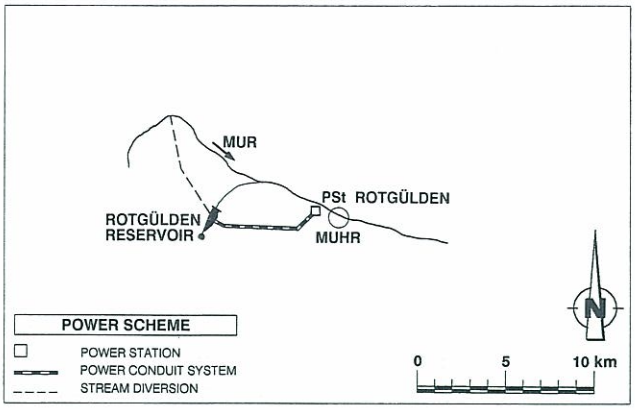
ROTGÜLDENSEE ROCKFILL DAM

Salzburg; Mur
Nearest town: St. Michael/Lungau





MAIN TECHNICAL DATA, Chapter K, 28 (2/22)

General

Power Station	Hintermuhr
Construction Period	1989–1991
Gross Head	601 m
Installed Capacity	2 x 34 = 68 MW
Mean Annual Generation	67 GWh

Dam

Maximum Height	45 m
Crest Length	273 m
Thickness at the Crest	5 m
Maximum Thickness at the Base	146 m
Volume: Excavation (overburden, rock)	99 400 m³
Embankment	346 000 m³

Reservoir

Catchment Area: Natural	9.78 km²
Inflow	16 hm³
Diversions	23.18 km²
Inflow	30.4 hm³
Normal Top Water Level (a.s.l.)	1 733 m
Minimum Operating Level (a.s.l.)	1 670 m
Gross Capacity	15.6 hm³
Live Storage	14.9 hm³
Area flooded by full Reservoir	0.43 km²

Appurtenant Works

Spillway, trough formed overflow spillway	
Capacity	79 m³/s
Bottom Outlet, steelpipe with plunger valve	
Capacity	18.5 m³/s
Power Intake	
Capacity	20.7 m³/s

Inspection gallery under the asphalt concrete core with seepage water monitoring and floodwarning systems

1 GENERAL

To ensure adequate electricity supplies for the Lungau district, Rotgülden power station was built in 1955–57 and the lower Rotgüldensee raised by approx. 15 m for the necessary pondage. Prior to that, in 1952/53, an 8 m high embankment of talus material had been built across the natural lake outlet to utilize the lake for remote storage for the Murfall hydropower plant built in 1922.

In view of the limited degree of electrification in the Lungau at that time and the absence of a high-voltage transmission line to the main consumer centres in Salzburg Province, no further hydropower development took place for many years. With the construction of a 110 kV transmission line for the province in the mid seventies and growing demand for electric energy, however, the decision was taken to achieve more effective utilization of the hydropower potential (as proposed by the water authority in 1957 already) and to submit plans for the Hintermuhr scheme.

Given the small natural catchment of only 9.78 km², this major hydropower development necessitated the incorporation of an extensive adduction system, comprising the upper reaches of the Mur (11.66 km²), the Muritzen (8.79 km²), and the Altenbergbach (2.73 km²), making a total catchment area of 23.18 km².

The first construction stage, from 1982 to 1984, comprised two water intakes and a 6.1 km long free surface tunnel for the Mur and Muritzen diversions. This was followed in a second stage in 1989–1991 by the construction of a rockfill dam to raise the reservoir by approx. 23 m and a bottom outlet, with a resulting increase in live storage to almost 15 hm³.

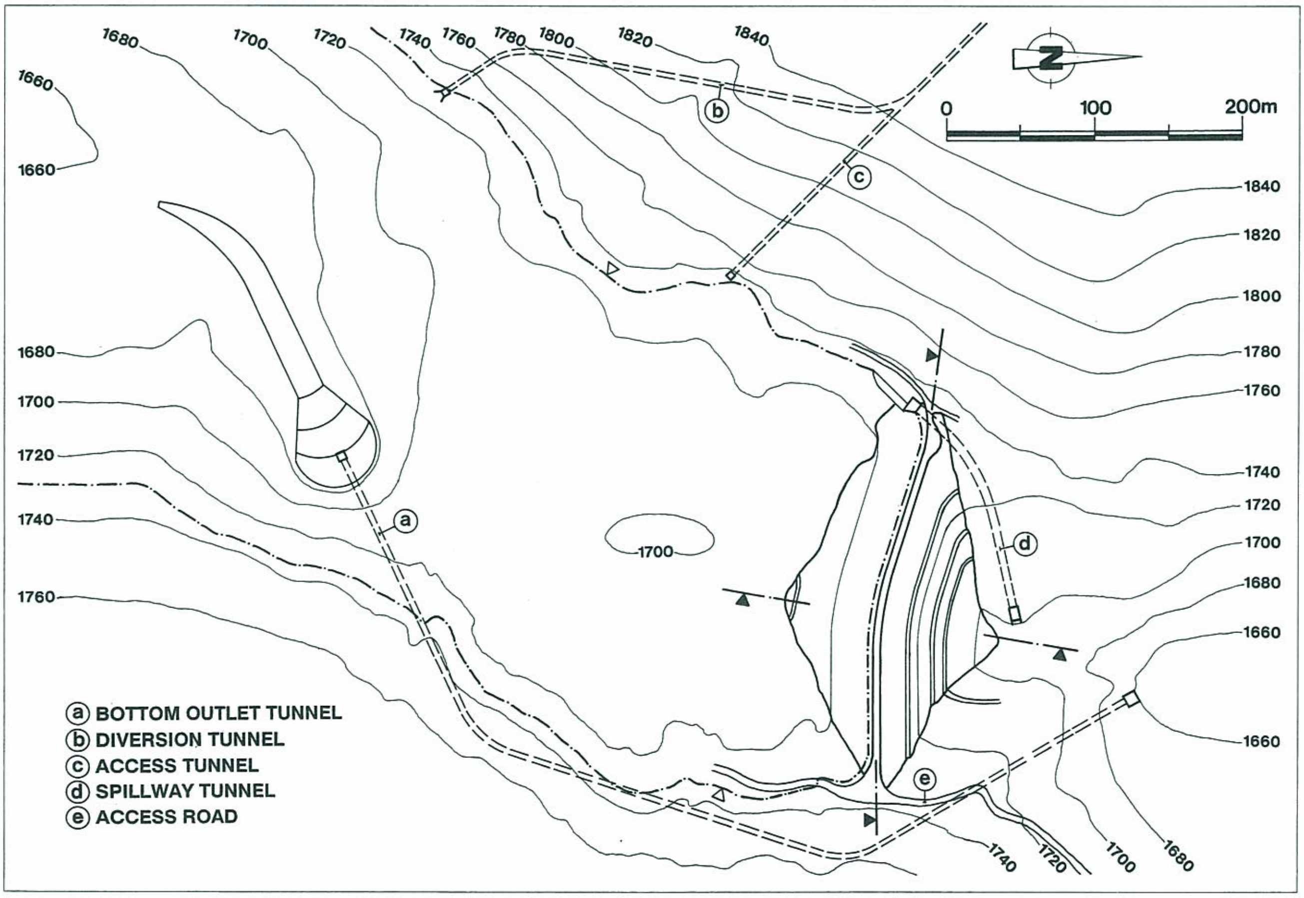
For the new scheme, the power tunnel and inclined steel-lined shaft now lead from the intake structure to an underground powerhouse at Hintermuhr instead of to the old Rotgüldensee power station. This makes for a gross head of 601 m compared with the original 389 m, while the increase in design capacity from 1.1 m³/s to approx. 14 m³/s is accompanied by a rise in generating capacity from 3.1 to 68 MW.

Of the present total annual generation of about 67 GWh, approximately 27 GWh is produced in the high-tariff winter period, representing an improvement in the relationship between winter generation and annual energy from 27% to 40%.

The development project was based on a figure of 1 500 mm for available runoff derived from various annual records and the mean runoff volumes for the upper reaches of the Mur, together with the results of SAFE's own monitoring activities.

As a result of the subsequent imposition of stricter conservation orders and changes to minimum streamflow requirements to reduce flood wave development in the tailwater, available runoff has since been provisionally reduced for the first five years of operation.

Figure 1 Plan

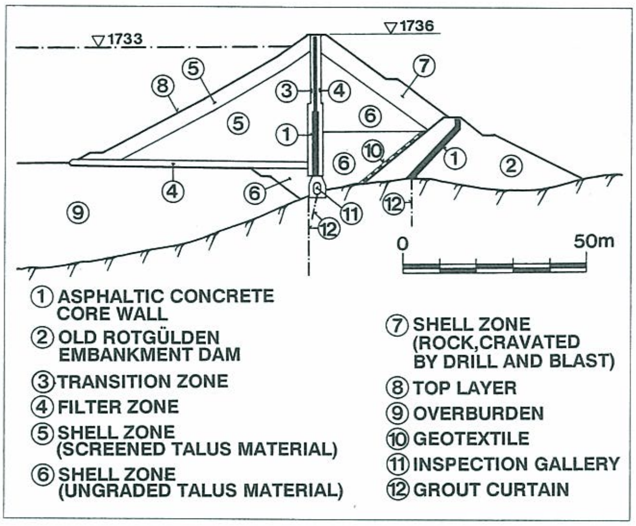


2 GEOLOGY

The dam and the various hydropower installations are located between the Schober and Silbereck mountains to the north of the Hafnereck in the area of the eastern extension of the Hohe Tauern. The geology of the rock massif of the site comprises fine to medium grained migmatite rock of the Rotgülden and Mureck gneiss types.

A natural rock sill with an undulating surface located at 1 700 m above sea-level and closing off an approx. 30 m deep glacial trough was used as the foundation for the old dam, which has now been incorporated in the new embankment.

Figure 2 Cross section



The dam foundation contact area is made up of migmatite granite gneiss with local inclusions of biotite amphibolites. As on the rest of the valley floor, the rock is very compact and almost completely free from discontinuities. On the right flank the rock dips NNW at a medium-steep to steep angle, becoming very steep on the valley floor and left flank.

The rock is sound, with significant signs of weathering limited to a few fault zones running parallel to the valley, which were excavated accordingly.

From the base of the inspection gallery, the upstream dam foundation area comprises dense deposits of fine and medium-grain sand with talus inclusions, reaching a depth of about 30 m at the outermost point of the dam toe. Primary settlement in these deposits under increasing overload during fill placement was only approx. 20–120 mm. At the margins of the glacial trough, sandy sediments alternate with coarse stream and mudflow debris, and talus material. Closer to the surface, the coarser grain fractions increase and include a few individual metre-size boulders. Organic material and zones of pronounced oxidation are also to be found here.

In view of the close proximity of the dam to the border of a seismic hazard zone I, dam stability was calculated on the basis of a response acceleration corresponding to 4% of acceleration g, i.e. for an earthquake intensity of 5 on the Mercalli-Sieberg Scale.

3 DAM

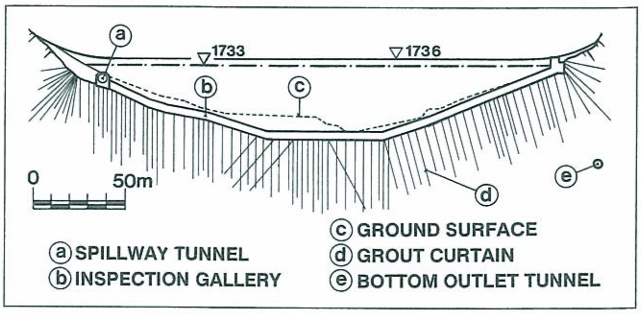
3.1 Design parameters

The original proposal was to raise the old Rotgüldensee rockfill dam built in 1957, which had an inclined asphalt concrete core and a curved boomerang shape on plan, by 30 m – subsequently reduced to 15 m – by adding material on the downstream side so as to permit the whole of the new embankment to be founded on the bedrock.

As a result of significant changes to operating requirements, however, e.g. the need to complete reservoir filling by 1 July, and the equally significant developments made in dam engineering in the meantime, the project was re-submitted with a 23 m higher normal top water level and upstream placement of the new embankment, even though that meant it would be partly founded on old, very dense lacustrine and glacial sediments. Only the new vertical asphalt concrete core and the subjacent asymmetrical inspection gallery are founded on the compact bedrock. Extensive preliminary tests were performed with the fill materials to determine the various parameters such as angle of friction, Proctor density, volume weight, water content, etc., and the results used to determine the required slope inclination, zone dimensions and lifts.

Near the left abutment there is a side channel spillway, which is partly covered both for avalanche protection

Figure 3 Longitudinal section



and for aesthetic reasons. The spillway discharges into an approx. 170 m long reinforced concrete tunnel, which is flat-bottomed with an otherwise circular section. The tunnel is completely covered with rockfill material and terminates at a natural rock face near the old lake outlet.

The approx. 700 m long bottom outlet is provided with a tower-shaped intake structure and an additional intake pipe at the level of the outlet tunnel. A concrete plug is employed for the bottom outlet to negotiate the dam sealing element. In addition to the 1 200 mm bottom outlet pipe, the concrete plug also accommodates the 600 mm power conduit to the old Rotgülden power station.

3.2 Rockfill material

The fill material for the dam body comprised both selected and random talus material in particle sizes of up to 500 mm. The material was excavated from a talus cone near the dam and also from the valley flanks below top water level in the upstream area of the reservoir, so that a quarry proper was not required for Rotgüldensee. Also, the blasted rock from the excavation for the underground powerhouse and for the low level outlet was incorporated in the downstream surface zone of the embankment. The transition and filter zones on either side of the asphalt concrete core were constructed with selected talus material in the grain fractions 0–100 mm upstream and 16–40 mm downstream.

The fill for the main zone of the downstream shoulder was random spoil from the excavation for the power conduit and, as already mentioned, it was covered with blasted rock from the excavation for the underground powerhouse. A riprap of boulders measuring 500–1 000 mm in length was laid on the upstream slope.

The fill material was of good quality, with high block strength and a good angle of friction.

3.3 Construction

Excavation of the sandy overburden in the area of the inspection gallery was followed by concreting for the gallery, with its limited tie-in with the bedrock. The next stage was placement of the fill for the embankment, using lifts of 0.20 m for the core transition zones and 0.8 m elsewhere. Compaction in the main dam body required 4–6 passes of a vibratory roller with dynamic compaction control. Continuous compaction tests were performed, with one test series planned for every 10 000–20 000 m^3 of fill comprising porosity tests, compaction testing with the water substitution method, and seepage testing to determine Kf in the case of impermeable or random fill materials.

Embankment tie-in with the sandy foundation on the upstream side was engineered in the form of a geotextile drainage blanket extending into the abutments. The same technique was employed for the transition to the old dam, with the original riprap left in place.

4 EXPERIENCES

4.1 Instrumentation

A full system of instrumentation was installed, with 12 earth pressure cells for horizontal and vertical loading respectively and a complete geodetic monitoring network comprising 32 surface monuments, 7 crest benchmarks and 10 base gallery benchmarks for regular monitoring, 5 settlement gauges and 3 horizontal plate gauges for monitoring dam behaviour on first impounding and drawdown, and 8 piezometers, 5 piezometer standpipes and 10 pore water pressure cells. Core seepage losses are measured section by section at 9 separate points and recorded together with seepage water in the downstream shoulder, first of all separately for the left and right flanks and then in total using a measuring weir and telemetry.

4.2 Monitoring and special events

At deep levels of the trough forming the natural lake, the granite gneiss bedrock scoured by glacial action is covered by fluvio-glacial sediments of mixed grain and thick layers of silty-sandy lacustrine clay sediments, which were a source of considerable difficulty in the construction of the bottom outlet. In the case of the upstream dam foundation area, on the other hand, the dense deposits of fine-to-medium sands were of such good quality that the planned consolidation measures (e.g. vibratory rolling) were considered superfluous. Nevertheless, with a sand foundation thickness of 18 m and a maximum embankment height of about 20 m, primary settlements of approx. 10 cm have been recorded. Most of these settlements have probably occurred at levels close to the bedrock. The monitoring results required for an analysis of long-term settlement behaviour (secondary settlement) have not yet been collected.

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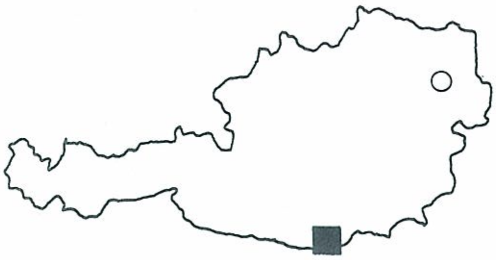
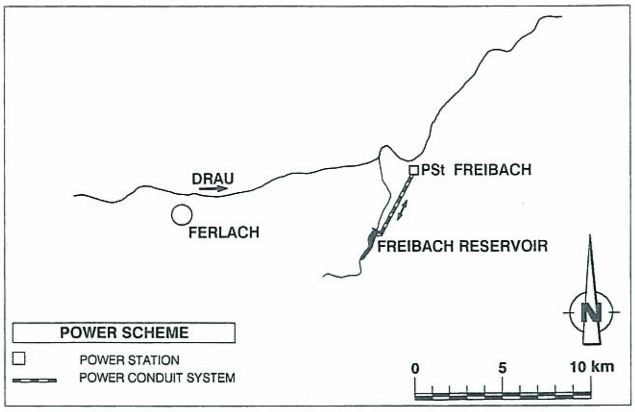
FREIBACH EARTHFILL DAM

Carinthia; Freibach, Drau
Nearest town: Ferlach





MAIN TECHNICAL DATA, Chapter K, 37 (2/24)

General

Power Station	Freibach
Construction Period	1957–1958
Gross Head	333.5 m
Installed Capacity	15 MW
Mean Annual Generation	36 GWh

Reservoir

Catchment Area	44 km^2
Normal Top Water Level (a.s.l.)	729.2 m
Minimum Operating Level (a.s.l.)	705.0 m
Gross Capacity	5.5 hm^3
Live Storage	5.1 hm^3
Area flooded by full Reservoir	0.39 km^2

Dam

Maximum Height	49 m
Crest Length	150 m
Thickness at the Crest	5 m
Volume: Moraine and Rockfill	230 000 m^3

Appurtenant Works

Spillway, uncontrolled, at dam crest	
Capacity	200 m^3/s
Bottom Outlet, gate-controlled pipe with stilling basin	
Capacity	8.20 m^3/s
Power Intake	
Capacity	5.75 m^3/s

1 GENERAL

The Freibach dam and reservoir were constructed in 1957–1959 to utilize the runoff from a catchment lying in the Karawanken massif. Prior to the construction of the project for the development of the Freibach stream, there had been several small hydro stations in this area.

Generation of energy is the only purpose of the Freibach development. The reservoir is operated on a seasonal basis. A limitation has been imposed on water level fluctuations during the summer months for reasons of environmental protection. The power conduit consists of a buried hillside penstock followed by a gallery and an above-ground penstock, which ends in the Freibach power station immediately next to the River Drau.

In the construction of the Freibach dam in the late fifties, it was mainly the substantial geological difficulties that constituted a great challenge, as the extensive foundation treatment requirements had to be met with what technology was available at that time. At the time of construction, the Freibach dam was one of the largest embankments in Austria.

With an average precipitation depth of as much as 1 800 mm p.a. on the slopes of the Karawanken mountains, the relatively small catchment area of 44 km² allows the generation of valuable peak energy, with about 250 hours of full-load operation p.a. The average annual runoff is about 55 million m³, of which more than 50%, i. e. about 30 million m³, occurs during the winter months.

2 GEOLOGY

To the right, the Freibach dam ties into the Jurassic rocks of the northern Karawanken mountains. The left abutment of the dam rests on a near-horizontal series of postglacial gravels, sands and silt deposited by the Drau glacier during the Würm glaciation.

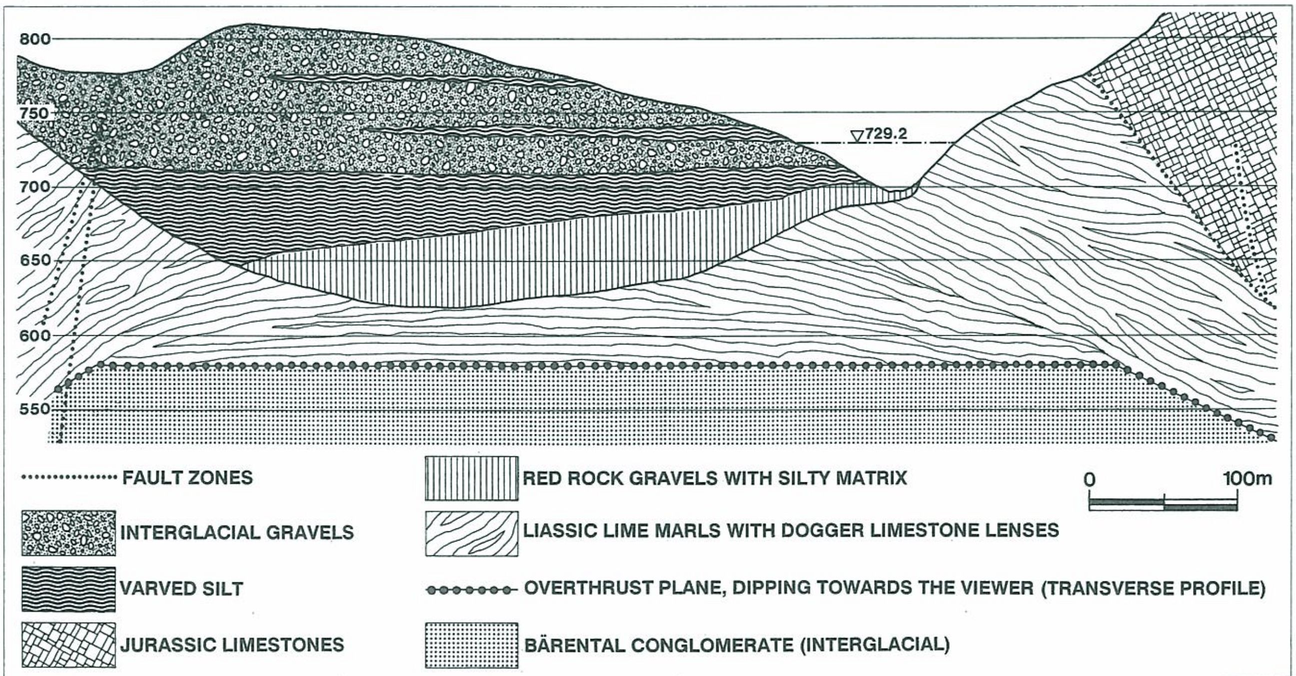
From the foundation of the right-hand dam abutment, in the solid Jurassic rock, two lines of grout holes were sunk to a depth of about 20 m. In the left dam abutment, a clay diaphragm was provided that extends from the spillway about 40 m in a westerly direction to ensure imperviousness in the gravel. Further to the west, this is followed by a grout curtain sunk from a gallery to a depth of 20 m. By means of these measures, it was possible to reduce underseepage through the gravel.

3 DAM

Geological conditions at the dam site precluded any alternative to an embankment. The type adopted was a rockfill dam with an earth core. The core material, screened for 50 mm maximum particle size, was improved by the addition of a maximum of 2% bentonite. The talus material for the adjacent filter and transition zones as well as ungraded talus material for the shoulders was obtained from a borrow in one of the reservoir slopes. A filter layer of gravel or screened talus material was placed on the downstream side. The dam surface was covered with top soil and seeded. For lengthening seepage paths, an impervious blanket of natural material improved with bentonite was placed upstream of the impervious core and cut-off wall. The upstream slope was protected with riprap with rock sizes between 100 and 800 mm.

Geological conditions at the dam site rendered foundation treatment particularly difficult. Whereas on the right-hand side it was possible to treat the rock with a grout curtain sunk from the cut-off wall, the region of rock debris extending approximately from the dam

Figure 1 Geology



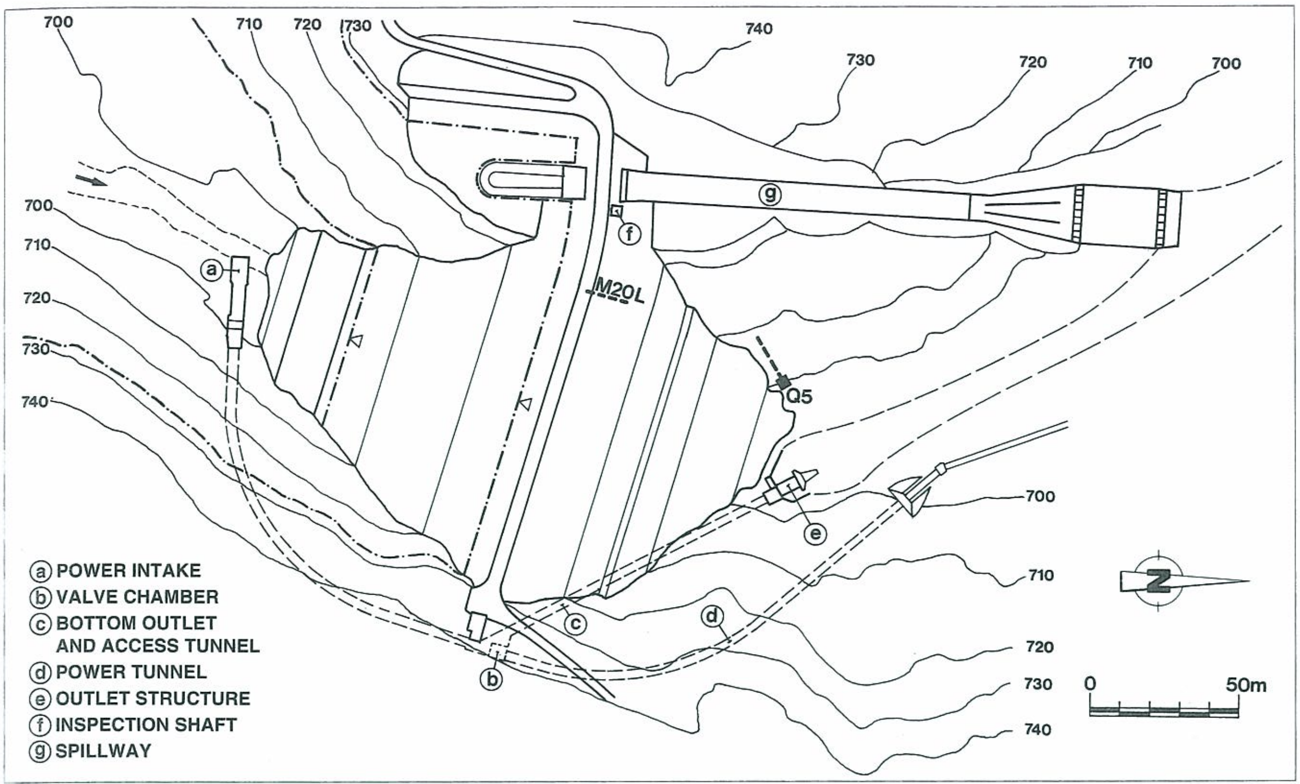
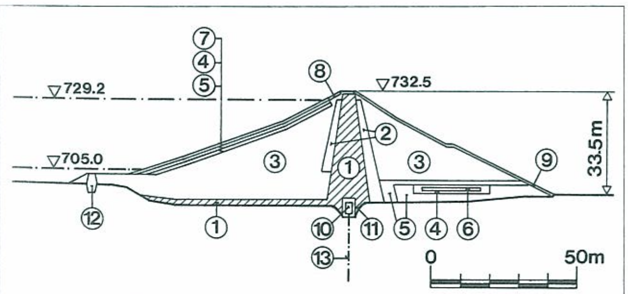


Figure 2 Plan

centre-line to the left called for the provision of a foundation cut-off constructed by mining methods from the bottom upwards. Starting at the spillway and extending to the left, a grout curtain and a silt-filled cut-off trench were provided as impervious elements. In the light of experience gathered during the phase of first filling, as further foundation treatment became necessary in the left-hand flank, an injection gallery was driven for further grouting.

Figure 3 Cross section



Appurtenant structures include a spillway designed to handle a 1 000-year flood with a discharge capacity of 200 m³/s. Both the overflow structure and the chute and ski-jump were studied on hydraulic scale models and have worked satisfactorily during 30 years of operation.

In order to allow rapid drawdown and as an emergency facility, a bottom outlet was provided as a branch from what originally served for diversion during construction and was then converted to a power tunnel. Its maximum discharge capacity is 8.2 m³/s (two 700 mm dia. steel pipes); it is operated by means of a sluice valve.

4 EXPERIENCES

The dam is instrumented with a number of pore water pressure transducers located in the cut-off wall and in the surrounding terrain. Springs whose yield is dependent on the reservoir water level for geological reasons and which are supplied from a very large body of ground water also serve as an indication of seepage conditions.

It has become apparent during operation that it is mainly the yield of the springs in the vicinity of the dam which provides evidence of potential changes in the foundation. But importance is also attached to the piezometers located in the left-hand flank and in the downstream shoulder of the dam as well as downstream of the grout curtain.

Long years' observation of springs, piezometers and pore water pressure transducers has shown the water level in the piezometers to be dependent on the reservoir level and spring yields to have a decreasing tendency.

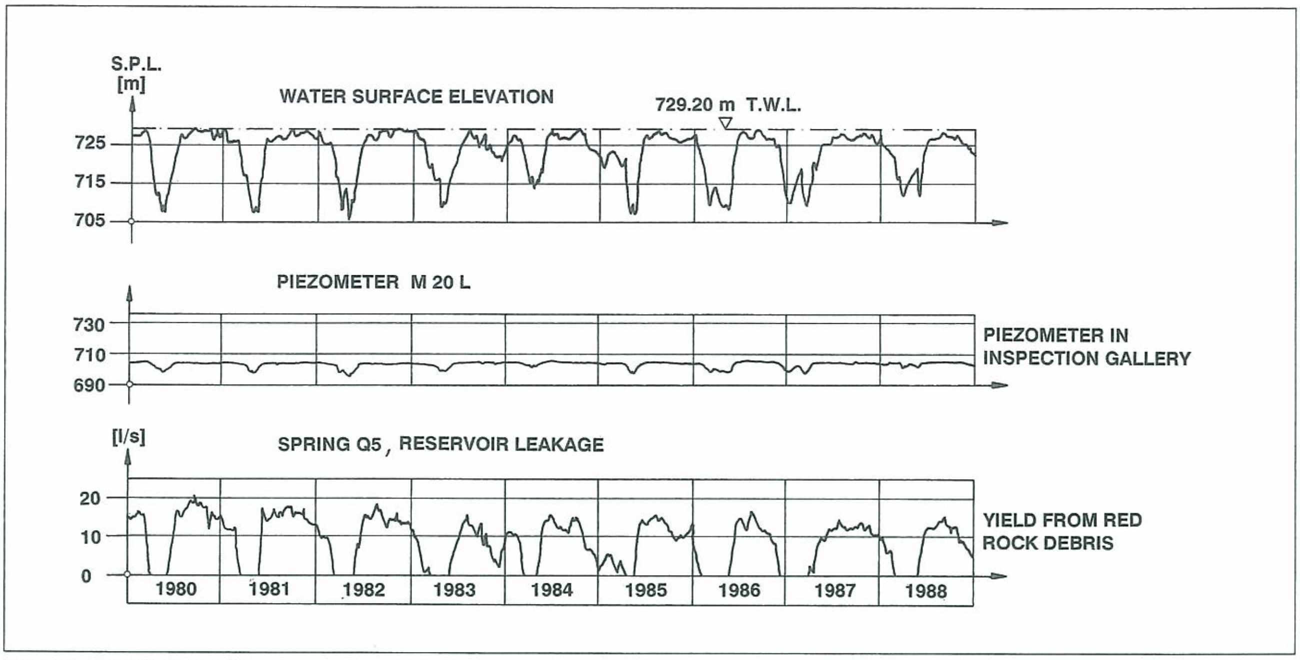


Figure 4 Measuring results

In addition, 10 benchmarks are available for the observation of dam deformations.

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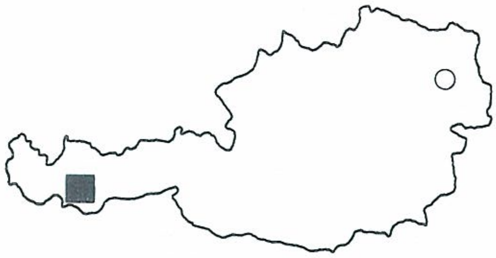
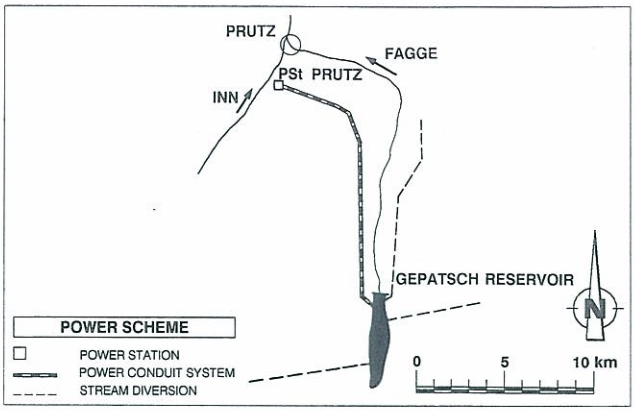
GEPATSCH ROCKFILL DAM

Tyrol; Faggenbach, Inn
Nearest town: Landeck





MAIN TECHNICAL DATA, Chapter K, 39 (3/14)

General

Development	Kaunertal
Power Station	Prutz
Construction Period	1961–1964
Gross Head	861 m
Installed Capacity	392 MW
Mean Annual Generation	620 GWh

Reservoir

Catchment Area: Natural	107 km²
Inflow	125 hm³
Diversions	172 km²
Inflow	177 hm³
Normal Top Water Level (a.s.l.)	1 767 m
Minimum Operating Level (a.s.l.)	1 665 m
Gross Capacity	139 hm³
Live Storage	138 hm³
Area flooded by full Reservoir	2.6 km²

Dam

Maximum Height	153 m
Crest Length	600 m
Thickness at the Crest	11.5 m
Maximum Thickness at the Base	420 m
Volume: Excavation (overburden, rock)	1 000 000 m³
Embankment	7 100 000 m³

Appurtenant Works

Spillway, morning glory	
Capacity	250 m³/s
Bottom Outlet, 2 slide gates	
Capacity	75 m³/s
Power Intake	
Capacity	48 m³/s

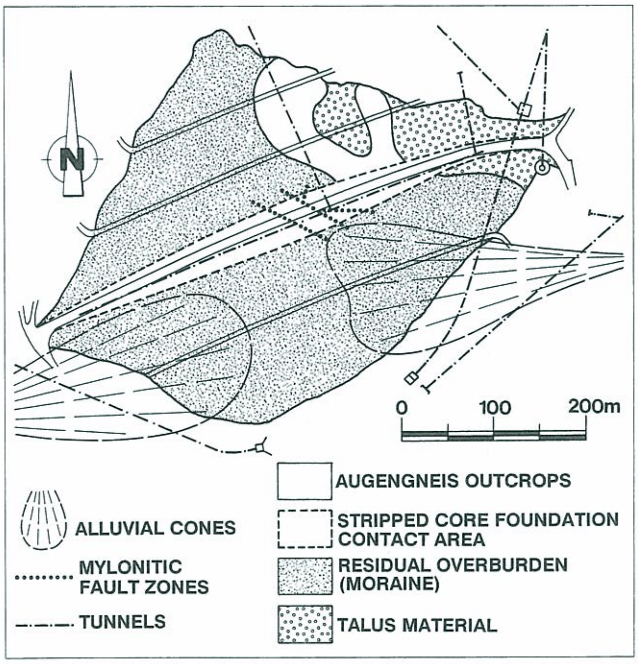
1 GENERAL

The Kaunertal power station is a high-head storage power scheme, with the Gepatsch seasonal reservoir located at the end of Kauner Valley and the Prutz power station in the upper Inn Valley. The scheme was constructed in 1961–1964 for the Tiroler Wasserkraftwerke AG (TIWAG), who produced the plans and supervised the works. A system of adducts and diversions was built to increase the natural catchment area of 107 km² to a total of 279 km², including extensive glacier fields, which ensure a fairly constant runoff throughout the year. The power stage has an average gross head of 861 m. It comprises a 13.2 km long power tunnel, a two-chamber surge tank with reverse-flow control throttle and a 1.9 km long pressure shaft. The 6 km long reservoir with an active storage of 138 million m³ is created by a 153 m high embankment dam with a central earth core of clayey talus material.

2 GEOLOGY

The Gepatsch reservoir is located mainly on a wide formation of foliated gneiss overdeepened to a depth of approx. 70 m as a result of glacial abrasion and terminating in a harder formation of Augengneis (eye gneiss). Following the last glacial period, extensive subsidence caused deformation to parts of the slopes of the valley basin. On first pilot filling, considerable movement occurred in the Hochmais slopes above the western shore, which largely ceased after a total settlement of approx. 7.5 m in the subsequent impounding period. The movement can be explained as re-activation of post-glacial slip in a flat rock body resting on moraine material caused by uplift forces acting on the foot of the slope submerged in the reservoir. In the case of the other subsidence zones, slight creep was also observed. Deformation measurements in the steep reservoir slopes, with annual increments of 0.2 to 3 cm, are indicative of a process of stabilization in the long term. In order to monitor slope movements, a comprehensive instrumentation network was installed. In addition, four exploration galleries were driven for purposes of geological investigation and the installation of monitoring equipment within the subsidence bodies. The geology of the dam site is illustrated in Fig. 1. The Augengneis (eye gneiss) formation traversing the valley, on which the dam is partly founded, is dissected into jointed bodies by flat-lying joints and steeply dipping joints along the length of the valley. Seepage paths were extensive in places and required careful treatment of the bedrock with contact grouting for the whole core foundation plus a grout curtain with a maximum depth of 100 m (grout take: 11.7 kg/m² for the contact grout- ing, and 26 kg/m² for the curtain). A steeply dipping 20–30 m wide mylonitic fault zone passes roughly through the middle of the valley. With the exception of the downstream zone towards the right abutment, the two shoulders of the dam are founded on the overburden, which is up to 30 m thick in places.

Figure 1 Foundation geology



3 DAM

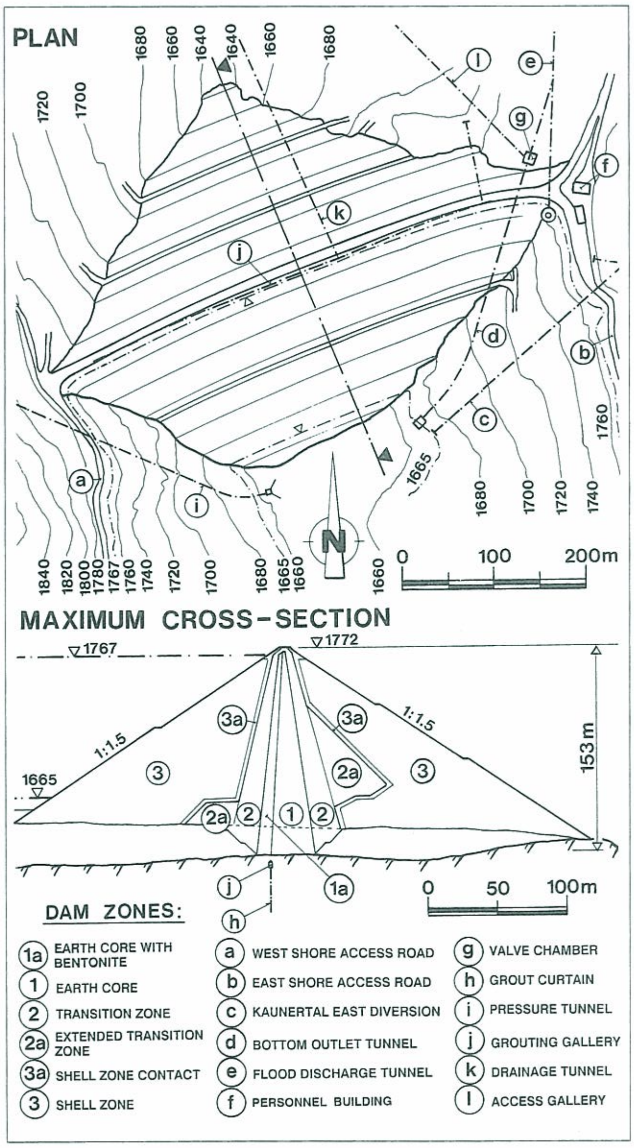
The Gepatsch rockfill dam is a three-zone structure with a central core. It has a maximum height of 153 m, a crest length of approx. 600 m, and a fill volume of 7.1 million m³. The dam axis curves upstream. The fill material for the shoulders was quarried in the immediate vicinity of the dam site. The earth core comprises talus materials screened for a maximum particle size of 80 mm for greater homogeneity and improved imper-

Table 1 Fill specifications

ZONE	MATERIAL	$d_{max.}$ [mm]	LIFT [cm]	FILL DENSITY [t/m³]	POROSITY n [%]	PERMEABILITY k [m/s]	ANGLE OF INTERNAL FRICTION
(1a) Earth core	Talus material with bentonite	80	30	2.34	18.3	5×10^{-10}	35°
(1) Earth core	Talus material	80	30	2.36	17.4	5×10^{-9}	35°
(2) Transition zone	Gravel	200	60	2.41	16.0	$>1 \times 10^{-5}$	40°
(2a) Extended transition zone	Gravel	200	60	2.44	14.0	$>1 \times 10^{-5}$	40°
(3a) Shell zone contact	Quarry-run material	500	200	1.90	28.8	$>1 \times 10^{-3}$	43°
(3) Shell zone	Quarry-run material and oversize particles	1700	200	1.90	28.8	1×10^{-3}	43°

meability. A drying plant was used to reduce the water content of the material to the optimum level for fill placement. For the upstream side of the earth core, 10 kg bentonite was added per ton of fill material in order to further reduce seepage from 5×10^{-9} m/s to 5×10^{-10} m/s. The core zone is protected against internal erosion by thick transition zones on both upstream and downstream faces. These zones comprise well graded sediment deposits taken from the reservoir area. On the downstream side, the transition zone had to be extended to compensate for a shortage of shoulder fill material. The fill material for the shoulders comprised hard Augengneis (eye gneiss) taken from a 200 m high quarry plus oversize particles from the screening plant used for the core material. The transition to the filter zones comprises quarry-run material and gravel mixed in situ. The steep embankments (1:1.5) were given a protective rock dressing for the upstream and downstream slopes. The earth core zones are founded entirely on bedrock, with a foundation contact comprising an approx. 10 cm manually laid layer of plastic clay. Before the clay contact layer was laid, the rock surface was carefully prepared by excavating overhangs, clearing out and grouting open joints, and cleaning with a high-pressure hose.

Figure 2 Plan and maximum cross section



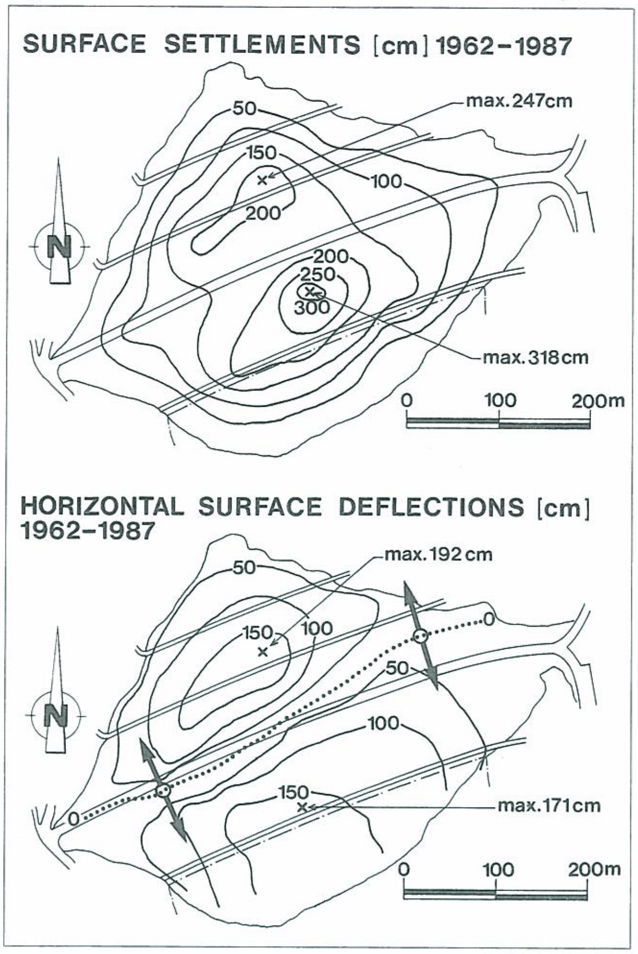
Compaction of the core material and transition zones, where the fill was placed in lifts of 30 and 60 cm respectively, produced low porosities of 14–18.3%. For the fill zones, the material was placed in lifts of 2 m and compaction performed with a 8.5-ton vibratory roller. Compaction tests revealed a porosity of 28.8%. Table 1 lists the specifications and placement data for the fill materials. Grouting work for contact grouting and the grout curtain was performed from a grouting gallery running the full length of the dam. This permitted all-year working. The bottom outlet has a discharge capacity of 75 m^3/s, and flood discharge capacity is 250 m^3/s. The two appurtenant structures are located in the bedrock of the right valley flank. During the construction period, the 1.3 km long bottom outlet tunnel served as a diversion tunnel. It was then converted to a bottom outlet by fitting a control gate with a lifting trashrack. As a safety precaution against silting up and obstruction of the intake, an upper intake, also protected with a trashrack, is provided. A valve chamber with two vertical lift gates is located approx. 320 m along the tunnel. After a further 91 m the tunnels from the bottom outlet and the flood discharge shaft converge, and the combined outlet discharges into a gorge. The bellmouth spillway connects with a shaft with a diameter of 14 m, gradually tapering to 3.4 m. An effective head of 197 m is available for flood discharge. The appurtenant structures also include a number of galleries, which permits all-year access to the grouting gallery and drainage tunnel and to the valve chamber.

4 EXPERIENCES

4.1 Dam monitoring

For the date of construction, unusually comprehensive monitoring equipment was installed to cover all major aspects of dam surveillance. Particular attention was paid to the difficult question of monitoring internal deformations. The horizontal plate gauges specially developed by TIWAG for this purpose have since been used in many subsequent projects. Surface monuments and benchmarks were installed to permit monitoring of surface deformations and crest deflection. In addition, a large number of earth and pore water pressure gauges were installed in order to monitor stress conditions within the fill and decrease of piezometric level within the core. Compaction of the rock foundations under dam load is measured by means of bar extensometers extending from the core foundation contact area into the grouting gallery. In order to be able to monitor the lowering in piezometric levels achieved with the grout curtain, holes were bored upstream and downstream in the rock from the grouting gallery and fitted with pressure gauge connections. The seepage waters draining through the whole of the downstream shell body from the dam core and rock foundations are channelled to a monitoring point for total seepage water loss. Seepage water losses are also recorded on a section by section basis. After 26 years of operation, all monitoring activities are still being continued, albeit to a reduced extent. Even after many years the dam monitoring system has continued to provide new and valuable insights.

Figure 3 ISO-settlements and radial horizontal deflections on plan



4.2 Special events

Given a dam cross-section with core zones that are prone to settlement in the central area flanked by stiffer gravel zones, stress transfer from the central area was to be expected. Extensive data from earth pressure monitoring show that the earth core transmits up to about 50% of its dead weight to the transition zones. This vaulting effect, which is found in varying degrees in almost all embankment dams of this type, has not caused any problems with dam behaviour. The deformations recorded for the dam surface and dam interior – some of them quite considerable with vectorial maxima of 343 and 498 cm respectively – can be explained by the low level of compaction achieved for the quarry-run fill material in the shoulders (Fig. 3). As a consequence of saturation settlements and changing load conditions corresponding to fluctuating reservoir levels, the upstream shoulder is much more prone to deformation than the downstream shoulder. The transverse strains in the central section of the dam have led to superficial longitudinal cracking at the crest, which is basically harmless. In the meantime, apart from a maximum creep of 2 cm in the upper third of the dam, the movements have practically ceased. On the basis of calculations performed with the low residual pore water pressures measured in the earth core, the Gepatsch rockfill dam can be described as being extremely stable. The present low figure of approx. 11 l/s for total seepage water losses from the core zone and the rock foundations also confirms the excellent behaviour of the dam.

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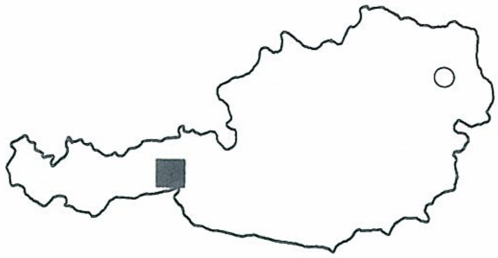
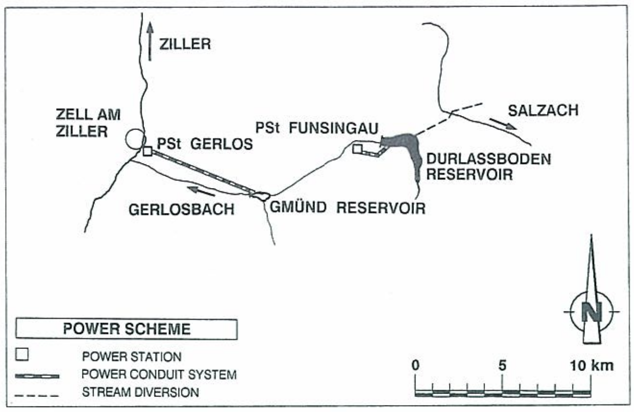
DURLASSBODEN EARTHFILL DAM

Tyrol; Gerlosbach, Ziller, Inn
Nearest town: Zell am Ziller





MAIN TECHNICAL DATA, Chapter K, 42 (3/18)

General

Development	Gerlos
Power Station	Funsingau
Construction Period	1965–1966
Gross Head	118 m
Installed Capacity	25 MW
Mean Annual Generation	25 GWh

Reservoir

Catchment Area: Natural	45 km²
Inflow	64 hm³
Diversions	30 km²
Inflow	35 hm³
Normal Top Water Level (a.s.l.)	1 405 m
Minimum Operating Level (a.s.l.)	1 360 m
Gross Capacity	53.5 hm³
Live Storage	52.0 hm³
Area flooded by full Reservoir	1.9 km²

Dam

Maximum Height	83 m
Crest Length	470 m
Thickness at the Crest	5.5 m
Maximum Thickness at the Base (including stabilizing fill)	340 m
Volume: Excavation (overburden, rock)	230 000 m³
Embankment	2 500 000 m³

Appurtenant Works

Spillway	
Capacity	200 m³/s
Bottom Outlet	
Capacity	130 m³/s
	(limited to 45 m³/s)
Power Intake	
Capacity	26 m³/s

1 GENERAL

1.1 Purpose

The Gerlos power station, in service since 1948, was initially operated on a weekly storage cycle, and in summer even as a run-of-river station. Addition of the annual storage reservoir has substantially improved the operation characteristics of this station, enabling it to meet peak loads all the year round. The winter percentage of generation has been increased from 22% to 47%.

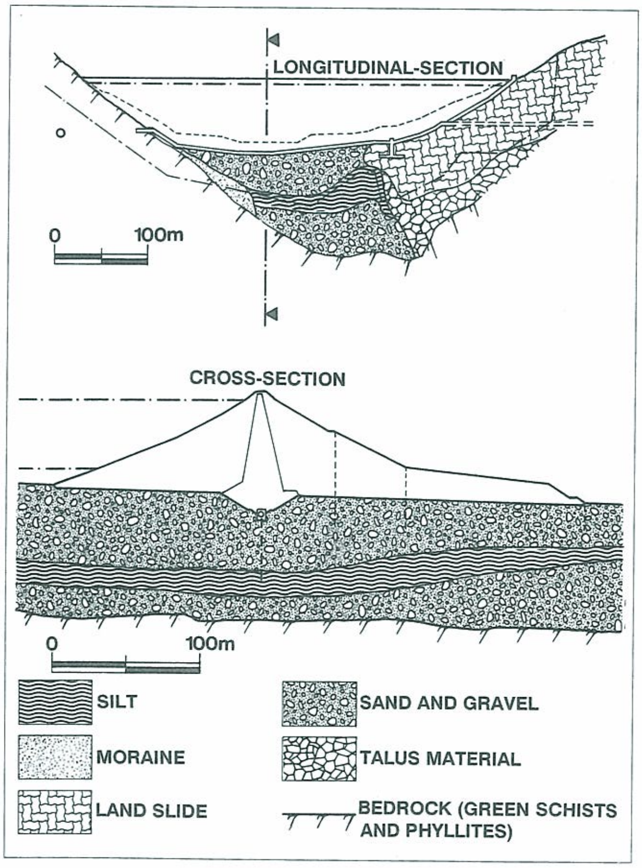
1.2 Hydrology

The natural catchment area ist augmented to 75 km^2 by the diversion of the Salzach and Nadernach streams (total runoff 95 million m^3).

1.3 Project development

Due to the low level of the bedrock at mid-valley (up to 136 m below the surface) the embankment type was the only feasible solution. The poor quality fill material available dictated a flat and sloped earthfill dam. For technical and economic reasons the earth dam with its impervious core in the centre and the grout curtain does not extend down to the rock at mid-valley but ends in the overlying silt layer, which is for the most part impervious. Underseepage into the subjacent gravel and sand layers was taken into account in the design of the dam. Since the site had been judged to be unsuitable for a dam 25 years before the Durlassboden dam is an example of the progress of dam technology.

Figure 1 Geology



2 GEOLOGY

The dam site is located on the northern slopes of the Austrian Central Alps at the northern fringe of the Pennine Tauern Window. At the southern valley flank the insitu rock consists of green schists, with medium steep dips to the north, and black phyllites, and at the northern valley flank it comprises pale green sericite phyllites with gypsum intercalations, and carbonate, quartzite, phyllite and calcareous schists. Over a depth of 136 m the valley is filled with several moraines, gravels and blocks of heterogeneous composition. At 30 m to 50 m below ground there is a 6–30 m thick sandy silt layer. The northern valley flank is characterized by an extensive and mainly firm landslide mass. The grout curtain covers the landslide mass and the sandy silt layer, extending to the moraines of the southern slope and down into the rock foundation.

The dam is located some 28 km SE of the seismically active Inn Valley in an area of low seismic activity.

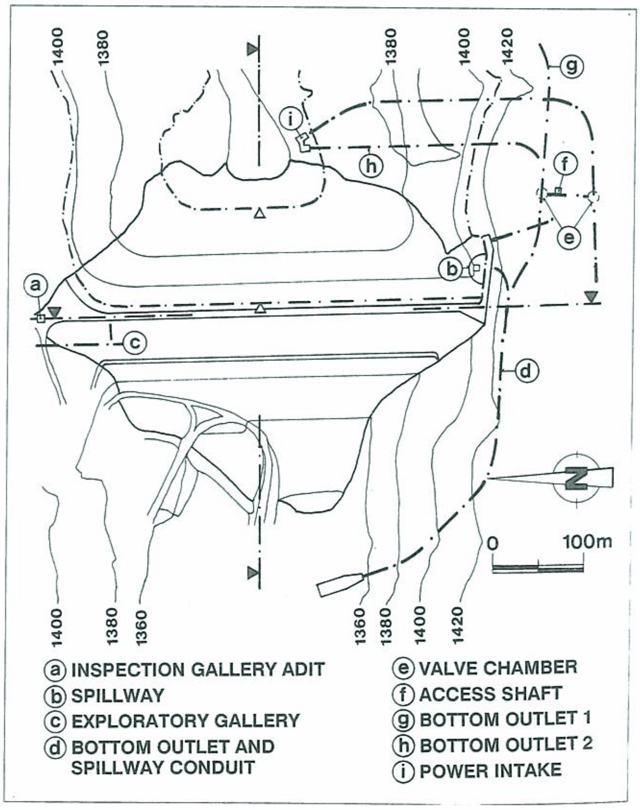
3 DAM

3.1 Site

The dam is founded on:

- exposed green schist with minimal overburden, which is strongly disturbed and heavily disintegrated, at the left flank
- talus material consisting of gravels, sand and silt exceeding 100 m in thickness in the middle of the valley

Figure 2 Plan



- an extensive landslide mass of heavily disintegrated black and calcareous phyllites at the right flank

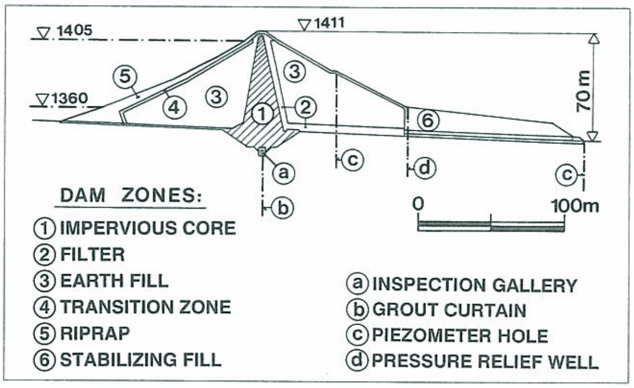
3.2 Zones and materials

Mixed-grained talus material with a maximum grain size of 80 mm was used for the impervious core and the impervious apron, 2 m thick, beneath the upstream shoulder. The core material was dried to a water content of 8%. In order to increase plasticity, 1–2% bentonite was added. The upstream shoulder consists of sandy gravel to El. 1 380 m, and of talus material above this level. Rip-rap using boulders of not less than 350 kg in weight protects the dam face against wave action. The rip-rap rests on a transition layer of mixed stone material. Downstream, a 3 m thick filter zone follows the impervious core. The downstream shoulder consists of talus material to a particle size of 1 m. A stabilizing fill with a filter layer beneath and 7 relief wells protects the dam from failure through piping.

3.3 Foundation treatment

The grout curtain, with an area of 10 600 m² extends 60 m deep into the foundation, ending in the horizontal silt layer. Grout consisted of a clay-cement or bentonite suspension with an Algonit gel admixture to preserve soil deformability. The grout holes were arranged in 8 lines, of which only three in the centre extended to full length. Injection was in 5 m sections, grout acceptance averaging 3.5 m³ per linear metre.

Figure 3 Cross section



4 EXPERIENCES

4.1 Dam monitoring

The effectiveness of the grout curtain is monitored by 16 piezometers and 22 pore pressure meters. For the measurement of hydrostatic pressures upstream and downstream of the grout curtain, 8 drill holes on each side were sunk from the inspection gallery, sloping upstream and downstream respectively. Further instruments, such as measuring weirs to measure the amount of seepage emerging in the north exploratory tunnel and in the inspection gallery, fluid level settlement systems, invar wire extensometers, pole socket extensometers, inverted plumb lines, clinometers, joint gauges, piezometers and geodetic measurement systems provide surveillance of the dam body. In the downstream foreland 12 relief wells and a great number of piezometers of varying lengths were installed to monitor underseepage and thus uplift.

4.2 Results

Readings in the area of the grout curtain show that headwater pressure is reduced by about 70% in the grout curtain. Since the grout curtain was not expected to assure 100% pressure relief, relief wells were provided downstream of the dam in the valley area, drainage holes were sunk from the bottom outlet tunnel at the left flank, and the north exploratory tunnel has been used as a drainage tunnel at the right flank. Seepage emerging in the relief wells slightly increased during the first few years of operation, and remained constant during the following 10 years, before it started to decrease about 5 years ago from 36 l/s to 28 l/s under full hydrostatic load.

Figure 4 Longitudinal section

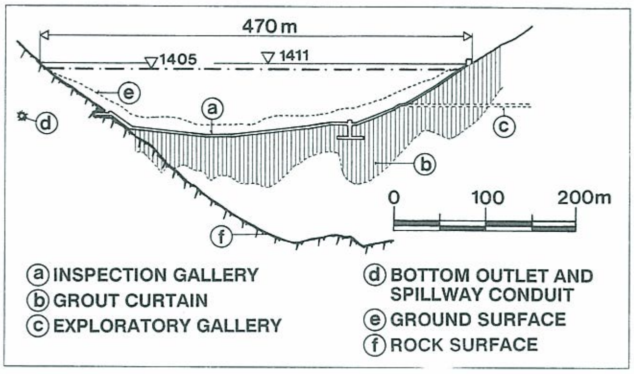
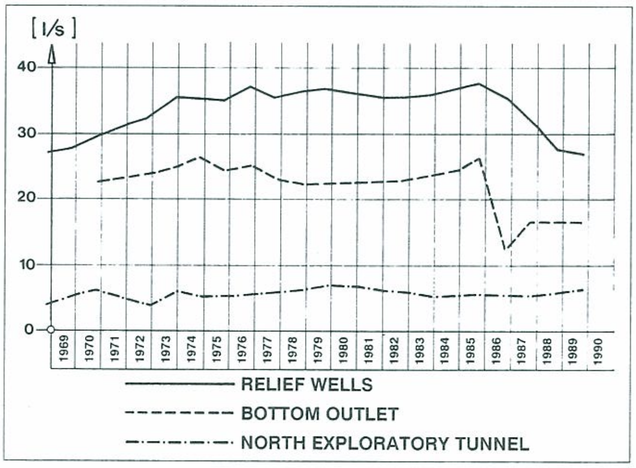


Figure 5 Seepage at top water level



The amount of seepage of < 2 l/s in the inspection gallery suggests practically no permeability for the dam body itself since completion. The foundation beneath mid-dam settled by 95 cm during construction, then by another 34 cm. The mean annual rate of settlement measured at the moment in the inspection gallery is about less than 1 cm and some 1.5 cm at the crest, declining only slowly. In contrast to this, the elastic vertical movements in the bottom gallery and at the crest,

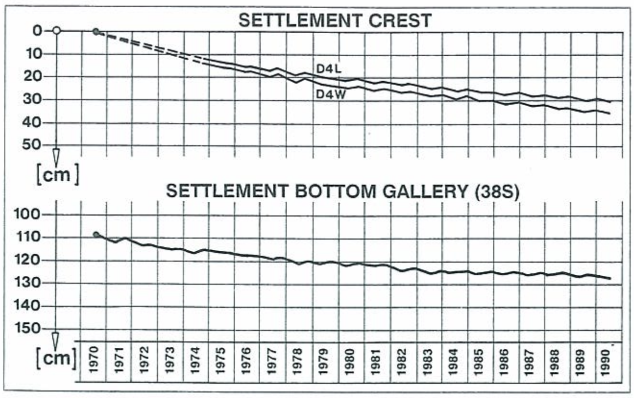


Figure 6 Settlement bottom gallery crest

which vary with the water level, have remained more or less constant at 2 cm during the whole period of operation.

Downstream horizontal displacement of the dam crest at mid-dam averages about 2.5 mm per year, whereas the amplitude, which varies with the water level, has remained practically constant over the last 20 years.

4.3 Special events

In accordance with the dam construction plans the seepage potential in the non-treated gravel-sand layers at great depth is not covered by the drainage facilities arranged in the overlying silt layer in the downstream dam area, or only to a very reduced degree. Consequently, no less than 14% of total potential is still present at a depth of about 20 m in the tailrace, approximately 330 m downstream of the impervious element and thus 120 m from the dam toe. According to the report from the piezometer drillings in 1985, the defined silt horizon disappeared more and more in this area, so that seepage ranging between 30 and 60 l /s between drawdown and topwater level in the mixed-grain gravel sand with low silt content and varying permeability emerges in the form of springs in the tailrace. Safety against failure through piping has been demonstrated, with 1.8 under full water load. In spring 1990 additional relief wells were provided in this area to increase safety against uplift along with a reduction in the number of springs emerging.

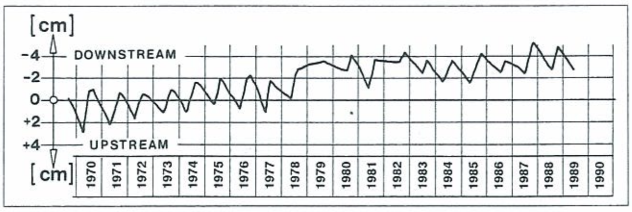


Figure 7 Horizontal displacement of dam crest

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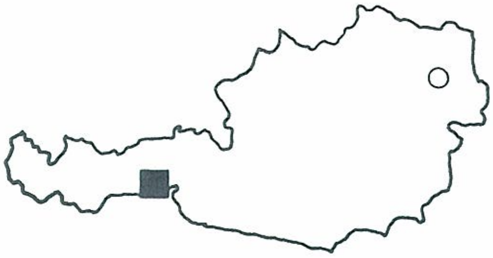
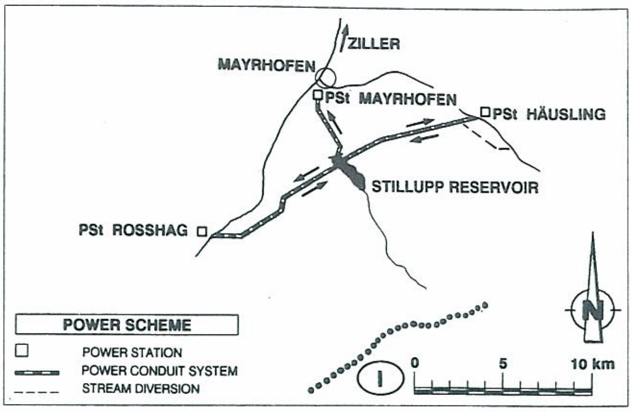
EBERLASTE (STILLUPP) EARTHFILL DAM

Tyrol; Stilluppbach, Ziller
Nearest town: Mayrhofen





MAIN TECHNICAL DATA, Chapter K, 44 (3/23)

General

Development	Zemm-Ziller
Power Station	Mayrhofen
Construction Period	1966 – 1968
Gross Head	470 m
Installed Capacity	345 MW
Mean Annual Generation	613 GWh

Reservoir

Catchment Area: Natural	61 km^2
Inflow	89 hm^3
Diversions and upper Stations	328 km^2
Inflow	478 hm^3
Normal Top Water Level (a.s.l.)	1 120 m
Minimum Operating Level (a.s.l.)	1 106 m
Total Storage	8.2 hm^3
Live Storage	6.9 hm^3
Area flooded by full Reservoir	0.6 km^2

Dam

Maximum Height	28 m
Crest Length	480 m
Thickness at the Crest	6 m
Maximum Thickness at the Base	158 m
Volume: Excavation	90 000 m^3
Dam fill	790 000 m^3

Appurtenant works

Spillway, elliptical with ungated crest	
Capacity at 2.6 m surcharge	450 m^3/s
Bottom Outlet	
Capacity at top storage level	55 m^3/s
Power intake	
Capacity	92 m^3/s

1 GENERAL

1.1 Purpose

Tailwater basin for the Rosshag and Häusling power stations, lower basin for superimposed pumped-storage operation, and weekly-storage reservoir for the Mayrhofen power station. The reservoir has an important key position between the upper station and the lower station of the power scheme, which supplies valuable peak-load energy.

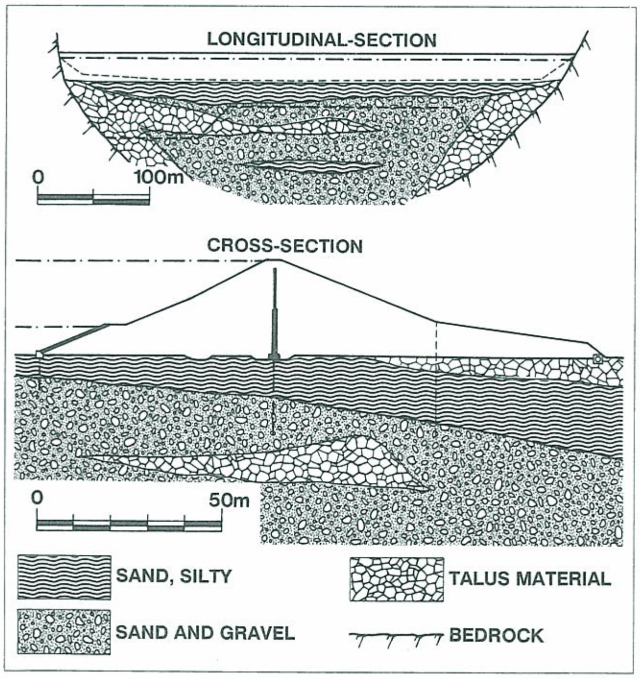
1.2 Hydrology

Runoff from the natural catchment of the Stillupp stream is augmented by that from the catchment above the Rosshag and Häusling power stations as the Stillupp reservoir serves as a lower basin for these stations. In addition four diversions enter the tailrace tunnel from Rosshag and two the tailrace tunnel from Häusling.

2 GEOLOGY

The flanks are composed of biotite and muscovite granitic gneiss and migmatite strata dipping at about 60° towards the middle of the valley, where drill holes attaining a maximum depth of 125 m did not encounter bedrock. The rock channel is filled with very heterogeneous sandy/gravelly river-deposited material which is interlocked with permeable talus material and boulders at the foot of the valley flanks.

Figure 1 Geology

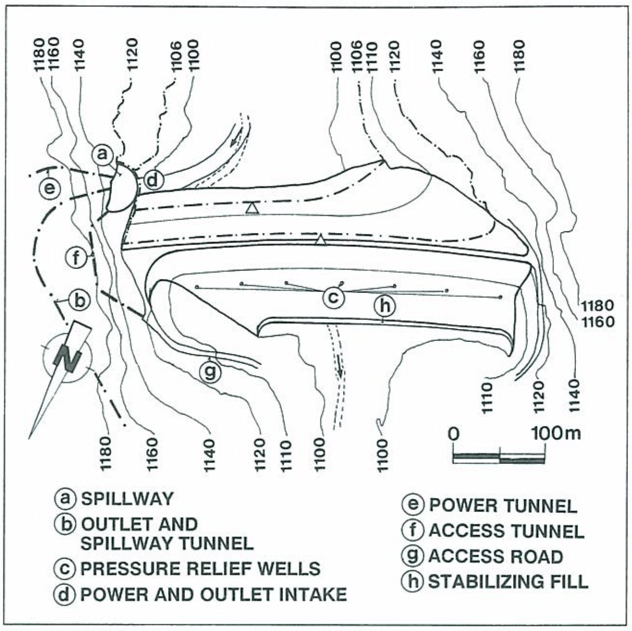


3 DAM

3.1 Site

The location of the dam was more or less dictated by that of the power conduit. Since the alluvium prevailing at the dam site extends down to a depth of more than 150 m, it was impossible to accomplish an economically acceptable connection to an impervious foundation horizon. The alluvium beneath the dam core was therefore sealed to a degree which prevents foundation erosion and reduces seepage.

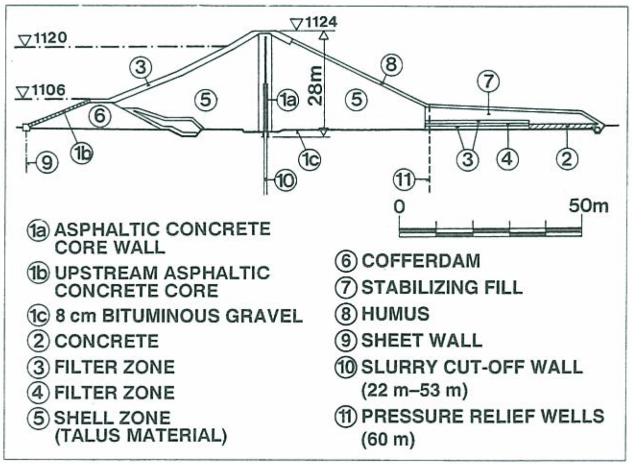
Figure 2 Plan



3.2 Zones and materials

Fill material was obtained from lateral talus cones. In the outer cones of the shoulders the talus material was placed in its natural state; in the interior a graded 0–200 mm size mix was used. The fill material was compacted by vibratory rollers.

Figure 3 Cross section



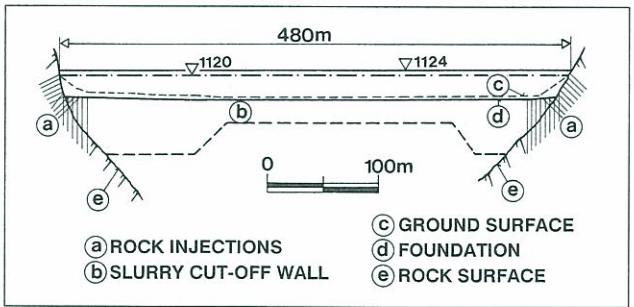
For the first time in Austria, an asphaltic-concrete impervious core 40 to 50 cm thick, and continuous with a slurry-trench foundation cut-off extending to a maximum depth of 52 m, was provided. The impervious core was subjected to substantial stresses due to differential settlement.

The asphaltic concrete was composed of 0–25 mm gravel with 8% limestone added and 7% type B 300 bitumen, and was placed in 25 cm layers by means of a crawler-type vehicle especially designed for this purpose. Placement of the 16 500 t of asphalt mix took 8 months in total. The slurry used for the foundation cut-off consisted of soil-cement with 0–40 mm aggregate, cement, bentonite, and a chemical admixture. Quality tests and material checks during construction were carried out in a special site laboratory.

3.3 Foundation treatment

The cut-off is intended to reduce seepage through the foundation to a reasonable degree. Complete imperviousness was not achieved because of the absence of soil layers of uniform density. In the middle of the valley, there is a 20 m deep surface layer of silts of sufficient density but subject to erosion. The cut-off is 22 m deep in this area. In the permeable cones interspersed with boulders prevailing at the foot of the slopes, the cut-off extends to a depth of 52 m. The slurry-trench cut-off was preferred over a grouted one because of the great heterogeneity of soil types and anticipated settlement. Only the contact between slurry-trench cut-off and bedrock on the valley slopes was grouted in places. 14 700 m² of slurry-trench cut-off were provided in total.

Figure 4 Longitudinal section



For the discharge of seepage water, 15 relief wells 0.25 m in dia. and 60 m deep were sunk on the downstream side of the dam. In order to accomplish adequate safety against ground failure, a 50 m long stabilizing fill was placed.

3.4 Flood relief works

- Elliptical spillway with ungated crest 60 m long, situated on the right hand valley flank, followed by 6.5 m dia. diversion tunnel terminating in the bottom outlet. Discharge capacity is 450 m³/s at 2.60 m surcharge.
- Bottom outlet in the right-hand slope, 580 m long, with intake at El. 1 098 m, 35 m in cross sectional area, equipped with double gate 3.5 m by 2.8 m. Discharge capacity at top storage level is 55 m³/s.
- Required total discharge is 450 m³/s corresponding to 5 m³/s.km² of natural catchment, plus 2 x 50 m³/s turbine discharge from the Rosshag and the Häusling power stations, plus 35 m³/s = 200 l/s.km² from the stream diversions.

3.5 Power Intake

The intake to the power tunnel is combined with the intake to the bottom outlet and the spillway. The intake area is 66 m². A 4.5 m by 3.8 m double gate serves as a shut-off. The power tunnel, 5.20 m in dia., conveys 92 m³/s.

4 EXPERIENCES

4.1 Dam monitoring

Measurements mainly relate to underseepage, which is monitored by 15 relief wells and 14 piezometers. Both the seepage rate – approx. 120 l/s at reservoir surface El. 1 116 m (4 m below top water level), i.e. about one-third of the forecast value – and the solid content are measured. The latter dropped to almost zero during the first year of operation. This self-sealing allowed the reservoir to be filled to top storage level at 1 120 m in 1971, as a result of which seepage losses rose to 150 l/s. In the following years the seepage dropped to 125 l/s due to self-sealing and is now constant. The hydrostatic pressures obtained from piezometer measurements varied but did not exceed the permissible limits. Vertical and horizontal dam deformations are monitored by geodetic means, with crest alignment, traversing and precision levelling. During construction, the foundation settled about 2.20 m in the middle of the valley. Since the end of construction work, secondary settlement of 20 cm has been measured. Settlement of the dam body itself has not exceeded 10 cm, i.e. 0.4% of dam height, since completion of the structure.

4.2 Special events

4.2.1 Pemeability in the slurry-trench cut-off

Seepage emerging from 15 relief wells is measured at every single well and conveyed to a pipe system which also serves as drainage for the downstream shell. Cumulative seepage measurement is also carried out. During the first years of operation the sum of the seepage volumes that emerged in the individual wells was in agreement with total seepage. From 1974 onwards cumulative seepage exceeded the individually measured volumes by 20 l/s. In 1975 this difference rose to 50 l/s. The content of matter suspended in the seepage water, which was measured every two months, had not changed and was below that of the water in the reservoir. One of the piezometers, however, revealed a dis-tinct pressure increase, so that the permeable section of the slurry-trench cut-off could be localized to a certain degree. In autumn 1976 the first drill holes were sunk through the dam down to a depth of 35 m and spaced 15 m apart upstream of the impermeable cut-off, and 20 observation gauges were installed along the axis of the relief wells. Tests with coloured water were carried out at the individual holes with a sufficient time lag and in 5 m sections. In an area exceeding 20 m in width near the original ground surface the shortest time, i.e. 10 minutes, taken to reach the downstream relief wells or observation gauges was recorded, whereas in the unaffected zones it took the coloured water about 8 hours to arrive. Thus the leakage was clearly localized and grouting could be carried out.

4.2.2 Sealing measures adopted

The permeable area was provided with a two-line grout curtain consisting of 41 drill holes extending to a depth of 36 m upstream of the existing cut-off. In the dam body the holes were equipped with solid pipes and in the foundation collar pipes were used. The two lines of holes were arranged at 2.5 m and 4 m from the dam axis. The holes of each line were spaced 3 m apart. On drawdown, which was required for other operational reasons between mid-October and mid-November 1976, the actual grouting was finally carried out. The grout material was a mix of cement, bentonite and water. For the last grout pass out of three 1–1.5% sodium silicate was added to achieve a mix of higher stiffness. Reservoir filling and operational experiences to date confirm the suitability of the measures taken. Piezometer P 5 again indicates normal water pressures, and the difference between total seepages measured at the individual wells and the volume obtained from the cumulative measuring location does not exceed the permitted tolerance.

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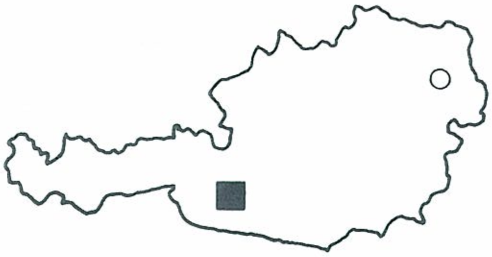
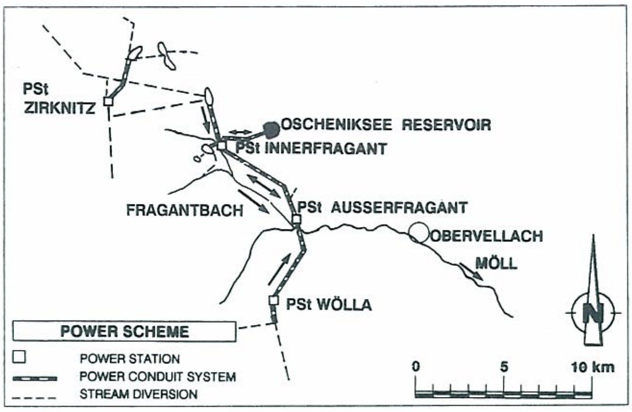
OSCHENIKSEE ROCKFILL DAM

Carinthia; Fragant, Möll, Drau
Nearest town: Obervellach





MAIN TECHNICAL DATA, Chapter K, 49 (4/7)

General

Development	Fragant-Oscheniksee
Power Station	Innerfragant
Construction Period	4 phases, 1971–1979
Gross Head	1 186 m
Installed Capacity	
Generating Mode	108 MW
Pumping Mode	100 MW
Mean Annual Generation	82.2 GWh

Dam

Maximum Height	116 m
Crest Length	530 m
Thickness at the Crest	5 m
Volume: Rockfill	2 100 000 m³
Upstream Asphalt Facing	41 000 m²

Reservoir

Catchment Area: Natural	1.7 km²
Diversions, Pumping	110 km²
Normal Top Water Level (a.s.l.)	2 391 m
Minimum Operating Level (a.s.l.)	2 245 m
Gross Capacity	34 hm³
Live Storage	33 hm³
Area flooded by full Reservoir	0.43 km²

Appurtenant Works

Spillway, uncontrolled diversion	
Capacity	6.4 m³/s
Bottom Outlet, gate-controlled pipe	
Capacity	1.5 m³/s
Power Intake	
Capacity: Generating Mode	10.2 m³/s
Pumping Mode	11.3 m³/s

1 GENERAL

The Fragant group of power schemes develops the runoff from the southern slope of the Hohe Tauern and the northern slope of the Kreuzeck mountains at several levels. Both the power stations and the main reservoirs were constructed in several stages designed to meet the increasing power requirements in the period from 1967 to 1984. At present, the Fragant group has a maximum capacity of 334 MW and generates an average of 549 GWh p.a. The development includes five power stations with five large reservoirs and several small basins.

The main dams are situated at altitudes between 2 300 and 2 500 m a.s.l. They are all embankments of more or less the same design as the Oscheniksee dam described below. The runoff from the Tauern massif is conveyed in tunnels and penstocks to the Zirknitz power station at El. 1 728 m. Passing through a compensating basin, the discharge from Zirknitz is utilized in the Innerfragant main station at El. 1 205 m, which also receives the discharge from the Oscheniksee power station. Discharge from Oscheniksee is finally utilized in the Ausserfragant power station El. 713 m.

Runoff from the Kreuzeck mountains is utilized in the Wölla power station and then may either be conveyed to the Ausserfragant power station for utilization or diverted to the Innerfragant power station for pumping. Thus, the Fragant group of power schemes offers a wide choice for utilizing the available runoff by storage, intermediate storage, direct utilization and pumping.

2 GEOLOGY

Grosser Oscheniksee, a natural lake, is situated on the south-western slope of the Sonnblick massif in the Hohe Tauern mountains. With a natural depth of 116 m, it is one of the deepest natural lakes in the Eastern Alps.

The geological setting of the dam site is formed by very compact granitic gneisses and granites. Fault zones cut through the lake basin in the north-east to south-west (longitudinal axis) and north to south directions. Due to the high altitude of the site, scree slopes have formed, which descend to the lake mainly from the north-eastern mountain flank. Fossil ice found among the boulders are glacial remnants.

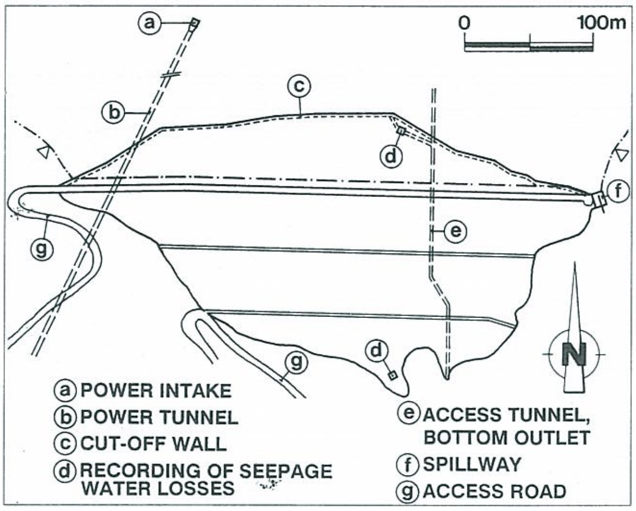
The rock bar at the outlet of the lake, actually the cirque threshold, was used as a foundation for the embankment. Oscheniksee dam rests over almost its entire central area on bedrock of flat-bedded granitic gneiss. A fault passing through the southern part of the dam foundation was closed with a concrete plug. Both sides of the dam tie into irregular relief formed by varying moraine materials.

A single-line grout curtain was sufficent due to the excellent mechanical properties of the rock. Its maximum depth is 25 m. About 70 to 75 kg of cement was pumped in per linear metre of grout hole.

3 DAM

The dam was constructed between 1971 and 1979 in four stages. The stage-wise construction scheme was adopted in view of the relatively short construction season at the high-level site and in order to enable immediate utilization of storage capacity. This concept was a determining factor in the choice of the dam type and the asphalt diaphragm as an impervious element that can be extended at any time.

Figure 1 Plan



Parts of the foundation surfaces of the first and second construction stages were covered to a maximum depth of 12 m by moraine material rich in boulders. The fill materials used for the first two construction stages, to El. 2 351 m and 2 358 m a.s.l., were first moraine material and then quarried granitic gneiss.

During the third construction stage between 1974 and 1976 – involving a fill volume of 1 million m^3 – quarried rock of high strength was the only material used.

For the final construction stage, completed in 1979, it was necessary to pump out the natural Kleine Oschenik Lake situated in the contact area of the dam. First a filter was placed in the lake basin. Then the fourth-stage dam toe was placed in moraine material with a maximum particle size of 1 m in maximum layers of 1.3 m. The main shoulder was constructed in quarried material and was also placed in layers of 1.3 m with a maximum particle size of 1 m. The rockfill was compacted by 450-kN (dynamic) vibratory rollers.

A filter of 50 cm thickness and a maximum particle size of 200 mm was generally provided between the main fill and the single-layer asphalt diaphragm. Before placing the binder course, the surface of the filter had to be

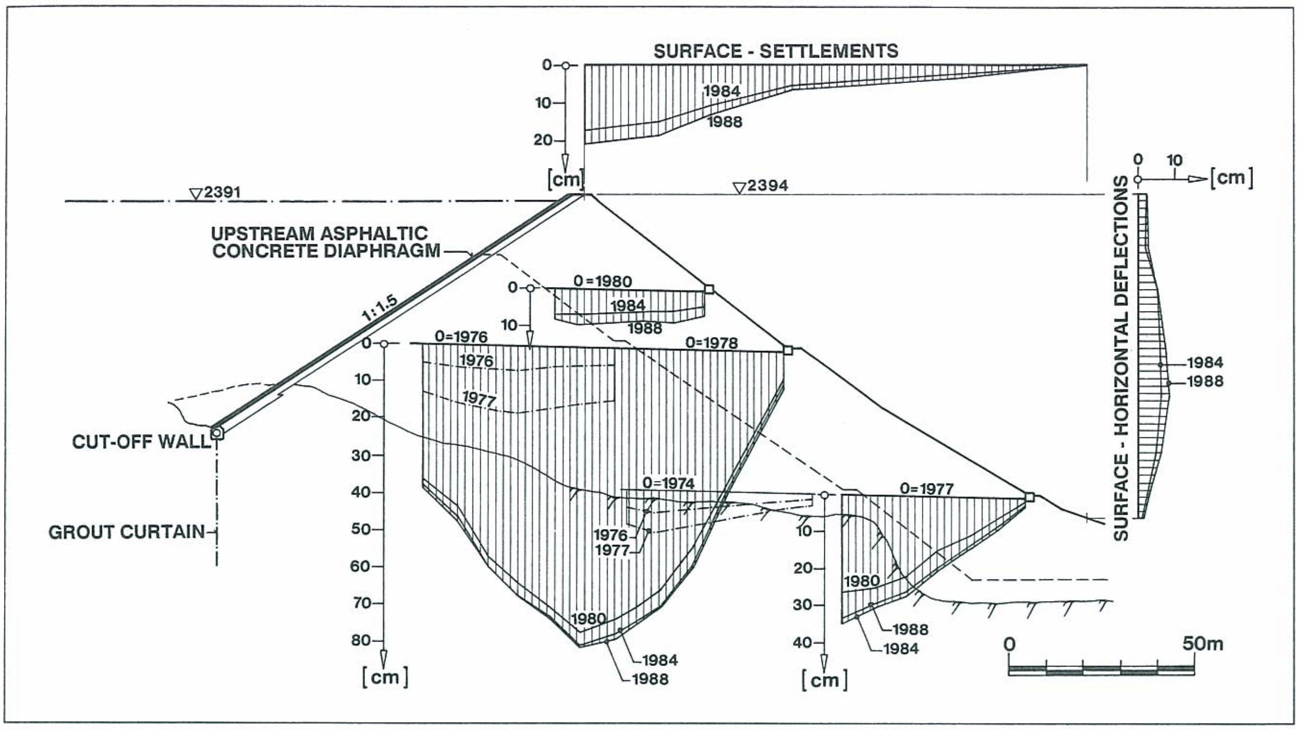


Figure 2 Cross section with settlements and horizontal deflections

levelled and shaped with a fine-grained levelling course. In the area of maximum dam height, the binder course is 12 cm thick at the base and decreases to 8 cm at the crest. The void content of the binder varies between10 and 15%; the bitumen content is about 5%. Finally the impervious diaphragm was applied from the top downwards, with the horizontal construction joints dictated both by the construction stages and by the finisher size. The maximum permissible void content was 3%. The bitumen content was approximately 7.5%, and diaphragm thicknesses varied between 12 and 7 cm. During the final construction stage, an additional impervious diaphragm was applied over the lower portion of the dam surface to an average thickness of 8 cm. This brought the total thickness of the diaphragm to 18 or 20 cm in the highest dam portion.

A concrete cut-off trench with a crawl space entirely tied into bedrock forms the transition between the impervious diaphragm and the single-line grout curtain.

Stability analysis for the embankment was based on the assumption of an angle of friction of 38° and compactness of 23 kN/m³ for the moraine material, and an angle of friction of 41 to 45° – depending on the depth of overlying fill – and compactness of 21 kN/m³ for the quarried material.

Particular attention had to be given to the steep and smooth natural ground surface in the area of the left-hand downstream dam base, where fill depths tended to be small for topographical reasons. By means of a rock profilograph especially developed at the Innsbruck University of Technology, the macro and micro roughnesses of the smooth rock surface were determined in order to establish its shear resistance. Reduced angles of internal friction of 35° without special roughening of the rock surface were assumed in the analysis.

The spillway was arranged in the left portion of the dam crest. It is an uncontrolled structure followed by a trapezoidal channel and is designed for a 24-hour rainstorm with a rainfall depth of 325 mm.

In addition, there is a discharge conduit consisting of an adit followed by a pipe embedded in the dam shoulder.

4 EXPERIENCES

Seepage is measured separately by dam sections and is discharged through the inspection gallery in the cut-off trench and the adit. Maximum seepage flow is at present about 5 l/s with a full reservoir. Six horizontal gauges were provided to monitor the internal deformation behaviour of the embankment. Maximum settlement about halfway up the dam is around 80 cm and mainly dates from the period of construction and first filling. Long-term settlements already show clear evidence of fading.

62 benchmarks are available for observing the downstream slope of the dam. Horizontal and vertical changes in position are measured twice a year. Maximum settlement at the dam crest has been 18 cm since completion of the structure, which corresponds to about 3‰ of the dam height of 60 m at the centre-line.

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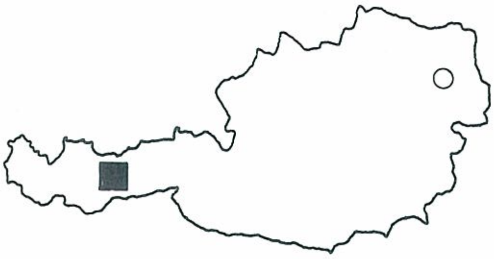
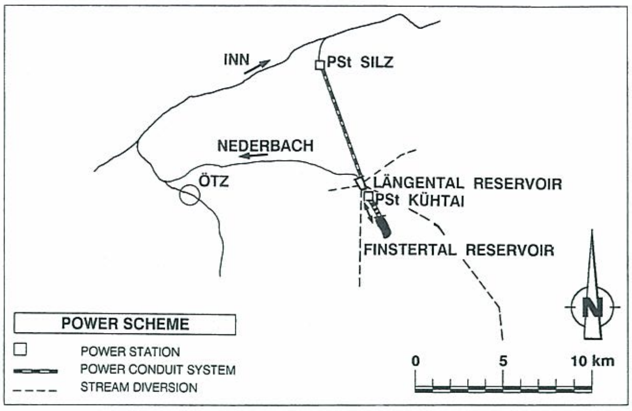
FINSTERTAL ROCKFILL DAM

Tyrol; Nederbach, Ötz, Inn
Nearest town: Ötz





MAIN TECHNICAL DATA, Chapter K, 57 (5/3)

General

Development	Sellrain-Silz scheme	
Power Station	Kühtai	Silz
Construction Period	1977	– 1981
Gross Head	394 m	1 250 m
Installed Capacity	292 MW	500 MW
Mean Annual Generation	57 GWh	45 GWh

Dam

Maximum Height	150 m
Crest Length	652 m
Thickness at the Crest	9 m
Maximum Thickness at the Base	388 m
Volume: Excavation (overburden, rock)	250 000 m^3
Embankment	4 600 000 m^3

Reservoir

Catchment Area: Upper Stage	6 km^2
Inflow	8 hm^3
Lower Stage	133 km^2
Inflow	153 hm^3
Normal Top Water Level (a.s.l.)	2 322 m
Minimum Operating Level (a.s.l.)	2 220 m
Gross Capacity	60.5 hm^3
Live Storage	60 hm^3
Area flooded by full Reservoir	1 km^2

Appurtenant Works

Spillway	
Capacity	3 m^3/s
Bottom Outlet, 2 butterfly valves	
Capacity	18.4 m^3/s
Power Intake	
Capacity: Generating Mode	80 m^3/s
Pumping Mode	68 m^3/s

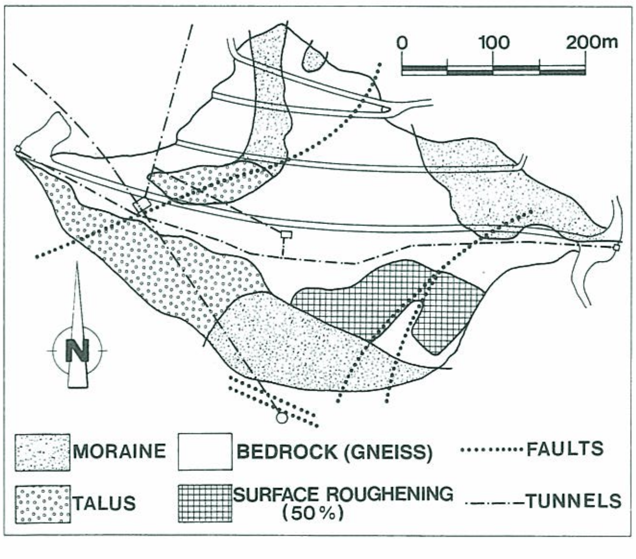
1 GENERAL

The Sellrain-Silz Power Scheme was constructed in 1977–1981 for the Tiroler Wasserkraftwerke AG (TIWAG), who produced the plans and supervised the works. The scheme is in two stages and utilizes high-level drainage areas (139 km²) with a total gross head of 1 678.5 m in the northern Stubai Alps to the west of Innsbruck. The main structure of the scheme is the 150 m high Finstertal embankment dam located at an altitude of 2 325 m above sea-level, which creates a seasonal reservoir with an active storage of 60 million m³. The extensive adduction system leads to the Längental storage reservoir at an altitude of 1 901 m, which is also the location of Kühtai power station and the 82 m deep shaft power station for the upper stage. The upper stage is provided with reversible pump-turbines for pumped storage operation. The power scheme incorporates two short pressure shafts. The two generating sets in the lower stage power station at Silz are among the most powerful hydropower installations worldwide (500 rpm rated speed, 352 MVA rated output per generator).

2 GEOLOGY

The glacial trough of the Finstertal lakes is traversed by a steeply dipping, south-plunging series of foliated gneisses, mica schists, hornblendic gneiss and granodiorite gneiss. Two rock sills are the dominant morphological feature of the reservoir area. The more southerly granodiorite ridge was quarried for rockfill material, while the sill to the north, which for the most part contains a high proportion of quartz and feldspar, passes

Figure 1 Foundation geology



through the dam foundation area in places. More than two thirds of the foundation contact area of the Finstertal embankment dam was only thinly covered bedrock, while the remaining area, and especially the left upstream abutment, was covered with an overburden of dense moraine material up to 20 m thick offering good bearing characteristics and with narrow zones of coarse-grained talus material. Two mylonitic fault zones with a maximum thickness of 3 m traverse the axis of the dam at an obtuse angle. On the basis of large-scale tests conducted to improve the key between the rockfill and the smooth bedrock polished by the abrasion of the glaciers on the steeply sloping right upstream abutment, notches were blasted into the rock, with a average depth of 70 cm, over an area of approx. 20 000 m². Although the foliated gneiss in the foundation area generally proved to be relatively impervious, a single-row vertical grout curtain up to 60 m deep was provided to intercept every single fracture or zone of increased permeability. Average cement uptake in the grout curtain was 16.5 kg/m². This reduced water inflow into the test boreholes to a maximum of 1 Lugeon. Contact grouting was also performed for the zone around the inspection gallery.

3 DAM

The 150 m high Finstertal rockfill dam was built in 1977–1980, with the difficult climatic conditions at an altitude of over 2 300 m permitting an average of only 105 placement days per year. Of the dam types investigated for the site, including concrete wall designs, a rockfill dam with asphaltic concrete core membrane and steep slopes proved to be the most cost-effective solution, quite apart from the ability of such a design to harmonize well with the mountain scenery. Optimum dam geometry with regard to embankment volume and stability was achieved by incorporating a slight upstream curvature of the main body across the valley followed by a short reverse curve to the right abutment. An inspection gallery, concreted more or less flush with the dam foundation in a groove blasted into the rock, runs the full length of the dam. During construction it served as the starting point for the asphaltic concrete core membrane and also for the curtain and contact grouting. With the location of the gallery only slightly upstream of the original lake retention sill, the core membrane, with its maximum height of 96 m, rests on a base measuring only 37 000 m².

The Finstertal rockfill dam is currently the highest dam in the world with an asphaltic concrete core membrane. Another innovative feature was the decision to place the 50–70 cm thick core at an angle over its full height – a significant further step in the development of this dam type. The inclination of the core also ensures that hydrostatic load is transmitted with optimum efficiency in terms of deformations and stress patterns to the larger portion of dam volume allocated to the downstream shoulder. Furthermore the design ensures that the core membrane receives greater support from the shoulders, so that the risk of undesirable deformation behaviour in the core membrane is largely excluded. The seven fill zones of the dam incorporate 80% blasted material, while the rest is moraine material from the overburden of the quarry area. The following Table 1 lists the various specifications for the fill material and details of lifts and compaction.

The 3 m thick upstream transition zone of screened moraine (100 mm max. particle size) is designed to re-

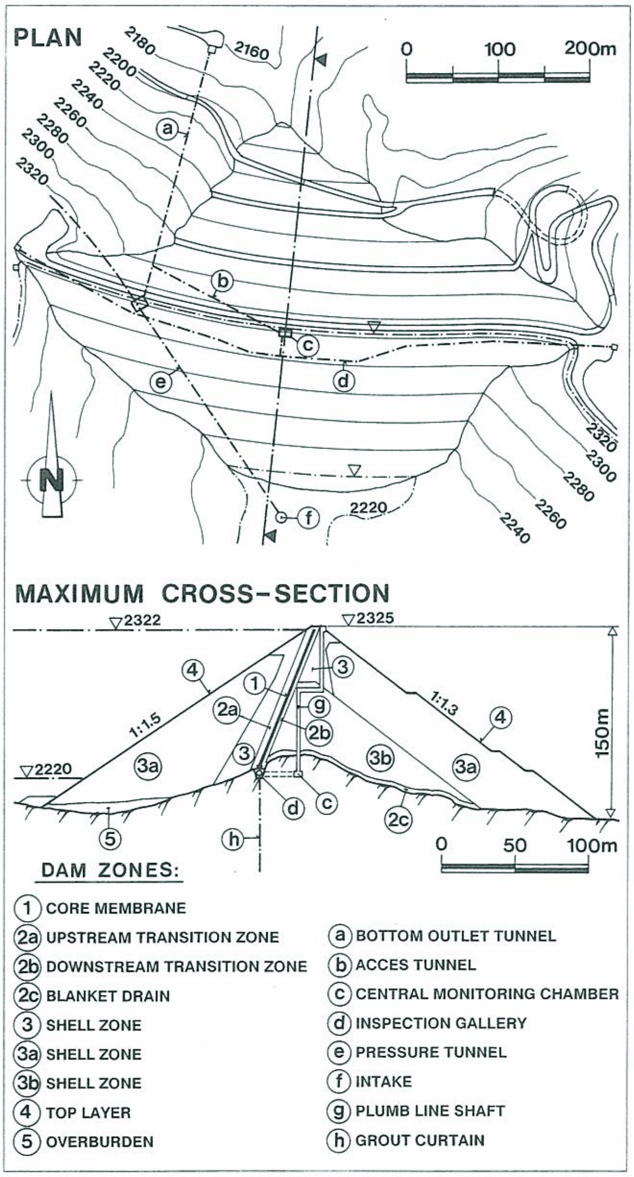
ZONE	MATERIAL	$d_{max.}$ [mm]	LIFT [cm] AND COMPACTION	FILL DENSITY [t/m³]	POROSITY n [%]	PERMEABILITY k [m/s]	ANGLE OF INTERNAL FRICTION	SHEAR RESISTANCE ON BEDROCK
(1) Core membrane	asphaltic concrete	16	25, vibrating finisher beam and 1t vibratory roller	i.M. 2.44	< 2	practically impermeable ($<10^{-15}$)	——	——
(2a) Upstream transition zone	screened moraine material	100	25, 2.2t vibratory roller	i.M. 2.36	i.M. 20	2×10^{-7}–2×10^{-9}	36.5°–41.4°	——
(2b) Downstream transition zone	screened quarry material	100	25, 2.2t vibratory roller	i.M. 2.16	i.M. 24	2×10^{-2}–2×10^{-5}	41.6°–46.0°	——
(2c) Blanket drain	quarry-run material	700	100 6×15t vibratory roller	i.M. 2.13	i.M. 23.5	$>10^{-4}$	41.6°–46.0°	——
(3) Shell zone	quarry-run material	700	75 6×15t vibratory roller (6t near core)	i.M. 2.22	i.M. 21	$>10^{-4}$	41.6°–46.0°	39.0°–41.5°
(3a) Shell zone	quarry-run material	700	100 6×15t vibratory roller	i.M. 2.13	i.M. 24	$>10^{-4}$	41.6°–46.0°	39.0°–41.5°
(3b) Shell zone	moraine material	700	100 6×15t vibratory roller (6t near core)	i.M. 2.41	i.M. 18	8×10^{-8}–2×10^{-9}	36.5°–41.4°	——
(4) Top layer	rock dressing	500–1000 long	placed	——			——	——
(5) Natural overburden	moraine material		rolled	i.M. 2.40	i.M. 22		36.5°–41.4°	34.9°–38.3°

Table 1 Fill specifications

duce seepage flow in case of leakage through the core membrane, to provide fines to plug minor leakages and to permit the drilling of grout holes if necessary.

Downstream from the core membrane is a 2 m thick zone of crushed granodiorite (100 mm max. particle size) with limited fines content, which serves to drain any seepage loss water and convey it by sectors to the inspection gallery. Apart from a 1.4 m wide section at the connection of the core with the inspection gallery, which was placed manually, the asphaltic concrete core was placed with the help of a special machine. The core and adjacent transition zones were placed in 25 cm layers and subsequently compacted simultaneously using a group of three vibrating finisher beams.

Figure 2 Plan and maximum cross section



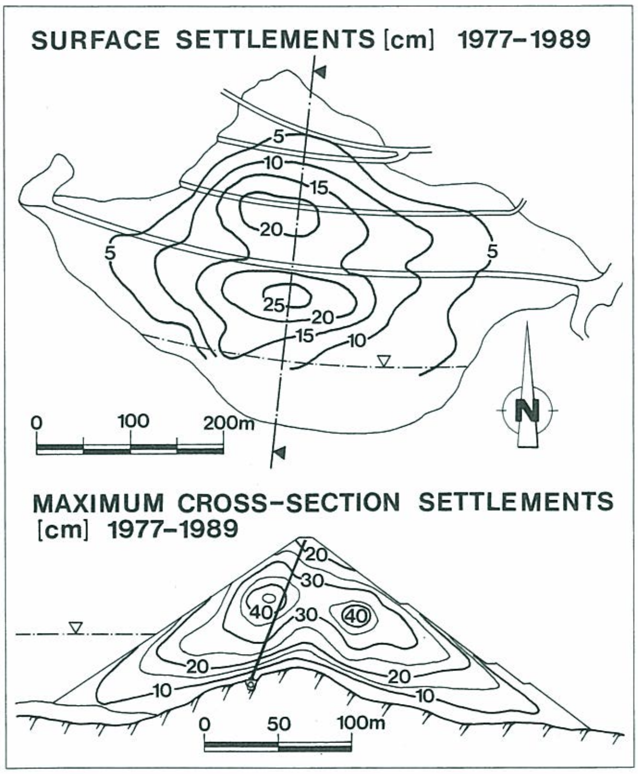
The four-component mix for the asphaltic concrete, comprising 75.9% by weight crushed granodiorite (18 mm max. grain size), 3% natural sand (0–3 mm), 8% limestone filler (0–0.09 mm), and 6.1%–6.3% B 65 bitumen, was calculated with the help of extensive experiments, laboratory tests and long-term triaxial tests, in order to achieve an optimum result with regard to imperviousness, crack-free deformation, shear strength, and weathering and aging stability. On the basis of large scale compaction tests, lifts of 0.75–1.0 m were selected for the main upstream and downstream fill zones, and compaction performed with 6 passes of a 15-ton vibratory roller. The result was a porosity of 18–24%, with increased compaction performed on the area adjacent to the transition zones so as to reduce the forces of deformation acting on the core membrane. To achieve better compaction and to partly anticipate post-construction settlement under the influence of fluctuating reservoir levels, the upstream shoulder was wetted with 0.5 m³ of water per m³ of fill. A full quality assurance and test programme was elaborated to ensure consistent results in the asphaltic concrete core and all fill zones. In view of the short construction period and the additional constraints imposed by frequent bad weather periods, the construction schedule was very tight, involving a labour force of up to 390 and a fleet of construction vehicles and machines with a total output of 24 700 kW. The average daily volume of fill placed was approx. 9 700 m³, with a maximum daily placement of 21 000 m³. For the asphaltic concrete core the average daily placement rate was 70 m³, with a maximum of 149 m³. In view of the small catchment area and its pronounced retention capacity, a flood water discharge capacity of only 3 m^3/s is provided in the form of a covered chute spillway at the left extremity of the dam crest leading to a concrete pipe. An intake tower 30 m upstream from the dam footing is designed for a maximum discharge of 80 m^3/s and pumped water throughput of 68 m^3/s. A 230 m long intake tunnel, from which the bottom outlet branches off, connects the tower with the combined valve chamber, which is fitted with two butterfly valves for each tunnel. The chamber can be reached via an access tunnel, and the tunnel mouth is the location of a control valve for the bottom outlet pipe laid in the tunnel floor.

4 EXPERIENCES

4.1 Dam monitoring

A full dam surveillance system was installed to provide reliable appraisal of the condition and behaviour of this high dam at all times. Instrumentation is located over five different levels in one principal section and five secondary sections and comprises a total of 793 measuring points for monitoring external and internal deformations, pore water pressures, earth pressures, temperature distribution, response acceleration, and seepage water losses. An accessible shaft was incorporated in the dam to permit surveillance of the core and surrounding areas. It is constructed of precast concrete rings embedded independently of each other in the fill in two staggered sections to take account of the inclination of the core membrane. The shaft accommodates two precision plumb-lines permitting continuous telemetering of crest and centre zone deflections and the orderly routing and arrangement of all lines and cables to the central monitoring chamber. Above all, it affords access to the central area of the dam over its full height, thus allowing precise monitoring of selected points in the core membrane by means of fluid level settlement devices, extensometers and a new type of magnetic deformation gauge developed by TIWAG in collaboration with Innsbruck University. The system permits changes in the thickness of the asphaltic concrete core to be measured to within a tenth of a millimetre. Any leaks or seepage in the core can be accurately localized thanks to structural measures permitting section by section capture of core seepage water in sections of 25 to 30 m.

Figure 3 ISO-settlements



4.2 Special events

The trouble-free operation of the Finstertal dam over the last ten years and the good results achieved with the asphaltic concrete core membrane are clear confirmation of the soundness of the design principles on which it is based. Surface deformations, with maximum settlements of 20–25 cm (Fig. 3) and maximum horizontal deflections of 15 cm, are of a magnitude that would hardly have been considered possible with earlier rockfill dams of such a height. As expected, the 2–4 cm thickening of the core observed during first filling quickly disappeared as a result of the support provided by the carefully compacted shoulders. No significant redistribution of stresses was observed. Total seepage water losses through a 37 000 m^2 core membrane surface were only 9 l/s on first filling, and declined to 3.5 l/s by the end of 1989.

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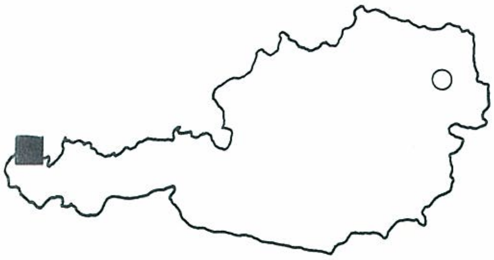
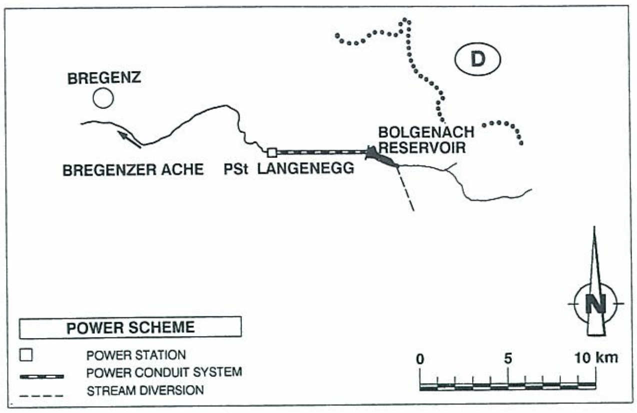
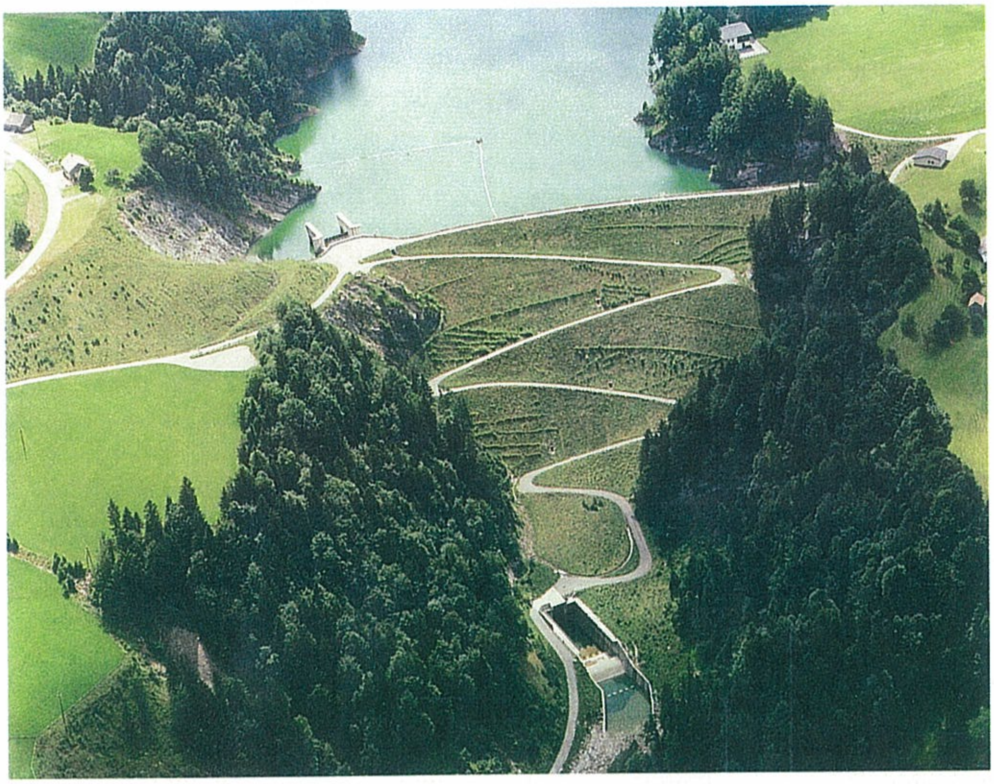
BOLGENACH EARTHFILL DAM

Vorarlberg; Bolgenach, Bregenzer Ache, Lake Constance
Nearest town: Bregenz







MAIN TECHNICAL DATA, Chapter K, 58 (4/23)

General

Power Station	Langenegg
Construction Period	1975 – 1979
Gross Head	280 m
Installed Capacity	80 MW
Mean Annual Generation	227 GWh

Reservoir

Catchment Area: Natural	89 km²
Inflow	169 hm³
Diversions	98 km²
Inflow	183 hm³
Normal Top Water Level (a.s.l.)	744.2 m
Minimum Operating Level (a.s.l.)	690.0 m
Gross Capacity	8.7 hm³
Live Storage	8.4 hm³
Area flooded by full Reservoir	0.3 km²

Dam

Maximum Height	102 m
Crest Length	240 m
Thickness at the Crest	6 m
Maximum Thickness at the Base	315 m
Volume: Excavation (overburden, rock)	150 000 m³
Embankment	1 350 000 m³

Appurtenant Works

Spillway, gated overflow weir	
Capacity	450 m³/s
Bottom Outlet, 2 slide gates	
Capacity	100 m³/s
Power Intake	
Capacity	32 m³/s

1 GENERAL

The Bolgenach dam and the reservoir impounded by it form part of the Langenegg development.

The decision on the construction of the Langenegg development was taken in 1971, following extensive planning studies. In the spring of 1973, Vorarlberger Kraftwerke AG as the utility responsible for electricity supply in the region signed a contract with Vorarlberger Jllwerke AG for the design and construction of the project. Work was commenced early in the summer of 1975. First power was generated in spring 1979.

The Langenegg power station develops the flows of the Bolgenach stream and the diverted Subersach stream under a total head of about 280 m. The long-term average annual runoff of 2 000 mm makes the catchment area above the Bolgenach reservoir one of the regions of highest precipitation in the Eastern Alps.

The Subersach stream is impounded at the intake to a depth of some 12 m. The weir is a gravity structure with a maximum height of about 19 m, diverting an average annual 183 million m^3 of water from a catchment of about 98 km^2 to the Bolgenach stream through an about 3.7 km long free-surface gallery.

The Bolgenach dam is an embankment with an impervious core constructed in a post-glacial gorge cut by the Bolgenach stream. Flood relief is provided by a bottom outlet as well as by a spillway consisting of an overflow section with a crest gate, an inclined shaft and a gallery. Energy dissipation is via a stilling basin.

Power water is conveyed from an intake works on the left-hand bank to a valve chamber equipped with a gate. This is followed by the Bolgenach pressure shaft about 270 m in inclined length, the low-level Rotenberg pressure tunnel about 5 370 m in length, the Langenegg pressure shaft about 45 m in inclined length and a manifold. The power conduit is about 5 920 m in total horizontal length.

The powerhouse is located underground on the right-hand bank of the Bregenzer Ache stream in a sandstone rib of the "Baustein" zone. This site was selected for geological reasons. The electrical and mechanical equipment consists of two vertical-shaft Francis turbines with a 43-MVA generator each. The power station is telecontrolled and monitored from the control centre at Bregenz.

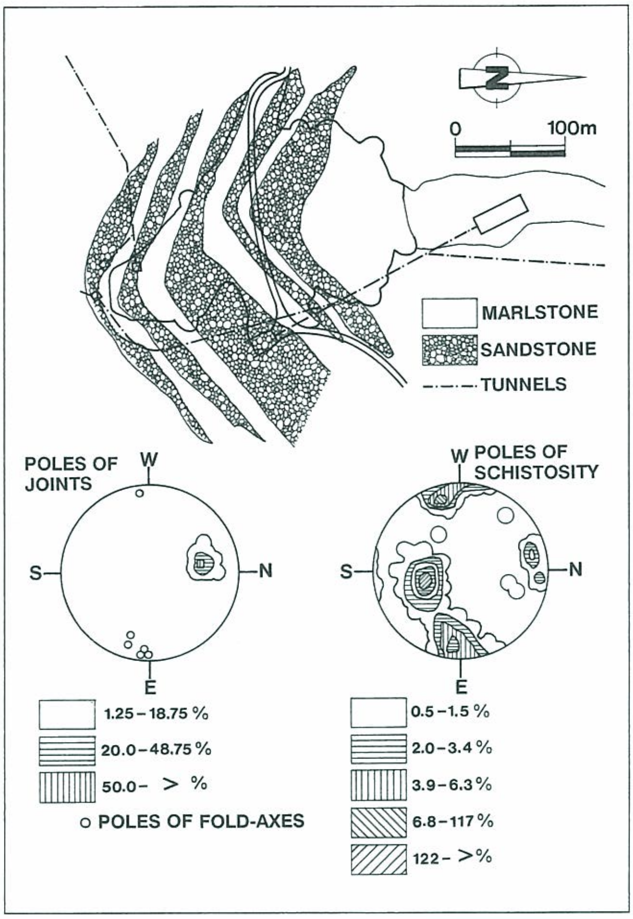
2 GEOLOGY

The reservoir and the greater part of the Bolgenach catchment above the dam as well as the pressure tunnel and the powerhouse are situated in the Subalpine Molasse of Vorarlberg. The narrow postglacial Bolgenach gorge was cut by the stream into a U-shaped valley carved by glacial erosion. It is for this reason that unconsolidated material as is normally left from the glacial period and its stages of retreat, such as moraines and subglacial sediments, is lacking in this gorge.

At the dam site, the gorge narrows due to the presence of five thick conglomerate and sandstone ribs separated by marl layers up to 20 m in thickness traversing the valley at the boundary between the marine molasse and the lower lacustrine molasse. Detailed mapping of the dam site included an accurate survey of structural features such as stratification and jointing. This gave the following sequence, listed from north to south (Fig. 1).

There is seismological evidence that the dam site has been affected by frequent earthquakes. The probable upper limit has been determined to be an earthquake intensity of 6.5° on the Mercalli-Sieberg scale with an average return period of at least 1 000 years.

Figure 1 Foundation geology



3 DAM

The design of the dam had to make allowance for some unusual features of the site. These were mainly the narrowness of the gorge and the difficult access, the alternation of rocks of different strength and permeability in the foundation and the irregular ground surface profile in the longitudinal direction of the valley. Additional limitations came from the relatively small storage available with the resulting filling and drawdown speeds, the impossibility of providing a lateral spillway as well as the forested catchment area resulting in floods with floating timber.

Fill material for the dam was obtained from unconsolidated material in the immediate vicinity of the site. The shoulders were constructed of gravel of low permeability, the core of dense ground moraine material. Both gravel and moraine are mixed-grained and lend themselves to compaction. The bulk densities reached during placement were 2.37 t/m³ and 2.22 t/m³ respectively, well above the simple Proctor density. The gravel was placed with a moisture content somewhat below optimum to reduce deformations, whereas the moraine material was placed with a moisture content slightly above optimum to enhance deformability. The upstream dam face was stabilized with conglomerate and sandstone riprap.

The impervious core of the embankment ties into a 20 m thick marl layer. For optimal embedding, the core is angled in elevation and curved in plan. The foundation contact of the core was provided with an about 10 cm thick layer of moraine material (less than 30 mm in particle size) with 3% bentonite added.

Overburden stripping for the foundation of the core in the gorge walls was carried out by caterpillars suspended from rope capstans. The high and partly overhanging sandstone and conglomerate cliffs had to be stabilized with prestressed anchors monitored by appropriate instrumentation. Over the foundation surface of the core, the waste mantle was carefully removed with an Alpine-Miner excavator and the freshly exposed ground surface was immediately protected from drying by placing the connection layer of moraine material with bentonite added.

A wobbler plant was installed to screen stones larger than 90 mm from the moraine material. This also proved efficient in ensuring uniform addition of water. The material was placed in layers 20 cm deep and compacted by 9-ton sheepsfoot rollers.

As placement proceeded, the planned grouting operations beneath the core were undertaken with particular care from the inspection gallery and the inspection shaft. Faulted marl areas that had been left in the foundation were included in the grouting scheme.

During construction, the Bolgenach stream was diverted through a gallery which subsequently became part of the bottom outlet and spillway structures. The authorities had required a 5 000-year flood to be assumed in the spillway design. The spillway structure consists of a central gate-controlled section and two lateral ungated overflow sections. The overflow structure is followed by an inclined shaft and a slightly inclined gallery ending in a stilling basin. A hydraulic scale model test was performed for the entire flood relief works to obtain a better idea of the complex hydraulic conditions involved.

Intermediate flood relief afforded by a cylindrical spillway tower 20 m in height and temporary closing of the diversion tunnel allowed partial reservoir filling and power station operation before the embankment was completed.

The bottom outlet is equipped with an inspection gate and a regulating gate and ends in the stilling basin of the flood relief structure.

4 EXPERIENCES

4.1 Dam monitoring

Figure 2 Plan

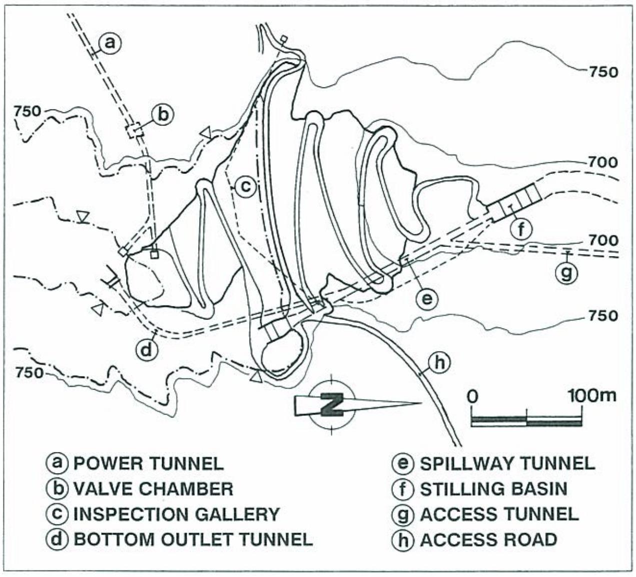
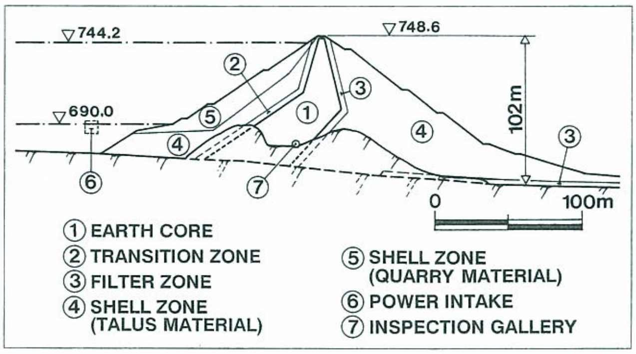


Figure 3 Cross section



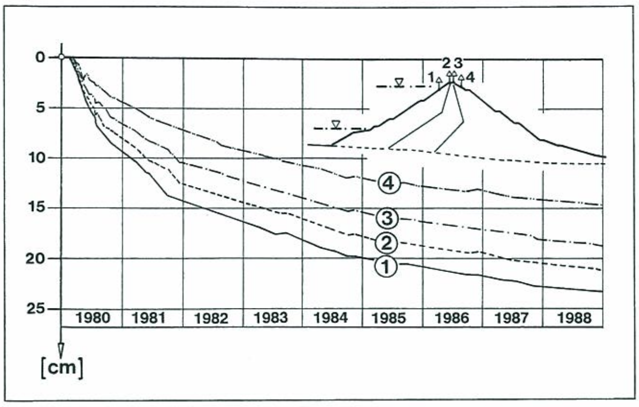
Due to the relatively small net storage of 8.4 million m³, substantial water level variations may occur within a short time. This may cause strongly alternating stresses acting on the embankment. In order to obtain detailed information on its behaviour, extensive instrumentation was provided. 37 pore pressure gauges and 90 earth pressure gauges were installed at 4 levels. Horizontal and vertical displacements are measured by 3 settlement gauges and 4 horizontal plate gauges.

The terminals of the inductive displacement measuring devices at the downstream dam face were integrated into the surveying system. The consolidation process has practically come to an end (see Fig. 4).

Extensometers were installed at critical points to measure relative movements between fill and in-situ rock.

From a system of pillars, points on the dam surface are observed for changes in position and level. The points situated along the dam crest provide information on surface movements and, through transmission by mobile inverted pendulums, they also allow observation of the moraine core.

Figure 4 Crest surface monument settlements



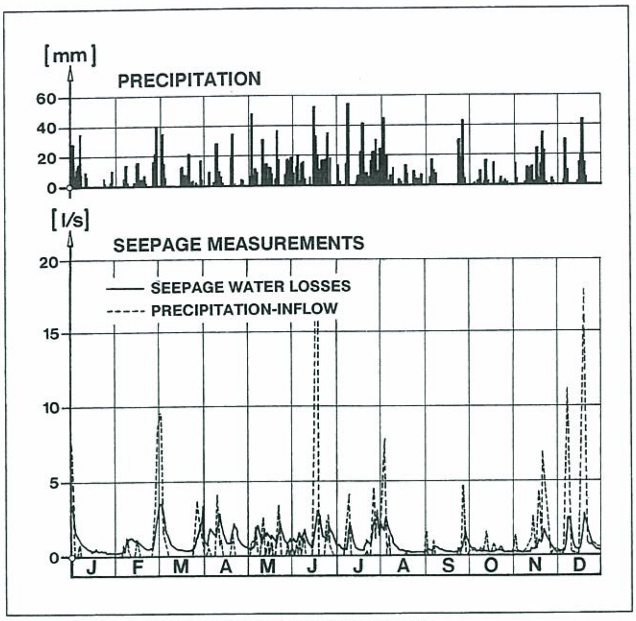


Figure 5 Precipitation and seepage water losses

Seepage flow, considered a main indication of dam stability, is measured by means of a weir and an automatic poise beam. Readings can be taken directly on a horizontal scale and are transmitted to the control centre at Bregenz, where they are recorded. The same method is used for the measurement and display of surface flows. Surface and seepage flows are mainly a function of precipitation depths (see Fig. 5). Seepage water also contains hillside surface water emerging from the core area. This explains the dependence of seepage flow on precipitation.

4.2 Special events

After several years' reservoir operation (top water level first reached in 1980), it may be stated that the core of the embankment and its foundation contact as well as the underlying marl layer are impervious. The anticipated redistribution of stresses due to bridging resulting from the irregular topography of the foundation surface and differential settlement between core and shells have remained relatively small due to the low deformability of the fill materials used. No indication of fissuring has been found on the embankment. With maximum settlements not exceeding 25 cm, dam behaviour is as expected.

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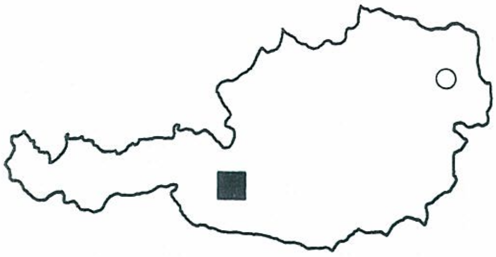
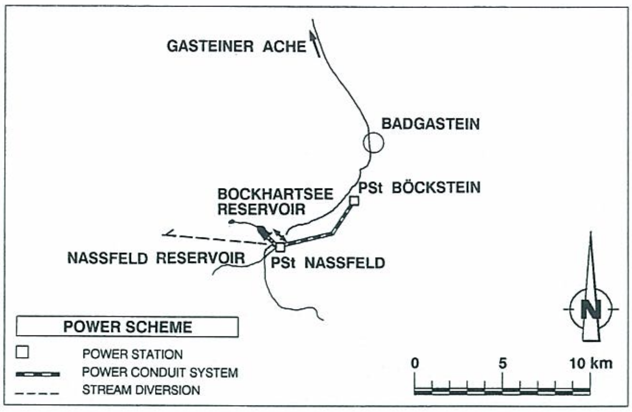
BOCKHARTSEE ROCKFILL DAM

Salzburg; Nassfelder Ache, Salzach
Nearest town: Badgastein





MAIN TECHNICAL DATA, Chapter K, 59 (5/7)

General

Power Station	Nass-feld	Böck-stein
Construction Period	1978 –	1983
Gross Head	280 m	452 m
Installed Capacity	29 MW	46 MW
Mean Annual Generation	120 GWh in total	

Reservoir

Catchment Area: Upper Stage	5 km²
Inflow	9.5 hm³
Lower Stage	58 km²
Inflow	92 hm³
Normal Top Water Level (a.s.l.)	1 872.5 m
Minimum Operating Level (a.s.l.)	1 812.0 m
Gross Capacity	14.8 hm³
Live Storage	14.2 hm³
Area flooded by full Reservoir	0.42 km²

Dam

Maximum Height	69 m
Crest Length	239.5 m
Thickness at the Crest	6 m
Max. Thickness at the Base	176 m
Volume: Excavation (overburden, rock)	70 000 m³
Embankment	230 000 m³

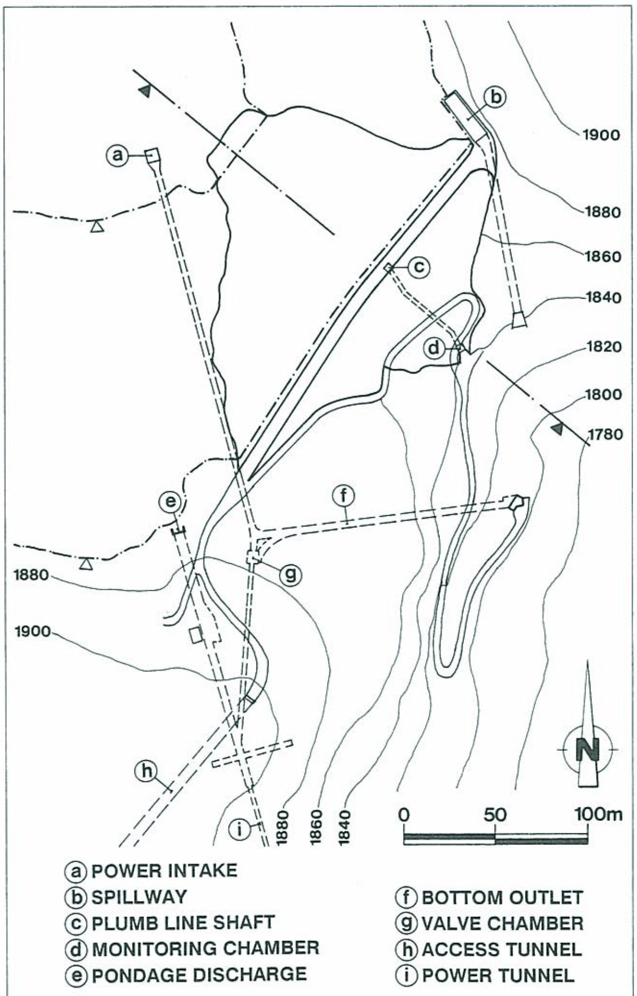
Appurtenant Works

Spillway, morning glory	
Capacity	44.3 m³/s
Bottom Outlet, 1 Howell-Bunger valve, 1 butterfly valve	
Capacity	11.6 m³/s
Power Intake	
Capacity: Generating Mode	11.3 m³/s
Pumping Mode	9.1 m³/s

1 GENERAL

The Bockhartsee power scheme, currently comprising the Nassfeld upper stage and Böckstein lower stage, was constructed exclusively for hydropower generation, although its side effects in terms of flood retention and control and its relevance for a burgeoning tourist indus-try are also worth mentioning. A third stage is also to be built to replace the Badgastein power station built in 1914, which has a head of only approx. 80 m and a design capacity of 3.5 m^3/s. When complete, this third stage will permit the turbine water from the Böckstein lower stage to be used for power generation with the additional head of the steep drop down to Badgastein at the mouth of the main valley. This will give the Bockhartsee power scheme a total head of approximately 1 000 m. The reservoir is based on a natural lake. Active storage from the lake was increased by locating the turbine water intake close to the deepest point and by building a dam for an extra 25 m of head. An Isogyre pump-turbine set is used to fill the reservoir and reduce replenishment times as well as for short-term pumping. A trans-basin diversion from the adjoining Rauris Valley adds an additional 21 km^2 to the 37.4 km^2 catchment of the Böckstein lower stage, the flow being limited to 2.5–3 m^3/s by the tunnel cross-section and available gradient. Minimum flow orders ensure an adequate volume of water in ecological terms for two noted waterfalls during daylight hours in summer and autumn. An active storage of 14.2 million m^3 permits the summer inflows to be partly stored for the winter peak tariff period. Nevertheless, out of a total annual generation of approx. 120 GWh achieved with the present two-stage scheme, only about 46 GWh is generated in the six-month winter period. Plans for the effective exploitation of potential capacity date back to the year 1949. The first project had to be shelved in 1954 in the face of resistance from the local residents and contradictory evidence presented on the subject of possible risks to the Gastein medicinal springs. Following further exploration of the source of the Gastein springs and a number of changes to the project, the Austrian water authorities gave the go-ahead in 1971, but work on the scheme was postponed once again because of rising prices in the construction industry. As a result, the project was modified once again to provide for higher capacities for both stages, and this required a corresponding revision of the planning permission issued by the water authorities. Finally, the decision was taken to implement the project on a partial basis, and work began on the lower stage in 1978. Before this was completed, however, it was decided to start work on the upper stage as well, and dam construction was begun. By December 1982, partial impounding had been completed, and the turbine went on line without pump working. It was not until the 110 kV power line had been built from Böckstein to Nassfeld in July 1984 that first filling could be continued to just below top water level with the pump-turbine working in the pumping mode.

Figure 1 Plan



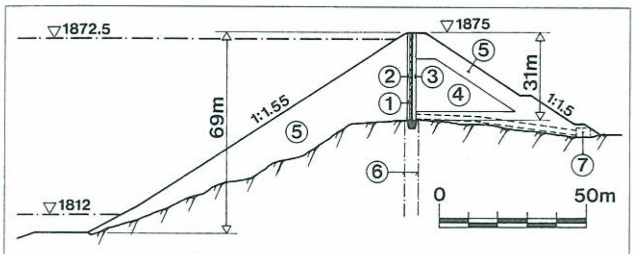
2 GEOLOGY

The Ankogel-Hochalm massif, which forms part of the Goldberg mountains, belongs to the Pennine tectonic system. The Bockhartsee reservoir is located in its entirety in siglitz gneiss (a hybrid granite gneiss with frequent albite predominance), which is one of the metamorphic mixed rocks of the central granite. Morphologically, the area is characterized by pronounced S-planes and by joint sets that are mostly very pronounced as well. The rock sill retaining the lake was formed not only by glacial action but also by moraine deposits in part. The local major tectonic fault zones, such as the steep WNW plunging Gastein Fäulen system or the ESE plunging ore joints, do not affect the area of the impounding rock sill. Although there are no areas of high seismic hazard in the Salzburg Region, it was decided to accept the suggestion of the Austrian Dam Commission to base stability calculations for the dam on a response acceleration of 4% of acceleration g, which is the equivalent of an intensity 5 earthquake on the Mercalli-Sieberg scale. The overburden on the rock sill was several metres deep, comprising moraine material, silty, fine-sandy sediment deposits, and in places detritus and organic peaty soil. This was stripped to provide a total dam foundation area of approx. 74 000 m² of consistently highly compact granite gneiss with only moderate jointing in the main (medium to widely spaced). The smooth rock surface created by the abrasion of the glaciers showed no signs of deep loosening or weathering phenomena. No difficulties were therefore encountered in sinking a contact spur for the core wall down to a depth of 2.5 m.

3 DAM

3.1 Design parameters

The rock sill formed on the southeast side of the Bockhartsee as a result of glacial action is relatively narrow, with high flanks that are very steep on the downstream side. It would therefore have been theoretically better suited to a concrete gravity dam design rather than a rockfill dam. All in all, however, the latter proved to be the more cost-effective approach and also more aesthetic. On the other hand, the original plan for an asphaltic concrete core was rejected during the actual construction of the dam and replaced with a concrete core wall. The work was carried out in the framework of a research project into high concrete core walls funded by the Austrian electricity generating industry and supervised by Innsbruck Technical University.



① CONCRETE CORE WALL d=60 cm WITH BITUMINOUS SLIP-LAYER
② UPSTREAM TRANSITION ZONE
③ DOWNSTREAM TRANSITION ZONE
④ SHELL ZONE (MORAINE)
⑤ SHELL ZONE (QUARRY MATERIAL)
⑥ GROUT CURTAIN
⑦ MONITORING CHAMBER

Figure 2 Cross section

3.2 Fill materials

The central – or siglitz – gneiss employed is very coarsely jointed and offers high rock strengths. The fill material quarried from a site in the vicinity of the dam was in the form of boulders up to 1 m in length. Fine-grained material was used for the transition zones on either side of the core, namely 2–100 mm for a 90 cm transition zone on the downstream side, and 0–30 mm for a 20 cm contact area followed by 0–300 mm for a 100 cm transition zone on the upstream side. The material, which offered good compactability, was placed in 1 m lifts for the shoulders, and in lifts of 0.5 m for the two transition zones. Compaction was performed with 8 passes of a 14.5-ton and 3-ton vibratory roller respectively. On the downstream side, moraine material and talus from a slope area adjoining the dam was also incorporated in the fill. To monitor the suitability of the placement criteria derived from compaction testing, a number of tests were performed dur-ing the construction phase. Compaction tests using the water substitution method were performed for every 5 m of lift in the shoulders, and ten further tests were carried out for the zone with the talus and moraine material. In addition, grading curves were produced and water content measured for every 1 m of lift in the transition zones.

4 EXPERIENCES

4.1 Dam monitoring

Geodetic monitoring is performed with six surface monuments located on the dam crest plus three monitoring sections extending from the upstream slope to the downstream slope with five to six benchmarks each plus a benchmark on the crest. Four settlement gauges, two on the upstream side and two on the downstream side, were used to monitor dam behaviour during first filling and first drawdown. During construction and the first few years of operation, the behaviour of the concrete core wall was monitored with the help of concrete and earth pressure cells, concrete strain gauges and thermometers, and sliding micrometers. The monitoring system also includes a plumb line installed in an accessible shaft, and nine triaxial joint gauges located in the shaft plus five on the dam.

4.2 Monitoring results and special events

During construction and on impounding, settlements at the upstream slope benchmarks were greater than on the downstream slope, namely 136 mm versus 37 mm or 3.67 : 1. On the other hand readings from the settlement gauge located on the dam crest were fourteen times higher on impounding than for the gauge on the downstream slope, namely 362 mm compared with 25 mm. Seepage water losses through the core wall on the one hand and surface water from the downstream shoulder plus ground water from the foundation area and abutments on the other are collected separately and in five sectors each. The monitoring chamber is located at the dam footing. The readings for ground and shoulder seepage water losses roughly reflect precipitation patterns. Total core seepage flows are discharged over a weir, where they are measured. The stilling basin located before the weir is fitted with a submersible electrode, which activates the warning system when flows reach a preset limit of 1 l/s (one litre per second). The records for seepage flows from 1985 to 1989 show core seepage water losses of between 0.0025 and 0.11 l/s. For other results of dam monitoring and their analysis you are referred to Volume 9 of the "Mitteilungen des Institutes für Bodenmechanik, Felsmechanik und Grundbau" (University of Innsbruck).

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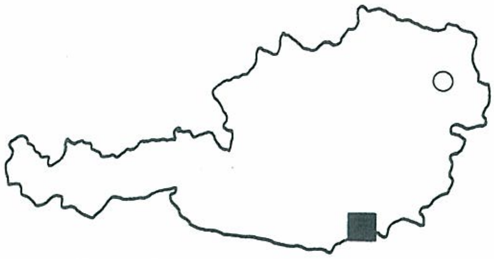
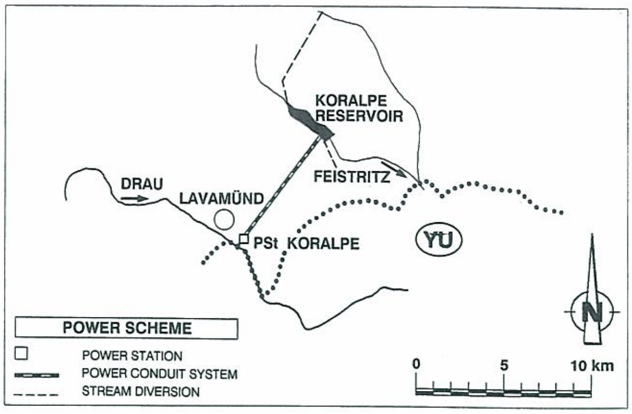
FEISTRITZBACH ROCKFILL DAM

Carinthia, Styria; Feistritzbach, Drau
Nearest town: Lavamünd





MAIN TECHNICAL DATA, Chapter K, 66 (.../...)

General

Development	Koralpe
Power Station	Lavamünd
Construction Period	1987–1990
Gross Head	735.5 m
Installed Capacity	50 MW
Mean Annual Generation	83.5 GWh

Dam

Maximum Height	88 m
Crest Length	370 m
Thickness at the Crest	12 m
Volume: Rockfill	1 600 000 m³
Asphalt Membrane	14 100 m²

Reservoir

Catchment Area: Natural	29.8 km²
Diversions	37.0 km²
Normal Top Water Level (a.s.l.)	1 080 m
Minimum Operating Level (a.s.l.)	1 053.5 m
Gross Capacity	22.2 hm³
Live storage	16 hm³
Area flooded by full Reservoir	0.87 km²

Appurtenant Works

Spillway, uncontrolled chute	
Capacity	105 m³/s
Bottom Outlet, gate-controlled pipe with stilling basin	
Capacity	15 m³/s
Power Intake	
Capacity	8 m³/s

1 GENERAL

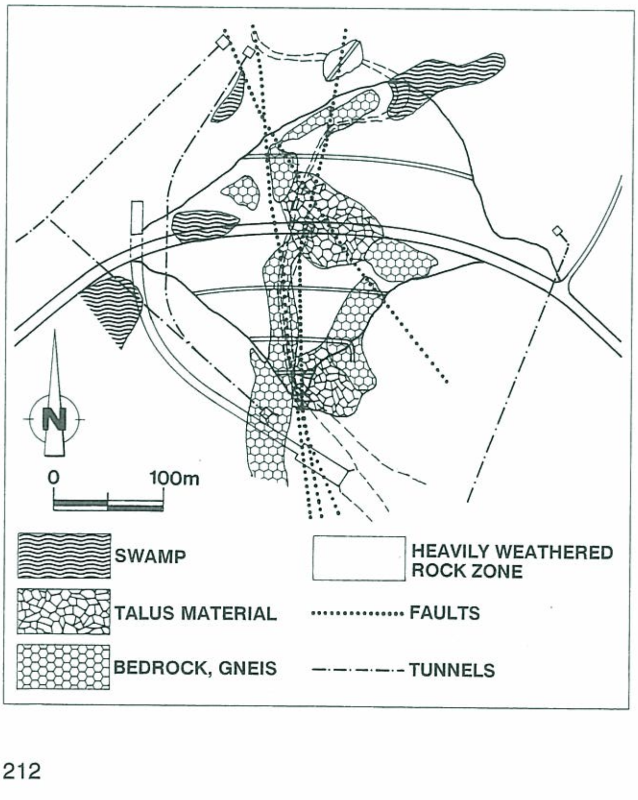
The Koralpe development, completed in 1990, utilizes the runoff from an area situated on the border between Styria and Carinthia for power generation. First development studies were undertaken in 1960, providing at that time for two major reservoirs with a pumped-storage facility supplied from the river Drau. The project finally adopted, however, consists of a large reservoir on the Feistritzbach stream with a diversion system from the Krumbach stream basin and has no pumping facility.

The only purpose of the development is the generation of energy. Its main feature is the Feistritzbach reservoir, which is operated on a seasonal basis. For reasons of environmental protection and planned recreation uses, a restriction has been imposed on reservoir operation to the effect that water level fluctuations are not allowed to exceed 1.0 m during the summer months. Another requirement is that the reservoir should be full by mid-June, which limits or even renders impossible turbine operation during reservoir filling in a year of average rainfall conditions. Special landscaping measures were taken in the bank zone affected by water level variations and on the downstream face of the embankment. Dyed concrete was used for the spillway and breakwaters.

Runoff from the catchment area of 66.8 km^2 is 50.3 million m^3 p.a., about two-thirds being accounted for by summer flow and one-third by winter flow. Minimum flow requirements are 65 l/s below the Feistritzbach dam, 90 l/s below the Krumbach stream intake, and 1 m^3/s for 10 months and 0.85 m^3/s for 2 months on the border with Yugoslavia.

The Koralpe power development was constructed as a joint venture by Kärntner Elektrizitäts-AG (80%) and Elektrogrospodarska Slovenje, Yugoslavia (20%), their shares corresponding to the ratio of heads of the Feistritzbach stream on Austrian and Yugoslav territory.

Figure 1 Foundation geology



2 GEOLOGY

The Feistritzbach embankment rests on the gneisses of the Koralpe Altkristallin ("old crystalline") formations. The steeper right-hand abutment has an even relief, with zones of compact rock alternating with varying, and in some places substantial, thicknesses of residual soil resulting from deep weathering. At the lowest point of the dam, a north-south striking fault line cuts through the dam site.

The left-hand abutment is composed of fairly uniform rock, but is covered almost throughout by severely disintegrated gneiss material interspersed with boulder zones.

Geological and soil mechanics investigations led to the identification of different zones within the up to 6 m deep overburden and in the transition to the bedrock. Therefore, large quantities of soil had to be removed before placing the dam fill, so as to ensure a stable foundation. Foundation treatment consisted of sinking a two-line grout curtain to a maximum depth of about 60 m. Several lines of contact grouting were provided in the transition zone between cut-off wall and bedrock.

All the appurtenant structures – bottom outlet, adit to power conduit and spillway – are located in the compact gneiss of the right-hand abutment.

3 DAM

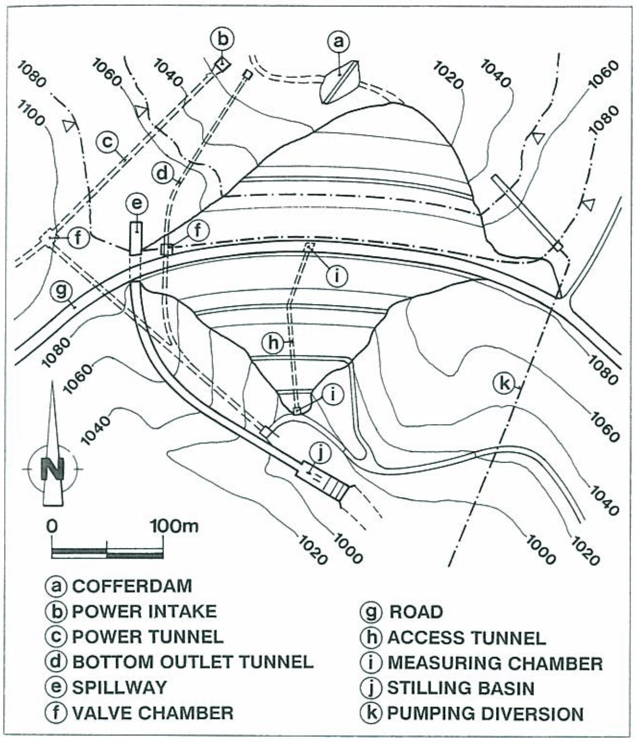
Several alternative designs of dam structure were studied. These included single and multi-layer asphaltic concrete facings placed in one or two construction phases as well as central asphaltic concrete membranes, vertical or inclined. Experience gathered and the results obtained from Finite Element analyses, cost estimates and topographical studies led to the adoption of a zoned embankment with a central vertical asphalt membrane inclined to the upstream over its upper 15 m. Quarried gneiss showing varying intensities of weathering was used as fill material. Weathering is particularly severe in near-surface zones and therefore determines the quality of the fill material.

The upstream shoulder was constructed in slightly weathered quarried material placed in layers of 60 cm maximum thickness and with a maximum particle size of 400 mm. Compaction was by vibratory rollers applying a dynamic load of about 32 kN. Although the required compactness was usually obtained by two or three roller passes, four passes were in general carried out.

The upstream face is protected by a 2 m thick riprap layer with a maximum boulder size of 70 cm.

Between the upstream main fill and the central asphalt membrane the embankment is structered as follows. A 3.5 m wide filter zone placed in 0.2 m lifts and with a maximum particle size of 65 mm is followed by a 2 m wide zone of fine material with a maximum particle size of 65 mm. The asphaltic concrete membrane was placed in 0.2 m layers in a single operation together with the downstream filter and the fine particle zone by means of a finisher. The membrane decreases in thickness from 0.7 m at the base to 0.5 m at the top. The bitumen content is about 6.5%; maximum particle size is approximately 16 mm and the required voids content is less than 3%.

Figure 2 Plan



On the downstream side, the upper limit imposed on particle size is 65 mm, and fine particles of less than 0.5 mm are limited to 10%. A horizontal filter layer 2 m in thickness was placed over the downstream portion of the dam base, and a toe drain installation of highly pervious material was provided on the downstream side.

Severely weathered rock that crumbled during quarrying and placement was used for the downstream shoulder, mainly in the portion below the berm at El. 1 050 m. The downstream dam face was covered with top soil and seeded.

Appurtenances include an uncontrolled trough spillway followed by a partly very steep chute ending in a stepped stilling basin. Hydrological analysis for determining the design flood in the catchment area of 29.8 km² above the dam site was based on two, six, nine and twelve-hour rainstorms with maximum rainfall depths of 90, 135, 155, and 185 mm. With allowance being made for the flood retarding effect of the reservoir, a six-hour rainstorm with a peak flow of 98 m³/s from the catchment was used as a basis for the design. Allowing for the discharge capacity of the Krumbach diversion, the spillway was designed for a capacity of 105 m³/s.

In addition, a bottom outlet was provided to allow rapid drawdown in an emergency. This facility, however, is not intended for the discharge of floods.

In the normal operating condition, the bottom outlet is closed by two valves located in the valve chamber, with the following 1 200 mm dia. steel pipe empty. The regulating gate at the end of the pipe line is closed before the outlet is operated. The pipe line is filled through a bypass in the valve chamber before the gate at the end of the pipe is opened. This gate also regulates outlet discharge. The outlet ends in a concrete conduit and stilling basin.

Figure 3 Cross section

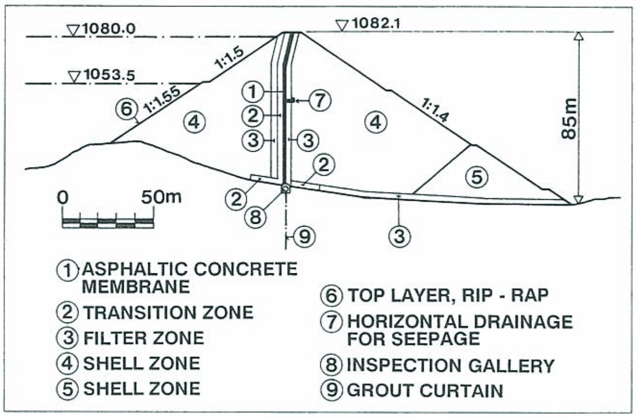
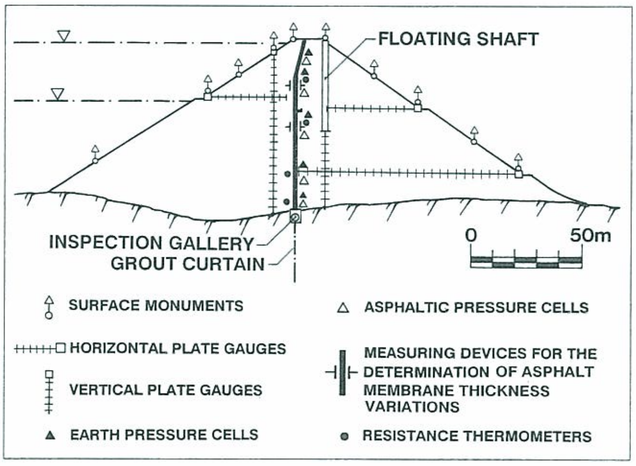


Figure 4 Monitoring equipment



The earthquakes underlying the design were assumed to have a horizontal acceleration of 0.04 g and a vertical acceleration of 0.02 g. To comply with additional wishes on the part of Yugoslavia, analyses were carried out for other safety criteria for a horizontal acceleration of 0.12 g and a vertical acceleration of 0.06 g.

Angles of friction depending on normal stress were assumed to be 36° to 44° for the severely weathered to sound rock and were entered in the stability analysis for the embankment.

4 EXPERIENCES

Instrumentation provided for the observation of the embankment includes a floating shaft in the upper half of the embankment, 3 vertical and 5 horizontal plate gauges, 39 benchmarks at the upstream and downstream dam faces, 2 instruments for measuring asphalt thickness, 72 earth and bitumen pressure transducers in the downstream filter and in the contact between concrete, drainage gallery and asphalt for measuring horizontal and vertical pressures, and in the lowest area of the downstream dam base, on the access structure, 9 extensometers on the upstream side for observing potential deformations in topographically critical cross sections, as well as additional instrumentation for seepage and turbidity.

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Chapter H

RIVER BARRAGES IN AUSTRIA

CONTENTS

RIVER BARRAGES IN AUSTRIA

By R. Fenz*

CHAINS OF POWER PLANTS

This report wants to give a description of the chains of power plants on the main Austrian rivers and to demonstrate the importance of this type of power stations for Austria's electricity supply, as well as to present the historical development and the different solutions of constructional details for these large river barrages. For the sake of clearness the scope of plant discussed here will be limited in terms of power station capacity i.e. stations with a max. capacity less than 10 MW will be precluded from consideration.

The importance of hydro power for electricity supply is very much a function of the geographical, hydrological and topographical conditions of a country and, hence, varies considerably. The hydraulic share of total electricity production is for instance 12% in U.S.A., 68% in Canada, 19% (mean value) in Europe. In Austria the contribution of hydro power was about 50% in 1918, increased to more than 80% in the years 1932 to 1938, then decreased under 60% by 1971/72. At present the percentage varies between 68 and 72%, even in dry years hydro accounts for almost two thirds of total generation.

In Europe large hydro shares are only found in Norway (99,7%) and Iceland, whereas Austria and Switzerland can be regarded as almost equally entitled to the attribute

Table 1
Comparison of Alpine hydro power countries Switzerland—Austria

	I	A	I/A	P	Spec. potential P/I	Spec. potential P/A
	Inhabitants (10⁶)	Area 10 km		Hydro-potential GWh/a	kWh/a/I	GWh/a/km
Switzerland (CH)	6.1	41	149	37 000	6 060	0.90
Austria (A)	7.5	84	90	53 700	7 160	0.64
Comparison (A/CH)	+23%	+105%	–40%	+45%	+18%	–29%

of "hydro power country". Both countries situated in the Alps are similar in many respects but there are some important differences in "hydro". Austria's developable hydro potential is 53 700 GWh/a and thus some 45% larger than that of Switzerland; the hydro resources per inhabitant are 18% larger but related to the area of national territory Austria's "hydro-density" is 29% lower than that of Switzerland (Table 1). The developed share of the developable potential is at present 93% in Switzerland and 60% in Austria.

In 1989 total electricity generation was about 50.2 TWh (i.e. 50 167GWh), of which 36.2 TWh (or 72 %) came from hydro stations and 14.0 TWh (28 %) from thermal stations. The above 36.2 TWh of hydro generation falls into 26.1 TWh from run-of-river stations and 10.1 TWh from storage schemes. The 26.1TWh of run-of-river energy thus accounted for 52 % of the total generation (hydro and thermal) of 50.2 TWh. For the sake of completeness, 5.9 TWh of imported electricity must be added, which brings the total energy to 56.1 TWh of which 8.3 TWh was exported and 47.8 TWh was used for domestic consumption.

Out of the total of 26.1 TWh/a of run-of-river energy 23.5 TWh/a or 90 % was generated by the seven series (chains) of powerstations on the rivers Danube, Drau, Enns, Inn, Mur, Salzach and Traun and 2.6 TWh/a or 10 % by other run-of-river plants.

Table 2
Austria's chains of run-of-river stations, 1990

River	Energy GWh/a	Capacity MW	Number of power stations (boundary stations)
Danube	11 696	1 916	(1/2) + 8
Drau	2 917	615	11
Enns	2 373	518	14
Inn	1 922	333	2 + (1/4 + 5/2)
Mur	1 155	216	12
Salzach	1 015	234	8
Traun	531	112	4
total	21 609	3 944	59 + (7)

Table 2 is a list of the seven river basins mentioned above, arranged in order of average annual energy in Gigawatt-hours (GWh/a) and shows also the maximum capacity in Megawatt (MW). For the rivers Inn and Danube only Austria's share of the output produced by boundary stations has been allowed for. The following description of river basins will be in the same order as shown in the table.

Austria's dense population, especially in the wide river valleys, and the topographical and civilisational conditions in this country allow practically no major impound-

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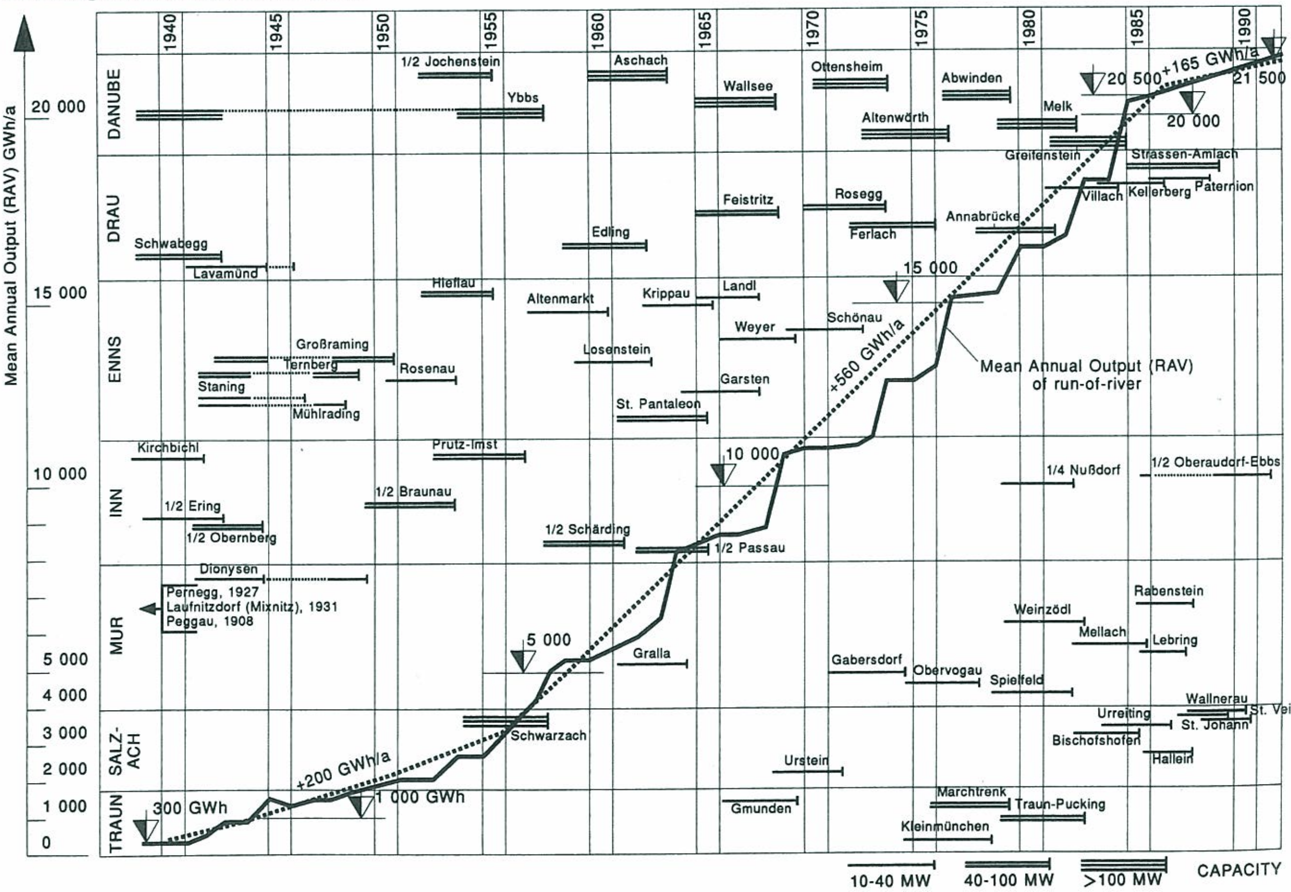
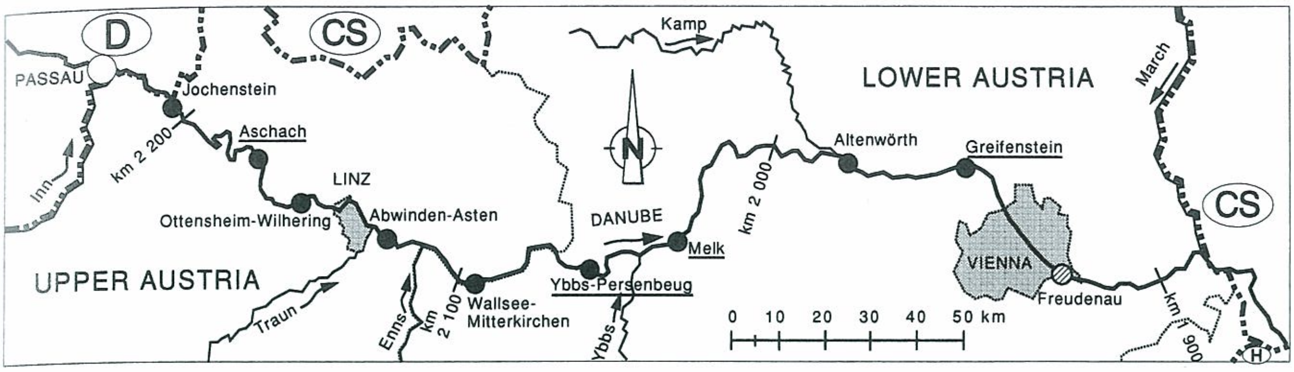
ment, because the resulting flooding of river banks would involve considerable relocations and loss in cultivated land and cultural assets. The most common type of run-of-river power station works under a limited head of between 8 and 16 m and, in reaches of very favourable topography, under heads of up to 20 or 25 m. A few power stations are of the diversion type with long tunnels, representing in fact a transitional type between run-of-river station and alpine high-head station. Apart from their primary activity of energy generation, many run-of-river projects in Austria carry out a number of other important activities. This is particularly true of the development of the Danube as a high-capacity waterway. But other run-of-river projects, too, may be called multi-purpose, especially with respect to flood protection for the banks, prevention of natural river bed degradation, and measures for handling sewage problems so as to contribute towards water pollution control and, as a consequence, to the creation of touristical resources, etc.

Table 3
Chronological table of run-of-river plants



DANUBE

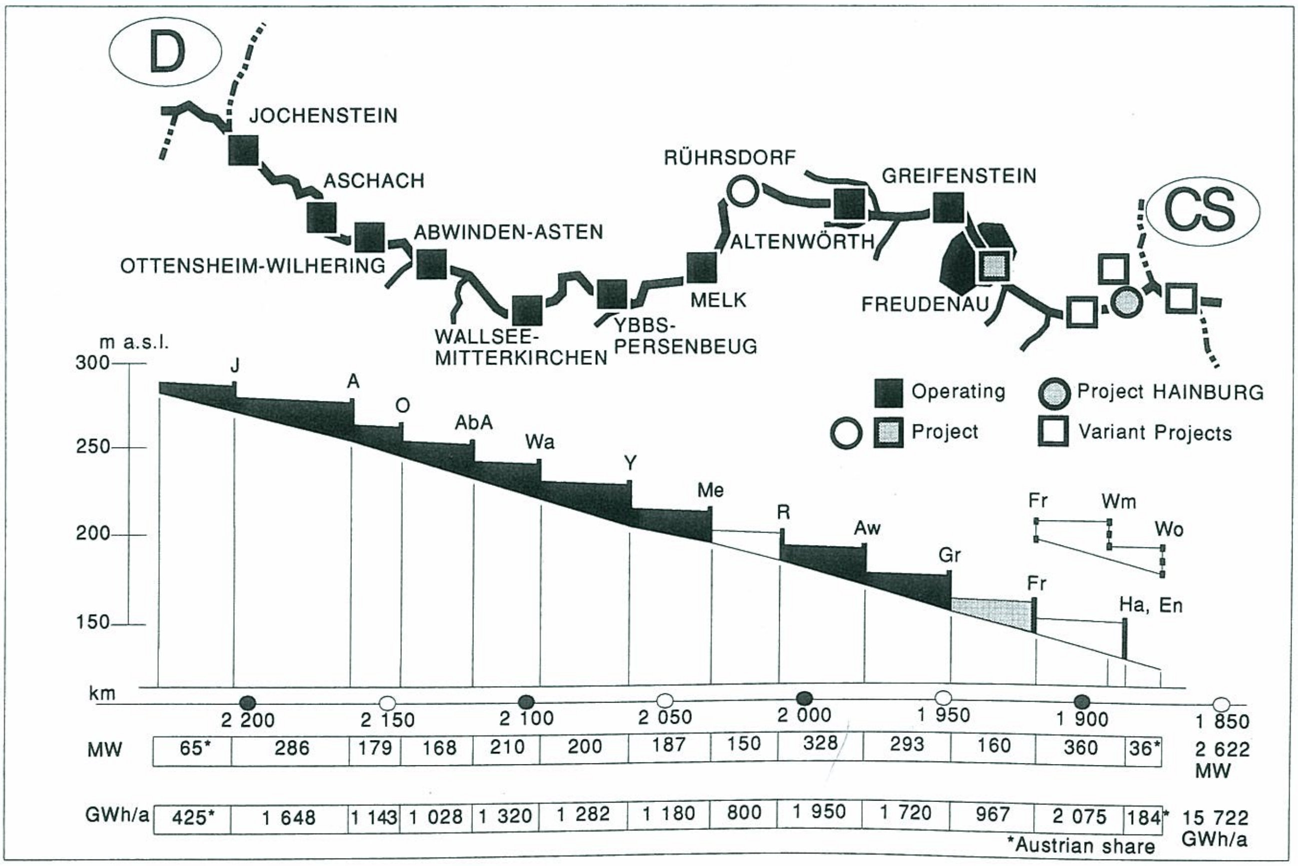


GENERAL

The Danube, Europe's largest river, crossing eight countries along its path from West to East, has not only been a factor of great historical and cultural consequence, but has for many centuries been a boundary and oftener still a link for Europe. Above all, however, the Danube has always been a traffic route and will in two years constitute an essential part of the Rhine-Main-Danube trans-European waterway. Along its course through Austria, the Danube is characterised both by a considerable gradient and by an abundance of water, resulting from the substantial flows contributed by the river Inn at Passau. Whereas the reach immediately upstream, in Germany has a gradient of only 0.02%, and the almost 1 800 km long eastern reach, down to the Black Sea drops at about 0.006%, except for the gorge development by the Djerdap power station (Iron Gate), the Austrian reach, more than 350 km in length, has a gradient of 0.04%. This involves not only a high energy potential for this country, but also a small channel depth due to the high flow velocities, especially during low flows. A multi-purpose plant on the Danube thus provides the required improvement for navigation as well as valuable and clean domestic energy production.

The great diversity of contributing catchments (Central Alps, foothills) ensures a most favourable distribution of river flow, so that the ratio of winter production to summer production by the Austrian power stations on the Danube

Figure 1
Masterplan of the Danube power-stations (by Österreichische Donaukraftwerke AG, August 1990)



is about 43% to 57%. In wet years (for instance in 1981), this may even be 49% to 51%. All the power stations on the Austrian Danube are multi-purpose installations, which, although planned, designed and constructed by an electricity-supply company, (Österreichische Donaukraftwerke AG [DoKW] in Vienna, "Donaukraft") constitute an essential factor in the improvement of the waterway. This implies that allowance is made for navigation requirements by providing locking facilities, and that on the other hand the government authorities responsible for the waterway payed a share of the total cost (approx. 20%).

Development of the Austrian Danube was commenced when the demand was consistent with the large energy potential available, which actually was not the case until the period after the Second World War. Only some preliminary studies and isolated projects had been prepared before that time. This led to the most welcome result that, prior to the construction of the first power project at Jochenstein, a master plan covering the whole Austrian reach of the Danube was available. Although this was subsequently modified in some respects to allow for the progress of technology – in particular, the number of planned power stations was reduced from fifteen to twelve by combining projects for reasons of economy – the Danube Master Plan continues to be used as a general guide. Its stage wise implementation has been in progress ever since 1953 to 1984, practically for 30 years, and has been characterised by an almost perfect continuity with respect to both time and especially staff. Concentration of planning and design, construction supervision, operation and administration in a single company and the continuity that has partly been accomplished in the execution of construction items and supplies have allowed extraordinary cost savings and reduction in construction time.

Among the power stations shown on the Masterplan (Fig.1) the first was Jochenstein, owned by Donaukraftwerk Jochenstein AG (DKJ), a bi-national company in which the Austrian and the German partners hold 50% of the shares each. The output being shared between the two countries (half and half). The remaining 11 or 12 power stations are owned, operated or planned for construction by Österreichische Donaukraftwerke AG (DoKW) Vienna, "Donaukraft".

The Masterplan covers a river length of 350 km and a total head of approx. 150 m; all the power stations, including half of Jochenstein are capable of a total power of 2 622 MW and a total energy of 15 722 GWh/a. The power stations Jochenstein, Aschach, Ottensheim, Wilhering, Abwinden-Asten, Wallsee-Mitterkirchen, Ybbs-Persenbeug and Melk, as well as Altenwörth and Greifenstein have been completed in the above mentioned perfect continuity of practically 30 years. This continuity suffered an unexpected interruption when work was due to start on a power scheme at Hainburg in the winter of 1984–1985. A powerful demonstration and occupation of the construction site by environmentalists risked to constitute a severe obstacle to the execution of the licensed and officially approved project. Under these circumstances, the government decided to discontinue construction preparation and ordered a "pause for reflection". At the same time, DoKW was ordered to prepare the Freudenau project (Vienna), which would follow downstream of Greifenstein, and to submit project possibilities for the reach between Vienna and the Austro-CSFR boundary, which would consider a proposed national park, the needs of navigation over this 40 km reach of the Danube and the electricity requirements in Austria.

Construction of the Freudenau project is expected to start in 1991, so that energy could be produced after 1995. At present, the Danube in Austria supplies 11 696 GWh/a (mean flow). In 1989, energy production amounted to 11 739 GWh and accounted for as much as 25% of the total domestic consumption of approx. 47 800 GWh.

Whereas Jochenstein, Aschach and Ybbs-Persenbeug are situated at narrow valley sections which the Danube has cut through the granitic gneisses of the Bohemian Massif, all the other stations, that is, from Ottensheim down to Greifenstein are situated in flat lowlands, where it was possible and expedient in each case to locate the main structure lateral to the natural river channel. The advantage of such an arrangement is that the whole project, consisting of powerhouse, spillway and lock, is constructed speedy in a single pit, safe from floods and interference from navigation, obviating the need for provisions to handle floods during construction. In the case of the former group of power projects, the narrowness of the sites allowed only stagewise construction in several successive construction pits. The resulting increase in flood water depth during construction was tolerable there.

Construction sites in the lowlands, with the main structure located besides the natural river channel, compared very favourably in terms of construction time and cost with the sites at the narrow vallery sections. Another essential difference concerns the effect of floods in the backwater areas. Whereas between the high banks the entire flows remain within the river channel even during floods, large-scale flooding is an essential characteristic of the riverine lowlands. In order to maintain this for ecological reasons, and for its flood retarding effect on downstream reaches as well as for economic reasons, overflow sections were provided at all the lowland stations except for Melk. For this purpose, some of the flanking embankments were constructed lower and designed so as to allow floods in excess of a given magnitude to flow out into the areas that used to be attained by the floods prior to development of the river. This prevents aggravation of the flood situation in downstream reaches. During an extreme flood wave, as much as 20% of the total flow may pass over the overflow sections, and as this drains off in the areas outside the dykes to return to the main river downstream of the power station, spillway design can be limited to 80% of the maximum flood.

A fundamental difference between high-bank and lowland power stations on the Danube lies in the geological conditions. Whereas Jochenstein, Aschach and Ybbs-Persenbeug are founded on granitic gneiss, the lowland stations had to be built mainly on sediments, mostly "schlier", a local variety of shale, covered by alluvial material about 10 to 14 m in thickness. Special foundation measures were required in places to prevent slope caving and to transfer horizontal forces safely into the schlier. At Melk, the structure was founded on densely packed sands, requiring a continuous cutoff, whereas Greifenstein had to be built on flysch of varying mechanical properties.

In the following paragraphs, some special design characteristics of the main features (lock, spillway, powerhouse and dykes) will be discussed in greater detail.

LOCKS

The dimensions of the locks to be provided to allow shipping on the river have been laid down by the International Danube Convention. This provides that twin locks 24 m in width and 230 m in effective length be built within the Austrian reach of the Danube.

In order to allow major trains of barges as are planned in the Eastern countries to go up to Vienna (and to the Korneuburg shipyard), one of the two lock chambers will be 34 m wide and 275 m in effective length at Vienna and downstream.

In spite of the different heads (between 9 and 16 m), depending on the respective river topography, and the resulting differences in lock filling flow, it is considered desirable that all the locks be designed for an uniform filing time of approximately 15 minutes so as to accomplish a uniform capacity of about 40 million t of annual navigation for each scheme on the river. It is only at Jochenstein and Ybbs that locks are filled from the upper approaches and emptied to the lower approaches, through the lock gates. Since the construction of Aschach in 1962, locks have been filled from the impounded headwater and emptied to the tailwater, but outside the approaches, so as to afford greatly improved navigation conditions. The locks including training and quay walls along the approaches cover a length of more than 1 000 m in the direction of flow and account for 50 to 60% of the total concrete volume needed for a power project. Lock chamber walls are comparable to concrete dams as to their statical function, with vertical side walls being a requirement and with the two faces of the middle wall alternatively acting as "upstream" and "downstream" faces in terms of statics. Crest height (in general, 2 m above water level) from lowest foundation is 33 m at Aschach. Wall thickness varies between 10.5 m and 15 m. As the gravel present in the riverbed is an excellent concrete aggregate, attempts to replace the gravity type by a different design have given no economical results. Rationalisation efforts have been aimed at using a cement-saving concrete by careful mix design and by allowing for the moisture contens of all the aggregate components to minimise the watercement factor.

Lock walls founded on rock are statically independent of the chamber floors and are designed to resist unilateral water or earth pressure and ice pressure (maximum pressure, 50 kN/m). For structures founded on the schlier (lowlands), the whole lock cross section (three walls, two floors) is regarded as an articulated chain, so as to include the chamber floors as supporting elements and above all to ensure safety against sliding with the chamber empty or partly empty. For this purpose, joints are grouted after the end of the setting process, as is practised in high concrete dams. Similarly, the extremely high pressures from the mitre gates are absorbed not only by the abutment blocks but also by the adjoining lock wall elements, which are bonded by joint grouting to form a statical unit.

Another important statical factor is the location of the main structural axis relative to the lock, which passes across the upper gate at Jochenstein and Ybbs and across the lower gate at all the other power stations on the Danube.

The walls of lock located in the tailwater (Jochenstein or Ybbs) will be pushed outwards during chamber filling, whereas the walls of a lock situated in the headwater will be pushed inwards when the chamber is empty. Concrete is better suited to meet the requirements of the latter design, which also allows the chamber floor to be included as a supporting element, which is especially desirable where the foundation is prone to sliding.

The overall concept of the power schemes on the Danube provides for the locks to be used besides the spillways for the passage of catastrophic floods, as navigation is stopped anyway in the case of large flows for reasons of flow velocity, clearance under bridges etc. The additional cost incurred for the lock, especially for the steel hydraulic gates, is only a small proportion of what additional spillway bays would cost.

SPILLWAYS

The spillway is intended to handle flows in excess of the turbine flow. Overflow over the lowered gates occurs rarely and then will at first be of a small magnitude. It is only in extreme cases that the spillway bays are used more or less for the discharge of floods. Up to 1974, all the spillways on the Danube were equipped with mechanically driven hook-type double leaf gates. Oil hydraulic driven tainter gates with flaps on top have been used since construction of Altenwörth. All the spillways have an uniform bay width of 24 m, i.e. the same as the lock chambers. An exception is Ybbs with a bay width of 30 m. The uniform width of 24 m allows the application of stoplogs both in locks and spillways and exchange among the power stations. Maximum hydraulic loading is highest at Aschach, equal to 11 000/7x24=66 $m^3/s/m$, or 11 000/6x24=77 $m^3/s/m$, depending on whether the maximum design flood is assumed to be handled by (n) or (n-1) openings. Piers, especially those founded on bedrock, are designed as single structures. Measures are taken in the spillway floors to ensure uplift relief in the case of unwatering for maintenance and repair. At the lowland stations founded on the schlier, spillway bays and piers are either combined to form frames or designed as drop-in girders. A massive concrete key under the weir sill of each station accommodates an inspection gallery which also serves for potential subsequent grouting, as an instrument gallery and for relief and observation of pore pressures, especially at the foundation contact. At Altenwörth, the key connects to a concrete trench cutoff 10 m deep to prevent seepage. Similar provisions had to be made at Melk. In the longitudinal direction, piers are shaped as stepped blocks with subsequently grouted joints.

Cut granite stone facings were first applied over the whole spillway bays (stilling basins) and piers, but were then reduced from project to project. At present, hard-aggregate or high-quality concrete facing is used almost exclusively. End sills are steel-lined. Due to the fairly continuous development of the Danube and main tributaries (Inn and Enns) by series of power stations, bed load transport through the spillways is expected to occur only under extreme conditions.

Apart from Jochenstein, whose spillway is curved in plan and not equipped with a crane runway, all the dams on the Danube are provided with a 120 t to 220 t capacity gantry crane running across powerhouse, spillway and locks. This serves for erection and for lowering stoplogs upstream of the spillway. Crane girders, first of the steel box type, have been of reinforced concrete since Wallsee. The same is true of the intraplant roadway bridges taken across the pier tops.

The piers, up to 41 m high above lowest foundation and only 6.0 m to 7.5 m wide, are outstanding features requiring solid keying in the bedrock, or studs in the "schlier", to resist the substantial water load. In addition, they have to withstand high linear loads where hook-type double leaf gates are present, and concentrated load in the case of tainter gates, with prestressing anchorage fixtures being used in the latter case.

POWERHOUSE

Power station design on the Danube has undergone changes in two respects, although the nine projects have been constructed in relatively rapid succession, between 1952 and 1984. One essential change relates to the selection of the type and number of turbines; these are in chronological order: five at Jochenstein, six at Ybbs, four at Aschach, six at Wallsee, all vertical-shaft Kaplan turbines; since 1970 (commencement of work at Ottensheim), nine horizontal-shaft Kaplan turbines (bulb turbines) each. It should be pointed out right away that, despite this variety and the very different heads involved, it has been possible to maintain a largely uniform inlet and outlet width so as to ensure universal use of stoplogs. The single-part inlets and outlets of the stations equipped with horizontal-shaft turbines are of a width corresponding to half the width of the inlets and outlets (divided into two) of the stations equipped with vertical-shaft Kaplan turbines, as for instance Ybbs, Aschach, Wallsee.

Partly due to the choice of turbines as described above – governed by economic aspects, especially in the structural sector – and partly as a result of the basic project idea, a second development took place in the form of a substantial reduction in powerhouse height above the impounded water level. Whereas this is as much as 22.4 m at Jochenstein, the only plant with the big crane provided in the powerhouse hall, powerhouse height is only 10.5 m at Aschach, 9.3 m at Ybbs and as little as 5.0 m at Wallsee, which does not even exceed the height of powerhouses equipped with horizontal-shaft turbines. It should be pointed out in this context that the level of the powerhouse roof, which serves as a runway for the universal gantry crane described in connexion with the spillway, is dependent to a high degree on the position of the main axis of the power station relative to the lock. Where the main structural axis crosses the lock at the upper gate, as at Jochenstein and Ybbs, the required clear height of 8.0 m above the impounded water level will have an important bearing on the choice of the roof level. At all the other stations, the lock is crossed at the lower gates, where the water level is lower. This also avoids the unfavourable visual effect of a lock wall towering over the tailwater level in its full height. Naturally, powerhouse statics is largely a function of the foundation conditions. As a feature common to all the power stations, there is a structural unit, separated from the adjoining blocks by joints, for each power unit; in this context, reduction of block width from a maximum of 32 m (Aschach) to a value between 17 m and 18 m for the bulb turbines has not only been a great advantage in terms of statics, but has helped to overcome construction problems and avoid cracking. In some cases, the joint separating the power unit blocks is deliberately connected to the adjacent water pressure.

Viewed in the direction of flow, powerhouses equipped with vertical-shaft units show a structural division into inlet, middle block and outlet, mainly to answer the geometrical requirements of spiral casing and draft tube bend. Mutual statical support between the three powerhouse elements is ensured. In the powerhouses equipped with bulb turbines, the inlet structure, much shorter in this case, directly passes into the middle part, which houses the power units.This is an essential advantage of this arrangement. Only the outlet structure is constructed as a separate element but then made to form a statical unit with the whole powerhouse cross section by means of joint grouting. It should finally be mentioned that it has lately been made a rule to place gravel fill on the draft tube for ballast.

At the power stations founded on the schlier, deep excavations (Wallsee) first presented some difficulties from slides; this gave rise to the development of keying structures in the form of concrete-filled wells acting as "studs" projecting downwards from the base of the structure. In addition, large-area excavation, especially in the powerhouse area, called for special precautions to protect the schlier, which tends to heave when exposed and later settles again when covered.

DYKES

It would be very much beyond the scope of this report to mention even the most important problems, their solution and the structural measures taken in the backwater areas above the power stations on the Danube. Therefore, discussion will be limited to the embankments in the backwater areas on the Danube. Whereas very few dykes were provided at the high-bank power sites, Jochenstein, Aschach and Ybbs, they constituted a very important structural and cost item at the power stations in the lowlands.

Having to retain the impounded water so as to protect low-level country behind (riverine lowlands or cultivated land), dykes have to answer high stability standards and must ensure watertightness both within the structure and more or less also in the foundation. In addition, waterside facings of new dykes must meet the requirements of a navigable river as well as the requirements resulting from ice formation, water level variations and ecological and biological needs. A special case are the overflow sections mentioned earlier in this report.

The development of dyke cross sections reflects technological and economic progress made in recent construction methods. A decisive feature is obviously dam height above ground level. Fill volumes at lowland power stations vary between 5.0 and 12.0 million m^3 and may well compete with large dam projects in terms of volume of earth moved.

At the first lowland power station, Wallsee, imperviousness was achieved by two light-weight sheet walls (35 to 40 kg/m^2). The lower one was vibrated into position from ground level down to the impervious schlier. Then a strip of bitumen was placed at the head of the sheet wall and the dam fill deposited on top. The second sheet wall was then carefully sunk by vibration through the dam fill and into the bitumen strip. In some very low dykes, impervious cores consist of a special mix of sandy gravel and riverside sand. Bank protection is by riprap everywhere on the Danube.

In the construction years 1970 to 1973, more economical methods were developed for the Ottensheim project. These included the use of thin diaphragm cutoffs, constructed by vibrating an I-beam some 20 m deep in alluvial material, then injecting a mix of cement, floursand and bentonite when gradually withdrawing the I-beam, so as to form an impervious diaphragm. In addition, bituminous concrete was used for the first time on the Danube for sealing a waterside slope. This was however limited to a bank length of 5.6 km, where site conditions allowed a gently sloping fill of natural gravel to be placed in front, which saved riprap and accomplished a flat bank of great biological value. For the rest, the development of impervious core material (gravel/sand mix) from natural sources, its processing and continuous test measurement during construction were systematically pushed ahead.

At Altenwörth, involving the largest amount of dyke fill the above-mentioned 12 million m^3, thin diaphragm were the only foundation cutoffs used. For low embankment heights, the diaphragm was connected to an impervious core. Where major fill heights were concerned reaching a maximum of 12 m above ground level at Altenwörth, the thin diaphragm was arranged at the waterside toe and connected to a bituminous slope facing carried to a level above the top water level. This facing, serving as a sealing and bankprotection element, was applied in a single 8 cm layer and, in the area of potential berthing impacts, in two 8 cm layers. Upwards from a berm provided on the bitumen facing below top water level, the facing was covered with riprap as a protection against wave action from navigation and potential damage from ice, as well as to allow men and animals to climb up and down, and above all to answer the requirements of stream biology in the important uppermost, light-filled, water layer; the latter is an important factor enhancing the self-purifying capacity of the dammed-up river.

From the economic point of view, the above mentioned bituminous slope facing was not acceptable for fill heights less than 8 or 10 m. Another important economic criterion is the availability within easy haul distance of quarries for the large riprap stone requirements. At Altenwörth, almost 1.8 million t of stones were placed in the main structure and in the backwater area, within a period of about two years.

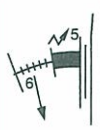
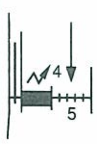
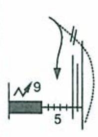
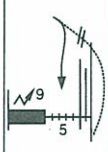
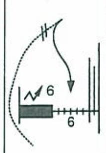
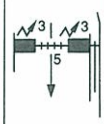
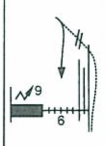
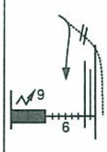
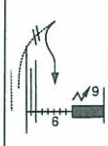
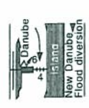
Cut-offs are carried down to the impervious schlier (some 10 to 14 m below ground level) only where the impounded water level is at least 3 or 4 m above ground level, or near towns and villages. Otherwise a short cut-off extending into the gravel foundation is sufficient to prevent seepage flow from reaching unacceptable magnitudes. An important element in dyke structures is the landside drainage channel (bottom some 1.0 m below mean water level, original condition), serving to maintain the inland water table at the original level or to control it as desired; special water release structures (inlets to convey stored water to riverside plains) are provided for this purpose.

A special design is required in the so-called overflow sections of the flanking embankments. Whereas all the other dykes at the power stations on the Danube are not overflown even during maximum floods and have unpaved landside slopes sown with grass seed, overflow sections allow part of flood water to flow out into the hinterland. For this reason, the heavy riprap protection is not limited to the waterside slope, but covers the crest, the landside slope and a widened channel along its toe, serving as a stilling basin. A lightweight sheet wall in the middle of the dyke, extending from the crest down to the gravel foundation or below the channel bottom prevents erosion within and below the riprap. Dyke cross section and stone size were tested and determined on hydraulic scale models.

As overflow sections are rarely in action (approx. every two or three years) and, if so, nearly never over their whole lengths, landside riprap-protected slopes are covered with riverside sand and seeded. Otherwise they would look barren and ugly. It has even been found out that this grass serves as an additional slope and crest stabilisation. The risk of potential destruction during floods is accepted.

It should be mentioned that in special cases plastic foil has lately been used as a sealing element in permanent embankments on the Danube. This development was spurred by the excellent experience made with sealing of construction-pits up to 1.4 km² in area by providing thin diaphragm cut-offs in the foundation and plastic foil in the fill. This method has completely replaced the former expensive sheet piling, and has contributed a great deal towards accelerating and rationalising the construction process.

Danube

Power station		Jochenstein	Aschach	Ottensheim-Wilhering	Abwinden-Asten	Wallsee-Mitterkirchen	Ybbs-Persenbeug	Melk	Altenwörth	Greifenstein	Freudenau
Owner		DKJ	DoKW	DoKW	DoKW	DoKW	DoKW	DoKW	DoKW	DoKW	DoKW
Operation since		1955	1963	1973	1979	1968	1957	1982	1976	1984	Project
Stationing	km	2 203.3	2 162.7	2 146.7	2 119.5	2 093.6	2 060.4	2 038.0	1 979.8	1 949.2	1 921.05
Storage level	m	290.3	280.0	264.0	251.0	240.0	226.20	214.0	193.5	177.0	161.35
Flow Q_{mean}	m³/s	1 430	1 450	1 450	1 600	1 730	1 750	1 807	1 830	1 882	1 700 *
Flow Q_{max}	m³/s	8 900	8 900	8 900	9 500	11 100	11 100	11 170	11 170	10 750	8 200 *
Flow Q_{rated}	m³/s	1 750	2 000	2 250	2 475	2 600	2 100	2 700	2 750	3 150	3 000 *
Head H_{mean}	m	10.20	15.30	10.70	9.30	10.90	11.0	8.4	14.80	12.6	8.5
Capacity (MC)	MW	(130) 65	286	179	168	210	200	187	328	293	165
Energy (AAE)	GWh	(850) 425	1 648	1 143	1 028	1 320	1 282	1 180	1 950	1 720	1 017
Layout											
Spillway/Weir											
Bays width	m	6×24	5×24	5×24	5×24	6×24	5×30	6×24	6×24	6×24	4×24
Pier width/height	m	5–6/31	7.10/41	7.50/37	6.0/37	7.50/37	7.50/34	6.0/31	7.0/37	6.0/31	6.0/33
Q max_{100}	m³/s	8 900	8 900	5 940	8 450	8 600	11 100	11 170	9 785	8 650	6 530
Gates		hook-double	hook-double	hook-double	tainter + flap	hook-double	hook-double	tainter + flap	tainter + flap	tainter + flap	tainter + flap
Powerhouse											
Construction, type		high	medium	low	low	low	medium	low	low	low	low
max height	m	52	53	39	40	42	42	39	46	44	47
Turbines, number and type		5 Kaplan ↓	4 Kaplan ↓	9 Kaplan →	9 Kaplan →	6 Kaplan ↓	6 Kaplan ↓	9 Kaplan →	9 Kaplan →	9 Kaplan →	6 Kaplan →
Backwater area											
Length	km	27	40	16	27	26	33	22	32	31	28
Overflow flood	m³/s	–	–	2 960	1 050	2 500	–	–	1 385	2 100	2 200**

* Danube River only

** New Danube – Flood diversion

YBBS-PERSENBEUG

River	Danube, at km 2 060.4	Operating since	1957
Nearest town	Ybbs, Lower Austria	Purpose	Hydropower Inland navigation Flood protection
Owner	DoKW Österreichische Donaukraftwerke AG Parkring 12, A-1010 Vienna		

MAIN CONSTRUCTION PARTS AND FUNCTION

Barrage	Weir: 5 openings of 30 m each Power station: 2 parts (south and north of weir), 2x3 turbines, 200 MW in total Locks: 2 chambers, 24x230 m

Apart from developing the river for power generation, an important purpose of the project was to improve navigational conditions in the Strudengau, a narrow part of the Danube valley, by flooding shoals and eliminating other obstacles such as cross-currents, one-way sections and the need to split up trains of barges.

HYDROLOGY

Catchment Area	92 464	km²
Q mean	1 750	m³/s
Flow Q max	11 100	m³/s
Q rated	2 100	m³/s
Storage Level (a.s.l.)	226.2	m
Mean Head	11.2	m
Capacity	200	MW
Energy Output	1 282	GWh/a

GEOLOGY

At the barrage site, the bedrock is composed of various types of gneisses cut up by a dense network of joints and minor faults, and two wide fault zones passing in the direction of flow. It is covered by varying thicknesses of gravel overburden. The high permeability of the rock made it necessary to provide a grout curtain and uplift relief downstream; effectiveness of these measures is checked at regular intervals.

The main part of the backwater area (10 km , 33 km in length) is situated within a narrow valley section where the Danube has cut through the granite formations. The most upstream part is bordered on both sides by overflow levees, closing off the low-lying areas from the backwater during low flow but providing retention space during floods, as was the case prior to construction of the barrage. Drainage of the polders is ensured by two automatic pumping stations (of a capacity of 16 m^3/s and 10 m^3/s respectively).

HISTORY AND DEVELOPMENT OF DESIGN

A project prepared by a Swiss engineer named Hoehn had been submitted to the water management authorities as early as 1928. Preparatory work was performed by Rhein-Main-Donau AG between 1938 and 1944, but was delayed and finally discontinued because of the war. The cofferdam from that period was then used for constructing the locks beginning in 1954. Some project plans had placed the barrage close to Persenbeug castle, but the final design located it further upstream at the upper end of the locks. The weir is at some distance from the long south wall of the lock and is flanked by the two parts of the powerhouse to ensure better flow conditions. The powerhouses are of the semi-outdoor type with low generator halls and two 135 t capacity gantry cranes traveling on the crest of the whole station (powerhouses, weir and locks). A public road bridge is located downstream of the crane track. Construction was performed in several successive pits in the river channel and the right bank, enclosed by double or single walled cofferdams (phases 1 and 2) and by cellular cofferdams (phases 3 and 4).

FOUNDATION

In view of the permeability shown by the granite/gneiss, it was necessary to provide a three-line grout curtain extending over the whole base, 27 m in depth, 6 400 m in total drill hole length. Average grout acceptance was 46 kg per linear meter of drill hole. Imperviousness of the right-hand bank is ensured by a slurry trench cut-off 145 m in length and 24 m in maximum depth, with a total area of 2 500 m^2. Connection to the bedrock on the left-hand bank is ensured by a short concrete wall constructed in an open pit.

INSTRUMENTATION

Geodetic monitoring of all structures. Jointing of the bedrock requires close observation of uplift pressure, performed by 114 instrument locations, 54 of which are equipped for seepage measurement.

CONCRETE

Aggregates were available from the overburden at the barrage site. They were washed and screened into five fractions to 105 mm in maximum size, with the addition of sand from a different source. Two concrete qualities were produced: bucket concrete with 220 kg of type 225 Portland cement per m^3, and pumped concrete with a maximum particle size of 65 mm and 260 kg of the same Portland cement per m^3. The water/cement ratio was 0.60. Concrete strengths reached at 28 days were 36 N/mm^2 in compression and 6 N/mm^2 in tension. Strengths, imperviousness and frost resistance were checked in the on-site lab. Locks were built of bucket concrete poured in blocks of 15 m in lengths and in lifts of 2.4 m. The powerhouses and weir were mostly made of pumped concrete using pumps for distances of up to 400 m.

FLOOD RELIEF WORK

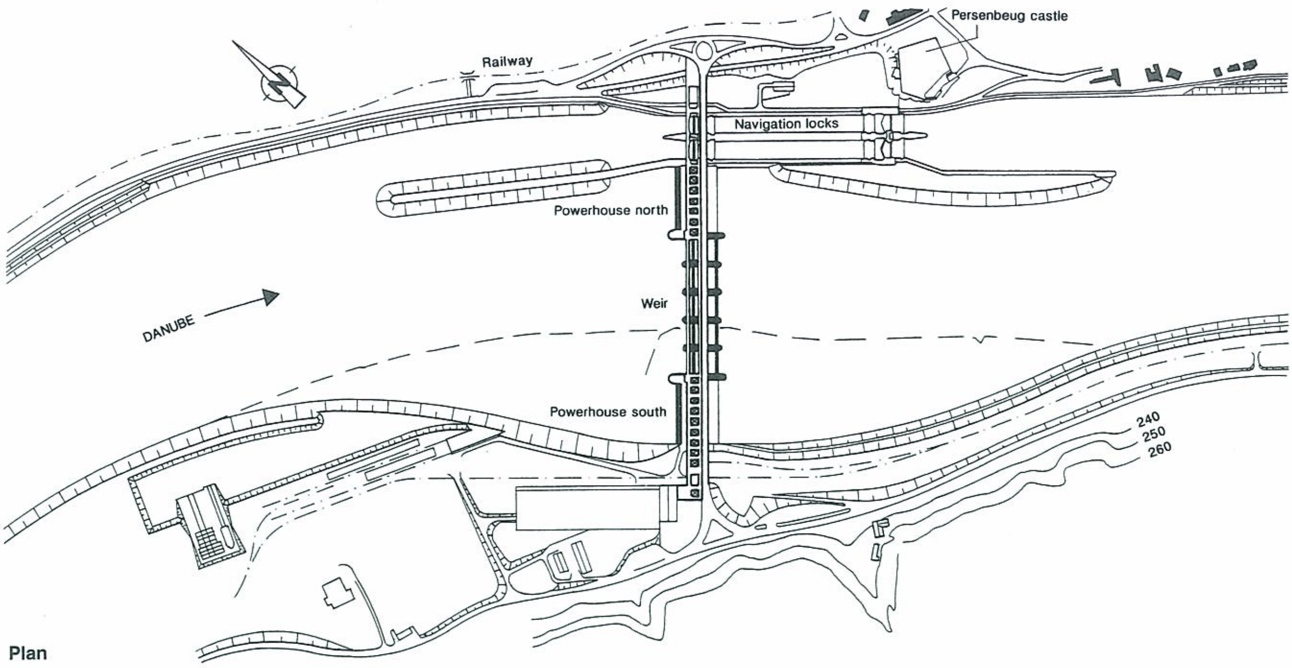
Weir with 5 openings of 30 m in clear width each, closed by 13.5 m high hook-type double-leaf gates; the inlet sill is at elevation 213 m. Granite lined stilling basin with end sill and scour protection at the outlet. Gate area is 1 960 m^2.

Locks are also used for flood discharge. For this purpose, the upper gates are equipped with hook-type double leaves with a total area of 620 m^2.

Total discharge of the design flood (Q max) of 11 100 m^3/s is through the weir and one lock chamber, ensuring that the natural flood level is not exceeded. With one weir opening blocked, the maximum unit discharge is 64 m^3/s per linear meter of weir and lock sill.

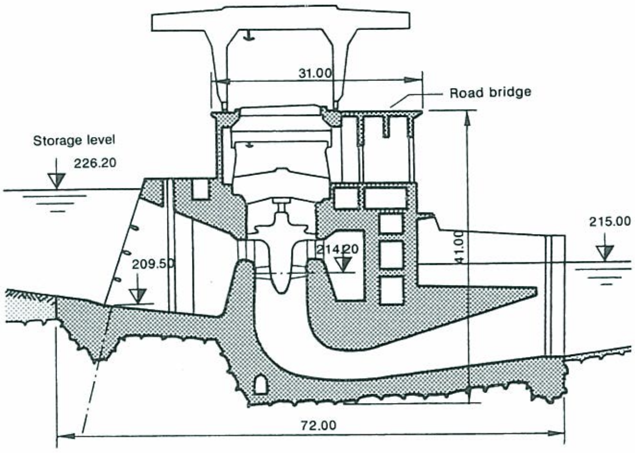
POWERHOUSE

The two powerhouses (north and south) flanking the weir are each equipped with three vertical-shaft Kaplan turbines of a rated discharge of 350 m^3/s and a maximum capacity of 35 MW. They are coupled to three-phase generators of 45 MVA capacity. The turbine runners, 7.4 m in diameter, are located at elevation 214.2 m, i.e. 0.8 m below mean tailwater level.

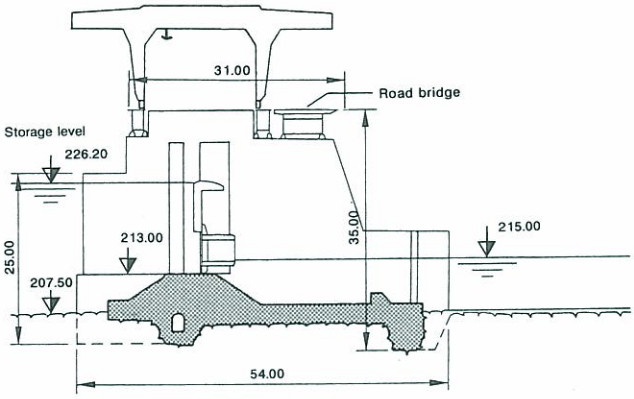


Plan

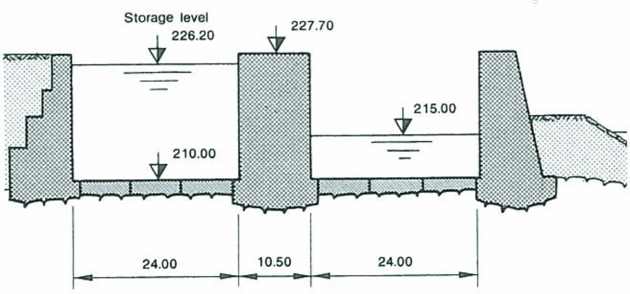
Powerhouse section



Weir section



Lock section



MAIN DIMENSIONS

	Weir	Powerhouse
Height	35 m	41 m
Length	5x30+4x7.5 = 180 m	2x93 = 186 m
	366 m in total	

The locks consist of two chambers with an effective area of 24x230 m each. Normal height is 23 m, at the gates more.

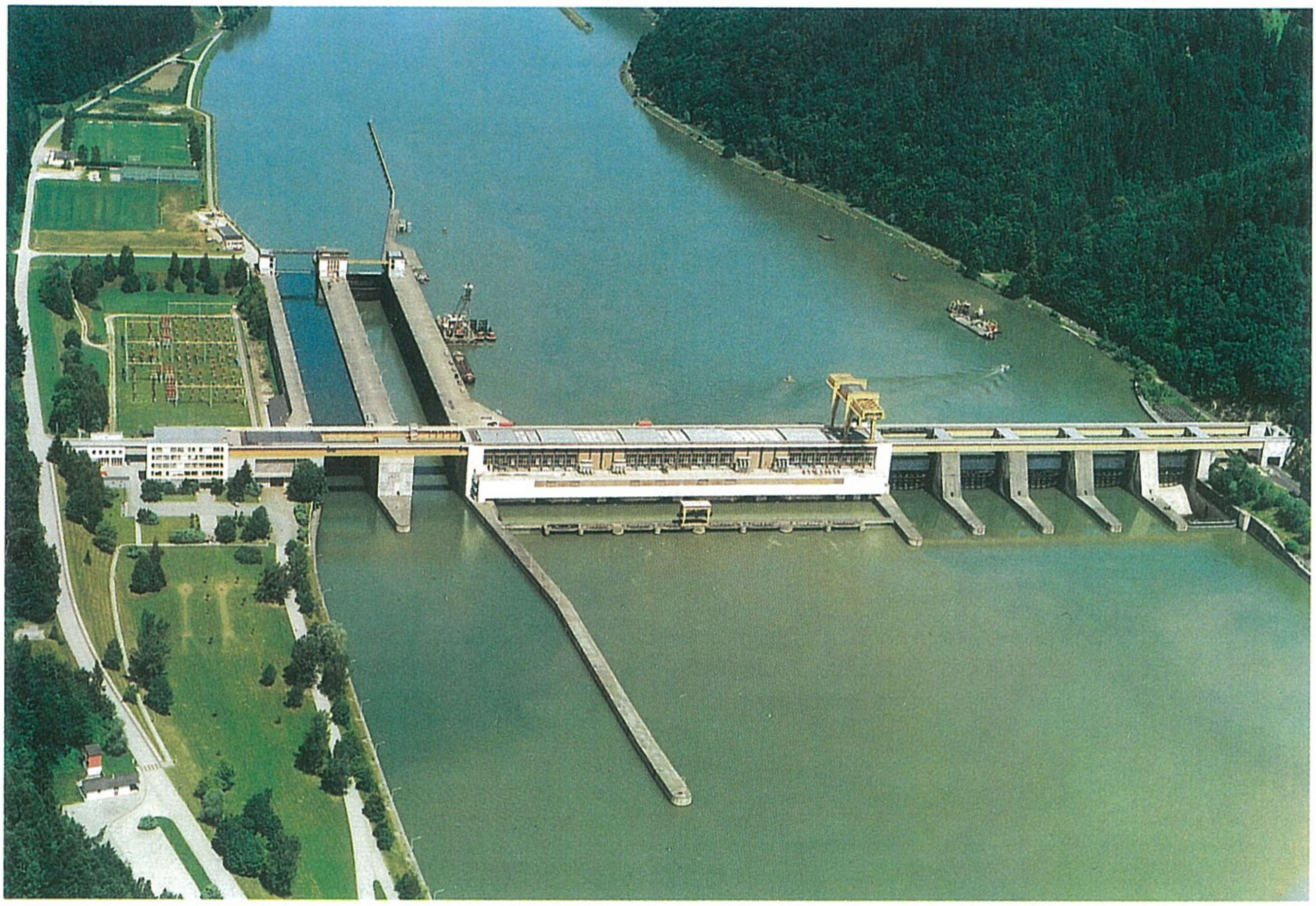
VOLUMES OF BARRAGE

Excavation	
Overburden	1 600 000 m³
Rock	280 000 m³
Concrete	690 000 m³

VOLUMES OF BACKWATER AREA

Embankment and Land Raising	2 900 000 m³
Rock excavation (incl. blasting of obstacles for navigation)	350 000 m³
Riprap	300 000 m³
Concrete Walls (roads)	45 000 m³

ASCHACH



River	Danube, at km 2 162.7	Operating since	1963
Nearest town	Aschach, Upper Austria	Purpose	Hydropower Inland navigation Flood protection
Owner	DoKW Österreichische Donaukraftwerke AG Parkring 12, A-1010 Vienna		

MAIN CONSTRUCTION PARTS AND FUNCTION

Barrage	Weir: 5 openings of 24 m each Power station: 4 turbines, 286 MW in total Locks: 2 chambers 24x230 m, filling and emptying system in the floor of the chambers

The station opened a 40 km stretch of the Danube between Jochenstein and Aschach for power generation and improvement of navigation in this very narrow part of the river valley. Power generation efficiency is enhanced by the large head and the high winter percentage of annual energy (almost 40%).

HYDROLOGY

Catchment Area	78 190 km²
Q mean	1 450 m³/s
Flow Q max	8 900 m³/s
Q rated	2 000 m³/s
Storage Level (a.s.l.)	280 m
Mean Head	15.3 m
Capacity	286 MW
Energy Output	1 648 GWh/a

GEOLOGY

Fine-grained granite largely free of faults is encountered at a depth of 8 m below the river bottom. Ice-age glaciers removed friable and weathered material with the result that foundation conditions are excellent. Some 90 drill-holes were sunk to determine the best place for the barrage, and a horizontal drillhole (300 m long) under the riverbed was carried out from a shaft on the left bank to check for fissures, faults or gorges which might put the planned barrage at risk.

The backwater area (40 km in length) extends to the upstream Jochenstein power station. It is situated in a very narrow granite valley, with no road for most of its length and a few small villages at the mouth of tributaries only.

HISTORY AND DEVELOPMENT OF DESIGN

After Ybbs-Persenbeug, Aschach is the second power station at the Danube constructed solely by Austria. It is situated near the downstream end of a narrow rocky stretch of the river extending upstream to the next power station at Jochenstein (owned equally by Austria and Germany) and on to Passau. Its length of 40 km provides for a head of nearly 16 m, which in turn translates into a high capacity. Planning started in 1957, construction in 1959. The arrangement of a powerhouse with four turbines in the middle of the river, flanked by the weir on the left bank and locks on the right bank, both partly behind the natural river banks, was tested hydraulically in a model of 1:80 scale.

The powerhouse is of the semi-outdoor type, equipped with a 16 t capacity indoor crane and a 220 t capacity gantry crane traveling on the crest across the whole structure.

Construction was in open pits, in three phases. The cofferdams consisted partly of circular cells, but mostly of gravel fill dams with a central sheet piling extending to the bedrock. Due to the substantial space requirements for the cofferdams, the transition block (between weir and powerhouse) which formed part of the longitudinal enclosure of phases 2 and 3 was built by pneumatic sinking. The powerhouse was constructed at the same time that the second lock chamber was completed, after diverting navigation to the southern lock 23 months after start of construction. The first turbine was operable after a total construction period of 45 months.

FOUNDATION

A double grout curtain of clay-cement mix with an area of 1 450 m^2 was injected. Average grout acceptance was 43 kg cement per m^2. On the left-hand bank the weir abutment is directly embedded in the bedrock, on the right-hand bank water tightness is ensured by single steel sheet piling.

INSTRUMENTATION

Leveling in the inspection gallery; approx. 130 instrument locations for uplift pressure and seepage. Effectiveness of uplift pressure relief is checked every two years.

CONCRETE

The aggregates were obtained for the most part from a borrow pit 5 km from the site, screened into 6 fractions of 120 mm maximum size, and hauled to the site by rail at a daily capacity of 8 500 tons.

Two types of concrete were produced: bucket concrete of 200–220 kg of cement per m^3 of concrete, and pumped concrete with 260 kg of cement. The concrete was mixed at a total rate of 330 m^3/h in special towers on both riverbanks, but mainly on the right-hand side for the locks and powerhouse. It was transported by trucks to the tower cranes for the locks and central pumping station. The maximum pumping distance was 200 m. Bucket concrete was poured in 2.4 m lifts.

FLOOD RELIEF WORK

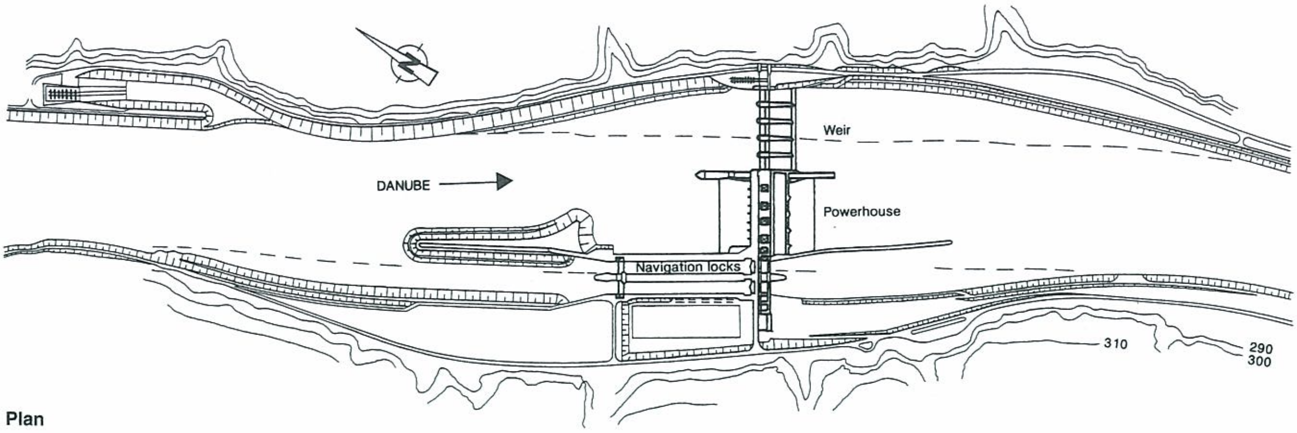
Weir on the left-hand bank with five openings of 24 m in clear width each, closed by hook-type double-leaf gates 15.8 m in height; the inlet sill was at elevation 261.0 m during phase 3 (powerhouse under construction and one lock only for navigation), and was then raised by 4 m to its final elevation of 265.0 m using stoplogs.

Locks also serve for flood discharge. The upper gates of both chambers are therefore of the same type as those at the weir. A special feature at Aschach is its filling and emptying system through canals and slots in the bottom of the lock chamber.

Total discharge of the design flood (Q max = 8 900 m^3/s) is possible through five weir openings and one lock, or four weirs and two locks. Q max is then 64 m^3/s per linear metre of opening. Together the seven openings (5+2) discharge 11 000 m^3/s.

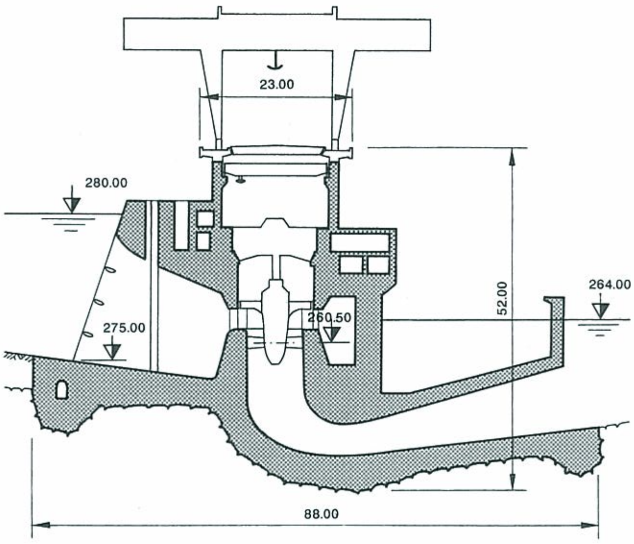
POWERHOUSE

The powerhouse is situated between the locks and weir and is connected to the switchyard and operational building by bridges above the tailwater. It is equipped with four vertical shaft Kaplan turbines for a rated discharge of 510 m^3/s and 72 MW maximum capacity each, coupled to generators of 85 MVA. The turbine runners are 8.4 m in diameter (at the time of their delivery in 1963 the largest in Western Europe). They are located at elevation 260.5 m, i.e. 3.50 m below mean tailwater level.

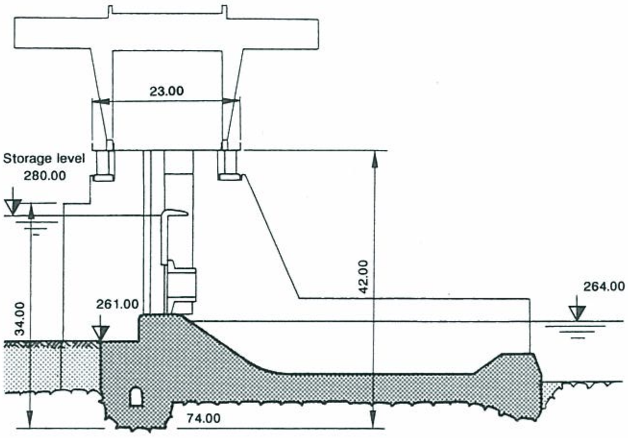


Plan

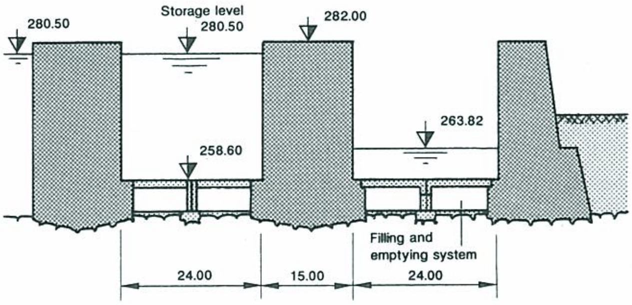
Powerhouse section



Weir section



Lock section



MAIN DIMENSIONS

	Weir	Powerhouse
Height	42 m	52 m
Length	5x24+4x9 = 156 m	164 m
	320 m in total	

The locks consist of two chambers with an effective area of 24x230 m each. Normal wall height: 33 m, normal width of the central wall: 15 m.

VOLUMES OF BARRAGE

Excavation		
	Overburden	2 800 000 m³
	Rock	280 000 m³
Concrete		1 050 000 m³

VOLUMES OF BACKWATER AREA

Land Raising	2 500 000 m³
Riprap, Slope	590 000 m³
Concrete Walls	45 000 m³

MELK

River	Danube, at km 2 038.0	Operating since	1982
Nearest town	Melk, Lower Austria	Purpose	Hydropower Inland navigation Flood protection Environment and ecology
Owner	DoKW Österreichische Donaukraftwerke AG Parkring 12, A-1010 Vienna		

MAIN CONSTRUCTION PARTS AND FUNCTION

Barrage	Weir: 6 openings of 24 m each Power station: 9 bulb turbines, 187 MW in total Locks: 2 chambers, 24x230 m

The backwater area is 22.5 km long and ends at Ybbs-Persenbeug. Most of the construction work concentrated on the lower 13 km, especially around Pöchlarn and some small villages nearby.

The Melk power station was built as a multi-purpose project with due consideration to all economic and ecological factors, to produce a harmonious balance between the two requirements. Major concerns governing its design and construction were blending the project with its vicinity dominated by Melk Abbey, energy generation, flood protection in the backwater area, improving conditions for navigation, and creating recreational facilities and nature preserves.

HYDROLOGY

Catchment Area	94 545	km^2
Q mean	1 807	m^3/s
Flow Q max	11 170	m^3/s
Q rated	2 700	m^3/s
Storage Level (a.s.l.)	214	m
Mean Head	8.4	m
Capacity	187	MW
Energy Output	1 180	GWh/a

GEOLOGY

The area covered by the project is characterized by a highly complex geological structure. A detailed image of the structure emerged only after long-term and comprehensive geophysical examinations (magnetic, seismic, gravimetric tests) and a great number of bores. The power station itself sits at the Diendorfer fault, one of the major geological faults at the southern rim of the Bohemian Massif.

After several relocations, a practical site was finally located that permitted founding the works uniformly on tertiary sediments (Melk sand). Only a few deep-lying parts (spur of the weir) reach down to the tertiary clay or the primary rockbase.

HISTORY AND DEVELOPMENT OF DESIGN

Melk closes the chain of power stations up to Jochenstein, as envisaged in the master plan. In addition to generating 1 180 GWh/a, it also eliminates several obstacles to navigation and shallow spots by its submergence. Embankments and cut offs built in the backwater area reduce the flooding risk for numerous settlements and in one case (Pöchlarn) ensure total flood protection.

Analogously to the Danube power stations erected before, Melk once again exploited the benefits provided by the situation in a flat terrain. The barrage was built on the right-hand bank in a single large-scale pit which cut down on construction time and did not impede navigation. The former course on the left-hand side was remodelled into a recreational and watersports area. By using bulb turbines, the barrage could be kept very low and thus blends harmoniously with its setting, without impairing the view of Melk Abbey.

FOUNDATION

The final site of the barrage on the right-hand bank at kilometer 2 038 is situated on an overburden of 10 to 16 m. Of this some 4 m is wetlands sand, and the rest is a conglomerate of sand and gravel intermingled with some large blocks. The structures are all founded on the Melk sand below this stratum.

An exploratory shaft was sunk to 10 m below the lowest foundation bottom before construction began. By that it was possible to extract some intact soil samples and obtain information on the compactness and permeability of the Melk sand. An apron (cut-off-curtain) underneath the barrage reduces underseepage to acceptable levels. A gallery is installed to allow subsequent grouting.

INSTRUMENTATION

Geodetic surveillance, monitors for settlement, uplift pressure, etc. permit continuous observation and monitoring of all structures.

CONCRETE

Aggregates were taken from the pit excavation material and screened into six fractions up to max. 64 mm at a gravel plant. A total of 900 000 m³ of concrete was produced, shipped to the site by truck dumpers (6 m³) and poured by telescope crane belt conveyors with a capacity of 40-50 m³/h each.

A lab operated by DoKW monitored aggregates, their contained moisture, cements and admixtures, concrete strength, frost resistance, etc. during construction time.

FLOOD RELIEF WORKS

Weir: The weir between the powerhouse and locks has six openings of 24 m in width each, spaced by five piers. The openings are shut by oil-hydraulic tainter gates with flaps on top. Upstream and downstream stoplogs are provided for maintenance purposes.

The six openings of 24 m each at Melk have the same effect as the five openings of 30 each of the Ybbs weir, the next link in the chain upstream of Melk, because both stations allow no flood relief from the backwater. Q max = 11 170 m³/s.

Locks: The locks can also serve for discharging extremely high floods; therefore the upper gates are the hook-type double-leaf gates with oil-hydraulic drive. The lower gates are fitted with mitre gates.

POWERHOUSE

The powerhouse on the right-hand bank accommodates nine turbo-generator sets, each consisting of a horizontal Kaplan bulb turbine with an output of 21 MW, a runner diameter of 6.3 m and a 24 MVA, 9 kV three-phase generator. The units each take up 18 m in width. The design water volume was defined as the 50-day flow Q rated = 2 700 m³/s. When the turbines are opened excessively, the station's absorption capacity can be boosted to 3 000 m³/s.

A turbine room crane with a carrying capacity of 25 t travels across the nine sets and assembly bays. A 120 t gantry crane running on a truck along the powerhouse roof covers the entire barrage structure (powerhouse, weir and locks) via crane bridges.

The use of bulb turbines permitted very low building heights to fit the structure into its surrounding. For scenic considerations, the open-air switchyard was moved 4 km away from the vista of Melk Abbey and linked to the powerhouse by high-voltage cables.

BACKWATER AREA

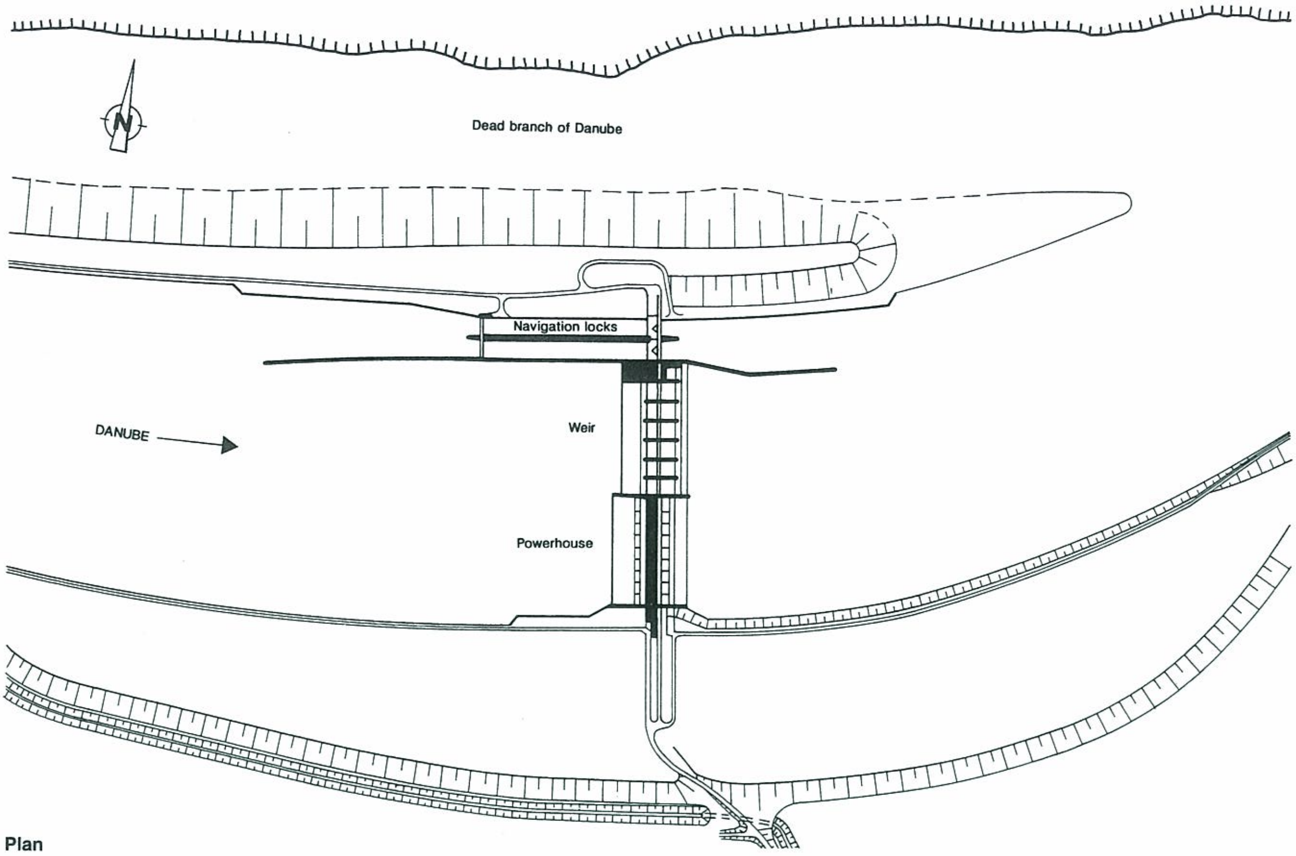
The Melk backwater extends over a length of 22.4 km from the barrage (at kilometre 2 038) to the Ybbs-Persenbeug power station (at kilometre 2 060.4). Most of the work concentrated on the lower 13 kilometres of the backwater area. Settlements on the right-hand banks now enjoy total flood protection, and the flooding frequency could be significantly reduced on the left-hand side.

Gravel and wetland sand for the embankment fillings were extracted from gravel banks in the Danube and from the main excavation pit. Cut-off was provided by diaphragms and some subterraneous curtains. Dam fills have a core of gravel/sand mixture (wetlands sand). The slopes were faced with riprap on the water side and harmonized with their surrounding on their off-water side. A system of parallel and drainage ditches on the land side of the slope ensures that the original mean ground water table could be maintained.

ENVIRONMENT AND ECOLOGY

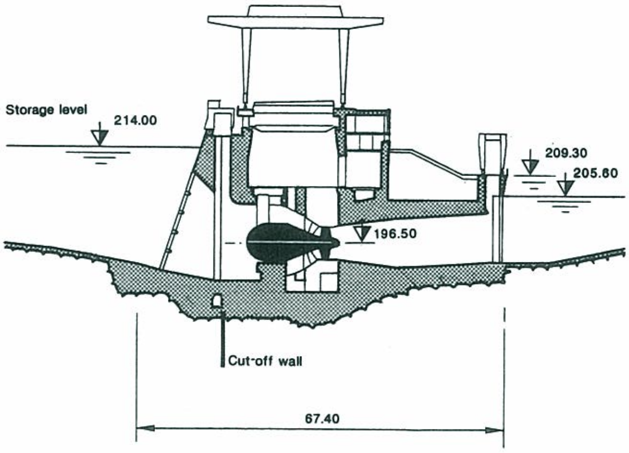
The Melk power station has succeeded in combining engineering and ecology into a harmonious whole by meeting recreational and leisure-time needs, ensuring flood control, protecting cultural monuments and taking into account both ecological objectives and technical requirements. The original riverbed of the Danube now on the left-hand bank was redesigned to serve as a recreational center, and a new yachting harbor was built at its downstream end. An aquatic bird sanctuary was created above the original course in cooperation with the Nature Conservation Board and provided with suitable vegetation and man-made gravel islands to serve as landing spot for water fowl.

Flood protection structures helped revitalize town cores of adjacent communities, particularly in the Nibelungen town of Pöchlarn. Landscaping activities on the flat embankment slopes have created picnic spots and walks. The water regime of the Melk Branch, an existing waterway next to Melk, was repaired by comprehensive solutions for waste-water problems and by building compensation water outlet works to supply the branch with fresh water. A sewage system and water treatment plant combine to achieve a year-round Danube-water quality of category II or better.

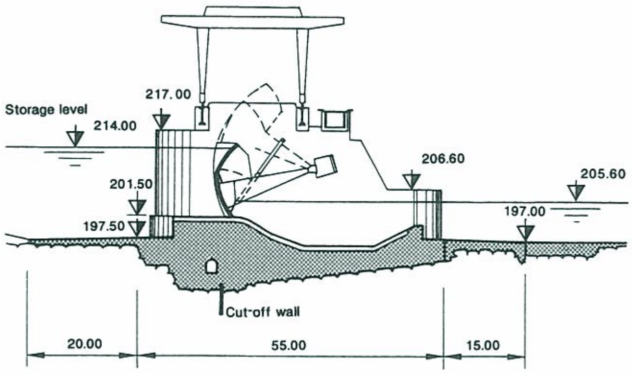


Plan

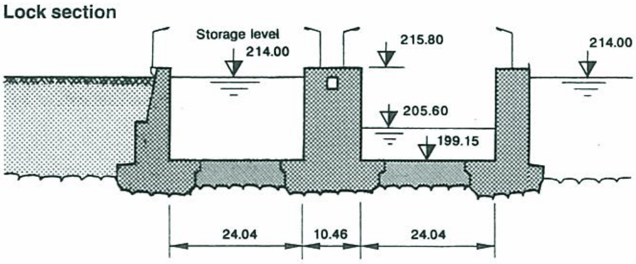
Powerhouse section



Weir section



Lock section



MAIN DIMENSIONS

	Weir	Powerhouse
Height	31 m	39 m
Length	6x24+5x6 = 174 m	9x18 = 162 m
	336 m in total	

The locks consist of 2 chambers of 24x230 m in effective area each. The middle wall has 10.5 m in width and 22 m in height.

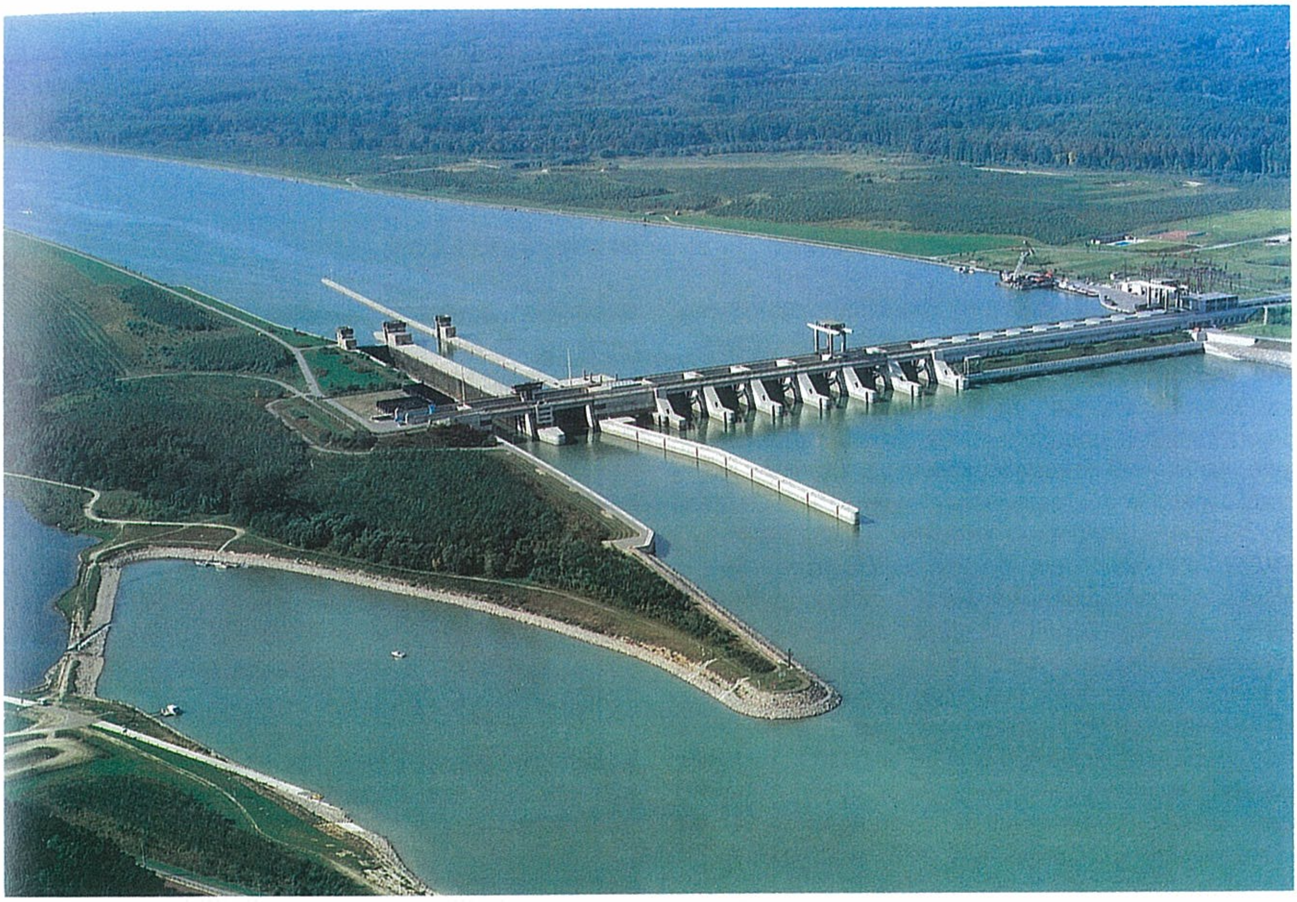
VOLUMES OF BARRAGE AND NEW RIVER BED

Excavation	
Gravel, Sand	14 000 000 m^3
Rock, Schlier	600 000 m^3
Concrete	900 000 m^3

VOLUMES AT BACKWATER AREA

Excavation	3 000 000 m^3
Diaphragm Walls	300 000 m^2
Cut off (concrete)	14 000 m^2
Riprap	460 000 m^3

GREIFENSTEIN



River	Danube, at km 1 949.2	Operating since	1984
Nearest town	Stockerau, Lower Austria	Purpose	Hydropower Inland navigation Flood protection Ecological effects (riverside forest irrigation system, "Giessgang")
Owner	DoKW Österreichische Donaukraftwerke AG Parkring 12, A-1010 Vienna		

MAIN CONSTRUCTION PARTS AND FUNCTION

Barrage	Weir: 6 openings of 24 m each Power station: 9 bulb turbines, 293 MW in total Locks: 2 chambers 24x230 m

The backwater area extends across the Tullner Feld basin, for a length of 31 km. Embankments and flood banks had to be constructed on both sides of the river. The town of Tulln on the right-hand bank is now fully protected against flooding.

The groundwater situation up-country to the left was reactivated by a generous network of ecological engineering activities, which improved the growth conditions of the riverside woodland for a terrain of more than 90 km^2.

HYDROLOGY

Catchment Area	100 100	km^2
Q mean	1 882	m^3/s
Flow Q max	10 750	m^3/s
Q rated	3 150	m^3/s
Storage Level (a.s.l.)	177	m
Mean Head	12.6	m
Capacity	293	MW
Energy Output	1 720	GWh/a

GEOLOGY

The Greifenstein power station is located directly to the west of the Danube gap, where the river leaves the Molasse zone of the Alpine foothills to enter the inneralpine Vienna basin through the hills of the Alpine flysch zone. With the latter overthrusting the rock of the Molasse zone, the barrage is located entirely in the flysch zone. "Flysch" is a clayey-sandy type of sedimentary rock of highly varying shares of sand or clay.

The backwater area is characterized by an alluvial sand and gravel layer of approx. 10 m in thickness overburdening the Oligocene "Schlier" (local name for a type of shale), which consists of more or less sandy and clayey strata and which is virtually impervious.

HISTORY AND DEVELOPMENT OF DESIGN

The master plan developed by DoKW in 1953–56 provided three power stations (Grafenwörth, Tulln and Klosterneuburg) for the stretch of the Danube between Krems and Vienna (Tullnerfeld basin). Positive experience acquired during construction of the Wallsee and Ottensheim plants, both of them located in flat lowlands, led to modifications in the master plan for the stretch, so that only two power stations were planned (Altenwörth and Greifenstein) which reduced the construction period to two thirds of the original schedule, produced major cost savings and significantly lessened the impact on the surrounding wetland forests. Another consideration was that two units with higher heads are more cost-efficient to operate with regard to energy generation and navigation than three barrages with lower heads each. Experience gathered in building Altenwörth (1976) showed that the higher embankments required for the backwater area posed no great difficulties.

The Greifenstein barrage was built in the chord of the bow of the original riverbed between kilometres 1 948 and 1 952, with its axis at kilometre 1 949.2, entirely in the left-bank wetlands vis-à-vis Greifenstein. Its location laterally to the Danube permitted running up the structures in a single large, unconfined and flood-protected pit without impairing navigation.

The retention water level for Greifenstein was set at 177.0 m to ensure uninterrupted impounding from Altenwörth in order to fully exploit the energy generating potential of the river and guarantee sufficient water depths for navigation. In line with recommendations by the International Danube Commission, the bridges must provide for a clearance of at least 8.0 m, which meant that the road and rail bridges in Tulln had to be raised by 2.0 m. Greifenstein again is a multi-purpose project whose main objectives include generation of energy (1 720 GWh/a), flood protection, improving navigation on the Danube waterway, comprehensive ecological measures in the northern hinterland, landscaping and a rich offer of leisure-time facilities.

FOUNDATION

The barrage is located so that all its structures are placed on Flysch on a varying stratification of clay and sandstone. The powerhouse and sluice head are founded on clayey layers, the weir and sluice tail are set on sandstone. The adjacent dams are cut off at the bottom by a diaphragm connecting to the Flysch. The dam fillings have a core of gravel/sand mixtures or a plastic sheet of 0.6 mm in thickness.

Upstream of the power station, excavations reached down to 6–8 m, while the downstream depth was 9–10 m from the original level of the left-hand bank. Excavation quantities for the power station and the two sections of the new river bed totalled 13.9 million m^3 of gravel and 2.4 million m^3 of Flysch.

INSTRUMENTATION

Geodetic surveillance, monitors for settlement, uplift pressure, etc. permit continuous observation and monitoring of all structures.

CONCRETE

The concrete aggregates were taken from the excavation material of the throughcut and the gravel banks in the Danube. Concrete was shipped to the site by truck dumpers (6 m^3) and poured by seven mobile telescope crane belt conveyors with a capacity of 40–50m^3/h each. The total volume of concrete used was 1.1 million m^3 (B 160, B 225, B 300).

A lab operated by DoKW on site performed regular material tests, and monitored manufacturing, charging and curing.

FLOOD RELIEF WORKS

Weir: The weir between the powerhouse and locks has six openings of 24 m in width each, spaced by five piers. The openings are shut by oil-hydraulic tainter gates with flaps on top. Upstream and downstream stoplogs are provided for maintenance purposes.

Flooding tests found that of Q max = 10 750 m^3/s about 8 650 m^3/s pass through the weir while the remaining 2 100 m^3/s flow through the left-hand hinterland into the tailwater downstream of Greifenstein (overflow sections of the embankments).

Locks: The locks can also serve for discharging extremely high floods; therefore the upper gates are of the hook-type double-leaf gates with oil-hydraulic drive. The lower gates are fitted with mitre gates.

POWERHOUSE

The powerhouse on the left-hand bank accommodates nine turbo-generator sets, each.consisting of a horizontal Kaplan bulb turbine with an output of 34 MW, a runner diameter of 6.5 m and a 38 MVA, 8 kV three-phase generator. The units each take up 19 m in width. The design water volume was defined as the 36-day flow Q rated= 3 150 m^3/s. An internal crane with a carrying capacity of 250 kN serves the turbine room and assembly bays. A 1 500 kN gantry crane travels on the crest of the barrage on a track running along the powerhouse roof and the crane bridges across the weir and sluice tail.

BACKWATER AREA

The Greifenstein backwater extends upstream to Altenwörth whose tailwater is slightly raised. The entire length of 31 km is within the Tullner Feld basin. Its embankments were placed on the bank terrain to meet static and hydraulic requirements. Where settlements (e.g. Tulln) reach close to the Danube it was, however, necessary to extend the fill into the river. The fill was taken from dredging in the Danube and lateral excavations. The embankments are cut off by diaphragms in the substratum topped by gravel/sand mixtures. Concrete cut-off curtains were used near settlements to avoid damage from vibrations. The slopes were faced with riprap on the water side and harmonized with their surrounding on their off-water side. A system of parallel ditches on the land side ensures that the original ground water table could be maintained.

ENVIRONMENT AND ECOLOGY

Ecologically, the measure of greatest impact was raising the phreatic surface which in turn improved growth conditions for a wetland area of more than 90 km^2 up-country to the left. The Danube training scheme implemented more than 100 years ago had produced a degradation of the bed which had lowered the main ground water levels and dried up the riverside forests.

At the heart of the environmental engineering measures to revitalize the wetlands is a system of irrigation channels 40 km in length. A naturalized, active course was achieved by incorporating ditches and hollows and by irregular routing of the channels. Fresh water is supplied by small tributaries, percolation water and seepage and, if necessary, by three compensation water outlet works. When the river reaches its flood level, an overflow section and flooding hollow located at the upstream start of the irrigation system gradually floods the entire forest area. The ground water table is controlled by opening and closing the 25 overflow cross-dams of the system (called "Giessgang").

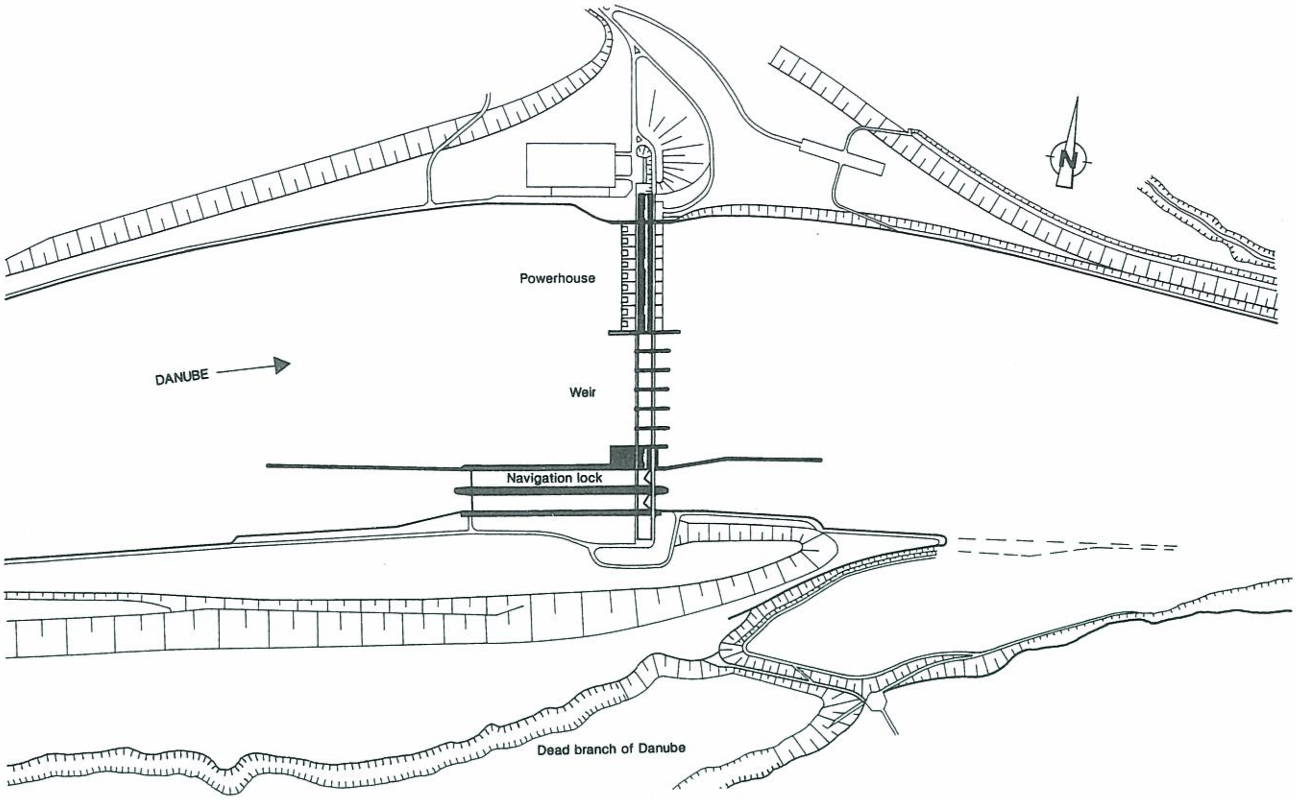
A few years into operation, the irrigation system has already come up to expectations and generated obvious ecological benefits.

The line of the embankments was varied by incorporating four bays, and some biotopes have been established within the overall system of the river wetland.

In Tulln, the right-hand embankment had the effect of savely sealing off the town against flooding which increased the degree of protection – and incidentally the value – of the settlements near the banks. By adapting the flood bank to existing structures it was possible to create a recreational area that enjoys great popularity.

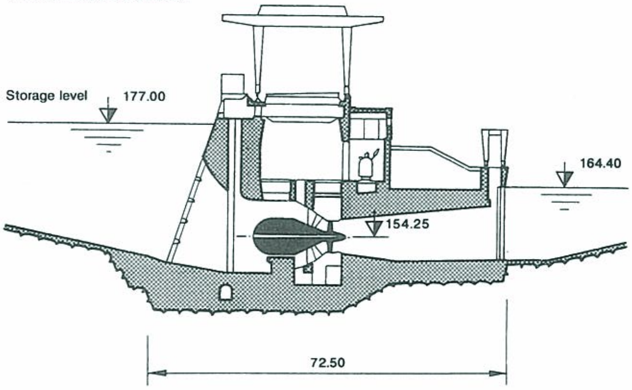
Yachting harbors, bays and flat river banks, surfaced dam-crown paths for bikers, bathing and watersports facilities at the original course of the Danube have enhanced the attractiveness of this stretch of the river for tourists and locals alike.

A comprehensive system of water treatment plants along the entire backwater area ensures that the water quality corresponds to or surpasses category II both in the Danube and in its former course near Greifenstein. Ecological measures implemented for the Greifenstein reservoir have thus improved the quality of life for the local population and at the same time created a valuable recreational area for the adjacent urban agglomeration of Vienna.

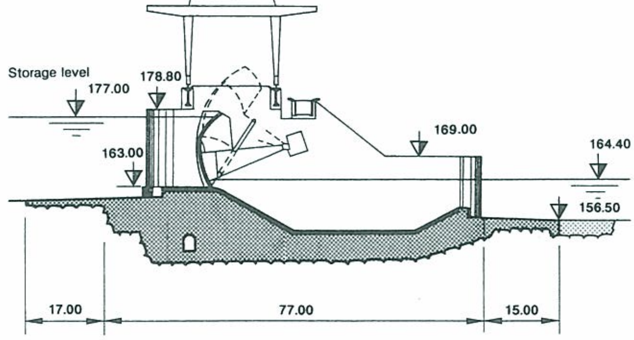


Plan

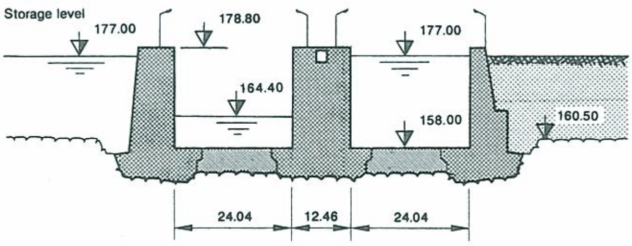
Powerhouse section



Weir section



Lock section



MAIN DIMENSIONS

	Weir	Powerhouse
Height	31 m	44 m
Length	6x24+5x6 = 174 m	9x19 = 171m
	345 m in total	

The locks consist of 2 chambers of 24x230 m in effective area each. The middle wall has 12.5 m in width and 27 m in height.

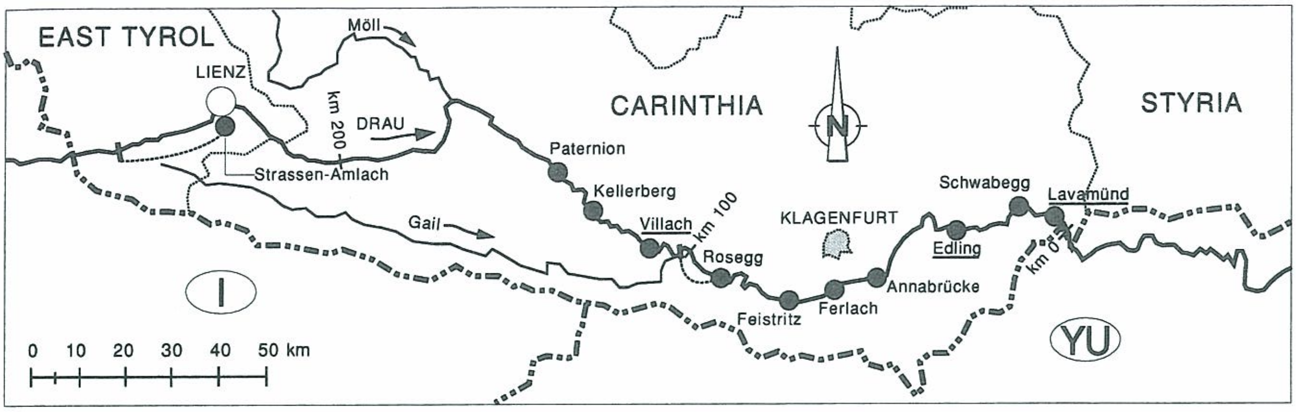
VOLUMES OF BARRAGE AND NEW RIVER BED

Excavation	
Gravel, Sand	13 900 000 m^3
Rock	2 400 000 m^3
Concrete	1 100 000 m^3

VOLUMES AT BACKWATER AREA

Excavation	9 900 000 m^3
Diaphragm Walls	380 000 m^2
Sheet Piling	4 000 m^2
Riprap	990 000 m^3

DRAU



The river Drau, flowing through East Tyrol and Carinthia in an easterly direction, collects the greater part of the run-off from the southern flank of the main ridge of the Alps and from the northern flank of the Southern Calcareous Alps. With a catchment area of 11 000 km² and a mean flow of 275 m³/s, it leaves Austrian territory near Lavamünd. Its adjoining reach in Yugoslavia is also developed for electricity production. Development completed by 1990 encompasses a 159 km long reach between the national border at Lavamünd and Paternion – halfway between Villach and Spittal/Drau – with a total head of 176 m. Upstream of the Parternion station, in operation since 1988, there still remains an undeveloped reach up to the outlet of the Malta lower stage at Möllbrücke, which is 21 km long and has a fall of 37 m. A master plan exists for this section which originally included two power stations, but has been modified to comprise four power stations with a total capacity of 89 MW and an annual energy of 346 GWh/a. Studies on the environmental impacts are in progress.

Independently of these reaches of the Drau in Carinthia construction of a run-of-river scheme at Strassen-Amlach was started in autumn 1984 and finished early in 1989. This is a diversion-type power station with a 22 km tunnel, utilizing a head of 370 m between Tassenbach (Strassen) and Lienz (Amlach) both in East Tyrol. Having a capacity of 60 MW, the power station generates 233 GWh/a.

For chronological reasons the description of power stations given in the following paragraphs will be against the direction of flow, proceeding upstream, from the Austro-Yugoslavian border. The series of developments on this lower and middle Drau (Lavamünd to Rosegg) comprises 7 power stations with a total capacity of 483 MW and an annual generation of 2 381 GWh/a. On the upper reach of the Drau the three power stations at Villach, Kellerberg and Paternion are completed (72 MW, 303 GWh/a in total). Four more run-of-river stations are in the project stage, their construction having so far met with yet unsolved ecological and environmental problems. These four projects would total 89 MW in capacity and 346 GWh/a energy. At the moment (1990), the total capacity (including Strassen-Amlach) is 615M W, producing 2 917 GWh/a., which makes the Drau developments the second largest chain of run-of-river stations in Austria.

Five of the seven power stations on the lower and middle Drau are barrages with spillways and powerhouses forming together a single structure. The uppermost station Rosegg - St. Jakob has a separate weir structure, a headrace canal, 3.4 km long designed for a discharge of 430 m³/s and a powerhouse located on the riverbank. The most downstream station, Lavamünd, was constructed as the first pierhead power station in the world (project Grengg-Lauffer) in 1942. This is equipped with vertical-shaft Kaplan turbines, originally 8.3 MW and, since 1985, 9.6 MW in capacity each, in the three weir piers. The same design was applied for most of the power stations following downstream, on the Yugoslavian reach of the Drau (as for instance Dravograd and Maribor).

Heads attain a maximum of 26 m which implies that the water level is raised substantially. This calls for dykes up to 24 m high along the backwater reaches. Recent dykes are almost uniformly of the gravelfill type with bitumen facings (concrete at Edling) on the waterside slope (inclined at 1:1.75) and with thin diaphragm and cast-in-situ walls as cut offs in the foundation as impervious elements. Weir piers and powerhouse structures designed in accordance with the surface level of the impounded water attain heads of 40 m. Some of the power stations are founded on rock. Those situated in flat valley reaches on conglomerate, which called for deep cut offs; at Annabrücke, where the foundation consists of sand-gravel layers, a grid of cut-off wall elements had to be provided. At almost all the stations there are inspection galleries in the weir sill and below the turbine inlet, rendering possible subsequent grouting.

Whereas Lavamünd and Schwabegg were constructed during the war, between 1939 and 1945, development was continued from 1959 onward and terminated with the completion of Annabrücke in 1981. It may be worth mentioning that the first two stations are equipped with hook-type double-leaf gates and all the following stations with tainter gates on the spillway structures. Concrete volumes vary between 130 000 and 170 000 m³ for a head between 20 and 24 m. The Lavamünd pierhead power station with a head of only 9 m, required only 52 000 m³ of concrete. The dykes on the banks of the backwater reaches, 6 to 24 m high have fill volumes between 0.4 and 5 million m³, depending on the local topography.

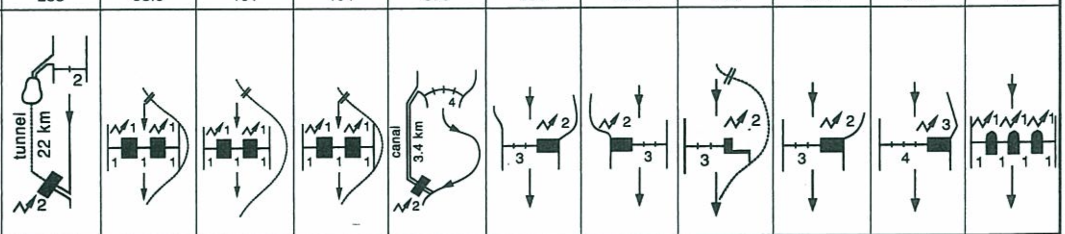
The Villach, Kellerberg and Paternion stations are pierhead power stations similar to Lavamünd, but with two power-unit-piers and three spillway bays each. They came in operation 1984, 1985 and 1988 and have a capacity of 24 MW and an output of approx. 100 GWh/a each. The next two stations are planned to be constructed to the same design, but for the remaining two, two design alternatives – pierhead or side plant – are being considered. Except for the Villach station which is located on a ridge of rock, the remaining stations had and have to be founded on fine-grained, poorly consolidated postglacial lake deposits, so that here the main structures must be placed on foundation grids of cut-off wall elements as a precaution against potential earthquake risks.

Backwater along these series of power stations had to be confined to the original width of the river, as the valley floor in this area is intensively cultivated. Except for the Villach station whose backwater area is situated between high natural river banks, continuous dykes had to be built on both banks almost everywhere. They are continued as flood protection dams beyond the upstream end of the backwater areas, so as to connect to a ground, natural or artificial, that is high enough to be safe from floods. The dykes of the same system as at the lower part of the river give a full flood protection, showing the multipurpose character of these power stations.

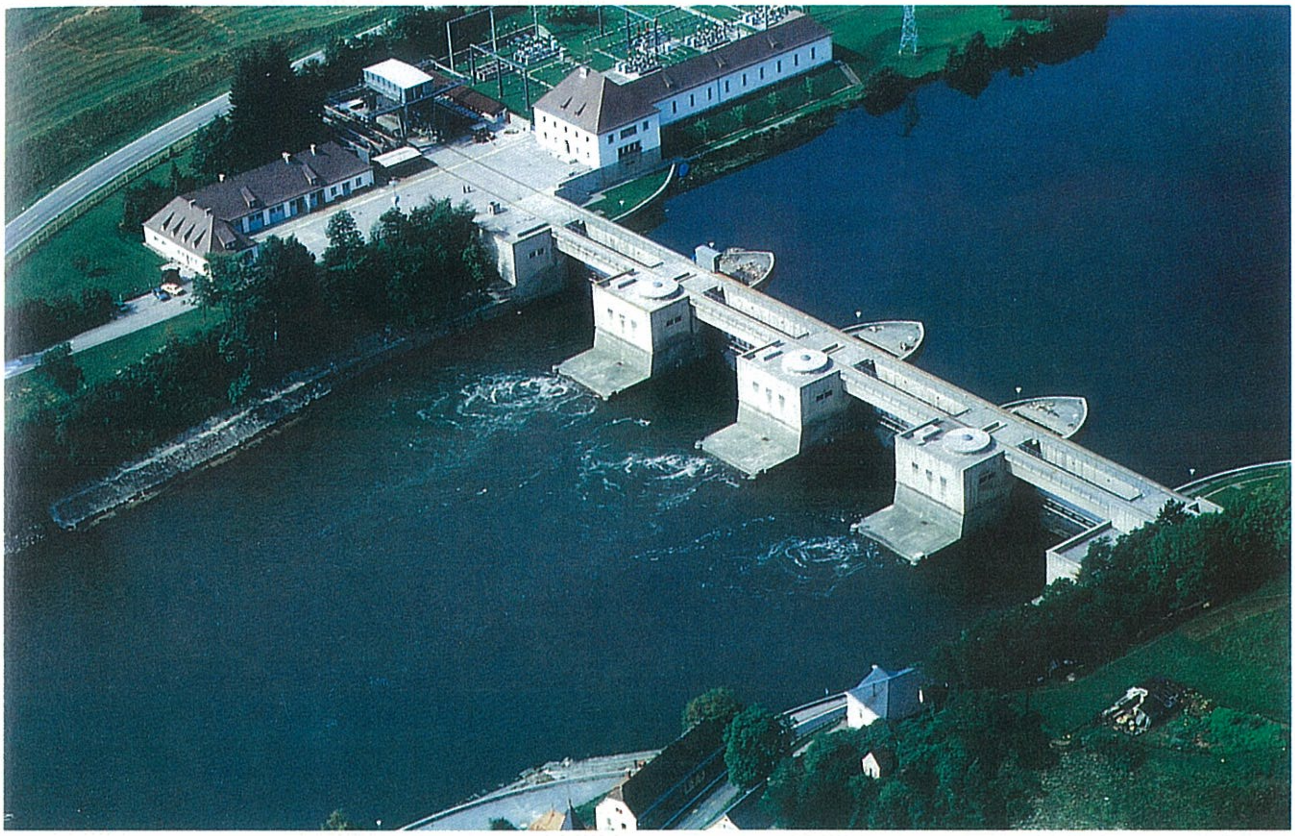
All the power stations in Carinthia on the river Drau are operated by Österreichische Draukraftwerke AG (ÖDK) Klagenfurt. Except for Schwabegg and Lavamünd they have also been designed and constructed by this company. The existing stations are operated on a pondage basis in accordance with the demand for electricity. The flow regime during the wintermonths is greatly enhanced by valuable contributions from upstream storage-schemes (Reisseck, Fragant, Malta), so that winter energy accounts for 37% and summer energy for 63% of the total annual generation.

The above-mentioned Strassen-Amlach power station on the Drau is situated in the most upstream reach in East Tyrol and has been constructed and is owned by Tiroler Wasserkraftwerke AG (TIWAG), Innsbruck. The Drau is diverted near the Austro-Italian border by a two-bay weir and a daily storage basin, from which a power tunnel some 22 km in length (design cross section ranging from 3.20 to 3.40 m) leads to a surge tank, pressure-shaft and above-ground powerhouse in the vicinity of Amlach near Lienz. With a rated discharge of 20 m³/s and working under a head of 370 m, the station has a capacity of 60 MW and produces an annual energy of 233 GWh/a (67% in summer and 33% in winter). This station was commissioned in 1988. The 60 km reach of the Drau between the outlet works near Amlach and the upstream end of the Spittal backwater reach is the subject of general development studies. It should finally be mentioned that the Malta lower stage, in fact also a run-of-river scheme (41 MW and 114 GWh), joins the Drau as a tributy at the upstream end of the Spittal backwater reach.

Drau

Power station		Strassen-Amlach	Paternion	Kellerberg	Villach	Rosegg-St. Jakob	Feistritz-Ludmannsdorf	Ferlach-Maria Rain	Annabrücke	Edling	Schwabegg	Lavamünd
Owner		TIWAG	ÖDK	ÖDK	ÖDK	ÖDK	ÖDK	ÖDK	ÖDK	ÖDK	ÖDK	ÖDK
Operation since		1989	1988	1985	1984	1973	1968	1975	1981	1962	1942	1944
Stationing	km	253/230	140 (75)	130 (85)	119 (96)	100/93 (115/122)	78 (137)	69 (146)	54 (161)	30 (185)	13 (202)	7 (208)
Storage level	m	1 069	515.0	505.3	496.1	485.5	461.5	437.5	416.4	390.8	369.0	348.7
Flow Q_{mean}	m³/s	10 (+1.4)	145	151	154	220	220	225	231	260	269	274
Flow Q_{max}	m³/s	210	1 660	1 690	1 700	2 300	2 300	2 300	2 450	2 700	2 700	2 700
Flow Q_{rated}	m³/s	20	320	320	320	430	420	450	440	440	405	405
Head H_{mean}	m	370	9.7	9.7	10.6	22.7	23.7	21.0	25.6	21.1	20.5	9.0
Capacity (MC)	MW	60	23.5	24	24	80	80	75	90	70	60	28
Energy (AAE)	GWh	233	98.0	101	104	370	390	336	416	375	340	154
Layout												
Spillway/Weir		Strassen				St. Martin						
Bays width	m	2×9	3×16	3×16	3×16	4×15	3×15	3×15	3×18	3×15	4×19	4×24
Pier width/height	m	2.0/9.9	20/27.5	20/24.5	20/25	4.3/22	2.4/39	4.6/40	5.0/40	5.7/36	5.0/33	16/17
Gates		tainter + flap	tainter + flap	tainter + flap	tainter + flap	tainter + flap	tainter + flap concr. beam	tainter + flap	tainter + flap	tainter + flap	hook-double	hook-double
Power conduit	km	tunnel 22	–	–	–	canal 3.4	–	–	–	–	–	–
Powerhouse		Amlach				Rosegg						
Construction, type		high	pier-head	pier-head	pier-head	high	high	high	high	high	low	pier-head
max height	m	34	27.5	27.5	28	44	47	42	40	38	34	28
Turbines, number and type		2 Francis ↓	2 Kaplan ↓	2 Kaplan ↓	2 Kaplan ↓	2 Kaplan ↓	2 Kaplan ↓	2 Kaplan ↓	2 Kaplan ↓	2 Kaplan ↓	3 Kaplan ↓	3 Kaplan ↓
Backwater area												
Length	km	1	5.9	10.6	10.1	15	15	9	15	24	17	6

LAVAMÜND



River	Drau, at km 208 (7)	Operating since	1944 (2 turbines), 1949 (3 turbines)
Nearest town	Lavamünd, Carinthia	Purpose	Hydropower
Owner since 1947	ÖDK Österreichische Draukraftwerke AG Kohldorferstrasse 98 A-9010 Klagenfurt		
before 1947	Alpen-Elektrowerke AG, Vienna		

MAIN CONSTRUCTION PARTS AND FUNCTION

Lavamünd is the first pierhead power station in the world. Weir: 4 openings, each 24 m in width (Q = 5 000 m^3/s), spaced by 3 piers, with one 9.6 MW turbine each, 28 MW in total.

Lavamünd was built as a link in the development chain from Völkermarkt to Maribor, solely to generate energy. To utilize the facility as soon as possible, engineers developed the pierhead type of power station which allowed operation already after the first stage of construction (1 pier, 2 weir openings) had been completed and when the backwater was not yet fully developed. The station was built in the river bed, the customary method at that time.

HYDROLOGY

Catchment Area	10 964	km^2
Q mean	274	m^3/s
Flow Q max	2 700 (5 000)	m^3/s
Q rated	405	m^3/s
Storage Level (a.s.l.)	348.7	m
Mean Head	9.4 (8.6)	m
Capacity	28	MW
Energy Output	154	GWh/a

GEOLOGY

At the barrage, the Drau river has cut into the bed rock several metres deep. The rock is part of the primitive clay schist interstratified by thin layers of quartz. The backwater area, whose level is only slightly higher than the original river bed, is at the border line between the schist formations to the north and the Triassic Karawanken range to the south. The Triassic dolomite on the right-hand (southern) bank is mostly covered by glacial gravel and moraines.

HISTORY AND DEVELOPMENT OF DESIGN

Apart from some preliminary concepts and construction of the Fala power station in the war-years of 1914–1918, Alpen-Elektrowerke AG made its first action plan for exploiting the hydropower potential between Völkermarkt and Maribor in 1938, which was soon followed by Schwabegg as the headwater station. Plans of following up with Mariborski otok as the most productive facility were waived in favor of Dravograd in 1941 and Lavamünd as the downstream station in 1942 in order to develop a closed chain. The first two turbogenerator sets became operable in April 1944 and August 1945 respectively, in spite of the difficulties caused by the war and post-war period; the third was added in April 1949.

The pierhead power station follows the philosophy developed by Grengg/Lauffer: arranging the turbogenerators "in the flow" provides for numerous benefits: lower space requirements by accommodating the powerhouse in the pier, the turbine inlet sill is always flushed, easy removal of floating debris in case of flooding, higher head from the flow velocity, uniform shape of the structures so that they blend better with their surrounding, shorter construction periods and lower building costs. (Model experiments carried out later at the Graz Technical University established that the benefits provided by a pierhead power station are highest at heads between 10 and 15 m.)

FOUNDATION

The barrage was entirely founded on the outcropping primitive clay schist. No cut off was necessary either at the bed or at the slopes. With regard to cut off for the pervious gravel at the upper flanks, experience with the upstream station at Schwabegg found that the Drau provided the necessary sealing by filling the cavities with fine and colloidal sediments physically and chemobiologically.

INSTRUMENTATION

A series of spindles has been placed at the weir to monitor structural displacements.

CONCRETE

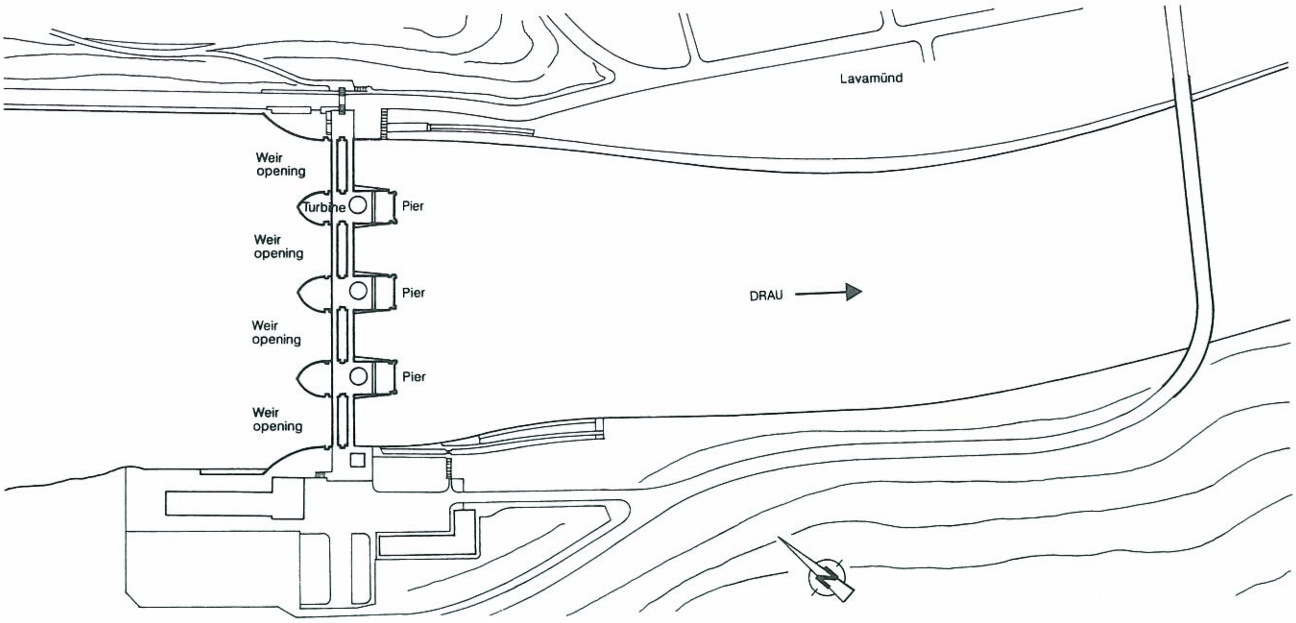
Aggregates were taken from nearby gravel terraces between the Drau and Lavant rivers. A mixture of 150 kg cement and 150 kg Thurament per m^3 of concrete was used for binders. The concrete was partly mixed by hand and poured with crane buckets and trolleys. In spite of impaired operations due to the war builders achieved adequate concrete qualities, B 225 at 23.5 N/mm^2 after 28 days and 32.5 N/mm^2 after 90 days.

FLOOD RELIEF WORKS

The weir has four openings of 24 m each, in total 96 m. It can discharge a probable maximum flood of Q = 5 000 m^3/s calculated at the time (reputedly measured in 1851). The openings are closed by hook-type double-leaf gates of 11 m in height. Stoplogs at the headwater and needles at the tailwater are provided as emergency gates and for maintenance.

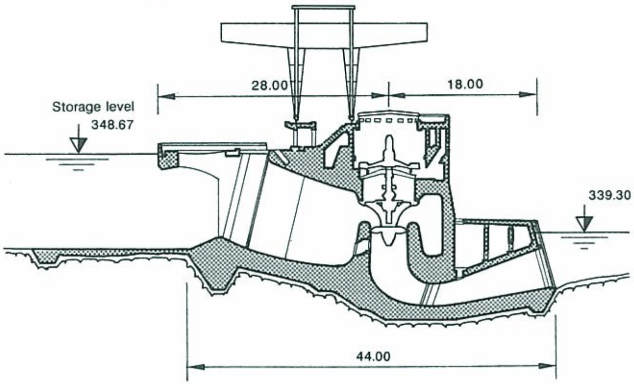
PIERHEAD POWER STATION

Each pier holds a turbogenerator set consisting of a four-blade Kaplan turbine and attached umbrella-type generator with a capacity of 9 580 kW and 12 500 kVA each. The inlets and outlets can be closed by stoplogs. A gantry crane with a total capacity of 140 t travels on the crest to serve the stoplogs, turbines and weir. Rake cleaning is performed by a unit covering the entire facility.

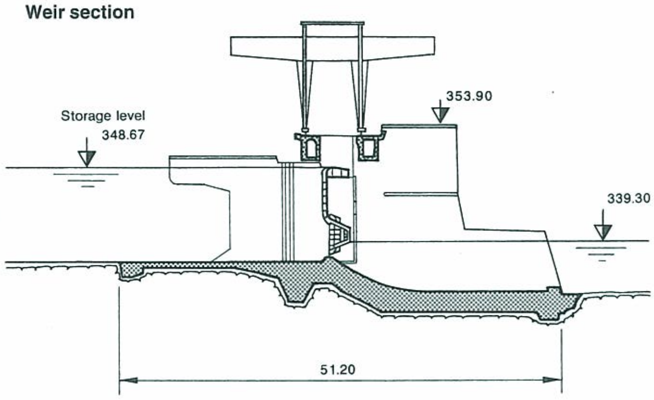


Plan

Section through pierhead power station



Weir section



MAIN DIMENSIONS

	Weir	Pier (turbine)
Heigth	25 m	28 m
Width	52 m	44 m
Length	4x24 m = 96 m	3x18 m = 54 m
	150 m in total	

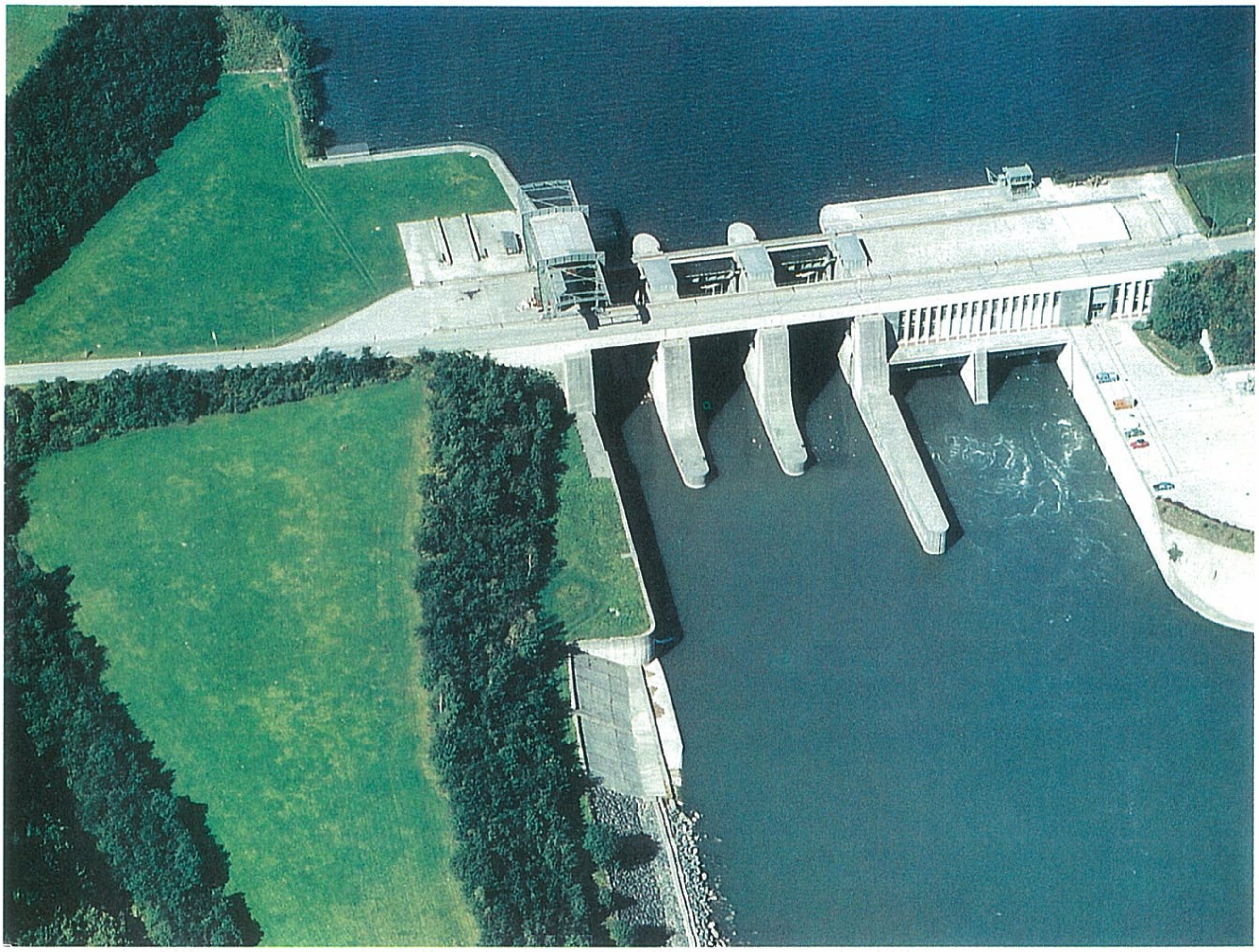
VOLUMES

Concrete	52 000 m³
Reinforcement Steel	1 120 t

BACKWATER AREA

Length	6 km
Max. Width	300 m
No embankments necessary	

EDLING



River	Drau, at km 185 (30)	Operating since	1962
Nearest town	Völkermarkt, Carinthia	Purpose	Hydropower Recreation area Flood control
Owner	ÖDK Österreichische Draukraftwerke AG Kohldorferstrasse 98 A-9010 Klagenfurt		

MAIN CONSTRUCTION PARTS AND FUNCTION

Weir: 3 openings, each 15.0 m in width.
Power station: 2 turbines of 35 MW each, 70 MW in total

Edling is the first power station built in the course of a development stretching between Schwabegg and Villach. It is directly upstream of the backwater of Schwabegg, which was finished in 1942.

HYDROLOGY

Catchment Area	10 656	km^2
Q mean	260	m^3/s
Flow Q max	3 500 (2 350)	m^3/s
Q rated	440	m^3/s
Storage Level (a.s.l.)	390.8	m
Mean Head	21.2	m
Capacity	70	MW
Energy Output	375	GWh/a

GEOLOGY

Within the site, the Drau is the general boundary between the schist formations to the north and the Triassic rocks of the Karawanken range to the south. At the barrage, the Triassic dolomites reach far across the Drau to the north so that the weir could be nested into the dolomites which, while full of fissures, are consistent thanks to a Tertiary overburden compacted in the Ice Age. A fault in the form of a marl band bounded by layers of ankerite, while impairing the stability of the pit slopes, does not reduce the soil bearing capacity. To the south of the pit, a thick layer of glacial gravel deposited in several thrusts with a strong flow of ground water covers the sloping dolomite. The ground water flow presses against the backwater, thereby reducing the otherwise considerable flow of percolation water.

To the east of Völkermarkt the backwater is located in a gorge-type valley cut out of the gravel deposits and dolomite by post-glacial melt water; to the west it is in a wide basin created by a post-glacial reservoir filled with fine sediments.

HISTORY AND DEVELOPMENT OF DESIGN

The rising demand for energy after the end of World War II made it necessary to start exploiting the energy potential offered by the Drau at around 1955. Projects dating from before 1918 and from 1940–44 were used as a basis for drawing up a new action plan covering Villach to Schwabegg/Völkermarkt (end of the backwater area). With the former headwater stage at Schwabegg silting up, Edling was realized as the first stage of the new development. At a remarkable 20 m, the storage level was then state-of-the-art. It produced a reservoir 21 km in length and more than 10 km^2 in surface and at that time was considered ecologically beneficial. Its construction method in a single pit laterally to the river was a notable first in Austria.

The project had a significant impact on the entire region because its access roads and bridges across the Drau improved the link between Völkermarkt and its wider surroundings.

FOUNDATION

The barrage, situated on the right-hand bank of the Drau, is founded on fissured but stable dolomite that does not show any karst features. No cut off was necessary as the dolomite is covered by a compacted Tertiary layer and the structural concrete connects closely to this layer.

The embankments of the backwater area and next to the barrage are set on gravel deposits of the Drau. They are sealed on the water side slope by concrete slabs, and in the substratum by various types of soil-cement walls, predecessors of today's diaphragm walls.

INSTRUMENTATION

Displacement of the embankments and weir are monitored geodetically. Percolation water flows in the embankments and abutting slopes of the weir are measured in stand pipe piezometers. Stability of the barrage is also monitored by measuring the uplift pressure.

CONCRETE

Aggregates were taken from a gravel terrace on the southern bank of the Drau and separated into four fractions to produce concretes from B 160 to B 450, mainly B 225. For binders, 85% cement 275 (230 kg) was used together with 15% flyash (40 kg) from the St. Andrä steam generating station. The concrete quality was 24.3 N/ mm^2 (31.7 N/mm^2) after 28 (90) days.

The embankments of the backwater area were filled with Drau gravel; their slope has a general incline of 1:2 at the off-water side and 1:1.75 at the water side. They are fortified by concrete slabs serving as sealing elements and erosion protection. Crest width: 3.5 m.

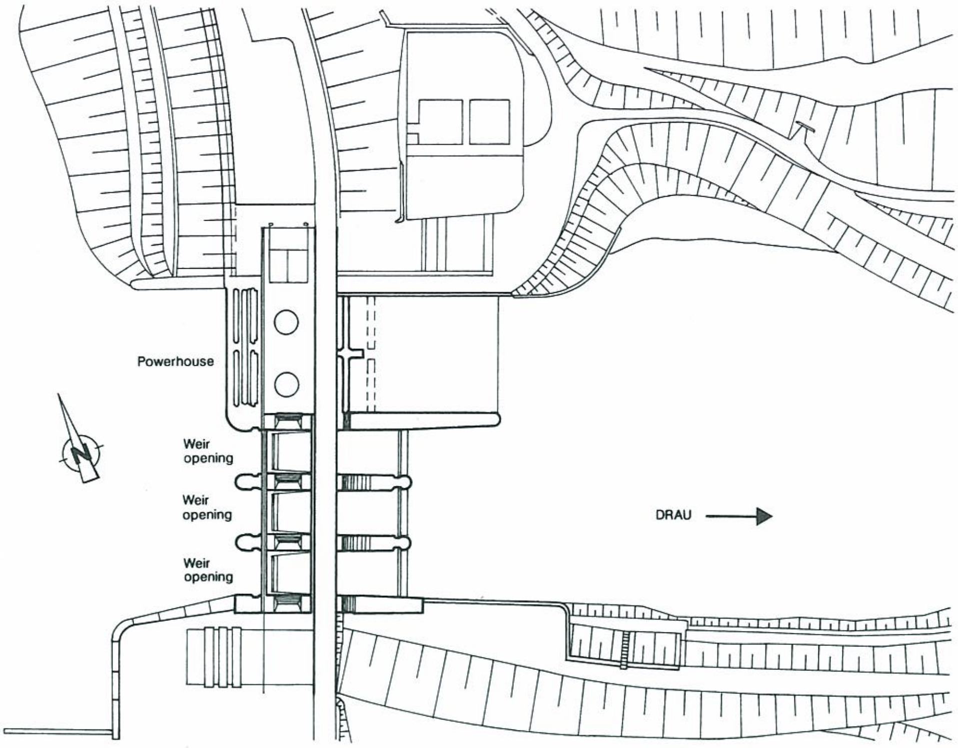
FLOOD RELIEF WORK

The weir has 3 openings of 15.04 m in clearance, which discharge HQ 5 000 = 3 500 m^3/s. If one opening is blocked the other two can discharge HQ 100 = 2 350 m^3/s. The gates are of the tainter type, 13.6 m high, with flaps 3.5 m in height. They are moved by chains and hoists installed at the pier crests. Stoplogs at the headwater and needles at the tailwater provide for emergency closing and maintenance.

POWERHOUSE

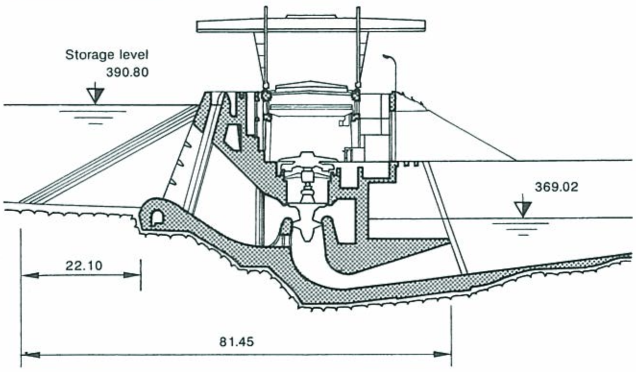
The powerhouse accommodates two turbogenerator sets on a vertical axis, consisting of six-blade Kaplan turbines with attached umbrella-type generators. The turbines have an absorption capacity of 220 m^3/s each and a capacity of 37.5 MW. The generators are designed for 45 MVA.

The inlets and outlets can be closed off by emergency gates (bulkhead gates). They are moved by a trestle crane of 54 t capacity which is installed on the crest. A crane with 2 x 90 t capacity is provided for assembly in the powerhouse. A separate unit cleans the rake. Four pump stations were built to drain the four polders in the backwater area. A total of 11 pumps of capacities of 200 to 1 500 l/s have been installed.

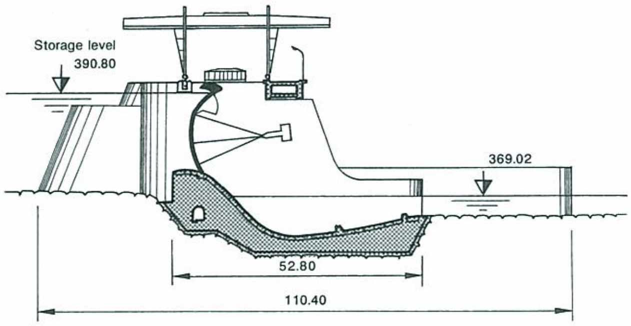


Plan

Powerhouse section



Weir section



MAIN DIMENSIONS

	Weir	Power station
Height	35.5 m	40.0 m
Width	52.0 m	60.0 m
Length	56.5 m	47.2 m
	103.7 m in total	

VOLUMES

Excavation (gravel)	590 000 m^3
Concrete	150 000 m^3
Reinforcement Steel	1 900 t
Embankments (dykes)	455 000 m^3

BACKWATER AREA

Length	26.0 km
Width max	1.2 km
Surface	10.7 km^2

VILLACH

River	Drau, at km 96 (119)	Operating since	1984
Nearest town	Villach, Carinthia	Purpose	Hydropower Flood protection for Villach
Owner	ÖDK Österreichische Draukraftwerke AG Kohldorferstrasse 98 A-9010 Klagenfurt		

MAIN CONSTRUCTION PARTS AND FUNCTIONS

Villach is the second pierhead powerstation on the Drau river. It was built 35 years after the first one at Lavamünd.

Weir: 3 openings of 16 m in width each, spaced by 2 piers, each with a 12.3 MW vertical Kaplan turbine, 24 MW in total

An embankment of 600 m in length was necessary on the left river bank.

HYDROLOGY

Catchment Area		5 164 km^2
	Q mean	154 m^3/s
Flow	Q max	2 500 (1 700) m^3/s
	Q rated	320 m^3/s
Storage Level (a.s.l.)		496.1 m
Mean Head		10.6 m
Capacity		24 MW
Energy Output		104 GWh/a

GEOLOGY

A tectonically formed cone projects from the Drau alluvions at the site of the barrage. Its rocks are of the Millstatt crystalline series and are characterized by a fluent transition of schistous gneiss, mica schist with a high content of biotite, chlorite schist and gneiss phyllite. In spite of the spider-web-like tectonic diversity of the contact surface, the rock has been found to be very stable. The small-scale and frequently laminate structures nevertheless translated into a higher excavation volume which assumed considerable dimensions at two fault zones so that an additional 3 000 m^3 of concrete were found to be necessary. The project was also affected by a tectonic node at the Villach basin, focus of severe earthquakes in Austria with an earthquake coefficient of E = 0.12-0.6. The structures thus had to be calculated taking into account a horizontal acceleration of g = 0.2.

The backwater area is embedded in the Drau alluvions. At some places it touches on the rocks of the Millstatt crystalline series (which also encloses thick layers of marble) and on its right-hand bank on the Triassic limestone of the Bleiberg mountain. The slopes are all considered stable.

HISTORY AND DEVELOPMENT OF DESIGN

The Drau as a source of hydropower was not utilized except around Villach by some floating mills operating in the 17th century. It was only the energy crisis of the 1970s, together with completion of the large-scale Malta storage power station in Western Carinthia, that gave the impetus for ÖDK to draw up a master plan for building power stations at the Drau river between Rosegg and the Malta tailrace stage upstream of Spittal, and to start construction at Villach.

The low storage levels and low water flow in the Drau without its tributary Gail have made it necessary to find highly economical modular solutions. Engineers thus used the pierhead type of power station whose "Lavamünd" prototype is operated by ÖDK and which has since found updated applications along the Inn.

FOUNDATION

The weir is founded entirely on a cone of crystalline rocks. The tectonic diversity of this rock necessitated removal of soft parts and replacing it with concrete. So far it has not been necessary to make cut-off groutings thanks to the mylonite veins in the system of fissures.

INSTRUMENTATION

Geodetic monitoring of structural displacements by spindle series, monitoring of the uplift pressure by piezometer tubes. Seepage at the embankment and site abutments is monitored by stand pipe piezometers. Percolation water at the dam is also measured by stand pipe piezometers, its volume is measured in parallel percolation water channels.

CONCRETE

Some 51 000 m^3 of concrete had to be poured into the barrage. Of this quantity, 3 000 m^3 were bed compensation and sealing concrete and 48 000 m^3 were used in construction with 2 570 t of reinforcing steel. This large quantity of steel was necessary in view of the earthquake risk.

The aggregate was supplied in five fractions by an existing gravel pit in the vicinity. Concrete was produced in the pit. Homogenized special cement PZ 275 FF with a flyash rate of 33% was used as a binder. The concrete was mostly of the B 225 type, to some extent also of categories B 300 and B 160.

Asphalt concrete was mixed with finishers and poured on the waterside slope to provide cut off. The ground was cut off by curtains and diaphragms.

FLOOD RELIEF WORKS

The three weir openings spaced between the banks and piers are designed for a maximum calculated flood of 5 000 year probability HQ 5 000 = 2 500 m^3/s and an HQ 100 = 1 700 m^3/s, which has to be discharged through two openings ("n-1 rule"). In both cases the headwater level will be slightly below the retention water level.

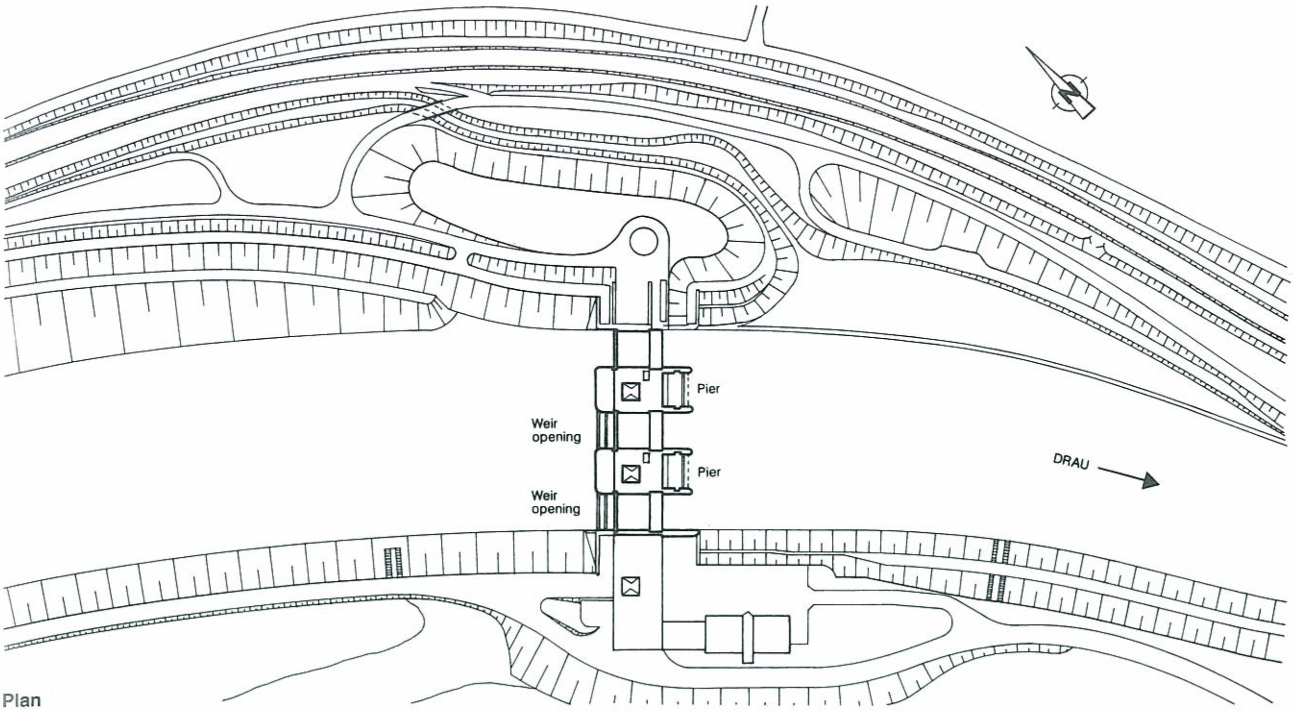
Tainter gates with flaps were used. Their dimensions are:

Opening Width	3x16 m	
Height of Tainter	7.75 m	10.80 m in total
Height of Flap (vertical)	3.05 m	

The openings can be closed by stoplogs in the headwater and by needles in the tailwater for maintenance.

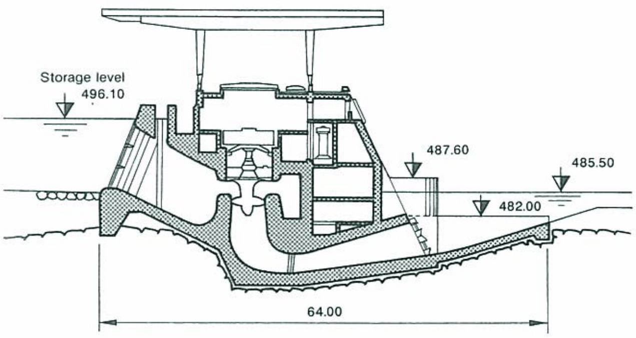
PIERHEAD POWER STATION

The two piers each hold a vertical turbogenerator set, a four-blade Kaplan turbine with attached three-phase generator of 12 MW output. The inlet spiral and draft tube can be closed off by stoplogs. The rake is cleaned by a special machine. A gantry crane of 80 t capacity runs on the crest of the weir and piers. The energy is fed into the 20 kV national grid via two 15 500 kVA transformers. The transformers and switchgear are included in the piers. Connection to the national grid is by cable lines.

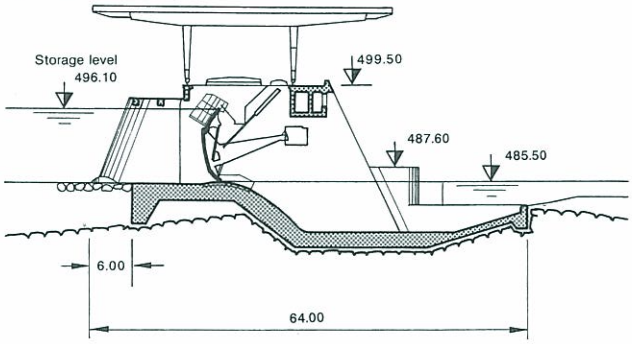


Plan

Pier section through pierhead power station



Weir section



MAIN DIMENSIONS

	Weir	Pier (turbine)
Height	24 m	27 m
Width	58 m	64 m
Length	3x16 + 2x4 = 56 m	2x20 = 40 m
	96 m in total	

VOLUMES OF BARRAGE

Excavation Gravel	630 000 m³
Rock	60 000 m³
Concrete	51 000 m³
Reinforcement Steel	2 570 t

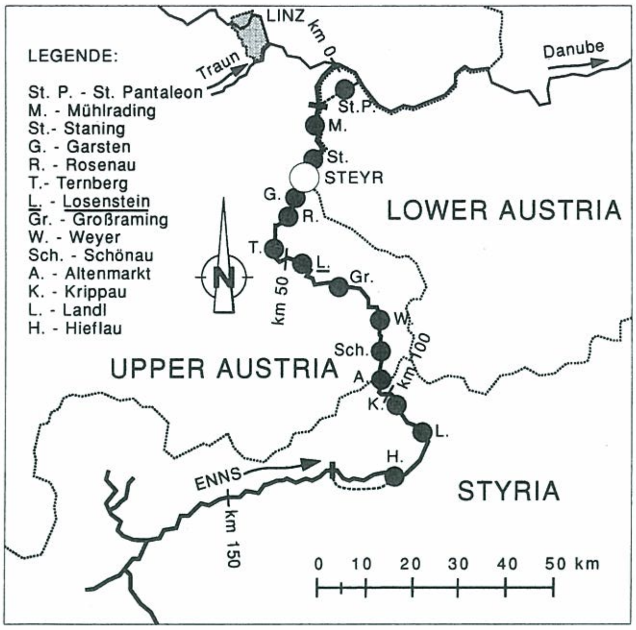
VOLUMES AT BACKWATER AREA

Excavation Gravel	450 000 m³
Riprap	30 000 t

VOLUMES AT TAILWATER AREA (OVERDEEPENING)

Excavation Gravel	1 130 000 m³
Riprap	230 000 t

ENNS



Rising in the northern part of the Alps (Niedere Tauern), the Enns is the only major river (catchment area is 6 100 km^2 and mean flow is 220 m^3/s) to flow on Austrian territory over its whole length. The main part of this river, that is its middle and lower course about 130 km in length and with a substantial fall, down to its mouth in the Danube, is developed by a continuous series of power-stations. It is only near the town of Steyr that a reach about 3 km long has remained unaffected. In this manner, a total gross head of 324 m is utilised by fourteen power stations with a total capacity of 518 MW and generating 2 373 GWh/a. This also includes the most upstream Hieflau, a diversion-type power station with an about 6 km-long tunnel and a combined surge tank and daily storage reservoir at Waag, developing the steep gradient of a gorge named Gesäuse.

This pilot power station is followed by stations at Landl, Krippau and Altenmarkt. These have weirs equipped with small turbines for water release to the reach from which flow is abstracted, as well as power tunnels and power stations (of which two are underground) equipped with one power unit each. This group of four power stations, situated along the Styrian reach of the river Enns, were placed into operation between 1955 and 1967 by Steirische Wasserkraft und Elektrizitäts AG (Steweag) in Graz. All the stations are located within a narrow valley, the Enns has cut through limestone and dolomite formations. No unusual foundation problems were encountered, except for foundation treatment in the weir areas to ensure imperviousness.

Downstream follow ten power stations with different layouts to suit local conditions. Schönau, Losenstein, Ternberg, Rosenau, Staning and Mühlrading show the usual side-by-side arrangement of spillway and powerhouse, with 3 to 5 spillway bays and 2 to 4 turbines. The Narrowness of the valley section at Weyer and additional difficulties from the presence of a railway line and a road called for a concept with 2 spillway bays and a power unit pier in between and with a second power unit located at the end of a short tunnel in the river bank about 1 km downstream of the weir site. At Grossraming, a symmetrical arrangement was adopted with a two-bay spillway in the middle and one power unit on each bank. At Garsten, the power station was built in a cut across a river bend, with a small turbine catering for water delivery to the dead branch. St. Pantaleon is a diversion-type power station with a 9 km-long asphalt- lined headrace, a powerhouse equipped with two power units and a downstream tailrace ending in the Danube. A remarkable feature of the two vertical-shaft Kaplan turbines in the St.Pantaleon powerhouse is the so-called spiral casing outlet. This consists of a gate device provided in the rear wall of the spiral casing. The gate opens when the unit is stopped, so as to prevent surge from developing in the long headrace. Another feature is the fact that one Kaplan unit each at Weyer and St.Pantaleon is designed for $16^2/_3$ Hz traction current, used by the railway. All ten power stations of the Enns are operated by Ennskraftwerke AG (EKW) in Steyr.

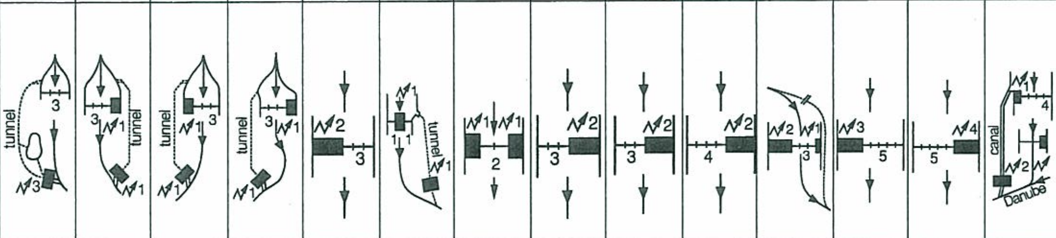
Development of the river Enns was started in 1941 and terminated in 1972. The characteristics of river course, geology, population pattern and traffic routes led to a multitude of interesting solutions. Many of the experiences gathered in this way have subsequently been used in other run-of-river developments. It should be mentioned in this context that the middle reach of the river Enns was the subject of discussions that went on for years. Besides the multi-stage project finally realized, an alternative was considered which provided for a single-stage scheme with a dam about 100 m high. The advantage of a large energy reserve, which would also benefit downstream power stations, conflicted with adverse effects on inhabited areas, railway lines and roads. In fact, almost 30 years ago, the problems created by a large reservoir with fluctuating water levels in an inhabited region in the foothills of the Alps (about 400 m a.s.l.) were already felt, perhaps even subconsciously, and the decision then taken appears to have been the right one in the ligth of our present consciousness of environmental impact.

The power and energy of the continuous series of power stations on the Enns, i. e.

Steweag	140 MW	681 GWh/a
EKW	378 MW	1 692 GWh/a
total	518 MW	2 373 GWh/a

has been available to the Austrian electricity supply since 1972, that is, for more than 18 years. Nearness of these power stations to the Upper Austrian and Styrian industrial areas affords considerable advantages, additional benefit resulting from pondage operation carefully scheduled to meet the requirements as they arise during the day. These past decades have also shown that nature and man have willingly accepted the inevitable changes involved. The scenic appearance of the backwater areas along the river is anything but that of destroyed nature. The river Enns is a very good example of how wrong it would be to assess environmental effects on the basis of the planning and design stage. It should also be mentioned in this context that hydro development was accompanied by the construction of roads and sewage treatment plants and the provision of recreation and sports facilities. In the long abstraction reach downstream of the weir diverting flow to the headrace of St. Pantaleon, a separate weir was constructed only to maintain the water level in the river bed.

Enns

Power station		Hieflau	Landl	Krippau	Alten-markt	Schönau	Weyer	Groß-raming	Losen-stein	Ternberg	Rosenau	Garsten	Staning	Mühl-rading	St. Pantaleon
Owner		Steweag	Steweag	Steweag	Steweag	EKW	EKW	EKW	EKW	EKW	EKW	EKW	EKW	EKW	EKW
Operation since		1955/56	1967/68	1965/66	1960/61	1972	1969	1950	1962	1949	1953	1967	1946	1948	1965
Stationing	km	126/117	114/111	108/101	98/91	86	77.5/76.5	64.4	55.7	47.9	40.2	34.3	20.0	13.8	8.1
Storage level	m	564.5	479.0	453.0	425.0	400.5	388.0	371.0	346.5	331.0	315.0	302.0	283.2	268.3	260.0
Q_{mean} Flow Q_{max} Q_{rated}	m³/s m³/s m³/s	78 1 000 90	94 1 100 120 + (20)	126 1 400 120 + (45)	129 1 400 106 + (18)	147 1 870 280	148 1 900 280	156 2 000 280	162 2 100 280	163 2 120 280	164 2 130 280	166 2 150 280 + 15	208 3 000 315	208 3 000 315	209 3 000 315 + 10
Head H_{mean}	m	78.4	21.4 (15.3)	23.0 (14.0)	23.9 (13.4)	11.2	15.7/16.1	23.5	14.8	15.0	12.7	12.3	14.2	8.0	18.8
Capacity (MC)	MW	63	25	29	23	26	37	65	38	40	28	32	37	21	54
Energy (AAE)	GWh	269	123	153	136	117	163	246	166	168	134	143	190	101	264
Layout															
Spillway/Weir		Gstatter-boden	Wandau	Groß-reifling	Essling										Thurnsdorf
Bays width	m	3×12	3×12	3×12	3×12	3×12	2×18	2×22.5	3×13.5	3×16	4×16	3×14	5×17	5×17.2	4×14
Pier width/height	m	4.4/19.5	4.7/21.0	4.1/20.0	3.0/19.2	4.0/25.5	18.0/35.0	4.0/37.0	5.2/31.0	5.7/32.2	5.5/28.0	4.0/23.0	5.0/26.0	4.8/23.0	4.0/21.5
Gates		tainter + flap	tainter + flap	tainter + flap	tainter + flap	tainter + flap	tainter + flap	flap bottom g.	hook-double	flap bottom g.	hook-double	tainter + flap	top leaf tainter	hook-double	tainter + flap
Power conduit	km	canal 0.5 tunnel 5.6	tun. 2.6	tun. 4.4	tun. 2.4	–	tun. 1.0								head 6.8 tail 2.2
Powerhouse		Hieflau	Landl	Krippau	Altenmarkt										St. Pantaleon
Construction, type		high	high	cavern.	cavern.	low	pier-head cavern.	high	low	high	med.	high	high	med.	high
max height	m	29.5	34.3	32.0	33.5	29.5	35.0/35.0	45.3	36.6	36.5	32.0	37.4	31.0	24.0	42.6
Turbines, number and type		3 Francis ↓	1 Kaplan + 1 Kaplan ↓	1 Kaplan + 1 Kaplan ↓	1 Kaplan + 1 Francis ↓	2 Kaplan ↓	1 Kaplan + 1 Kaplan ↓	2 Kaplan ↓	2 Kaplan ↓	2 Kaplan ↓	2 Kaplan ↓	2 Kaplan + 1 Kaplan ↓	3 Kaplan ↓	4 Kaplan ↓	2 Kaplan + 1 Francis ↓
Backwater area															
Length	km	2.0	2.5	3.0	2.9	7.0	8.9	12.1	8.7	7.8	7.7	5.9	10.0	6.2	5.7

LOSENSTEIN

River	Enns, at km 55.7	Operating since	1962
Nearest town	Steyr, Upper Austria	Purpose	Hydropower
Owner	EKW Ennskraftwerke AG Resthofstrasse 2, A-4400 Steyr		

MAIN CONSTRUCTION PARTS AND FUNCTION

Barrage — Weir: 3 openings of 13.5 m each
Power station: 2 turbines of 19 MW each, 38 MW in total

Losenstein acts as a link in the planned chain of power stations lining the middle and lower Enns, closing the gap between the Grossraming (1950) and Ternberg (1949) power stations.

HYDROLOGY

Catchment Area	4 840 km^2
Q mean	162 m^3/s
Flow Q max	2 100 m^3/s
Q rated	280 m^3/s
Storage Level (a.s.l.)	346.5 m
Mean Head	14.8 m
Capacity	38 MW
Energy Output	166 GWh/a

GEOLOGY

The Losenstein power plant is located in the Upper Austrian limestone Alpine foothills. It is founded on a rocky threshold of Vils limestone (upper Jurassic malmstone) ranging across the river at a width of 70 to 80 m. The walls of the valley are remains of a glacial terrace. Its bottom consists of gravel deposits from the last Ice Age.

HISTORY AND DEVELOPMENT OF DESIGN

The first plan for developing the energy potential of the Enns river from the provincial border between Styria and Upper Austria to its confluence with the Danube was drawn up by Österreichische Wasserkraft- und Elektrizitäts AG as early as 1925. The plans and pre-paratory work for the Losenstein project were passed to Ennskraftwerke AG (EKW), Steyr, a company founded in 1947, where they were adapted and harmonized with the downstream Ternberg power station and Grossraming, a project already in progress upstream of Losenstein. Construction was started in 1958 and completed in 1962.

FOUNDATION

The barrage is founded on a threshold of limestone rock shaped so as to allow proper cut-off connection to the limestone. The limestone threshold is enclosed up- and downstream by marl which would have been much less suitable for foundation work.

INSTRUMENTATION

Geodetic monitoring comprises settlement and position measurements.

CONCRETE

The concrete used at Losenstein (approx. 86 000 m^3 in total) was mixed with aggregates taken from the excavation pit for the tailrace channel of the St. Pantaleon power station planned next to the mouth of the Enns, as no suitable gravel deposits could be found nearer to the site. Supplementary to pure Portland cement, 15% pozzolana was added to some concretes as a binder.

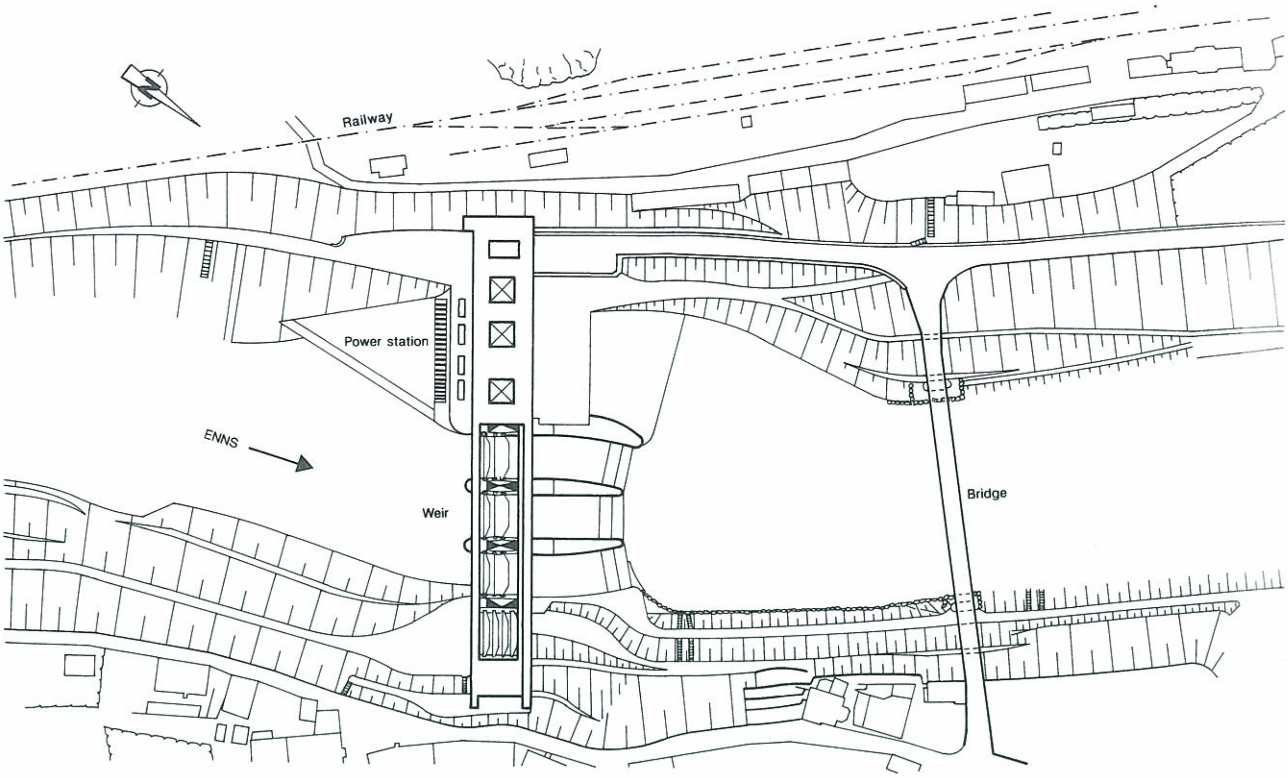
Coarse concrete B 160/225, max. grain: 100 mm
Binder: 200–250 kg/m^3
Pumped concrete B 225, max. grain: 80 mm
Binder: 280 kg/m^3

FLOOD RELIEF WORK

Weir with three openings, 13.5 m clearance, 16.2 m closing height. The openings are closed by hook-type double leaf gates. Q max is discharged through two openings of the weir. Seven stoplogs serve as an emergency gate.

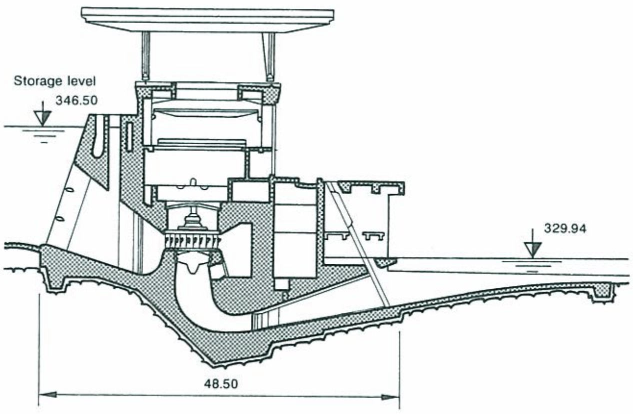
POWERHOUSE

Two turbo-generator sets with vertical Kaplan turbines for a rated flow of 140 m^3/s each, three-phase generators (20 MVA). The intake is fronted by a rack of about 180 m^2 in surface and a bulkhead gate. Each set is provided with a sluice gate fitted stationarily as a tailrace gate at the end of the draft tube. A gantry crane is provided for hoisting and moving equipment.

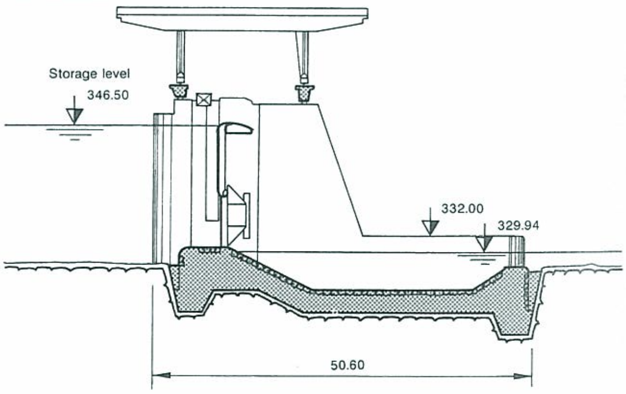


Plan

Powerhouse section



Weir section



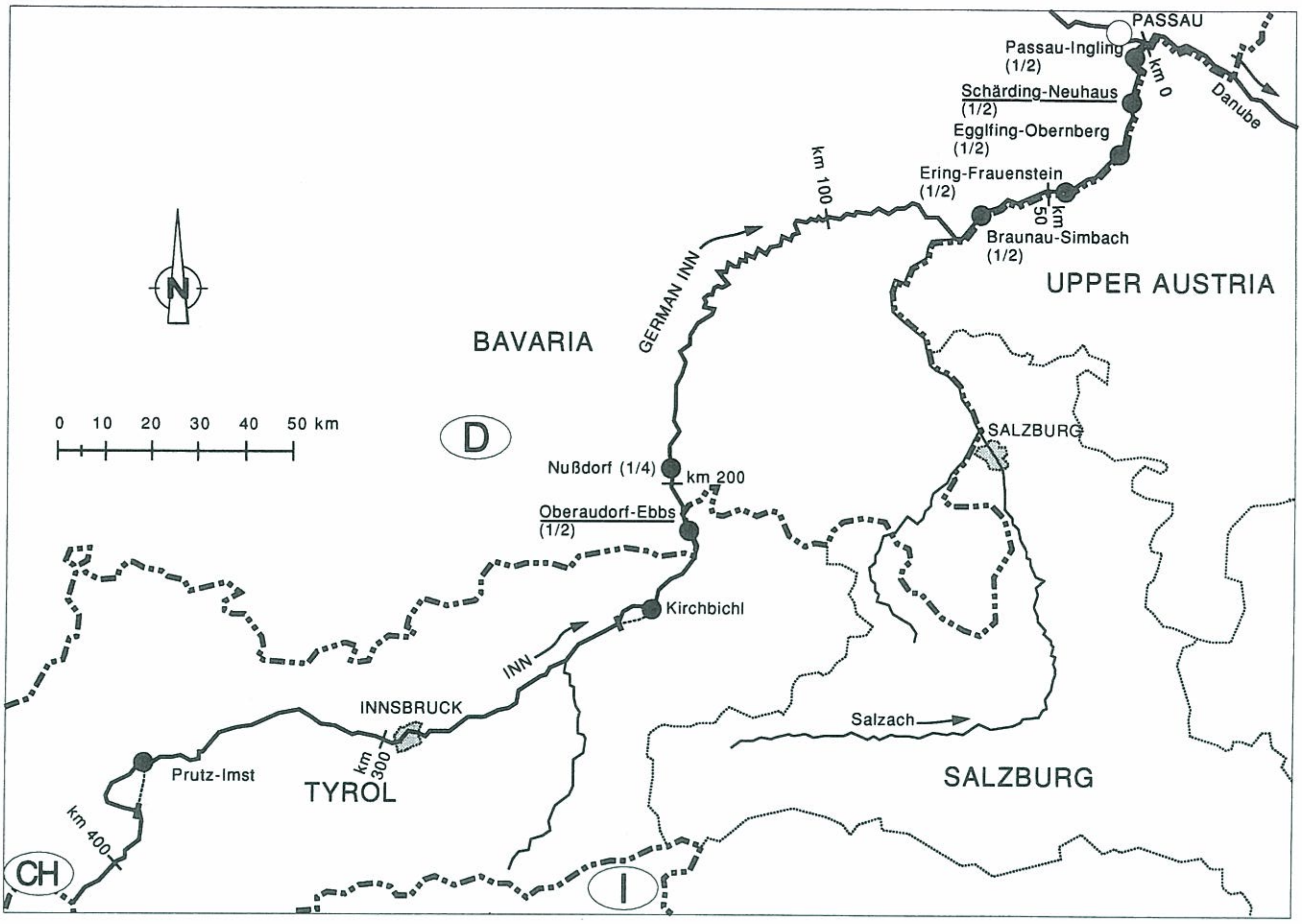
MAIN DIMENSIONS

	Weir	Powerhouse
Height	31 m	38 m
Width	47 m	48 m
Length	51 m	50 m
		101 m in total

VOLUMES

Excavation	
Gravel	160 000 m^3
Rock	90 000 m^3
Concrete	86 000 m^3
Riprap (backwater)	70 000 m^3

INN



Apart from the Danube, the Inn is the largest river in Austria. Its uppermost catchment area lies in south-eastern Switzerland. Then it flows through Austria over a length of 220 km, from the Tyrolean-Swiss boundary near Pfunds (Hochfinstermünz) to the boundary with Germany near Kufstein. It is only after a further length of 150 km in Germany that, from its junction with the Salzach coming from the central Alpine region in Austria, it forms the Austro-German boundary and then, near Passau, discharges in the Danube , which at this point is divided between Germany and Austria. Catchments and mean flows at the above mentioned points along the Austrian boundary are as follows:

Boundary	Catchment	Q m
(a) Switzerland/Austria	2 700 km^2	75 m^3/s
(b) Austria/Germany	9 400 km^2	305 m^3/s
(c) Germany/Austria (incl.Salzach)	22 900 km^2	700 m^3/s
(d) Germany/Austria (Passau)	26 000 km^2	750 m^3/s

The 220 km-long upper course of the Inn (Tyrolean Inn) is at present utilized for energy production by not more than two power stations, Prutz-Imst and Kirchbichl. Both are isolated power stations owned by Tiroler Wasserkraftwerke AG (TIWAG), Innsbruck, described later in this report.

In the German reach of the river Inn, systematic development was carried on, with some interruptions, ever since the construction of a diversion-type power station was started at Töging as far back as 1919, and was finally completed with the commissioning of Nussdorf in 1982. This developed reach comprises 10 power stations – one diversion-type power station and nine stations in the river bed – of which the youngest and most upstream station, Nussdorf, is situated entirely on German territory (Bavaria), whereas backwatering partly extends into the right-hand bank area, which is Austrian, so that Austria's share of this stations's output accounts for 23.5%.

Upstream of Nussdorf follows the site of the Oberaudorf-Ebbs project, also jointly owned (one half each) by Germany and Austria, with the powerhouse and backwater area being situated along the upper Boundary Inn, which extends as far as Kufstein. Construction of this project, originally planned to start in 1983, was actually not started until 1989, as very complex legal aspects had to be solved with the responsible German and Austrian water authorities, the main issues being the expected environmental impact and acceptance by the affected communities. The difficulty of appraising the amount of damages to be paid was the main cause of the delay. Besides, attempts were made by groups of environmentalists to prevent the realization of the hydro project altogether.

In the boundary section ("Grenz-Inn"), covering a length of 70 km between the mouth of the tributary Salzach and the junction with the Danube, development as a continuous series of power stations was started in 1939 and completed in 1965. Five power stations with heads of between 9.6 m and 11.6 m were constructed. A fairly uniform general concept was maintained, in particular on the projects realised between 1951 and 1965, Braunau-Simbach, Schärding-Neuhaus and Passau-Ingling. Together with the Ering-Frauenstein and Egglfing-Obernberg power stations, constructed in the years of war 1939 to 1944, this group develops about 70 km of river length with a head of about 53 m, with a generating capacity of 435 MW and an annual energy of 2 470 GWh made available in equal shares to Germany (Bavaria) and Austria.

The importance of the Inn in terms of water resources and energy potential is illustrated by the following characteristic values of the Inn at its junction with the Danube near Passau, where it substantially exceeds the Danube in flow:

	Inn	Danube
Catchment Area	26 100 km^2	48 200 km^2
Annual Volume of Flow	$23 \cdot 10^9$ m^3	$21 \cdot 10^9$ m^3
Q min	195 m^3/s	165 m^3/s
Q mean	750 m^3/s	630 m^3/s
Q max	7 400 m^3/s	3 700 m^3/s

All the power stations along the Boundary Inn are equipped with vertical-shaft Kaplan turbines (14 to 24 MW each) and show a completely uniform design for reasons of economy. The same applies to the spillways, which consists of five bays of 23 m width and 6 m-wide piers, all closed by hook-type double-leaf gates, at four power stations alike. Only Ering has six 18 m-wide spillway bays and 5 m-wide piers. All the stations are founded on schlier localy called "flinz", except for Passau-Ingling, which is founded on granitic gneiss. The flinz present in the Inn basin is a very stable, impervious and ultra-fine grained sediment perfectly capable of withstanding the structural loads involved. The statical system of the spillways consists partly of a continuous slab with piers placed on top and partly of independent spillway bays and piers (on the granite at Passau-Ingling). In the latter case, bays are fixed with rock anchors to absorb uplift forces. The spillway and powerhouse structures are provided with sheet piling cut-offs and/or appropriate keys. Concrete volumes are all between 150 000 m^3 and 180 000 m^3 per power station. All the power stations were constructed in the original river bed in two successive pits, with diversion of part of the river flow. Pit-enclosures consis-ted of caisson walls as well as gravefill dams with sheet piling cores or partly anchored sheet piling or sheet pile cofferdams.

The structural design of the whole development is called "Inn-type" and is characterised by flat buildings serviced by a main gantry cran moving over the whole barrage.

Structural measures in the backwater areas included embankments (dykes) up to 9 or 10 m high, especially at Braunau-Simbach and Schärding-Neuhaus. Waterside slopes (1:1.75) are protected with poured in-place concrete blankets with wave breakers on top and sheet piling at the toes. This shows that construction of thin diaphragms, later to be applied on the Danube and on the Drau, was not used before 1960. Recipient and drainage channels and, where required for topographical reasons, pumping stations had to be provided outside the dykes.

As mentioned above, Nussdorf, the second power station downstream of Kufstein, near the boundary, has been in operation since 1982. Nussdorf is of the pierhead type. The foundation is made up of largely impervious fine sand. Sheet piling and heavy spillway floors safe from uplift were required. Dykes received concrete blankets on the slopes and thin-diaphragm cut offs as impervious elements. Nussdorf is capable of 48 MW and generates 226 GWh/a. Austria's share accounts for 23.5% (i. e. 11.3 MW and 53 GWh/a).

The Oberaudorf-Ebbs station will be fairly similar to Nussdorf, except that horizontal-shaft Kaplan turbines (bulb turbines) will be provided in the powerhouse- piers. Generation from this power station (expected to be in operation in 1991/1992) will be devided in equal shares between Germany and Austria (30 MW and 134 GWh/a each).

By way of summery, it can be said that the almost completed development of this boundary Inn is evidence of successful technological efforts and a perfect example of how river development is realized on an international basis through the phases of design, construction and operation. Essential problems of ecology arising from the hydro projects have been solved in a perfect way for the Inn, in particular for its boundary section. The fact that the backwater areas on the river Inn have been made an international nature reserve for wildlife (waterbirds and biocenose) many years after the completion of the hydro projects shows that technology and ecology need not necessarily represent conflicting interests.

All development since 1950 has been undertaken by Österreichisch-Bayerische Kraftwerke AG (ÖBK) having its headquarters at Schärding, Austria.

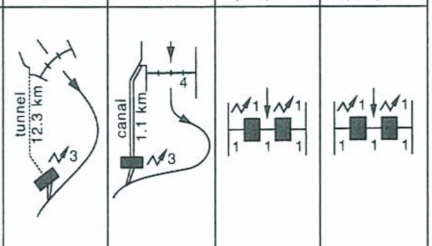
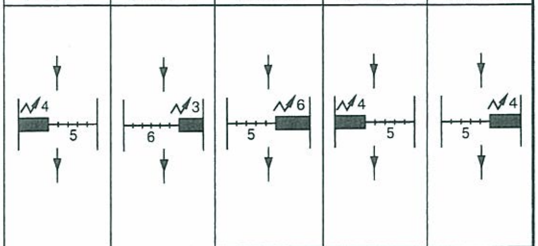
The Prutz-Imst power scheme, mentioned at the beginning of this chapter, is situated in the Tyrolean reach of the river Inn and utilizes the fall of the river near Landeck by cutting across a large river bend by means of a 12.3 km long tunnel; it was completed in 1956. The station is designed for a discharge of 75 m^3/s, the developed head is approximately 140 m. The weir is situated at a narrow section of the valley near Prutz, the site of the large Kaunertal station with the Gepatsch reservoir, and consists of three bays arranged polygonally in plan to allow hydraulically favourable discharge, and closed by hook-type double-leaf gates. Desilting basins next to the weir form the transition to the power tunnel. The underground power station is equipped with three Francis turbines (total 82 MW). Annual energy-production, enhanced by the storage effects from the Kaunertal scheme and from reservoirs in the upper course in Switzerland, is 500 GWh/a.

The second power station on the Tyrolean Inn, Kirchbichl, was constructed as a run-of-river station 1938–1941. Its headrace canal 1 km long, cuts across a pronounced river bend upstream of Kufstein. The four-bay weir structure, equipped with hook-type double-leaf gates is capable of a Q max of 1 800 m^3/s. Next to it is the headrace inlet for 250 m^3/s; head is 8.5 m. Foundation of the weir on course gravel called for a large amount of sheet piling. Kirchbichl has a capacity of 23 MW and generates 134 GWh/a.

At present the Austrian electricity supply derives from the river Inn: 50% of the lower boundary-Inn (Braunau to Passau), 23.5% from Nussdorf and 100% from Kirchbichl and Prutz-Imst. In total: 217+11+23+82 = 333 MW and 1 235+53+134+500 = 1 922 GWh/a. The Oberaudorf-Ebbs power station will add 30 MW and 134 GWh/a.

Planned projects for the Tyrolean part of the Inn include a diversion-type power station utilizing the 160 m head of the Austrian-Swiss boundary section and the adjacent purely Austrian section down to Prutz. Several run-of-river stations, some upstream and some downstream of Innsbruck and one near Kufstein form part of a general masterplan.

Inn

Power station		Prutz-Imst	Kirchbichl	Oberaudorf-Ebbs	Nußdorf		Braunau-Simbach	Ering-Frauenstein	Egglfing-Obernberg	Schärding-Neuhaus	Passau-Ingling
Owner		TIWAG	TIWAG	ÖBK	IWAG 53% ÖBK 47%		ÖBK	IWAG	IWAG	ÖBK	ÖBK
Operation since		1956	1941	constr. (1991)	1982		1953	1942	1944	1961	1965
Stationing	km	387/360	233/230	211	198		61	48	35	19	4
Storage level	m	858.5	497.0	477.4	464.4		348.5	336.2	325.9	314.9	303.0
Flow Q_{mean} Q_{max} Q_{rated}	m^3/s m^3/s m^3/s	80 600 80	320 1 800 250	305 2 150 580	320 2 400 550		706 6 200 1 000	715 6 400 1 040	721 6 600 990	732 6 800 1 000	746 7 400 1 000
Head H_{mean}	m	141.5	8.5	11.7	11.6		11.6	9.6	10.5	11.2	10.0
Capacity (MC)	MW	82	23	(60) 30	(48) 11		(96) 48	(72) 36	(84) 42	(96) 48	(86) 43
Energy (AAE)	GWh	500	134	(268) 134	(226) 53		(554) 277	(428) 214	(468) 234	(540) 270	(480) 240
Layout						German reach of Inn: IWAG L = 123 km, H = 104 m 9 powerstations (Rosenheim to Stammham) 296 MW, 1 781 GWh					
Spillway/Weir		Runserau									
Bays width	m	3×13	4×20	3×16	3×18		5×23	6×18	5×23	5×23	5×23
Pier width/height	m	5–1/16	5/17	20/35.3	25/30.4		6/30	5/31	6/28	6/25	6/27
Gates		hook-double	hook-double	tainter + flap	tainter + flap		hook-double	hook-double	hook-double	hook-double	hook double
Power conduit	km	tunnel 12.3	canal h 1.1	–	–		–	–	–	–	–
Powerhouse		Imsterau									
Construction, type		cavern	high	pier-head	pier-head		low	low	low	low	low
max height	m	30	37	36.3	30		34	34	27	32	32
Turbines, number and type		3 Francis ↓	3 Kaplan ↓	2 Kaplan →	2 Kaplan ↓		4 Kaplan ↓	3 Kaplan ↓	6 Kaplan ↓	4 Kaplan ↓	4 Kaplan ↓
Backwater area											
Length	km	3	9	11	13		14	13	13	17	15

SCHÄRDING-NEUHAUS

River	Inn, at km 18.8	Operating since	1961
Nearest town	Schärding, Upper Austria	Purpose	Hydropower Flood protection
Owner	ÖBK Österreichisch-Bayerische Kraftwerke AG Münchnerstrasse 48, D-8265 Simbach 50% held by Austria and Bavaria respectively		

MAIN CONSTRUCTION PARTS AND FUNCTION

Barrage: Weir: 5 openings of 23 m each
Power station: 4 turbines, 96 MW in total

Development of the Inn on the border between Germany and Austria was continued by a run-of-river station largely similar to that previously built at Braunau-Simbach (1953) and constructed in line with the same economic principles. This chain was completed four years later when the Passau-Ingling power station went into operation in 1965.

HYDROLOGY

Catchment Area	24 430	km^2
Q mean	732	m^3/s
Flow Q max	6 800	m^3/s
Q rated	1 000	m^3/s
Storage Level (a.s.l.)	314.9	m
Mean Head	11.2	m
Capacity	96 MW	50% each for Austria and Bavaria
Energy Output	540 GWh/a	50% each for Austria and Bavaria

GEOLOGY

The granite subsoil sloping from the right-hand bank towards the river is overburdened by impervious marl layers (flinz) interstratified with thin layers of fine sand. They are covered by alluvial river gravel up to 10 m in depth, interspersed by some blocks. At the selected site, both the weir and the powerhouse are founded on flinz whose permeability and shear resistance were established by in-situ testing and lab tests. The backwater area (17 km in length) on the right-hand (Austrian) side is bounded by a natural high bank, while the left-hand (Bavarian) boundary is formed by an embankment extending to the Obernberg station situated upstream, to provide flood control for an area of 8 km^2. The backwater has an average width of 300 m, widening in places depending on the line of the high bank.

HISTORY AND DEVELOPMENT OF DESIGN

Two power stations (Ering-Frauenstein and Egglfing-Obernberg) were built on the stretch of the Inn along the border during the war. New stations were added between 1951 and 1965, at Braunau-Simbach, Schärding-Neuhaus and Passau-Ingling. These stations have been planned, built and operated by ÖBK, a binational company whose shares and electricity generation are owned in equal parts by Austria and Bavaria (Germany).

The structural design of this type of station is called "Inn model". It is characterized by flat buildings serviced by a main gantry crane traveling across the powerhouse and weir for assembly and maintenance. Construction of the barrage was carried out in two successive pits in the river channel widened at places.

FOUNDATION

Imperviousness of the weir and powerhouse foundations is ensured by cut-off walls extending deeply into the flinz. Grouting was found to be unnecessary. The stability analysis was based on the full uplift pressure, taking into account the mechanical properties of the soil. The stilling basins were reinforced against uplift occuring during repairs.

INSTRUMENTATION

Standard geodetic monitoring. Piezometers in each weir pier, between powerhouse and weir and in the left quay wall for monitoring uplift pressure.

CONCRETE

Concrete aggregates obtained from the river and its banks were screened into five fractions (70 mm max.). Two types of pumped concrete were produced with 250 and 300 kg of Portland cement per m^3. Compression strengths achieved at 56 days were 28.5 and 33.0 N/mm^2 respectively. The concrete was poured in 2.4 m lifts by means of concrete pumps over distances of up to 180 m. Stone lining for the weir openings was not necessary, with surface protection afforded by a 30 cm vacuum concrete facing.

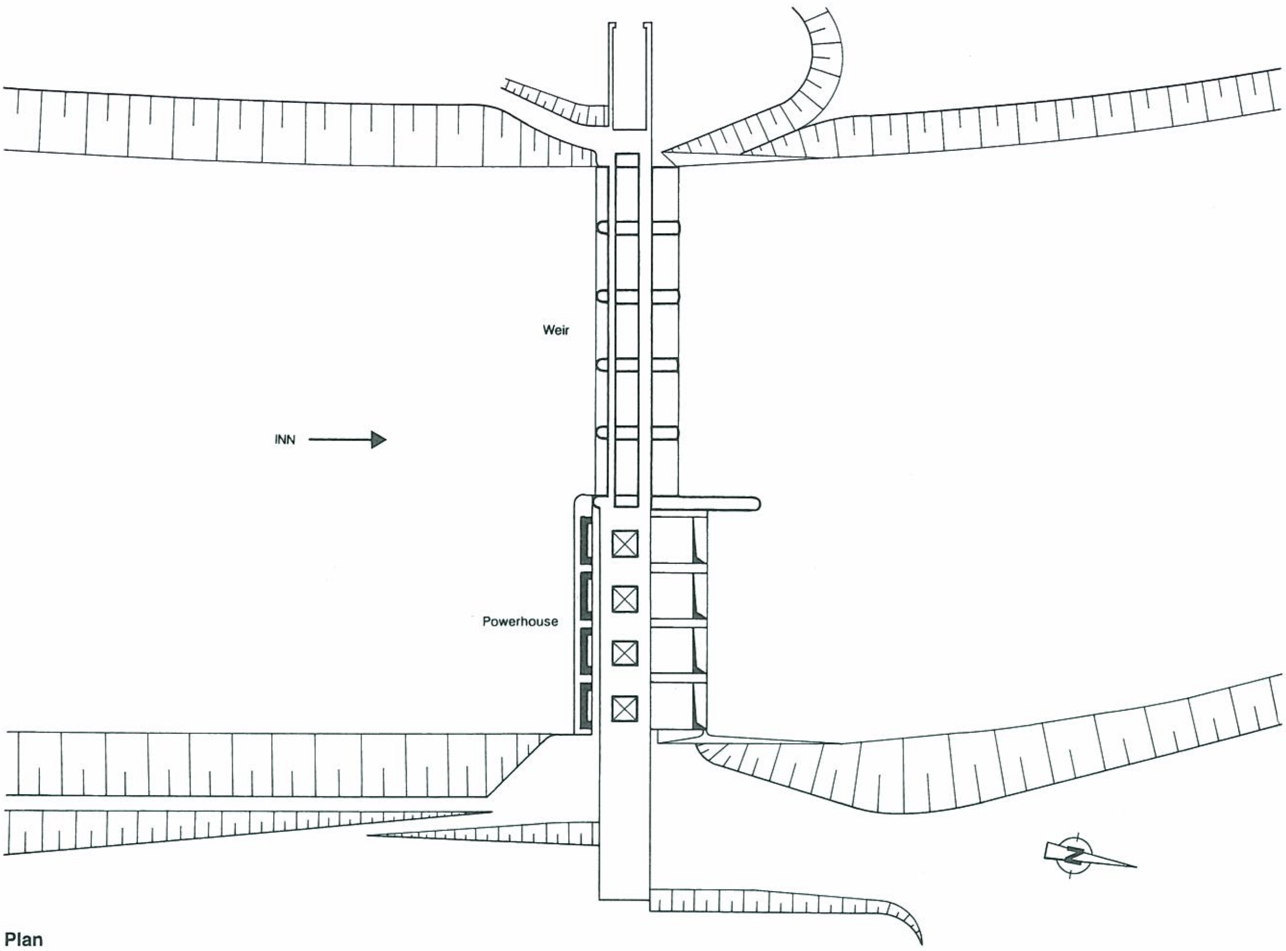
The left-hand embankments, 3.50 m wide at their crest, with a slope of 1:1.75 at the water side and 1:1.67 to 1:5 at the off-water side, consist of gravel of 100 mm maximum size, placed in compacted layers. The water-side slope is protected by a concrete slab of 16 cm in thickness and a wavebreaker. A narrow trench was excavated to allow the concrete facing to be carried down to the top of the 40 500 m^2 steel sheet piling which extends through the gravel overburden into the flinz.

FLOOD RELIEF WORK

Weir with five openings of 23 m in clear width, with hook-type double-leaf gates 13.8 m in height. The design flood of 6 800 m^3/s can be discharged even with one weir opening blocked and with the normal headwater level maintained. A total of 7 530 m^3/s can be discharged when all gates are open. Maximum unit discharge of the design flood is 59 m^3/s per linear metre of weir.

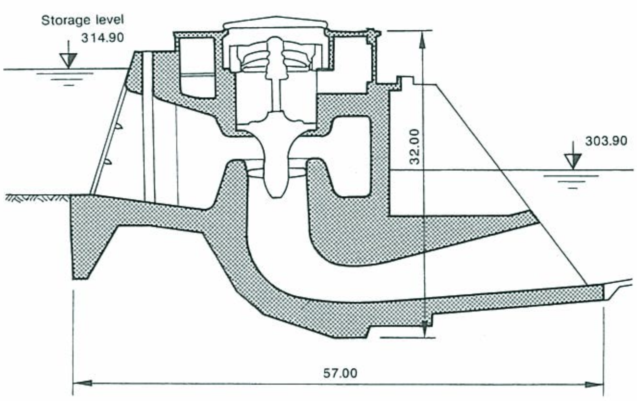
POWERHOUSE

Equipped with four vertical shaft Kaplan turbines with a rated discharge of 250 m^3/s and a capacity of 24 MW each, coupled to umbrella-type generators of a capacity of 30 MVA. The intake is protected by a three-part rack of 265 m^2 in surface. Gantry-type stoplogs are provided in front of the rack, and stoplogs are fitted in a slot behind the rack.

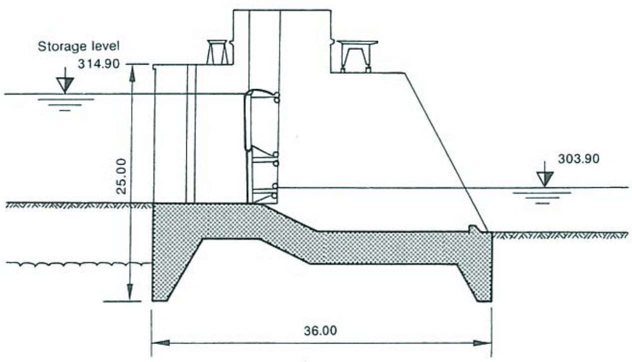


Plan

Powerhouse section



Weir section



MAIN DIMENSIONS

	Weir	Powerhouse
Height	31 m	32 m
Width	36 m	57 m
Length	139 m	102 m
	241 m in total	

VOLUMES OF BARRAGE

Excavation	
Gravel	310 000 m^3
Flinz	75 000 m^3
Concrete	168 000 m^3

VOLUMES AT BACKWATER AREA

Left Bank

Embankments (10 m high max.; 16 km long)

Volume	2 400 000 m^3
Concrete	45 000 m^3
Sheet Piling	40 500 m^2

Right Bank

Natural Slope Stabilization

Concrete	35 000 m^3

OBERAUDORF-EBBS

River	Inn, at km 211.4	Operating	Scheduled as of February 1992
Nearest town	Kufstein, Tyrol	Purpose	Hydropower Flood protection Prevention of riverbed degradation
Owner	ÖBK Österreichisch-Bayerische Kraftwerke AG Münchnerstrasse 48, D-8265 Simbach 50% held by Austria and Bavaria respectively		

MAIN CONSTRUCTION PARTS AND FUNCTION

Barrage	Pier-head power station Weir: 3 openings of 16 m width each, spaced by 2 piers of 20 m width and one 30 MW bulb turbine each, 60 MW in total

Final stage of the combined German/Austrian hydropower development on the upper reach of the Boundary-Inn downstream of Kufstein.

HYDROLOGY

Catchment Area		9 400 km²
Q mean		305 m³/s
Flow Q max		2 150 m³/s
Q rated		580 m³/s
Storage Level (a.s.l.)		447.4 m
Mean Head		11.7 m
Capacity	60 MW	50% each for
Energy Output	268 GWh/a	Austria and Bavaria

GEOLOGY

After sinking a total of 21 exploratory drillholes at the barrage area, it was found that no rock or impervious ground could be reached so that the weir and piers had to be erected on gravel and layers of sand. The pit had to be sealed by a diaphragm 20 m in depth in the gravel combined with a 10 m cut-off-curtain above. To cope with the unexpected inflow of water (seepage), mostly from the bottom, it was necessary to raise the pump capacity at almost 3 m³/s. The Inn power station at Nußdorf, only 12 km away, which was built in 1980 under similar geological conditions, required only 0.1 m³/s in pumping capacity. Even then, original plans for Oberaudorf-Ebbs had already provided for pumps of a total capacity of 1 m³/s.

HISTORY AND DEVELOPMENT OF DESIGN

Oberaudorf-Ebbs is the final link in the chain of Inn power stations downstream of Kufstein. It will complete the development of the 220 km stretch of the river until its confluence with the Danube in Passau. With it, the Inn now accommodates seven power stations operated jointly by Austria and Bavaria plus nine more along its stretch on German territory.

When the Nussdorf power station started operation in 1982, plans provided for an immediate realization of the Oberaudorf-Ebbs project directly upstream, i. a. to halt degradation at the free-flowing stretch of the river. The water management approval procedure was, however, retarded by objections voiced by the adjoining communities and in particular by Kufstein, by environmentalist concerns and by divergent statements submitted by the two countries involved. Accordingly construction, which had been scheduled to start in 1983, did not commence until February 1988 and then only for diverting the Inn for the excavation pit, only to be suspended four weeks later. It took fully one year (until February 1989) before work could be resumed after solutions had been found for the legal difficulties encountered.

Oberaudorf-Ebbs is a pierhead power station of a type similar to Nussdorf except that it has horizontal bulb turbines in its piers.

Its backwater area stretches for 10 km and requires embankments of up to 8 m in height, although at some places natural high-level river banks could be used. Diaphragm walls (max. 20 m) are provided for cut off. The embankments are made of gravel from the "Schanzler Lahn" (rock debris) mixed with wetland sand. Biotopes of a total extension of 125 000 m² will be created on both banks of the Inn.

FOUNDATION

The weir and piers are provided with a shallow foundation on the sand/gravel layers. Cut off against excessive underseepage is provided by a reinforced concrete curtain. No additional grouting work is planned. The uplift pressure is accounted for in the stress calculations. The weir links up laterally with the embankments by wing walls founded on concrete curtains.

The barrage is being built in a single pit after diverting the Inn on its left-hand (Bavarian) bank for the construction period. The diversion channel, designed for Q = 2 150 m³/s, required excavation of 200 000 m³ of gravel and sand.

INSTRUMENTATION

Geodetic monitoring, piezometer tubes for uplift pressure and percolation water measurement.

CONCRETE

Excavation material produced from the diversion channel is taken and processed for use as aggregates. A total of 150 000 m³ of concrete will be poured. The sill and stilling basin will be surfaced with granolithic concrete.

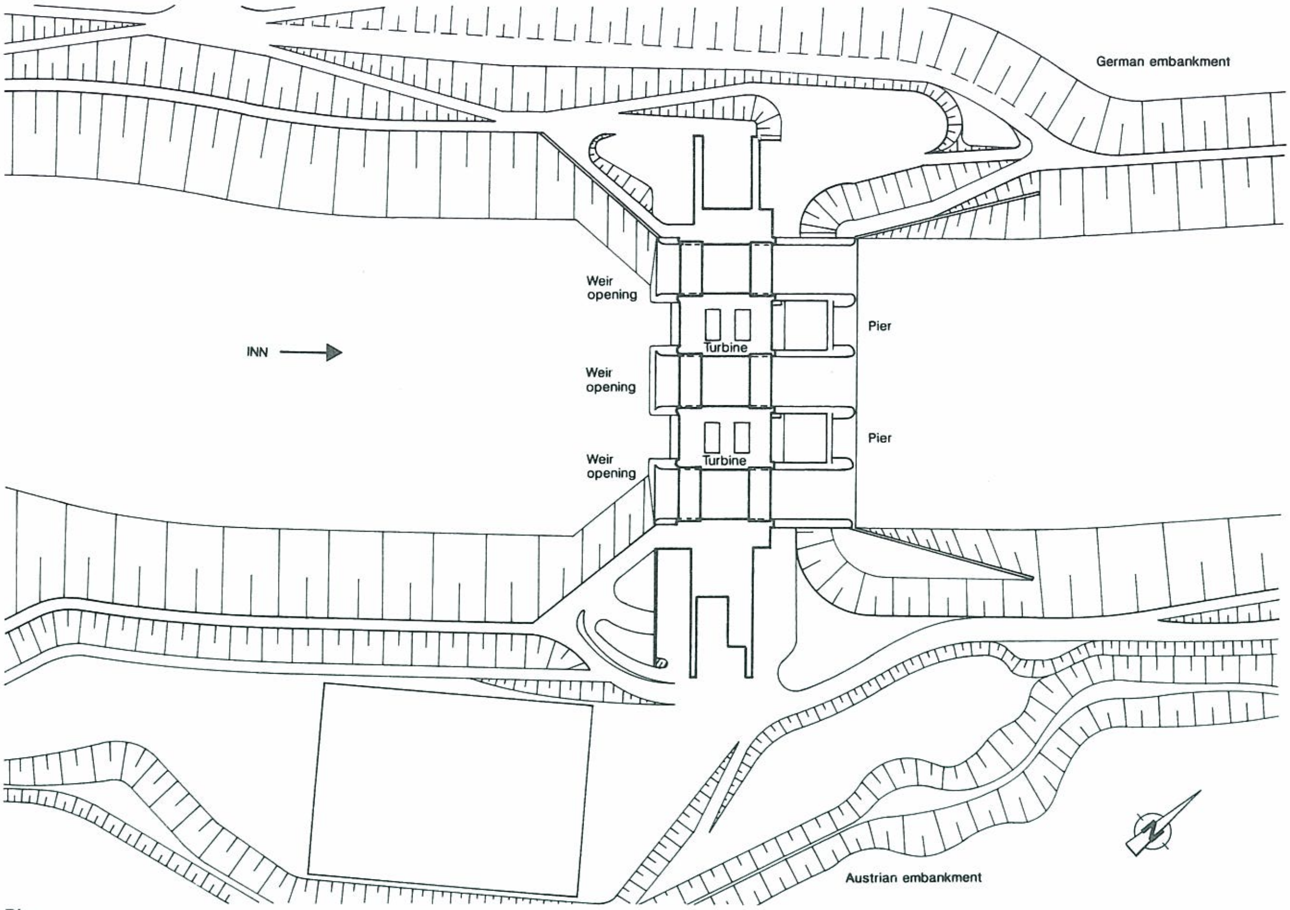
FLOOD RELIEF WORKS

Weir with 3 openings of 16 m width each, closed by tainter gates with flaps (15.5 m in total height). Q max = 2 150 m³/s can be discharged by 2 (n-1) openings; all 3 openings can release Q = 2 700 m³/s.

POWER STATION

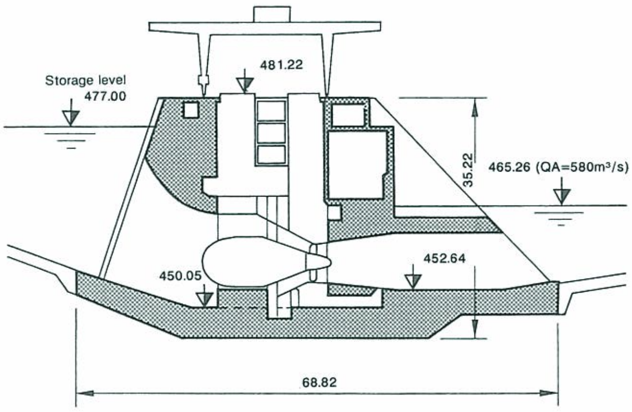
Pierhead power station with 2 piers, each 20 m in width and 36.3 m in height, 1 bulb turbine of Q rated = 290 m³/s, 6.10 m in diameter, 30 MW, 35.3 MVA generator in each pier.

Two gantry cranes of 60 t capacity each are provided for maintenance. They run across the crest of the weir and piers and are also used for moving stoplogs.

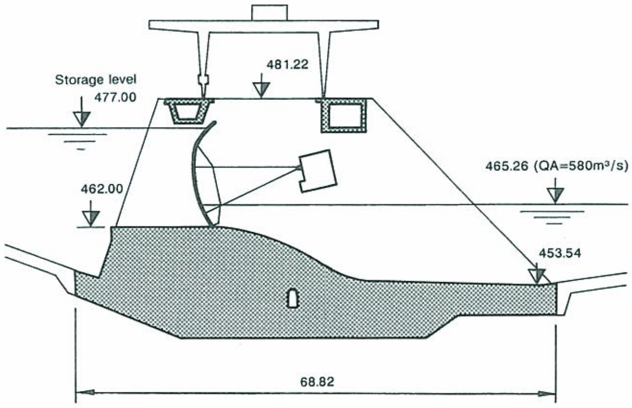


Plan

Section through pierhead power station



Weir section



MAIN DIMENSIONS

	Weir	Powerhouse Pier
Height	35.3 m	36.3 m
Length	3 x 16 m	2 x 20 m
	88 m in total	

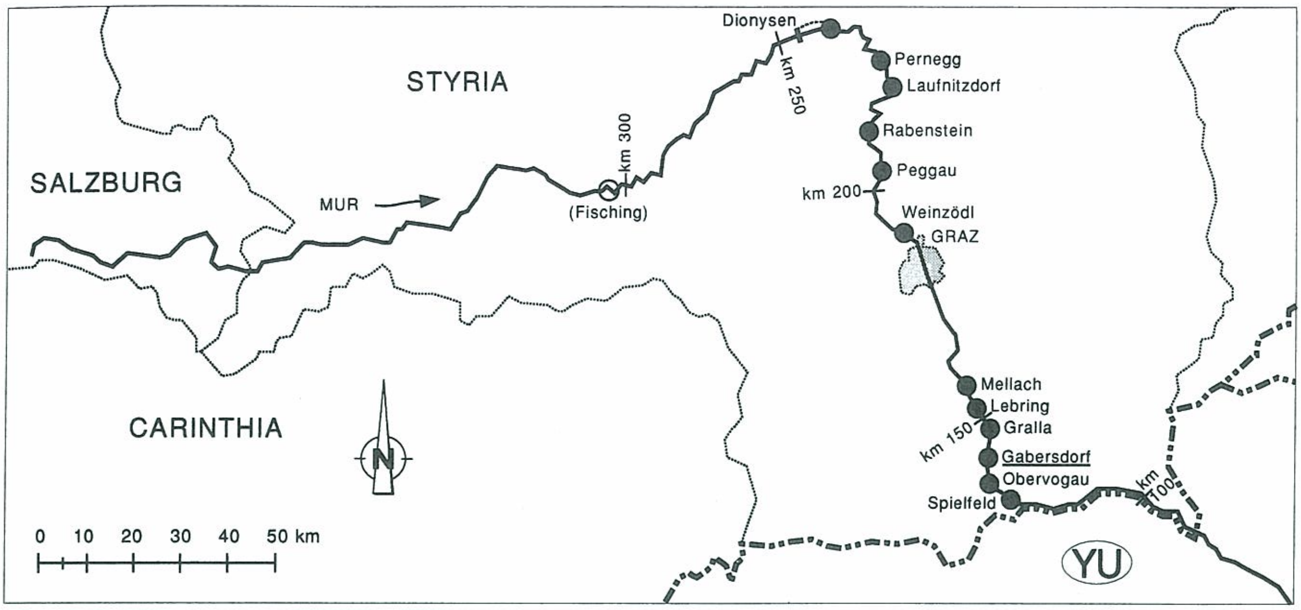
VOLUMES OF BARRAGE AREA

Excavation (sand/gravel)	2 000 000 m^3
Concrete	150 000 m^3

VOLUMES AT BACKWATER AREA

Embankments	1 600 000 m^3
Riprap	210 000 m^3

MUR



The river Mur rises on the southern flank of the main ridge of the Alps (Niedere Tauern). It first flows in an easterly direction, then continues southward, passing through the city of Graz and finally reaches the Austro-Yugoslav boundary zone. Its catchment area there is approximately 10 300 km² and its mean flow 158 m³/s.

Development of this river began before 1904, then was carried on at large intervals and is now nearing completion in the middle reaches upstream and downstream of Graz. Over the last few years development has been startet in the uppermost reach, which is partly on Salzburg territory. There, enlargement and upgrading of the Rotgüldensee hydro scheme is under construction, the power station on the Zederhaus, a tributary of the river Mur, is completed and Einach (28 MW, 125 GWh/a) is in the project stage. Further downstream, where the Mur enters Styrian territory, there is a small power station at Bodensdorf (Mur: 7 MW, 34 GWh/a.; Paal: 27 MW, 86 GWh/a), which has been in operation since 1982, and one at St. Georgen (6 MW, 32 GWh/a), in operation since 1985. Construction of a power station at Fisching (18 MW, 74 GWh/a), due to start in 1990, is meeting with some resistance on the part of environmentalists.

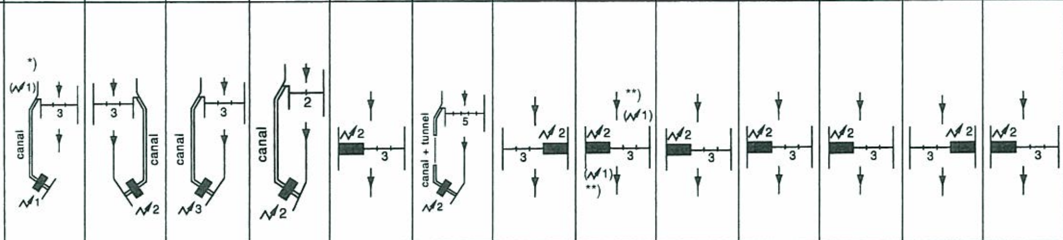
Over the last two decades development has proceeded stage-wise, mainly in the reach between Graz and Spielfeld (where the boundary Mur begins). All the projects are run-of-river stations with three-bay spillways and adjacent powerhouses equipped with 2 bulb turbines each. Development of the reach upstream of Graz, up to the Dionysen station, dates back to the period between 1908 and the time of the Second World-War. In accordance with the state of engineering at that time these power stations are mainly of the diversion type, with long open headrace canals, which allowed heads of up to 19 m to be accomplished. Construction and operation is in the hands of 2 companies: Steirische Wasserkraft und Elektrizitätsgesellschaft (Steweag), Graz, and Steiermärkische Elektrizitäts AG (StEG), Graz.

The six major power stations (of more than 10 MW each) situated upstream of the provincial capital Graz, at Dionysen, Pernegg, Laufnitzdorf, Rabenstein, Peggau and Weinzödl have a total capacity of 89 MW and generate 492 GWh/a. This total output is increased by existing minor power stations in the reach upstream of Graz, owned by comunities, industries, private owners with a total of approximately 23 MW and 150 GWh/a. Downstream of Graz the 6 power stations at Mellach, Lebring (new), Gralla, Gabersdorf, Obervogau and Spielfeld have nearly the same output i. e. 91 MW and 448 GWh/a.

As mentioned before almost all the stations in the upper reach are of the diversion type, equipped according to the state of engineering at the time of their construction: vertical-lift gates, roller drum gates and hook-type double-leaf gates; concrete-lined trapezoidal canals, later partly replaced by bituminous concrete. Powerhouses have been equipped with Francis turbines, most of them have meanwhile been replaced by Kaplan turbines. The newer stations are equipped with bulb turbines, only Weinzödl has two Straflo units, where the generators are attached around the turbines.

With about 216 MW and 1 155 GWh/a the chain of power stations on the river Mur, although counting among the smaller developments, is interesting in conveying an idea of hydro-development in Austria over the whole century. Many valuable impetus has come from engineering hydropower on the Mur, both in the past and in our times. The continual growing demand for renewable and clean energy, to be produced in the own country, urges the further development of the hydropotential on the river Mur, which is estimated about 3 000 GWh/a in total and only 1/3 is used until now. Substantial hydro researches are still available, both in the upper course and along the reach forming the boundary with Yugoslavia over a length of about 36 km with a fall of 50 m.

Mur

Power station		Fisching	Dionysen	Pernegg	Laufnitz-dorf	Rabenstein	Peggau	Weinzödl	Mellach	Lebring (neu)	Gralla	Gabers-dorf	Ober-vogau	Spielfeld
Owner		STEWEAG	STEWEAG	STEWEAG	STEWEAG	StEG	StEG	StEG	STEWEAG	StEG	STEWEAG	STEWEAG	STEWEAG	STEWEAG
Operation since		constr.	1944	1927	1931	1987	1908	1982	1985	1987	1964	1974	1977	1982
Stationing	km	307.7/304.3	244.2/239.9	229.6/226.7	222.7/214.8	207.5	205.2/200.8	184.0	159.1	151.6	147.6	142.6	137.6	132.3
Storage level	m	681.0	504.3	467.3	448.3	418.8	410.0	363.0	305.5	292.0	281.0	271.5	262.0	254.0
Flow Q_{mean}	m³/s	48	80	105	108	115	115	115	124	135	135	135	140	157
Flow Q_{max}	m³/s	620	1 200	1 500	1 500	1 250	1 250	1 250	1 250	1 250	1 250	1 250	1 250	1 500
Flow Q_{rated}	m³/s	86/16*)	85	140	110	180	110	180	180/12/9**)	200	200	220	240	240
Head H_{mean}	m	22.3/11*)	16.5	16.7	18.6	8.2	14.4	9.0	9.6/7.4/7.4**)	10.2	8.3	8.2	7.1	7.0
Capacity (MC)	MW	16.5/1.5*)	12	18	17	13	13	16	15/0.75/0.55**)	19.4	14.5	14.5	13	13
Energy (AAE)	GWh	67/7*)	70	105	108	62	78	68	76/6/2**)	89	71	68	60	76
Layout														
Spillway/Weir			Mötschlach	Pernegg	Mixnitz		Peggau							
Bays width	m	3x10	3x15	3x15	2x25	3x15.5	2x14/3x12	3x16.5	3x15	3x15.5	3x15	3x15	3x20	3x20
Pier width/height	m	3.0/21.0	3.8/15.5	5.0/27.0	5.0/13.5	3.0/18.0	2.0/11.0	3.0/18.0	2.5/18.0	3.0/22.0	4.0/16.0	4.0/19.0	2.4/16.0	2.4/16.0
Gates		tainter + flap	hook-double	hook-double	roller-drum	tainter + flap	leaf	tainter + flap	tainter + flap	tainter + flap	tainter + flap	tainter + flap	tainter + flap	tainter + flap
Power conduit	km	canal h 1.1 t 0.1	canal h 3.7 t 0.4	canal h 2.3 t 0.3	canal h 7.0 t 0.2	–	canal 2.1 tunnel 1.0	–	–	–	–	–	–	–
Powerhouse			Dionysen	Kirchdorf	Laufnitzdorf		Peggau							
Construction, type		covered	high	high	high	high	high	high	high	high	high	high	high	high
max height	m	40	33	32	31.4	20.4	17.0	23.0	27.0	23.7	29.9	25.7	25.0	26.0
Turbines, number and type		1 Kaplan ↓ + 1 Kaplan*) →	2 Kaplan ↓	3 Francis ↓	2 Kaplan ↓	2 Kaplan ↓	2 Kaplan ↓	2 Kaplan Straflo →	2 Kaplan → 2 Kaplan**) →	2 Kaplan →	2 Kaplan ↓	2 Kaplan →	2 Kaplan →	2 Kaplan →
Backwater area														
Length	km	4.9	1.5	5.3	4.0	4.0	2.5	1.2	3.4	4.9	3.8	5.4	4.6	5.3

GABERSDORF

River	Mur, at km 142.2
Nearest town	Leibnitz, Styria
Owner	STEWEAG Steirische Wasserkraft- und Elektrizitäts AG Leonhardgürtel 10 A-8010 Graz
Operating since	1974
Purpose	Hydropower Limited flood protection Biotope

MAIN CONSTRUCTION PARTS AND FUNCTION

Barrage	Weir: 3 openings of 15 m each Power station: 2 bulb turbines, 14.5 MW in total

Link in the development between Gralla (1964) and the Austrian-Yugoslavian border at Spielfeld. The barrage was designed analogously to the Gralla power station. Primarily it serves as a source of hydropower and provides limited flood protection. With the backwater area silting up, it is expected to become a biotope similar to that at the Gralla backwater (which is already a bird sanctuary).

HYDROLOGY

Catchment Area	8 204	km^2
Q mean	135	m^3/s
Flow Q max	1 250	m^3/s
Q rated	220	m^3/s
Storage Level (a.s.l.)	271.5	m
Mean Head	8.2	m
Capacity	14.5	MW
Energy Output	68	GWh/a

GEOLOGY

The relatively flat Mur river bed cuts through an alluvial gravel field thinly covering a tertiary system that consists mainly of sand, clay and marl transitions. The grayish brown tertiary substratum called "Tegel" is sufficiently stable and impervious. Exploratory drilling found that the tertiary layers crop out at some 4–6 m below the original ground, so that it was no problem to provide adequate foundations.

The backwater area is enclosed by embankments of 6.5–7 m on both sides to account for a reservoir level of about 5–5.5 m above the original surface. Their length of about 3.3 km on both sides was filled with material from underwater excavations which was interspersed with a high rate of the Tegel which had to be excavated as well. Adjacent to the weir, diaphragms of 400 m in length were built on both banks to link up with the compact tertiary system. No cut off was necessary at the upstream backwater area, either for the embankments or for the recent gravel between the dam contact surface and the tertiary layer, because the Mur carries considerable amounts of suspended particles (paper and woodpulp mills) which effectively seals the barrage area.

HISTORY AND DEVELOPMENT OF DESIGN

The first stage of a development project spanning the Mur to the south of Graz was begun by STEWEAG in 1962 with its Gralla power station (completed in 1964). The rising demand for energy and gradual degradation of the Mur bed in the Gralla tailwater made it necessary to construct the Gabersdorf station. The construction method and plant type used for Gralla were reemployed for Gabersdorf and later for the next stations in line, Obervogau (1977) and Spielfeld (1982), with the exception of the turbines (bulb type) and the architectural structure of the powerhouse.

FOUNDATION

The weir and powerhouse are founded on sufficiently stable and impervious Tegel, at places anchored by studs. No cut-off structures were necessary except for the embankments abuting the power station (for a length of 400 m on each side).

Stability was calculated for a standard uplift pressure (linear decrease to the tailwater) to take into account the soil mechanical properties even though the area downstream of the stilling basin is drained. The bottoms of the stilling basin have no static effect on the weir.

INSTRUMENTATION

Usual geodetic monitoring, drainage water is measured in monitoring galleries, oxygen content of the river water is checked at a turbine, ground water monitoring along the backwater area and tailrace.

CONCRETE

Aggregates were taken from gravel pits of the Graz and Leibnitz fields. Bulk concrete parts such as the weir and underground structures for the powerhouse used B 225 with at least 250 kg of Portland cement per m^3 of concrete, while B 300 with 300 kg of Portland cement was used for the finely structured parts such as bridges, the powerhouse building, buttresses and pulling heads for the tainter gate bearings.

In view of the relatively minor bed load transport (usually small-grained) there was no need for comprehensive armoring at the weir or any special surface hardening measures at the weir concrete. Accordingly, only a small area of the weir sill has been armored.

The water-side slopes in the backwater area and tailrace have an incline of 1:2. They are secured by slope plates near the power station and riprap at the other places.

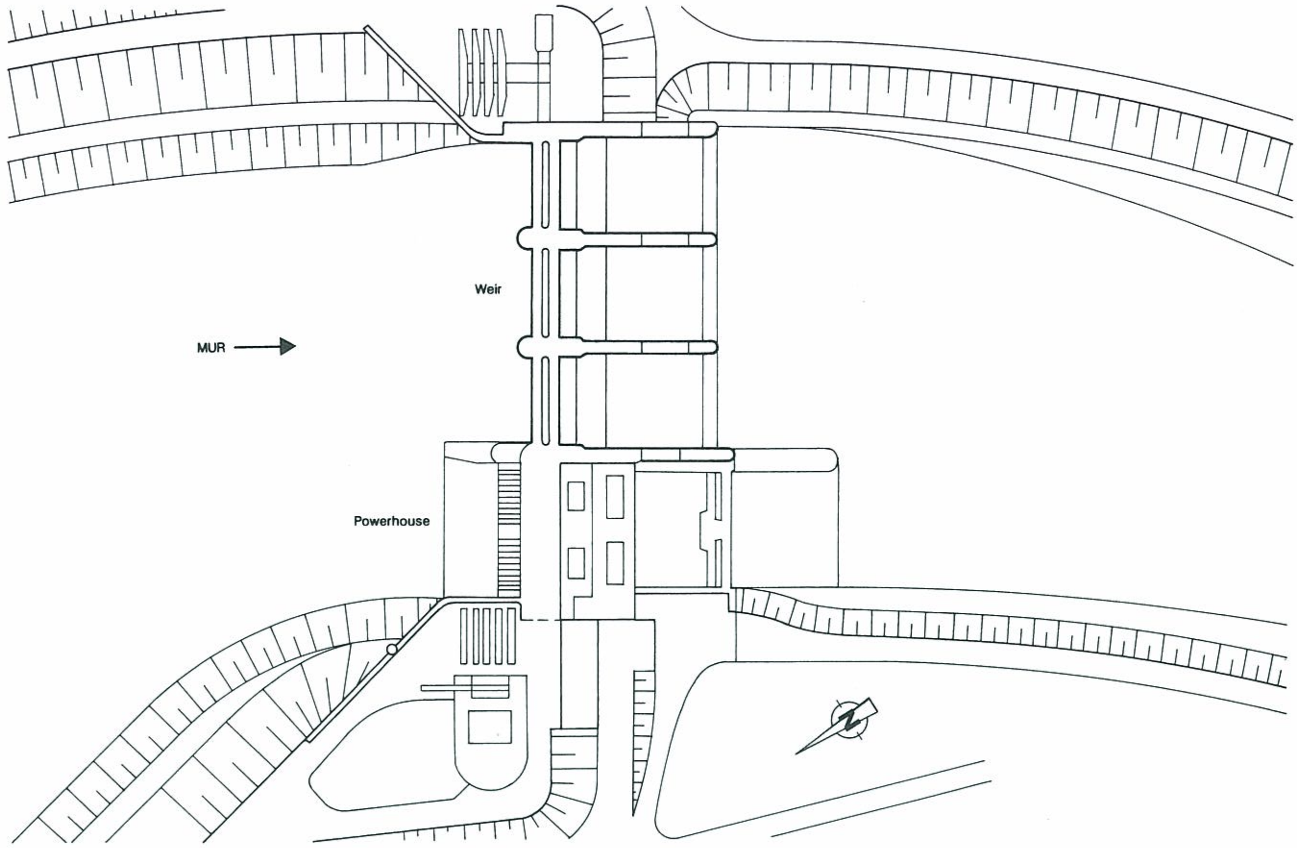
FLOOD RELIEF WORK

Weir with 3 openings, each 15 m in width and with a gate height of 8 m. The tainter gates and flaps are moved hydraulically.

The maximum flood of 1 250 m^3/s can be discharged even when one gate is blocked and the top water level is retained. Some 1 800 m^3/s can be discharged when all gates are opened and the top water level is retained.

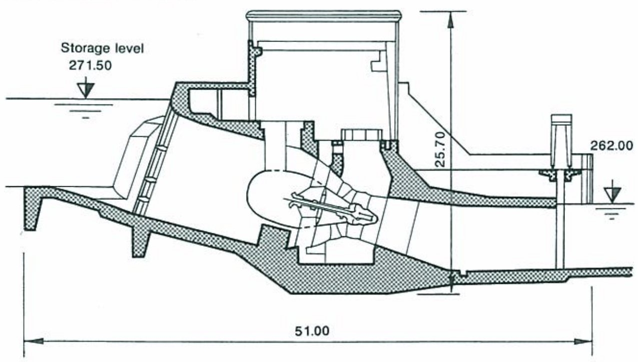
POWERHOUSE

The powerhouse accommodates two bulb turbines on a shaft inclined by some 14°. The rated flow is 110 m^3/s each. At a rated head of 8.6 m the capacity is 9 MW each. The stoplogs for the turbine inlets are moved by the rack cleaning unit.

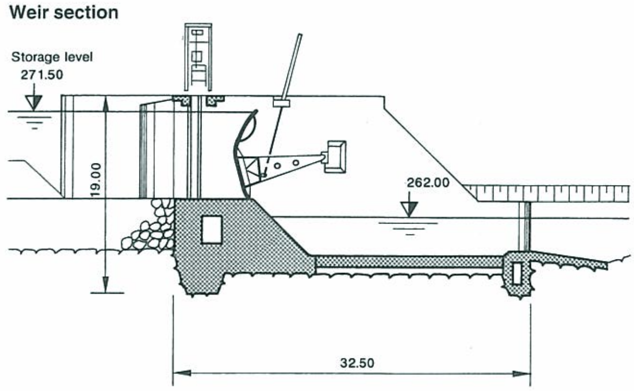


Plan

Powerhouse section



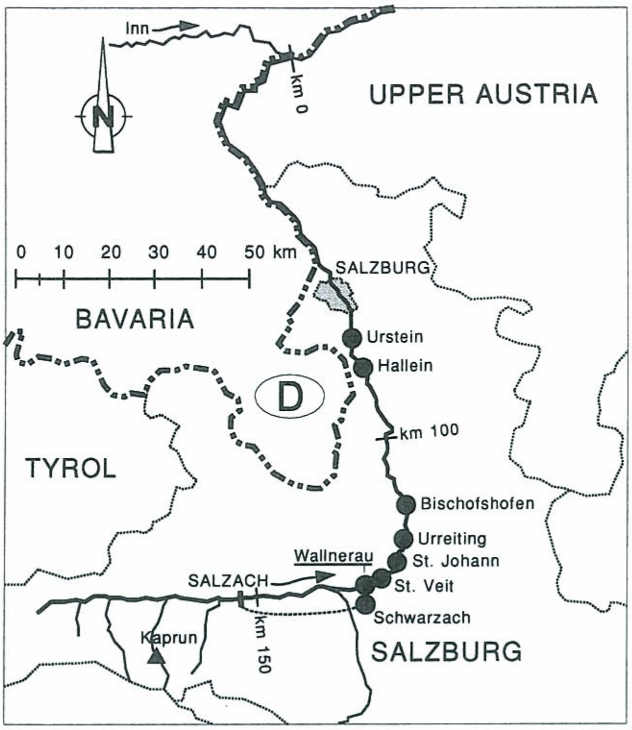
Weir section



MAIN DIMENSIONS

	Weir	Powerhouse
Height	19 m	26 m
Width	51 m	51 m
Length	53 m	25 m
	78 m in total	

SALZACH



The Salzach drains the northern flank of the Hohe Tauern and flows over almost its whole length of 225 km (source to junction with the river Inn) on the territory of the province of Salzburg; only its lowest reach of 59 km forms the boundary between Bavaria (Germany) and Austria. At its junction with the Inn it has a catchment area of 6 734 km^2. In spite of the substantial energy potential of this river only a small station of 3 MW capacity was constructed for a papermill at Hallein in 1928.

It was not before the construction of Glockner-Kaprun with its large storage schemes and of the schemes owned by the Austrian Railways in the Stubach valley that a reach with a high gradient between Bruck and Golling was worth developing. The main reason was the increased streamflow in the Salzach during midwinter months due to these upstream reservoirs (February 11 m^3/s to 27 m^3/s, or March 15 m^3/s to 30 m^3/s). This effect – very similar to the river Inn (power station Prutz-Imst) – made it useful to start construction of the power station Schwarzach 1953 as the main station of the Middle Salzach chain. The three-bay weir, capable of discharging a maximum flood of about 1 100 m^3/s, diverts a maximum flow of 90 m^3/s. After passing through sand traps (Dufour) the water is conveyed through a tunnel 16.9 km in length and 5.5 m in diameter, which ends in a daily storage reservoir (1.5 million m^3) for peak load. The power station is located downstream of the reservoir and is equipped with 4 Francis turbines of 30 MW each, generating total 480 GWh/a.

It was for the construction of the long and large diameter power tunnels for the Inn (Prutz-Imst 1953–56) and the Salzach (Schwarzach 1954–58) that the most outstanding progress in modern tunnel engineering was achieved, now universally known as the NATM (New Austrian Tunnelling Method) which is based on the principle of immediate application of shotcrete and rock bolting or anchorage to mobilize the rock mass as a load-bearing element.

Following the Schwarzach power station, study of a great number of project possibilities led to the selection of a Middle Salzach series of 7 power stations (total length 22 km, head 70 m) to be constructed in several phases. The names of the 7 plants – in direction of flow – are Wallnerau, St.Veit, St.Johann, Urreiting, Bischofshofen, Kreuzbergmaut and Pfarrwerfen. Work was started with Bischofshofen (1985) followed by Urreiting immediately upstream and of practically equal design (1986).

The stations St.Veit (1988) and Wallnerau (1990) followed next. The fifth station, St. Johann, is under construction and will be completed 1990/91. By that time more than 70% of the planned total of 112 MW and 504 GWh/a (30% in winter and 70% in summer) will be completed. Each station, except Wallnerau, consists of a powerhouse block with 2 horizontal bulb turbines and an adjacent three-bay spillway block. The design of Wallnerau makes allowance for the particular project situation at the confluence of the river Salzach with the tailrace from the Schwarzach power station. It consists of one powerhouse with 2 bulb turbines (16 m head) for the river Salzach and one powerhouse with 3 S-turbines (6 m head) for the tailwater of the Schwarzach station. This part can be overflowed to enable independent operation of Schwarzach in any case. The spillway for the river Salzach consists of 2 bays, each equipped with a tainter gate and a flap and a concrete beam between.

Most of the relatively low dykes of all the other stations have an asphaltic facing covered with dry stone pitching and a thin diaphragm cut-off extending almost 20 m into the foundation for imperviousness. A great deal of work was done to preserve the nature, landscape, flora and fauna in cooperation with ecological specialists and in accordance with the desires of the population. The next stations to be built will be Kreuzbergmaut and Pfarrwerfen downstream of Bischofshofen.

In the much flatter reach between Golling and the river's junction with the Inn only 2 power stations have so far been built. Between Hallein and the city of Salzburg the first one was Urstein (1971) with a capacity of 20 MW and an output of 107 GWh/a. In terms of stream engineering Urstein has to fulfill the additional function of preventing further degradation of the riverbed. A similar project was realized at Hallein, where a sill dating from the year 1972 was reconstructed as a power station, which was put in operation in 1987. It has a capacity of 12 MW and generates 66 GWh/a.

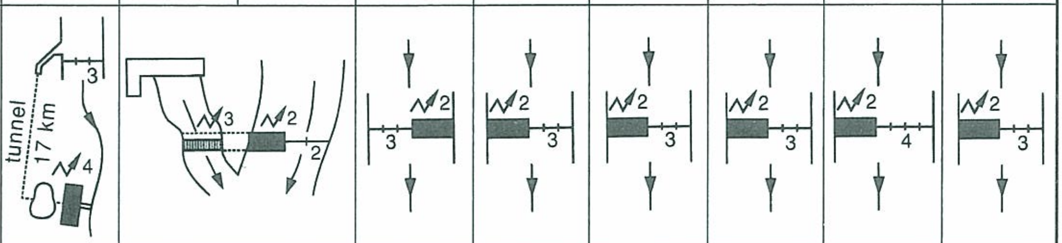
For several decades plans have been considered, which provide the construction of a series of power stations in the reach downstream of the city of Salzburg, where the

Salzach forms the boundary with Bavaria. A definite date for the implementation of this project is not known.

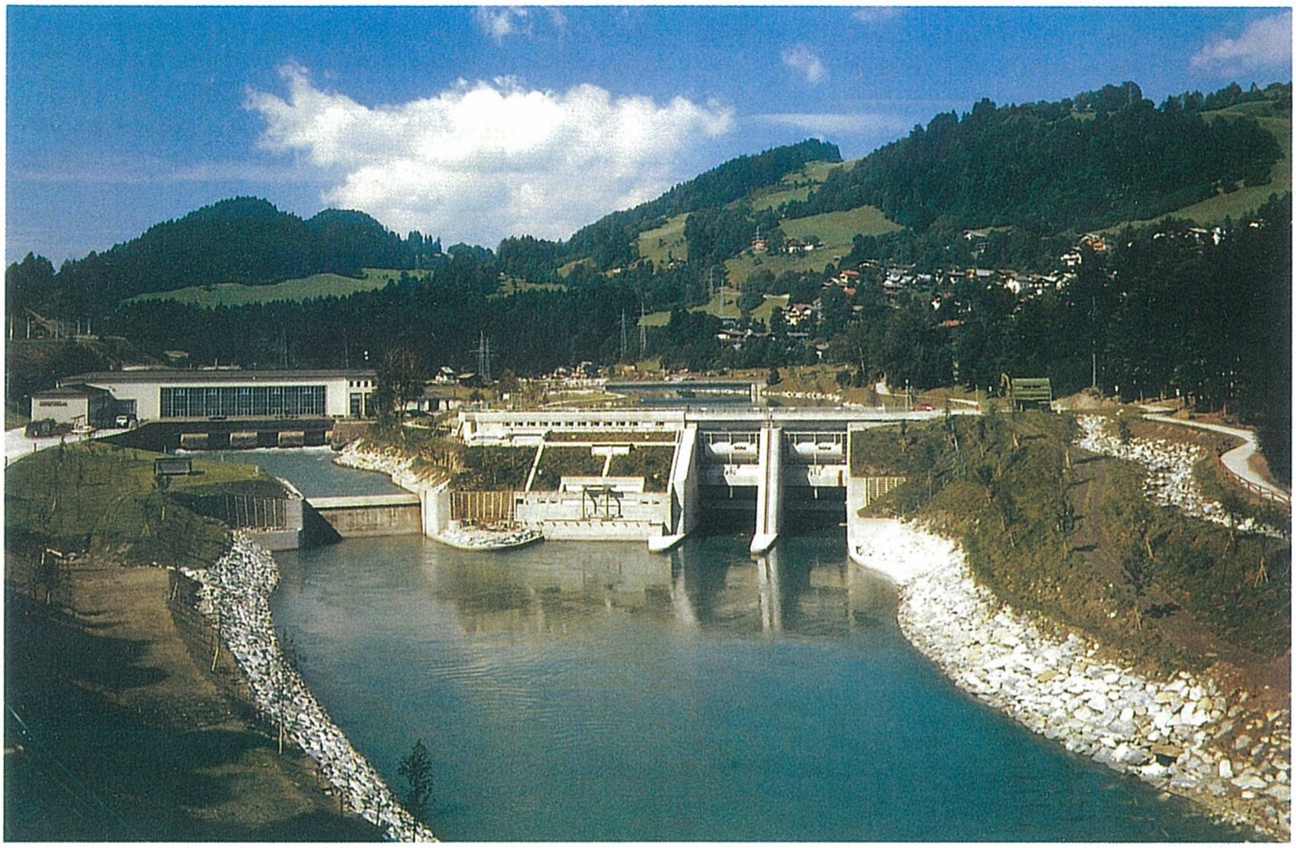
Schwarzach, Wallnerau and St. Veit were constructed and are owned by Tauernkraftwerke AG (TKW), Salzburg, Construction of the other schemes on the Middle Salzach is jointly undertaken by TKW and SAFE (Salzburger AG für Elektrizitätswirtschaft, Salzburg), whereas Urstein and Hallein were constructed by SAFE alone.

At the time of completion of St. Johann (1990/91) the then 8 power stations, Schwarzach to Urstein will produce 234 MW and 1 015 GWh/a.

Salzach

Power station		Schwarzach	Wallnerau		St. Veit	St. Johann	Urreiting	Bischofs-hofen	Hallein	Urstein
			tailrace	Salzach						
Owner		TKW	TKW		TKW	SAFE/TKW	SAFE/TKW	SAFE/TKW	SAFE	SAFE
Operation since		1958	1990		1989	1991	1986	1985	1987	1971
Stationing	km	154/135	134.8		131.7	128.6	124	120	80	75
Storage level	m	738.0	588.5	598.0	582.0	570.6	559.2	547.8	440.7	434.0
Q_{mean} Flow Q_{max} Q_{rated}	m³/s m³/s m³/s	60 1 100 90/110	46.4 – 105	41.2 1 134 95	89 1 150 183	97 1 255 190	103 1 320 186	106 1 356 202	154 2 200 220	174 2 730 250
Head H_{mean}	m	132	6.1	15.4	10.8	10.4	10.8	9.7	6.7	8.9
Capacity (MC)	MW	120	5.1	12.0	16.5	16.5	16	16	12	20
Energy (AAE)	GWh	480	19.3	43.2	71	75	80	73	66	107
Layout										
Spillway/Weir		Högmoos								
Bays width	m	3x10	–	2x10	3x10	3x10	3x10	3x10	4x25	3x16
Pier width/height	m	3.5/15.0	–	3/25.3	3.0/20.5	3.0/20.9	3.0/21.7	3.0/21.7	1.5/18.6	4.5/21.0
Gates		tainter + flap	–	flap concr. beam tainter	tainter + flap	tainter + flap	tainter + flap	tainter + flap	flap	tainter + flap
Power conduit	km	tunnel 17	–	–	–	–	–	–	–	–
Powerhouse		Schwarzach								
Construction, type		high	submerged	low	low	low	low	low	low	low
max height	m	25.0	15.4	28.0	27.3	27.3	27.3	27.3	29.0	26.0
Turbines, number and type		4 Francis ↓	3 S-Turb.	2 Kaplan ↘	2 Kaplan →	2 Kaplan →	2 Kaplan →	2 Kaplan →	2 Kaplan →	2 Kaplan ↘
Backwater area										
Length	km	1.5	0.2	1.4	3.1	3.5	4.5	4.7	2.4	5.3

WALLNERAU



River	Salzach, at km 134.8	Operating since	1990
Nearest town	Schwarzach, Salzburg	Purpose	Hydropower Flood protection
Owner	TKW Tauernkraftwerke AG A-5020 Salzburg, Rainerstraße 29		

MAIN CONSTRUCTION PARTS AND FUNCTION

Two independent power stations at the point where the tailrace canal from the Schwarzach power station discharges into the Salzach provide the link-up of the Middle Salzach development to the Schwarzach power station.

DOUBLE POWER STATION

a) at the Salzach: Weir with openings of 10 m in width each,
Power station with 2 bulb turbines, 12 MW in total

b) Schwarzach tailrace: Power station with 3 S-turbines 5.1 MW in total

HYDROLOGY

	a)	b)
Catchment Area	2 161 km²	—
Q mean	41 m³/s	46 m³/s
Flow Q max	1 134 m³/s	—
Q rated	95 m³/s	105 m³/s
Storage Level (a.s.l.)	598 m	588.5 m
Mean Head	15.4 m	6.1 m
Capacity	12 MW	5.1 MW
	17.1 MW in total	
Energy Output	43 GWh/a	19 GWh/a
	62 GWh in total	

GEOLOGY

At the barrage, the rocky outcrop of the greywacke zone (graphitic schist) slopes from the right-hand wall (17 m) towards the centre of the valley (50 m). The layers topping the primary rocks consist of a displaced base moraine, strata of fine sand with varying rates of silt, and sandy/pebbly sediment. The soil-mechanical parameters for the subsoil were identified by in-situ and lab tests. The backwater has a natural high terrace as its left-hand flank. The primary rocks crop out to the surface at places on both banks. The rock is normally covered by densely packed layers of little imperviousness which offer excellent cut-off conditions for the backwater area (diaphragm at the right-hand bank).

HISTORY AND DEVELOPMENT OF DESIGN

The St. Veit power station, originally planned to serve as the first stage of the development, was extended by an upstream stage at Wallnerau after it was found that the additional overdeepening to be provided at the tailwater of the Schwarzach power plant proved difficult to implement both from the engineering and economic points of view. Several options for locating the barrage at Wallnerau were examined. Damming up the tailrace canal of the Schwarzach power plant would have raised the phreatic surface and would have been detrimental to the powerhouse area, so that it was decided to divide the retention water level for the Wallnerau power station into 588.5 m at the tailrace canal and 598.0 m in the Salzach bed. This rather uncommon solution meant lower land requirements, alleviated hydraulic, geological and engineering problems and at the same time improved efficiency.

FOUNDATION

The barrage and the two power stations have the usual surface foundations on gravel and sand. In sections characterized by unstable layers the soil had to be replaced. A subterraneous curtain reaching down to 15 m below the foundation bed serves as an apron abuting the cut offs on both banks at the headrace of the weir and the two power stations.

INSTRUMENTATION

Standard geodetic monitoring of all parts. Piezometers downstream of the two weirs and at the contact surfaces of the power stations above and below the apron.

CONCRETE

A total of 12 different types of concrete (ready-mix concrete) of varying proportions of the five aggregates (up to 63 mm) and admixtures were prepared. The cement used was a special type "Salzach" FAS 30, with 30% flyash (230 –350 kg/m^3) and Portland cement 375 (340–380 kg/m^3). Compression strength after 28 days varied between 28.0 N/mm^2 and 62.0 N/mm^2, depending on the quality of the concrete.

FLOOD RELIEF WORK

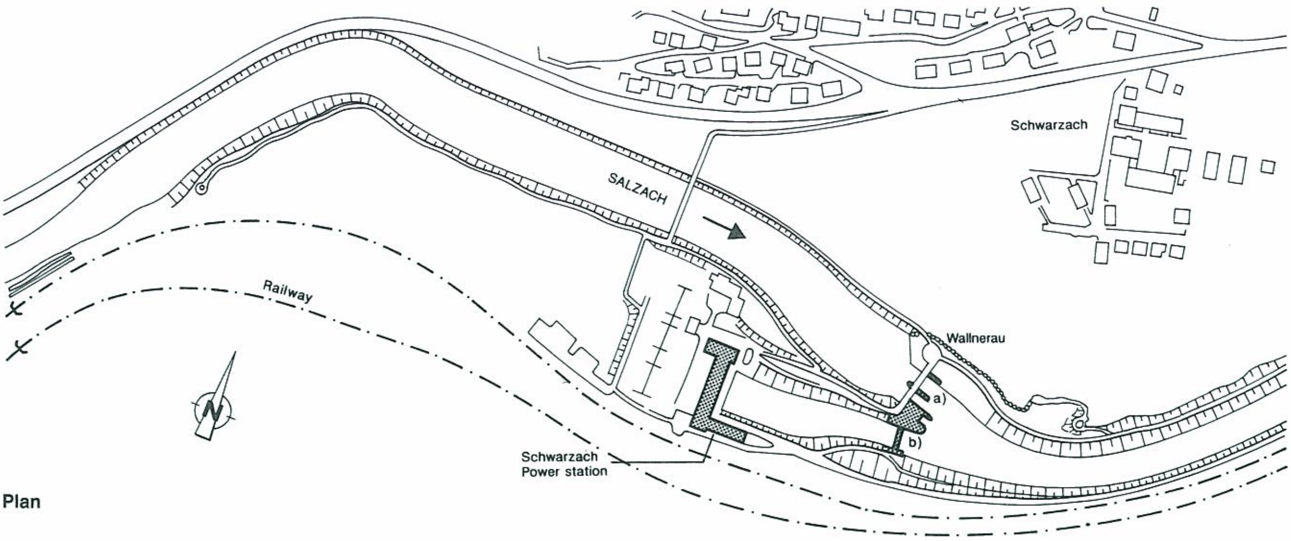
Weir: 2 openings of 10 m in width each
lower gate: tainter gate
stoplog (concrete): 5.5 m in height
upper gate: flap gates
} Height: 13.9 m in total

HQ 100 can be discharged even if one of the four gates is blocked. The discharge is 1 134 m^3/s when all gates are opened. In this case the stilling basin receives 56 m^3/s.m.

POWER STATIONS

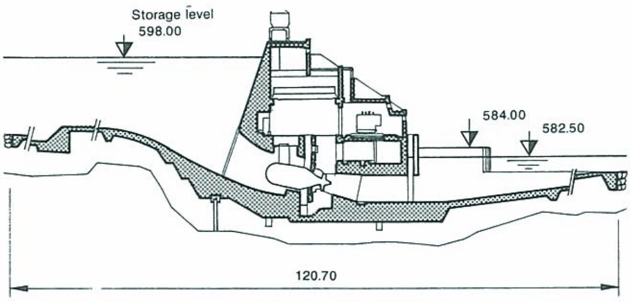
a) At the Salzach: two Kaplan bulb turbines (axis inclined by 10°) of 47.5 m^3/s and 6 MW capacity, runner diameter: 2.70 m; 2 three-phase generators of 8 500 kVA rated output each; turbine axis at elevation 578.7 m.

b) At the Schwarzach tailrace: 3 S-turbines of 35 m^3/s and 1.7 MW capacity each, runner diameter: 2.2 m, 3 gear-coupled generators of 2 000 kVA rated output each; turbine axis at elevation 581.0 m. The powerhouse can be submerged by the water from the Schwarzach power station.

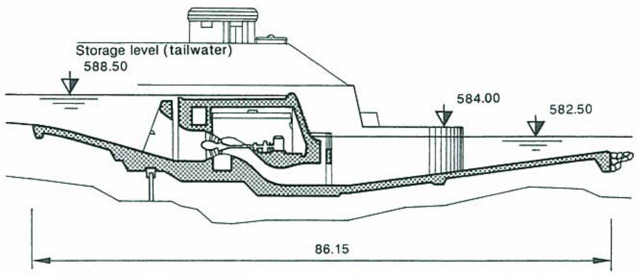


Plan

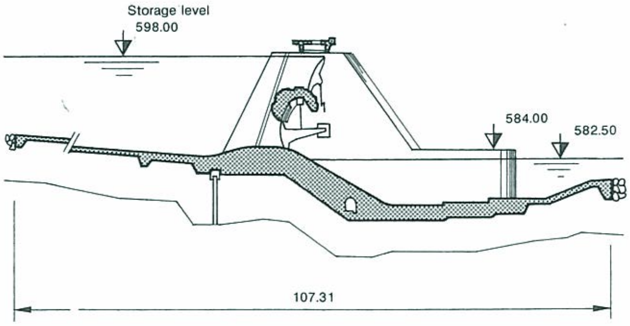
a) Powerhouse section, Salzach



b) Powerhouse section, Schwarzach tailwater



a) Weir section



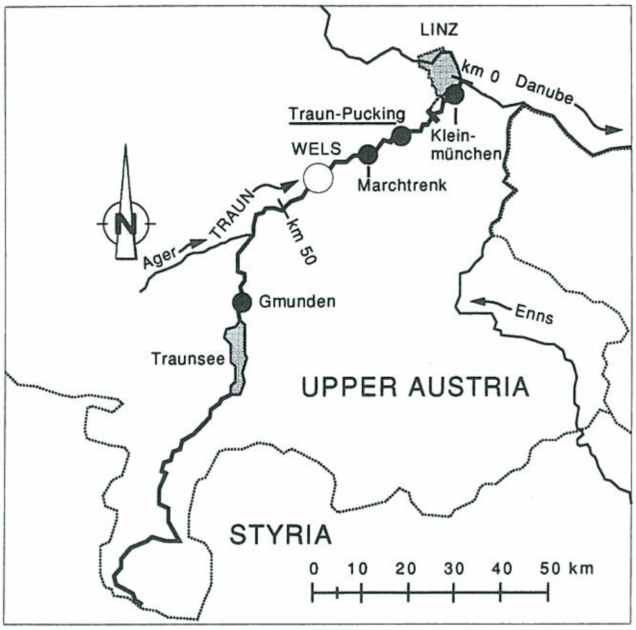
MAIN DIMENSIONS

	Weir	Power station a)	Power station b)
Height	27.3 m	28.0 m	15.4 m
Width	29 m	34 m	31 m
Length	48 m	35 m	29 m

VOLUMES

Excavation	
Barrage	165 000 m³
Deepening	230 000 m³
Embankment	150 000 m³
Concrete	37 000 m³

TRAUN



Rising in the limestone and dolomite mountain region of Aussee and Dachstein in the Salzkammergut the river Traun flows through the Hallstätter and the Traunsee lakes. Leaving the Traunsee at Gmunden it is joined by the Ager at Lambach, the Ager being a tributary with severe quality problems. The Traun then flows through the lowlands near Wels and discharges in the Danube right below Linz. At Gmunden, where it leaves the Traunsee the catchment area 1 490km^2 and mean flow is 72 m^3/s. 75 km further downstream, at the river's junction with the Danube, the catchment is 4 300 km^2 and mean flow is 138 m^3/s. The lakes in the upper reach of the river and its tributaries, numbering more than 50 and totalling 124 km^2 in surface area, have an equalizing effect on the flow regime and help to retain floods.

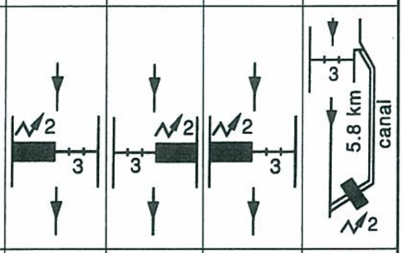
In the past, the Traun was used as a waterway, especially for the transport of salt coming from the salt mines in the Salzkammergut region. At the falls, a bypass canal was constructed for navigation as early as 1552. Water power was used a long time ago for small industrial mills (paper, wood, etc.). A 32 km long canal connecting the Traun at Wels with the Danube was constructed in the Middle Ages.

First small hydro stations were built near Steyrermühl, Laakirchen and Lambach. All of them had relatively low heads and rated discharges and hence limited capacities. Subsequent reconstruction rendered possible the installation of larger power units. The Traunleiten power station, owned by the Wels electricity supply company, was substantially enlarged as recently as 1970. Between 1969 and 1982 major run-of-river stations were commissioned on the Traun. These are Gmunden (Traunfall, however of only 8.8 MW capacity), Marchtrenk and Traun-Pucking constructed by Oberösterreichische Kraftwerke AG (OKA), Linz, and further downstream Kleinmünchen by the second provincial company of Upper Austria, ESG, Linz. Further hydrostations – partly replacing existing small plants – are planned to be built in accordance with a masterplan that has been prepared for the river Traun by the OKA.

An interesting fact from the engineering point of view is the installation, for the first time (1968) in Austria, of two bulb turbines in an inclined position at the Gmunden powerstation. The recent stations in general have 3-bay spillway structures and adjacent powerhouses. It is only at the most downstream station, Kleinmünchen, that an open headrace canal, some 5.8 km in length had to be constructed.

Impounding by the three lower power stations has greatly improved the groundwater conditions in the Welser Heide plain and in the whole area extending between Wels and Linz, where unacceptable riverbed degradation had occurred. The degradation was as much as 15 cm per year (max.) and had a total amount of 7 m (!) at the city of Wels in this century (1900–1980). Severe groundwater lowering was stopped and it became possible to stabilize or

Traun

Power station		Gmunden	Marchtrenk	Traun-Pucking	Klein-münchen
Owner		OKA	OKA	OKA	ESG
Operation since		1968	1979	1982	1978
Stationing	km	72	24	14	8/2
Storage level	m	422.6	309.5	289.3	262.5
Flow Q_{mean}	m³/s	72	128	128	138
Flow Q_{max}	m³/s	1 300	2 300	2 300	2 300
Flow Q_{rated}	m³/s	150	244	206	124
Head H_{mean}	m	9.8	19.7	25.5	10.1
Capacity (MC)	MW	12	43	46	11
Energy (AAE)	GWh	48	181	222	80
Layout					
Spillway/Weir					St. Martin
Bays width	m	3×22.5	3×13	3×13	3×18
Pier width/height	m	2.0/19.5	4.8/36.0	5.0/44.8	4.0/12.0
Gates		flap	tainter + flap	tainter + flap	lifting gate
Power conduit	km	–	–	–	canal 5.8
Powerhouse					Linz
Construction, type		low	medium	medium	high
max height	m	23.5	38.5	41.3	26.0
Turbines, number and type		2 Kaplan ↘	2 Kaplan ↓	2 Kaplan ↓	2 Kaplan →
Backwater area					
Length	km	Traunsee 14	8	10	6

even increase the phreatic level. In addition development of this reach of the river Traun gave the decisive impetus for tackling the severe wastewater problems mainly caused by the paper- and cellulose- industries on the Traun, Krems and Ager. The development of the Traun demonstrates that run-of-river powerprojects may very well fulfill several functions at the same time and can solve ecological problems in a perfect way.

Electricity generation at the Traun in the four power stations at Gmunden, Marchtrenk, Pucking and Kleinmünchen is 531 GWh/a with a capacity of 112 MW. Addition of the two stations of slightly less than 10 MW capacity, Traunfall and Traunleiten (Wels) would bring this to 659 GWh/a and 130 MW. Including the small industrial power stations the river Traun produces more than 700 GWh/a. This will substantially be increased to about 1 060 GWh/a and 260 MW, when the masterplan is realized. Construction of the Lambach and Saag power stations is planned to be started during the next years.

TRAUN-PUCKING

River	Traun, at km 13.8	Operating since	1982
Nearest town	Linz, Upper Austria	Purpose	Hydropower Prevention of further degradation of the Traun river bed Stabilization of the phreatic surface
Owner	OKA Oberösterreichische Kraftwerke AG Böhmerwaldstrasse 3 A-4020 Linz		

MAIN CONSTRUCTION PARTS AND FUNCTION

Barrage	Weir: 3 openings of 13 m in width each Power station: 2 turbines, 46 MW in total No navigation lock

Within the scope of the General Traun Development Plan, the hydropower station at Traun-Pucking was built after completing the upstream Marchtrenk power station in 1979. Traun-Pucking is a combination of two stations originally planned for this stretch of the river. It went into operation in 1982.

HYDROLOGY

Catchment Area	3 647	km^2
Q mean	128	m^3/s
Flow Q max	2 300	m^3/s
Q rated	206	m^3/s
Storage Level (a.s.l.)	298.3	m
Mean Head	25.5	m
Capacity	46	MW
Energy Output	222	GWh/a

GEOLOGY AND ENVIRONMENT

A thick stratum of highly compacted siltstone called "Schlier" is situated at the barrage site underneath the post-glacial river gravel. The main construction parts (weir and powerhouse) are founded on the Schlier whose surface is about 8 m below the original ground surface. The backwater area stretches for about 10 km, bordered by embankments on both sides for a length of 7 km. The dams are continued by a cut reach of the bed to the tailwater of the Marchtrenk station. The tailwater section of Traun-Pucking is shaped by an overdeepening with a maximum drawdown of 9.0 m extending over 4 km to the storage level of Kleinmünchen.

Both sides of the embankments and the draw-down tailwater are provided with draining flumes and rivulets to stabilize the groundwater level. The system ensures that the water level is maintained at some oxbows, seven large and several small ponds, and has restored the ecological balance of the surrounding woodland which had been endangered by the rapid drop in the groundwater level in the last 80 years after the general river training program caused straigthening and rapid drawdown of the riverbed to a depression of up to 7 m.

HISTORY

The Traun river, traditionally long ago an important waterway for Upper Austria because of its geographical situation, hydraulic gradient and relatively high flow, was considered as a potential source for hydropower from an early date.

After World War II, a scheme was developed to build a chain of power stations from its outflow from Lake Traun to its confluence with the Danube. The plan was based on long-term scientific research into the river regime. The economic, biological, geological, botanic and engineering aspects of the research project were collected in a General Traun Development Scheme in 1955–1961, and served as the foundation for actual development although some parameters such as number and dimensions of the power stations had to be amended to take into account recent progress in engineering. The first power station to be realized under the scheme was Gmunden, whose construction was started in 1967 and which went into operation in 1968.

From the 1970s, development has concentrated on the lower stretch of the Traun, from the Ager tributary to the backwater area of the Kleinmünchen power station near Linz. Of the four stations planned, Marchtrenk was built in 1977–1979, and Traun-Pucking in 1980–1983, while two more stages are to be located near Lambach.

FOUNDATION

Thanks to its high compactness, the Schlier at the barrage site is impervious and shows excellent bearing capacity. The weir and powerhouse were nevertheless secured against uplift by enclosed horizontal concrete aprons and cut-off walls. Static safety is ensured for all parts.

INSTRUMENTATION

Geodetic monitoring of the barrage and embankments. Continuous measurement of uplift by piezometres.

Inclinometers and settlement monitors complete the measuring instrumentation.

CONCRETE

Standard procedure

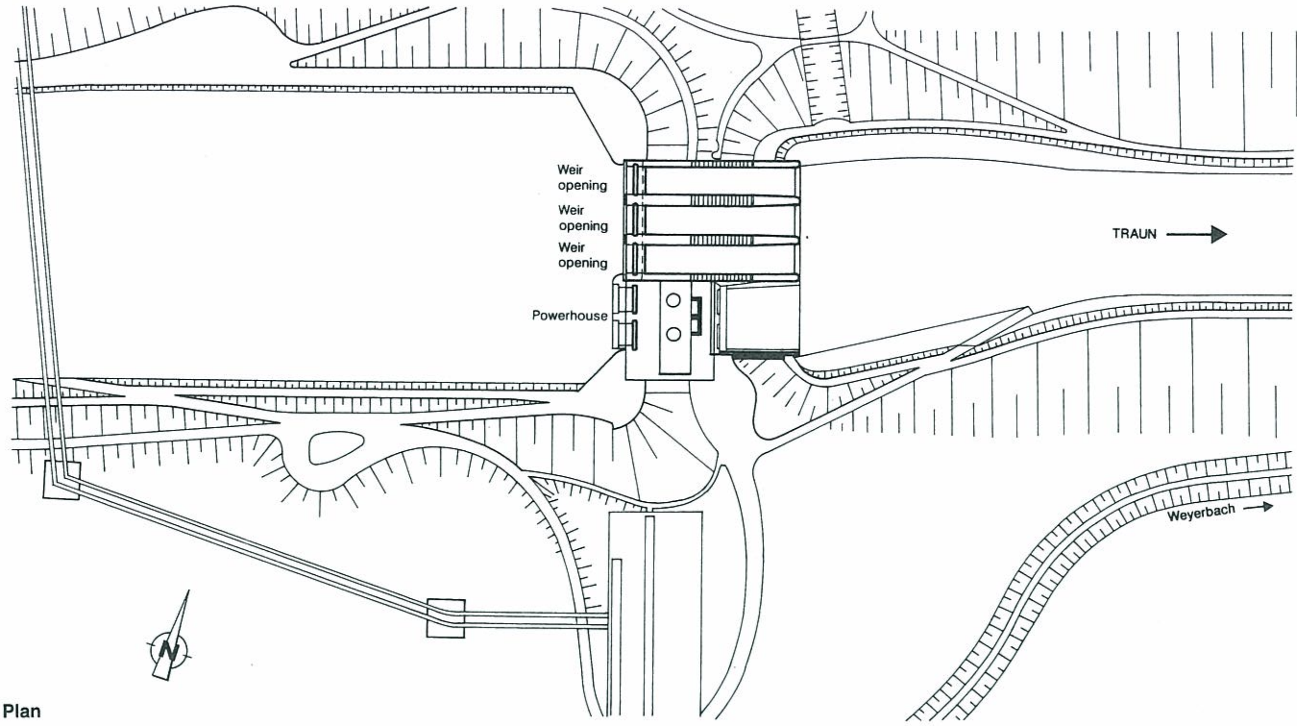
FLOOD RELIEF WORK

The weir has three openings of 13.0 m in width each, closed by tainter gates with upset flaps of 15.5 m in overall height.

All three openings are used to discharge the maximum probable flood (RHHQ) of 2 300 m^3/s, while two openings are sufficient for a 100 year flood. Maximum discharge is 59 m^3/s per metre of width.

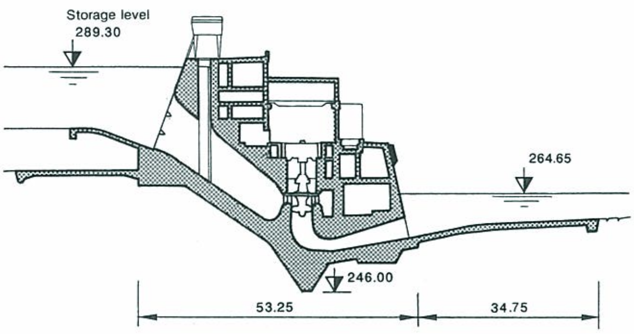
POWERHOUSE

The power station is equipped with two vertical-shaft Kaplan turbines of a rated discharge of 103 m^3/s and a capacity of 23 MW each, directly connected to three-phase generators of 26 kVA and transformers of 32 MVA each. The turbine runners, 4.7 m in diameter, are situated at elevation 262.0 m, i.e. 2.2 m below mean tailwater level.

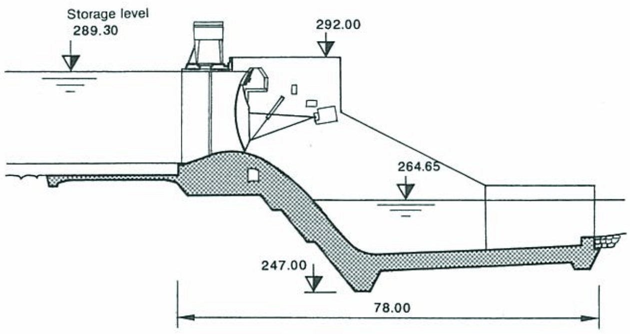


Plan

Powerhouse section



Weir section



MAIN DIMENSIONS

	Weir	Powerhouse
Height	45 m	46 m
Width	90 m	65 m
Length	48 m	55 m
	103 m in total	

VOLUMES

Excavation of Siltstone ("Schlier")	1 460 000 m^3
Excavation of Loose Soil	3 890 000 m^3
Embankment Materials	2 550 000 m^3
Concrete	108 000 m^3
Diaphragm Walls (cut-off)	300 000 m^2
Bituminous Facings of Embankments	173 000 m^2
Riprap and Pitching	490 000 t
Reinforcement Steel	6 400 t

Chapter J

DAM CONSTRUCTION IN AUSTRIA

A CONTRACTOR'S VIEW

CONTENTS

DAM CONSTRUCTION IN AUSTRIA

A CONTRACTOR'S VIEW

By W. Reismann and R. Petter*

Hydro power holds a place of particular importance in Austria. In fact, two out of three kilowatts supplied to the consumer come from this inexhaustible resource. For a long time, this inestimable value was fully appreciated, the more so as the conversion process has no detrimental environmental implications. Recently, however, there has been a change in the public view of hydro power development, and this change in values, although hard to explain by rational arguments, has almost entirely paralysed hydro plant construction.

1 THE IMPORTANCE OF HYDRO POWER DEVELOPMENT IN AUSTRIA TO THE CONSTRUCTION INDUSTRY IN THE COURSE OF THE PAST FOUR DECADES

The utilization on a major scale of the Austrian hydro potential by hydro power stations started around the middle of this century. Generating plant constructed in the four decades between 1950 and 1990 produces about 30 000 GWh p.a., corresponding to 55 or 60% of Austria's maximum developable potential. Out of this, about 70% is accounted for by run-of-river plant and 30% by storage plant. The construction efforts required have been of great importance to the Austrian construction industry. Both technological and economic impetus has come from power plant construction and has had an immensely stimulating effect on the nation's economy as a whole.

Construction process engineering has developed along with the construction of these power projects by the use of ever more refined equipment so as to bring about increasingly radical rationalization effects. These are felt both for the run-of-river stations on our rivers and for the storage developments in the mountains. The two types of power developments, although fundamentally different in terms of construction method, have many features and problems in common with respect to the development of the construction methods.

In the following, the development of construction techniques will be described for the series of power stations on the river Danube as well as for selected construction sites of storage developments. This does not mean, however, that the remaining hydro projects have not contributed their share to this development. Valuable impulses have come from the construction sites on the rivers Enns, Drau, Mur and Inn and from a great number of medium-sized and large storage developments in all parts of Austria.

The run-of-river stations constructed on the Danube between 1950 and 1990 are Ybbs-Persenbeug, Aschach, Wallsee-Mitterkirchen, Ottensheim-Wilhering, Altenwörth, Abwinden-Asten, Melk and Greifenstein.

The large concrete dams dating from this period are those at Limberg, the two Mooserboden dams, the dams at Kops, Schlegeis, Kölnbrein and Zillergründl, the large embankment dams at Gepatsch, Durlassboden, Eberlaste, Innerfragant, Finstertal, Längental and Feistritz.

2 CONSTRUCTION SITE DEVELOPMENT AND INFRASTRUCTURE

The success of construction operations is dependent on a careful development of the construction site and an efficient supply with material and equipment, including the necessary precautions against natural influences as e.g. floods on the rivers and avalanches and mudslides in the mountains.

A common feature of all the construction sites is that during the first one-third of the period under study rail-mounted vehicles were the dominating means of transport. At Ybbs-Persenbeug and Aschach, railways led to the very centres of construction activities, and a railway also led to the main store of the Kaprun high-mountain construction site. In addition, to provide access to the high-level sites of the Kaprun and Reisseck projects, continuous ropeways, shuttle ropeways and inclined hoists were provided to handle the required passenger and material transports. The disadvantage was that, with their limited capacities, these facilities caused the bottlenecks that determined the length of the respective construction period.

During the mid-sixties and onwards, as large construction equipment was developed and road construction techniques were improved, roads of increasing capacity were constructed to provide direct access from the public road network. For the high-level sites, veritable mountain roads with turning bends and often with a great number of tunnels were constructed. Apart from the problems of road alignment in difficult

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terrain, allowance had to be made for avalanche and rockfall risks. The construction of two-lane access roads boosted construction work progress and was a prerequisite for a great number of rationalization measures and the development of tourist trade that followed.

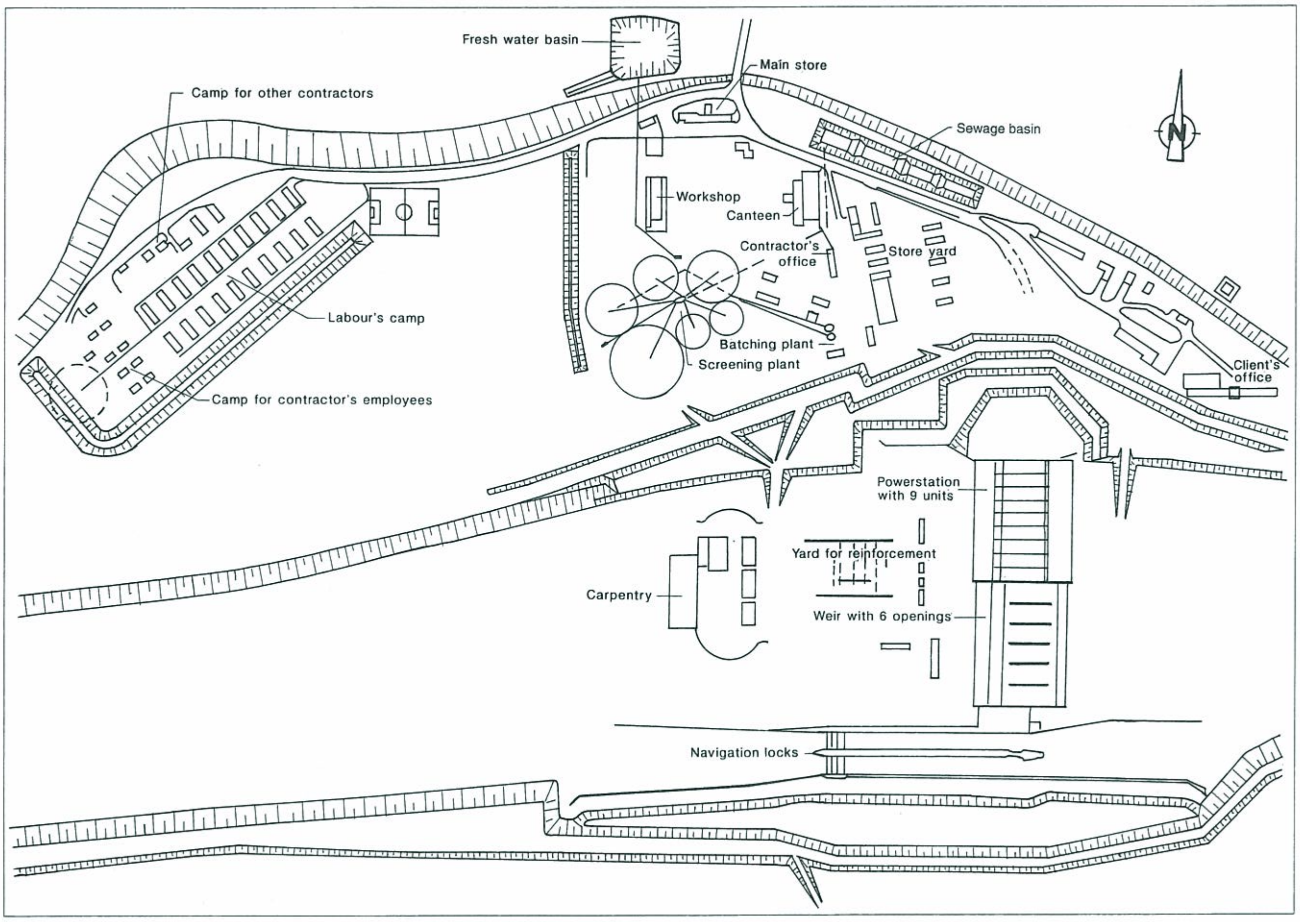
Apart from the access roads, there are other facilities needed to ensure supplies to the sites. They may be summarized under the term infrastructure. This includes electricity and water supply, sewerage, facilities for staff accommodation and catering workshops, stores and offices with their telecommunication equipment.

This is another field where a fundamental change has taken place over the past 40 years. The growing use of construction equipment has allowed increasing cuts in construction activities workshops outside the project site.

The standard of staff accommodation has definitely improved, although the dominating type of building was the wooden hut through almost the whole period. It is only recently that accommodation containers have come into use.

Full use has also been made of the merits of technological development for the other facilities of infrastructure as e.g. telecommunications, and the requirements have been adjusted to the growing necessities. Thus, sewage disposal regulations are becoming increasingly severe, and ground storage makes allowance for environmental aspects to the largest possible extent.

Figure 1 Location plan of site installations for Greifenstein power project on the Danube



number of personnel – from about 3 000 workers at Kaprun to about 500 at the Zillergründl dam, from about 3 000 workers at Ybbs-Persenbeug to about 1 000 at the Melk run-of-river station, and from about 1 000 workers at the Gepatsch embankment to 200 workers at the Finstertal embankment.

In spite of that, motor capacity per man has also increased from 12 kW to about 20 kW. But this does not necessarily mean that any greater expenditures have been required in the field of workshop equipment. Whereas in the earlier days of dam construction it was necessary to provide self-sufficient workshops, it has become increasingly possible to include in the overall

Figs. 1 and 2 are location plans showing site installations at a run-of-river station (Greifenstein) and at a storage project (Kölnbrein), respectively.

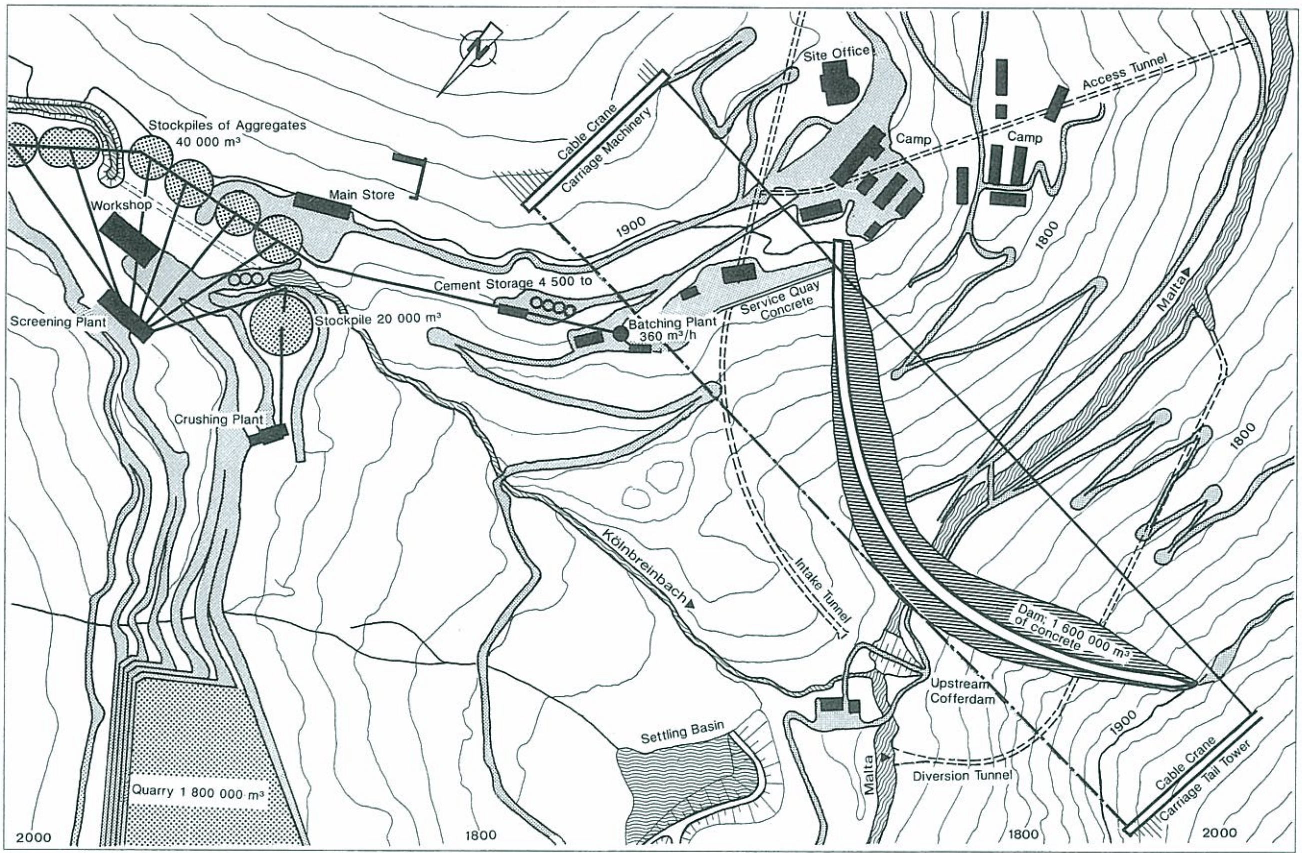
3 COFFERDAMS

This chapter mainly deals with run-of-river stations, although the foundations for large concrete dams may also call for huge excavation volumes. At the Zillergründl concrete dam, stream engineering techniques using bored cut-offs and bored circular cells were applied to stabilize the slopes.

During the first two decades of the period under review, steel sheet piling was practically the only method in use for stabilizing excavation walls. These were provided as required, in the form of simple sheet piling with fills of gravel or rock, wattle work rolls or coarse-gravel gabions. For the larger flow velocities, double-walled cofferdams made of sheet piling with coarse gravel fill, walings and anchors were used. Where very large flow velocities were present, as e.g. in the centre excavations of the first power projects on the Danube, circular cell enclosures were needed. 12-m diameter cylinders filled with coarse gravel were sunk around the excavation (Fig. 3).

Driving was at first carried out with clumsy tubular scaffold equipment mounted on rails, which was then gradually replaced by cable-operated crawler-type excavators. For the pile hammers, development led from steam operation to hydraulic operation.

Figure 2 Location plan of site installations for Kölnbrein dam project



The driving operations were carried out during low-flow periods from filled-up ground or in some cases from a floating platform. The narrowed river channel represented a constant potential flood risk, whereas on the other hand working across the whole cross section was not possible due to the necessity of maintaining a certain minimum of river flow. Therefore, these operations held little possibility of rationalization.

A new concept, which provides for locating the barrage and power station in a cut across the neck of a river bend in combination with thin diaphragm cut-offs and plastic film, has led to new ways of excavation design. The outstanding feature of this concept is the possibility of constructing the whole project in a single large excavation safe from floods. These substantial advantages for the construction operations in general have also led to cuts in construction time and cost.

Especially on the Danube, where it was once necessary to split up the work into up to four excavations and four construction phases to safeguard the maintenance of navigation, the advantage of carrying out the work in a single excavation is obvious (Fig. 4).

Following completion of the barrage and power station within an excavation that is protected from floods by levees, the river is diverted into its new bed. After flooding the construction pit, dam fill is placed from both banks so as to cut off the original river bend, which now becomes a dead branch.

This concept of dry-pit construction for run-of-river stations implies that earthworks are assuming an ever increasing importance. The development accomplished in this field has greatly contributed to the success of the dry-pit method.

4 EARTHWORKS

Earthworks are an item of particular importance in power station construction. Geologically speaking, the earthworks cover a large variety of ground extending from loose soil to rock. Earthwork operations are needed for the provision of a construction pit, for excavating the foundation for the structure itself as well as for obtaining fill material and concrete aggregates.

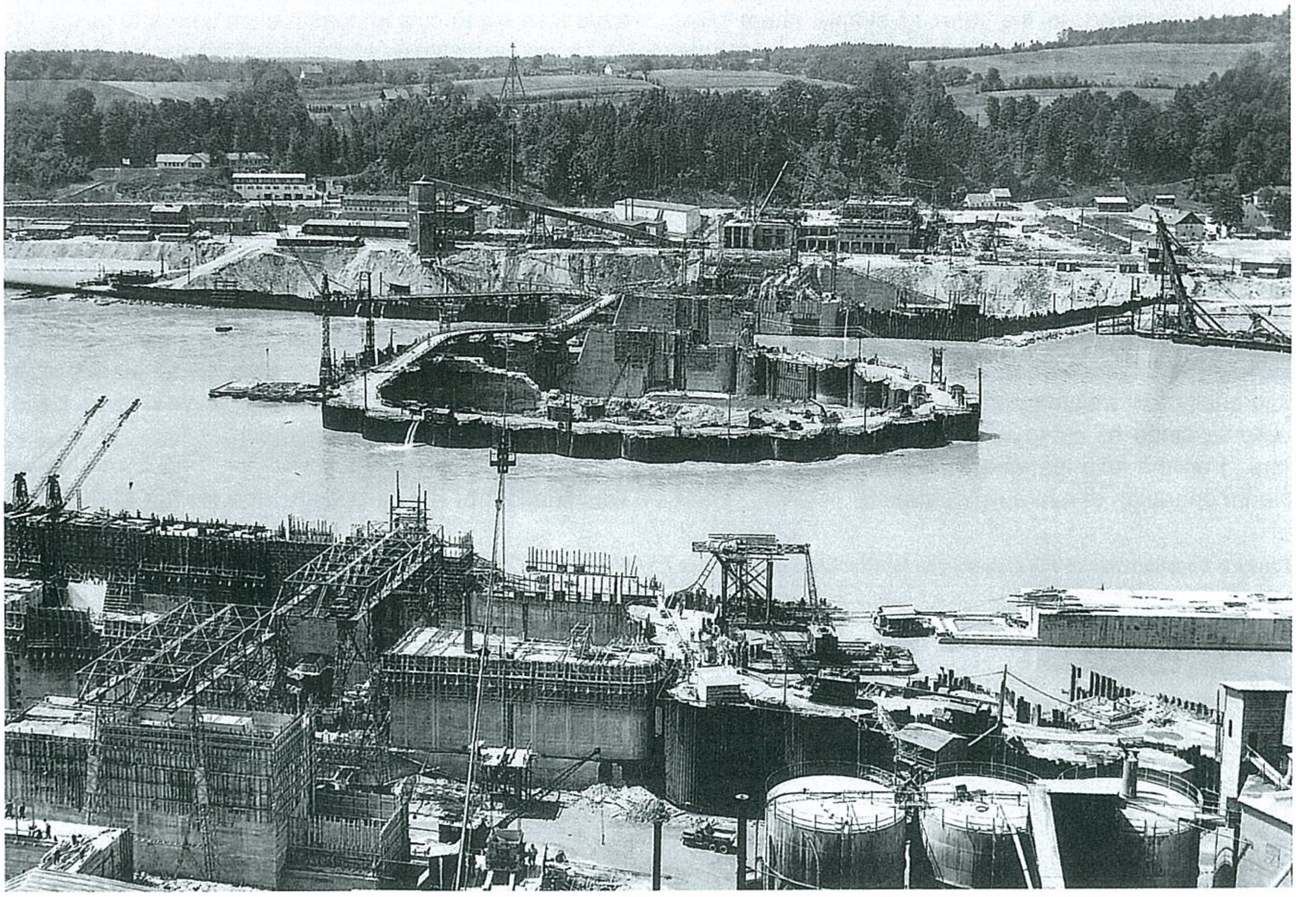


Figure 3 Circular-cell enclosure of excavation for two spillway bays of Ybbs-Persenbeug power station on the Danube, 1956

The following paragraphs give a description of how a number of selected aspects in the large field of earthworks have developed over the past four decades.

4.1 Earthworks in soil

Typical works covered by this item are the stripping of humus, topsoil, sand and gravel at run-of-river station sites and of talus material and gravel from valley bottoms for storage schemes at high-level locations.

At the beginning of the period under study, rail-mounted equipment was still in common use. Thus, for the excavation of the Limberg dam foundation, steam-operated grab excavators were used which loaded field railway wagons pulled to the dumping sites by steam engines (Fig. 5). Then followed the era of cable excavators and haulers. First, at the Mooserboden high-mountain site and at the first run-of-river station sites, diesel-operated cable excavators with bucket capacities of 1 or 2 m³ and hauling units of 12 to 15 t payload were used. In fact, it was not before the early sixties that with the construction of the Gepatsch embankment dam a change took place in the character of construction equipment. The cable-operated excavators, driven by diesel engines or electric motors, became substantially heavier, reaching bucket capacities of up to 3.6 m³, while the haulers had payloads of as much as 25 t. The earthwork equipment used at the Gepatsch dam site totalled 30 000 kW in capacity.

In the years that followed the trend continued to be towards larger capacities, with the hauling units reaching a payload of 35 t at the high-level sites of Kölnbrein and Zillergründl, and of 50 t at the hydro power sites on the Danube.

Another landmark in the history of earthwork machinery was the replacement of cable-operated excavators by hydraulic excavators and wheel loaders. A further step in this direction was finally made by the introduction of self-propelled scrapers with capacities of 24 m³, which were used at Altenwörth on the Danube in 1973.

Table 1 Relationship of equipment cost of loosening, loading, hauling and placement of unconsolidated soil by means of earthwork equipment

Type of equipment	Total cost ratio	Percentage of equipment cost
1.7-m³ cable-operated excavator 38-t off-road hauler 103-kW bulldozer	1.00	70
4.5-m³ cable-operated excavator 56-t off-road hauler 126-kW bulldozer	0.78	80
5.4-m³ wheel-loader 56-t off-road hauler 126-kW bulldozer	0.63	80
24.5-m³ self-propelled scraper	0.40	86

For reasons of economy, the use of cable-operated excavators has since been limited to dredging with bucket equipment. For all other uses, the much faster hydraulic excavators or wheel loaders are superior in economy, whereas the use of scrapers depends on whether the material to be worked lends itself to this method and whether the areas to be worked are large enough to be economical.

Table 1 is a list of several groups of equipment which shows the cost aspect of construction machinery in earthworks and in particular demonstrates the substantial magnitude of this item of capital expenditure. In this way, continuity in the use of such equipment becomes a crucial problem in construction management. Between 1965 and 1985, this continuity was ensured by an uninterrupted series of power plant construction sites on the river Danube. These were Wallsee-Mitterkirchen, Ottensheim, Altenwörth, Abwinden-Asten, Melk and Greifenstein.

Earth moved within this period amounted to a volume of about 100 million m³, with about 50 earthmoving machines working at a time, which corresponded to a present value as new of about 200 million Austrian Schilling. With the service life of a machine being 20 000 operating hours, each machine reached some 10 000 operating hours at each site and thus had to be replaced twice during this 20-year period. This continuity, which has yielded optimal results, has for the time being been brought to a sudden end by the ban on the Hainburg project.

Conditions to be faced by earthwork at high-level dam sites are much less favourable. Material to be moved in the valley bottom usually contains stones and boulders. Excavation dewatering and slope stability present further difficulties.

Overburden stripping on the flanks of dam sites poses problems because of the steepness of the terrain. At Limberg, the greater part of the overburden was stripped by hand. It was only later that hydraulic excavators and crawler pushers were introduced.

As compared with low-level sites, earthwork at mountain sites has been calculated to cost roughly twice as much in the valley bottom and about three times as much on the flanks.

Figure 4 Excavation for Greifenstein power project on the Danube, 1983

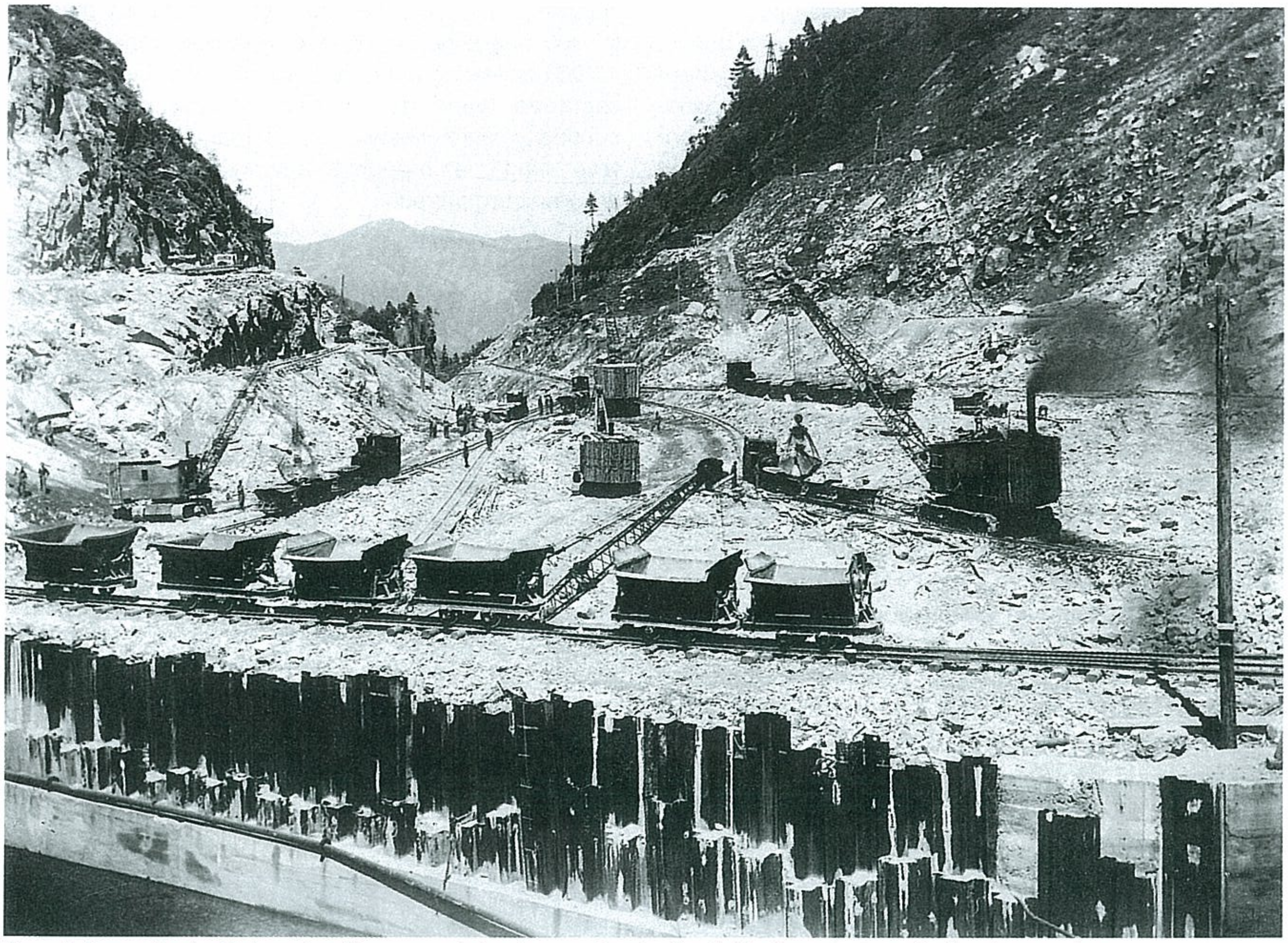


Figure 5 Excavation for Limberg dam. Steam-operated grab excavators loading field railway wagons pulled by steam engines

4.2 Construction in rock

Open cuts in solid rock were required at some river sites in the lowlands for the tie-in or foundation of structures. Rock needing blasting was rarely encountered at the construction sites on the Danube. At Ybbs-Persenbeug and Aschach, shot holes were hand-drilled into the foundation rock within the excavation before blasting.

It was not before Greifenstein that bedrock was encountered in the form of flysch. Work at that site was carried out by 86-t bulldozers more than 500 kW in engine capacity and equipped with ripper teeth. The three bulldozers used there ripped 1.7 million m^3 of flysch, with a maximum monthly performance of 300 000 m^3.

Dam construction in the mountains always involved the cutting of rock for the foundations and the quarrying of materials for embankment construction and aggregate production. This has also led to a certain continuity in the development of construction methods. Whereas at the dam sites of the Kaprun development shot holes were largely hand drilled, crawler mounted wagon drills equipped with heavy air hammers were already used at the Gepatsch quarry in order to drill the shot holes for excavating the 20-m high benches by auger mining methods. The same method was used for the excavation of the foundations for the dam projects which followed.

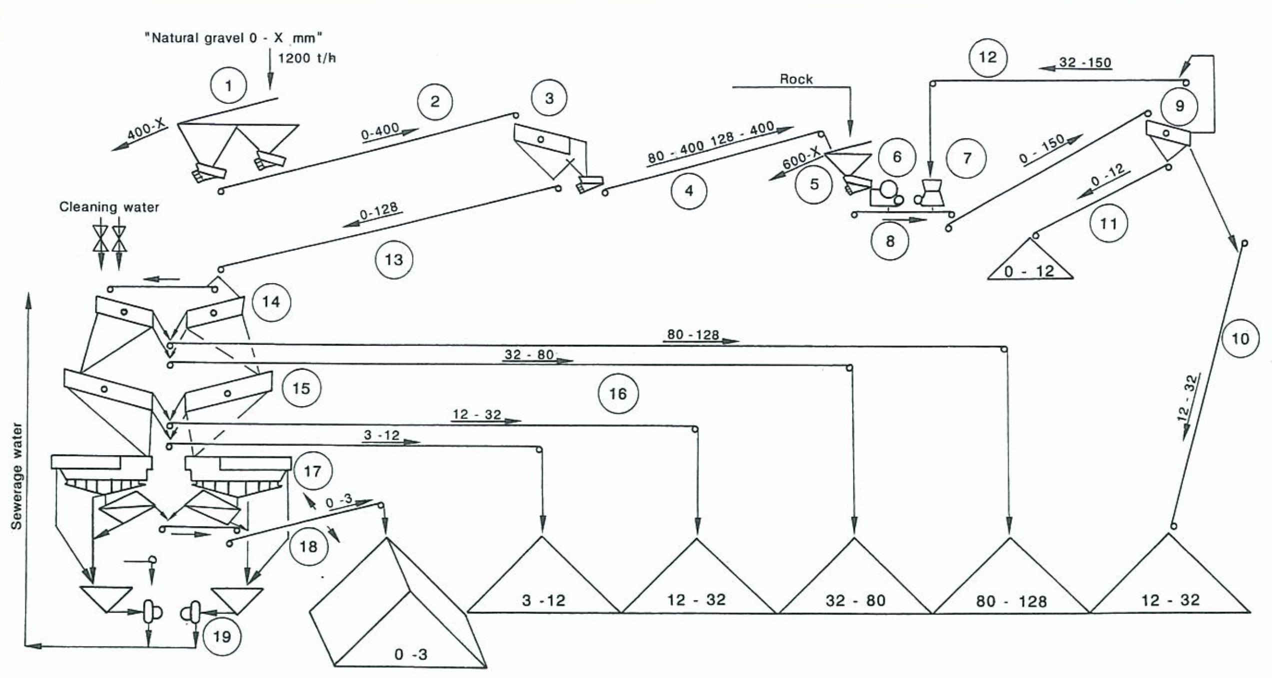
For the first time at Kölnbrein, the smooth blasting and presplitting methods were used by sinking parallel perforation drillings for the excavation line in the flanks. In this way blasting damage to the foundation rock was minimized, so that less treatment of the rock surface was needed.

An important cost factor in foundation excavation is the removal of shot rock. As access to the higher levels of rock flanks is hardly possible, vehicles must be loaded near the valley bottom, which calls for large amounts of pushing work. The crawler pushers used at Schlegeis were 250-kW CAT D8 machines. At Kölnbrein, 340-kW CAT D9 were also used, while at Zillergründl CAT D10 machines of 52 kW capacity were employed.

Another important landmark in the development of construction in rock was the use of hydraulic drilling equipment, which allowed the drilling performance of the individual machines to be doubled. Such equipment was first used in 1978 in the quarry for the Finstertal embankment and in 1982 for the Zillergründl arch dam.

4.3 Equipment used in earthworks and rock excavation

The effects of rationalization in earthworks and rock excavation in the course of time is best demonstrated by relating equipment capacity, the product of rated engine capacity and number of operating hours, to the volume of excavation work accomplished. Mean values of kWh/m^3 for several construction sites and types of work are listed below:



1 Screening feeder   2 Conveyor belt   3 Screening machine   4 Conveyor belt   5 Screening feeder   6 Crusher   7 Cone crusher   8 Conveyor belt

9 Screening machine   10-13 Conveyor belt   14-15 Screening machine   16 Conveyor belt   17 Sand classifier   18 Conveyor belt   19 Sewerage waterpump

Figure 6 Material flow chart of preparation plant for moraine gravels with a feed capacity of 1000 t per hour at Zillergründl construction site

Rationalization effect in earthworks:

Wallsee-Mitterkirchen run-of-river station on the Danube: for about 10 million m³ of excavation, 18.8 kWh/m³ on average

Greifenstein run-of-river station on the Danube: for about 15.7 million m³ of excavation, 8.0 kWh/m³ on average

Rationalization effect in rock excavation:

Gepatsch quarry for fill material: for about 4.6 million m³ of rock, 23.0 kWh/m³

Kölnbrein quarry for aggregates: for about 1.6 million m³ of rock, 18.0 kWh/m³

Finstertal quarry for fill material: for about 3.2 million m³ of rock, 10.0 kWh/m³

Rationalization effect in pushing rock from flanks:

Kölnbrein	12.5 kWh/m³
Zillergründl	7.0 kWh/m³

Rationalization effect in rock drilling:

Kölnbrein	10.8 kWh/m³
Zillergründl	2.5 kWh/m³

The above figures show on the one hand that mountain sites require the larger equipment capacities, and on the other hand that the technological development that has taken place is reflected by the decreasing values of kWh/m³.

5 MATERIAL PREPARATION TECHNOLOGY

Whereas preparation plants for the impervious cores of embankment dams were used only at Gepatsch, Bolgenach and Durlassboden, aggregate preparation equipment was provided at all the concrete dams and at the large run-of-river stations on the Danube, so that we can look back on a 40-year period of development in this field.

In the course of the decades, this development has mainly been influenced by concrete technology aiming at minimizing cement contents in order to reduce heat build-up and save costs. This has been accomplished by an increased accuracy in separating the size fractions, screening out fine particles smaller than 0.06 mm, washing the coarse fractions and by exact weighing and mixing in the fabrication of concrete.

A novelty introduced in gravel preparation in 1951 at Mooserboden was sand separation at 1 mm by decanting. 30 years later, at the Zillergründl arch dam, use of a sand processing plant allowed the production of sand of any desired grading. Exact separation of size fractions was accomplished by increasingly sophisticated vibratory screens and by spraying the screening decks. The large water requirements of 2 m³ per ton of aggregate called for the provision of large pumping and water supply facilities (Fig. 6).

Since 1989, for the construction of the supporting structure for the Kölnbrein arch dam, water clarifiers have for the first time been used for purifying waste water from gravel preparation. Apart from facilitating the decanting process in a settling basin, this method recovers about 90% of the service water and limits the supply of raw water to 10%.

Since the construction of the Kölnbrein dam, handling of gravel between the preparation plant and the concrete mixing plant has been converted to belt-conveyor operation both on the Danube and at concrete dam sites, with the gravel being drawn from the stock pile heap into conveyor belt galleries placed underneath. This has superseded the cableway installations used at Kaprun or road haulage as practised at Schlegeis.

For concrete production, mixing towers with hoppers and gravity mixers were used at the construction sites of both run-of-river stations and storage schemes. It was only at Limberg, up to 1950, that continuous-flow pug mill mixers with feed-type weighers were used.

Concreting performance was boosted by increasing the size of mixing towers to an output of 360 m³ per hour, as at Kölnbrein and Zillergründl, or by providing two towers, as practised on the Danube. Improvement of concrete quality was mainly achieved by the constant improvement of weighing techniques as well as by providing automatic control and supervision for the plants. An important step in this direction was the introduction of nuclear measurement of the water contained in the sand and the automatic reduction of water addition as was introduced at the Ottensheim dam. This aided in achieving a substantial improvement in the uniformity of concrete consistency. Computer technology has been introduced in the mixing towers for optimizing economy and concrete quality and ensuring complete supervision of concrete quality by checking each individual batch.

Constant development work in the field of concrete technology has accompanied dam construction and has also influenced construction operations. As to the cement, efforts have been directed at reducing heat build-up in the mass concrete. This has obviated the need for providing cooling slots, which had necessitated a second concrete pouring operation at Kaprun, and it has become possible to pour larger blocks with higher lifts of up to 3 m.

At first, blast-furnace slags were ground together with the cement (as e.g. at Schlegeis) until, at Kölnbrein, flyash was added for the first time. To ensure the appropriate mixing of cement and flyash, special blending plants were needed at Kölnbrein and Zillergründl. Recently, the flyash has been added already at the cement works, which now supplies the flyash cement as a finished product.

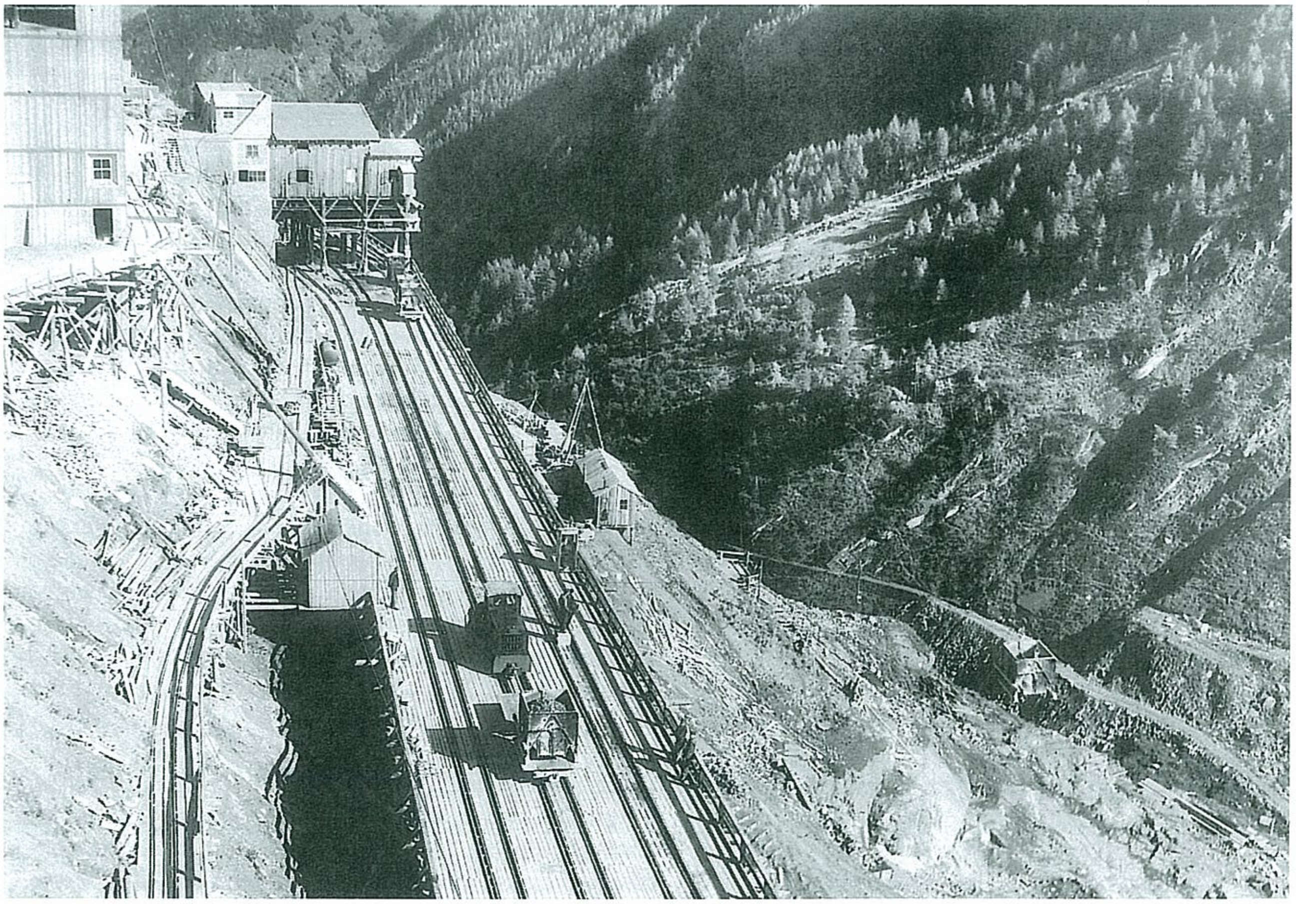
At Zillergründl, an ice production plant was used for the first time. This method uses flake ice instead of water for concrete production in order to reduce the placing temperature. The maximum temperature of the mass concrete of the Zillergründl dam was thus reduced to 35 °C and that of the concrete for the Kölnbrein supporting structure even to 29 °C.

6 CONCRETE CONSTRUCTION

6.1 Concrete placement

The treatment of mass concrete holds a prominent place in the construction techniques for both large concrete dams and large barrages. This is why great

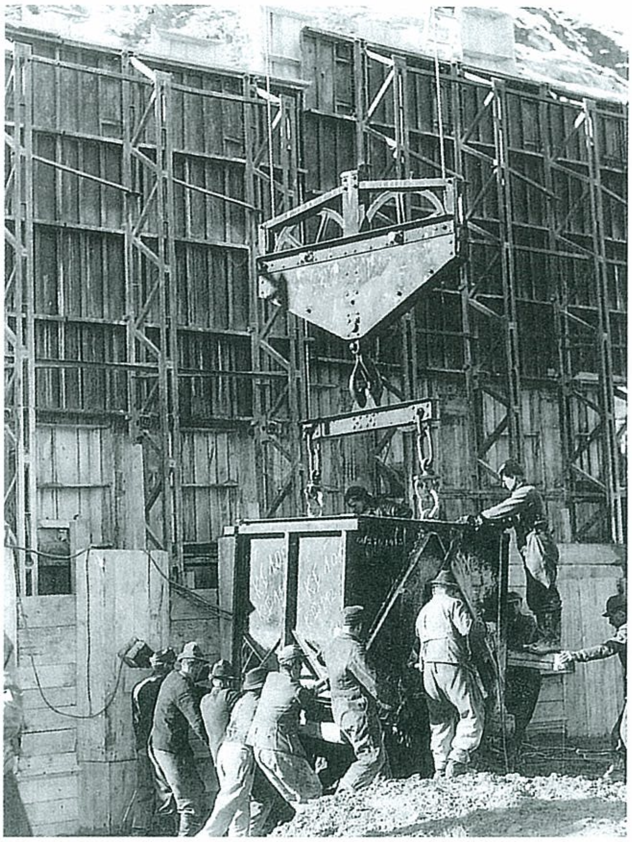
Figure 7 Transport of freshly-mixed concrete for Limberg dam in 1948. Diesel engines pulled two dollies each on three tracks from the three continuous mixers to the three 9-t capacity blondins



efforts have always been spent on the development of the construction techniques and the corresponding machinery in this particular field.

The concrete placement operations at high-level sites are mainly determined by the fact that the working season is limited to six months a year. This renders any increase in pouring performance twice as important, so that it was at a relatively early stage of development – in 1947 at Limberg – that blondins were used for concrete pouring. Buckets holding 3 m^3 of freshly mixed concrete were transported on dollies drawn by diesel engines from the mixing tower to the blondins, where they had to be hung onto the crane by hand (Fig. 7).

Figure 8 Manpower-intensive concrete placement at Limberg dam in 1948



At the Mooser dam, this time-consuming manipulation of the buckets was replaced by the concrete being directly filled into the buckets by diesel dumpers.

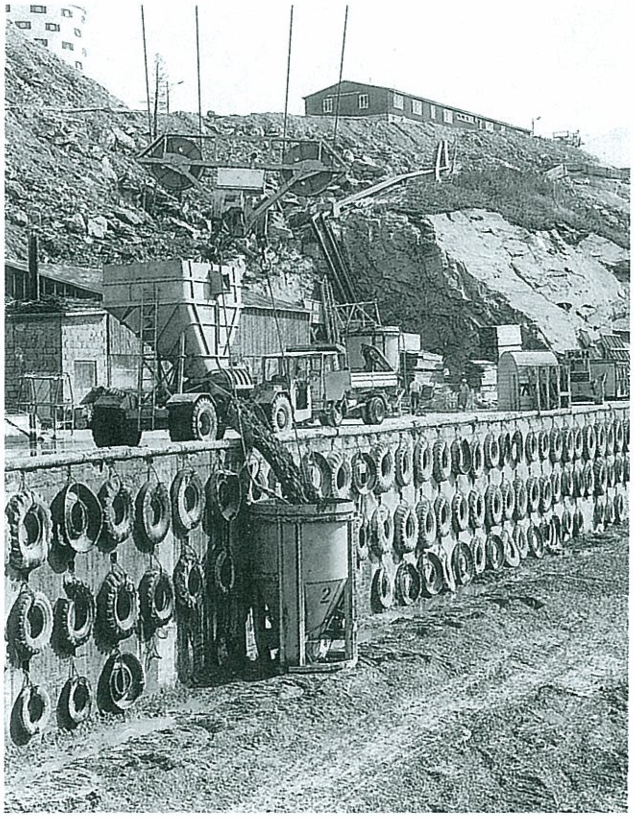
A particularly critical operation was the manual opening of the buckets. The "bucket jumpers" on the blocks of the dams of the Kaprun development were quite famous, but their job was extremely hazardous (Fig. 8).

At Schlegeis, it was already 6 m^3 of concrete by crane cycle that was transported on two parallel craneways, the buckets being already opened hydraulically with the help of a pull rope. Rail-mounted bulk transporters pulled by electric locomotives filled the concrete into the buckets on the concrete quay. Although this represented a considerable improvement, the rail-mounted system still lacked flexibility.

The latest development in this field took place with the introduction at Kölnbrein of the trackless concrete quay with centre pivot steered diesel-driven hopper lorries. The capacity of the hopper lorries and pouring buckets had meanwhile increased to 9 m^3 (Fig. 9). The modern 26-t capacity blondins also allowed higher driving speeds, so that a substantial increase in pouring rates was accomplished. The maximum daily performance at Kölnbrein was 7 200 m^3 of concrete placed by two blondins (Fig. 10).

The high placement rates called for high compaction rates. While at Kaprun the concrete had been spread and compacted mainly by hand, crawlers were used at Schlegeis for concrete spreading. Concrete compaction at Schlegeis was accomplished by crawlers equipped with four electric heavy-duty immersion vibrators. At Kölnbrein, hydraulic-driven vibrators were used instead of the electric vibrators, so that the particularly troublesome electric cable was no longer needed. At Zillergründl finally, eight hydraulic vibrators per crawler ensured optimal and sufficient concrete compaction.

Figure 9 Transport of ready-mix concrete by 9-m^3 capacity hopper lorries as were used at the Kölnbrein and Zillergründl construction sites



The problem of concrete placement has been entirely different at the large barrages of the run-of-river stations. At the first hydro schemes on the Danube, concrete was placed by pumping. However, this extremely efficient method of concrete placement had to be abandoned for reasons of concrete technology, as the high cement content needed for the pumping led to excessive heat build-up in the mass concrete.

At the Wallsee-Mitterkirchen power station on the Dan-

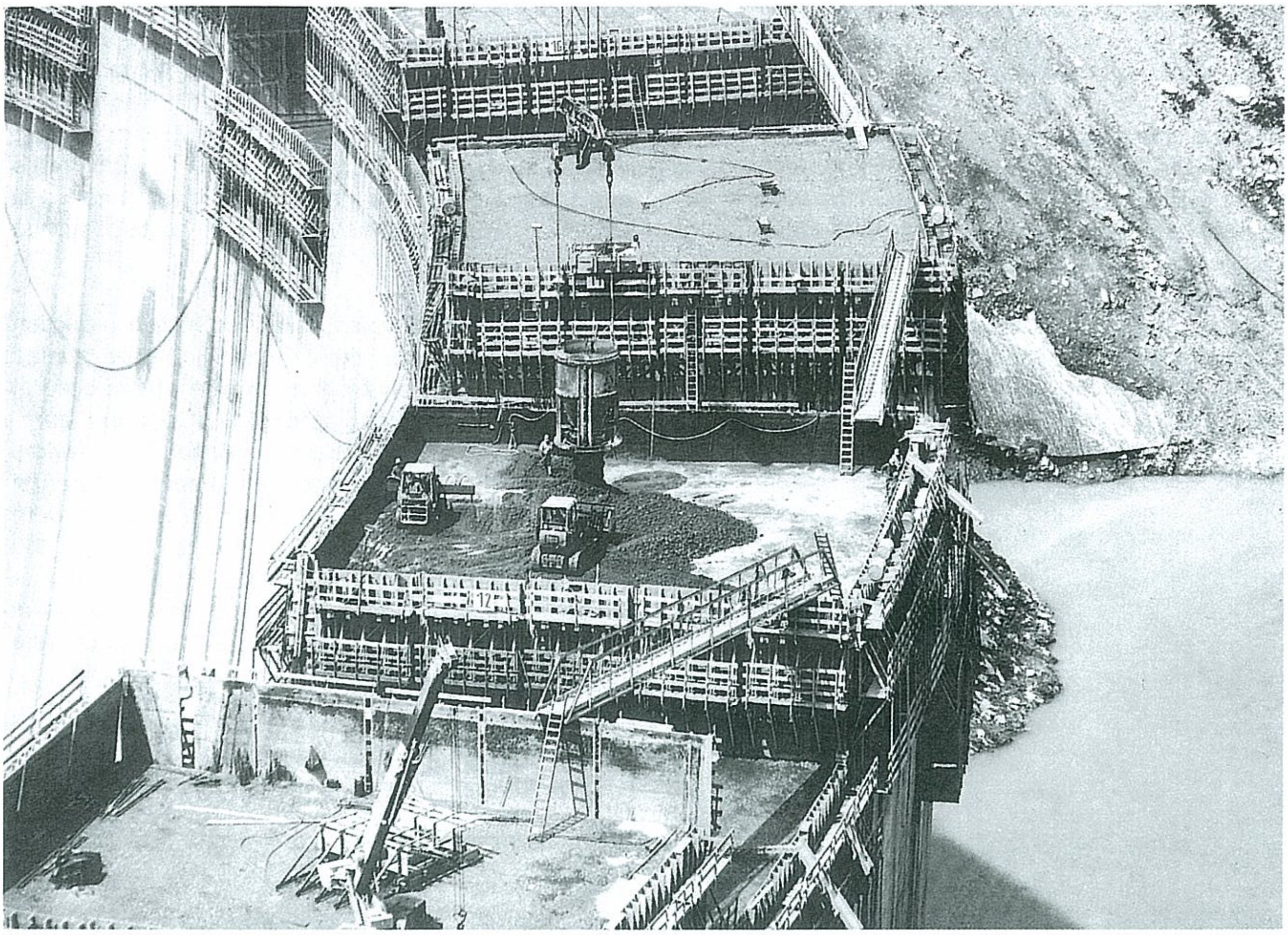


Figure 10 Concrete placement at Zillergründl dam by vibrating and spreading crawlers in 1983

ube, heavy tower cranes with a load moment of 2 500 kNm were used for the first time, with the concrete being placed by buckets. For the reinforced blocks in the spillway area and above all for the powerhouse, hoppers with breeches pieces had to be used for pouring in order to avoid segregation. Complex supporting frames holding these hoppers were an outstanding feature of such construction sites. The development then led to the use of high-velocity belt conveyors mounted on the booms of telescopic cranes. At Greifenstein, as much as 70% of the total concrete quantity was already placed by conveyor belt. With the concrete being placed in the block by means of a hose, frames for the hoppers were no longer needed (Fig. 11). While at first the freshly mixed concrete was hauled to the tower cranes and conveyor belts by normal lorries, with the transfer of the concrete being carried out by tipping silos, special concrete bowls were later used which allowed direct loading.

6.2 Formwork technology

Great progress has also and above all been made in the field of formwork technology. From the very beginning, large-panel formwork with units of some 10 m^2 in area were used for the mass concrete. At first, mainly steel formwork was used. At the large river barrages, formwork was pulled into place by tower cranes, whereas at the construction sites of the Kaprun dams formwork was lifted into place by devices consisting of winches and racks. At Schlegeis, it was possible by means of the 20-t capacity blondins to lift mobile cranes onto the blocks. Carpenter-produced plate-lined wooden climbing formwork was used there. In the years that followed an Austrian timber construction firm developed in cooperation with dam construction firms a climbing formwork that was designed to answer the special requirements of the mass concrete. Starting from the Kölnbrein dam and the Altenwörth run-of-river station construction sites, DOKA climbing formwork for dams entered the path towards worldwide application (Fig. 12). By using this type of climbing formwork it was also possible to accomplish a substantial reduction in the number of hours spent per square metre of formwork.

7 THE USE OF BITUMINOUS ELEMENTS IN HYDRAULIC ENGINEERING

Bitumen as an impervious element was used already in antiquity. Its use in hydraulic engineering is closely associated with the development of laying machines in asphalt road construction. The first construction project in Austria where asphalt was used in the form of an impervious blanket was the Schwarzach storage basin in 1956. Material properties with respect to imperviousness and stability were studied by means of model tests. Placing on slopes was by means of converted road surfacing finishers and winch-driven vibrating rollers and or base plates.

The mixes applied were rich in bitumen and fine particles to permit adequate compaction by means of the equipment available. Stability was accomplished through the addition of fibrous material.

As laying machines have been improved, leaner asphalt mixes with smaller percentages of fine particles have become feasible. The result of this development is a well graded mineral aggregate with a small proportion of voids, which are filled with bitumen. The increased internal friction of the mineral constituents and their cementing with the bitumen ensure the required stability.

Besides, owing to the use of modified and up-to-date asphalt finishers, there has been a steady increase in laying performance.

Outstanding landmarks in this development are the Hochwurten, Grossee, Oscheniksee and Gösskar embankment projects.

Especially at high-level sites, impervious slope facings are subject to mechanical and thermal loadings during water level fluctuations, mainly as a result of ice formation and insolation.

Also, asphalt surfaces in mountain landscapes tend to be an irritating sight to mountain lovers.

This led dam designers to seek alternative solutions in the use of asphalt in hydraulic engineering. By the introduction of asphalt core membranes, the impervious element was moved from the surface to the interior of the dam. The construction of such asphalt membranes called for the development of special equipment capable of placing a narrow asphalt structure by layers, while the asphalt mixes only needed some modification, as the functional criteria remained the same.

Figure 12 DOKA climbing formwork used at Kölnbrein and Zillergründl construction sites

Particular attention must be directed at the course joints to ensure the continuous imperviousness of the structure. Pretreatment of the preceding course helps to accomplish a satisfactory bond. Uniform and continuous filling on both sides is required to support the

Figure 11 High speed conveyor belts on telescoping cranes pouring concrete at the construction site of the Melk power project on the Danube in 1980

Figure 13 Placement of asphaltic-concrete core membrane at Finstertal embankment dam

membrane. Although this construction method was first applied as early as 1967 at the Eberlaste reservoir, a spectacular breakthrough was not achieved until the construction of the about 100 m high Finstertal embankment dam (Fig. 13) with an asphalt core membrane. A special finisher was used for the first time which placed and compacted the asphaltic concrete and the supporting shells in layers up to 30 cm deep in a single operation. As evidenced by the extremely small seepage flows, the performance of this impervious element is excellent. The latest application of this design is the 80 m high Feistritzbach dam constructed in the year 1989–90.

Development studies on construction equipment for asphalt membranes have not so much been aimed at boosting output, as this is a quantity defined by the filling operations, but at reducing the concrete-pouring crews and at improving compaction.

8 CONSTRUCTION SITE ORGANIZATION

An essential factor in the good functioning of construction operations is adequate project management and organization. As almost all the large hydro power projects are constructed by joint ventures, the first step has always been to conclude a joint-venture contract between the participating firms. Preparation of a joint-venture standard contract by the Vereinigung Industrieller Bauunternehmungen Österreichs (association of industrial construction enterprises in Austria) has led to a large measure of standardization of these contracts over the last two decades. In this way, the conclusion of such contracts has become routine work, and administration has become highly efficient. This has mainly been accomplished by the appointment by name of one or several persons to be entrusted with the responsible management of contract implementation.

A clear site management organization chart with the

Site Manager at the top and a subdivision of spheres of competence at the second level has been successfully used for the execution of the individual construction projects. The organization chart for the Greifenstein power scheme on the Danube is given as an example (Fig. 14).

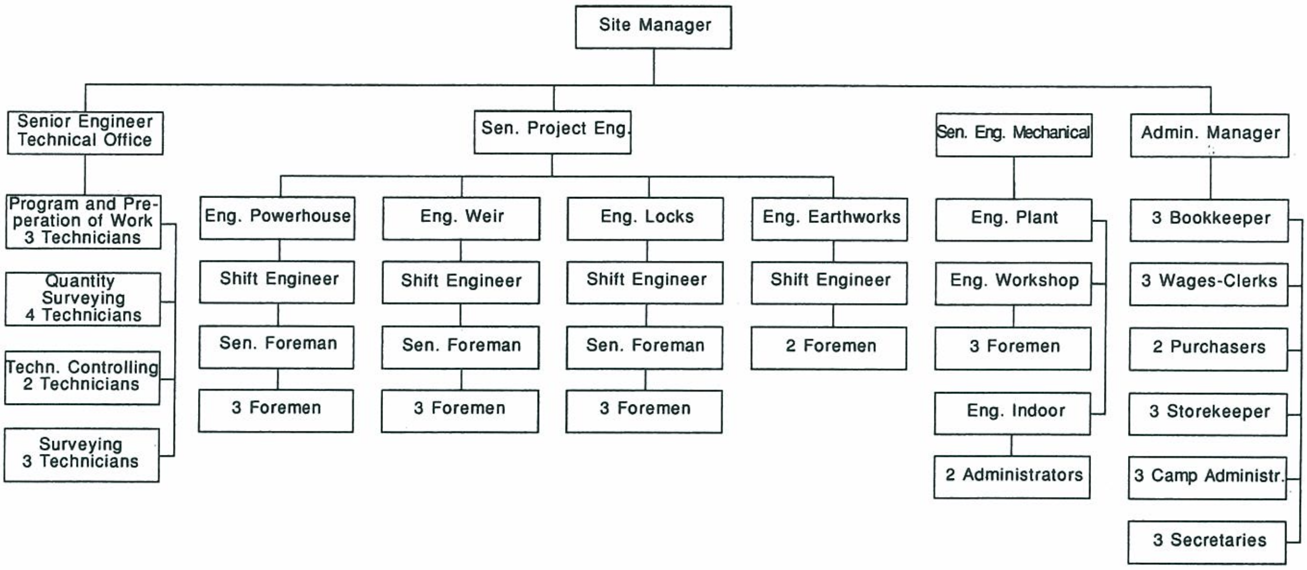
Due to the more or less uninterrupted sequence of construction projects, firms and individuals have had the opportunity to cooperate at several sites so as to acquaint themselves well with the work and acquire a certain routine. On the other hand, however, new firms and persons that have joined the team have brought the necessary renewal and have facilitated, by their new ideas, the progress of rationalization. It is also such measures that have brought about substantial reductions in the number of senior staff needed at a site. The number of employees has steadily been reduced in this way.

illustrate the repercussions of the above developments in power project construction as reflected by rationalization.

In view of the relatively high cost of labour in Austria, rationalization efforts have primarily been directed at this cost factor. The use of more and better equipment and the simplification of construction methods have resulted in a progressive reduction of the number of hours spent per unit of work done. Careful and ample work preparation as practised to an increasing degree in power project construction has also helped to accomplish a large measure of rationalization.

The concrete pouring operations for large dams exhibit particularly pronounced industrial features. The manpower and machinery employed at a site provided with appropriately dimensioned equipment are a nearly constant quantity. In this way, the increase in work done

Figure 14 Organization chart at the climax of construction operations at the Greifenstein project site on the Danube



An incentive wage system and working hours attractive to the workers have also contributed a great deal towards improving economy.

At remote high-level sites, work organization on a fortnight basis holds great attraction to the construction worker. As this also presents advantages in terms of site management due to the reduced number of interruptions, the fortnight system has been adopted at practically all high-mountain sites. It is only lately that there has been a growing opposition to this system on the part of the authorities and trade unions, so that, despite the workers' resistance, construction operations will soon return to the seven-day system. Then continuous operation schemes as practised e.g. at Kölnbrein to make optimal use of the short concrete pouring season will become impossible.

9 RATIONALIZATION OF CONSTRUCTION OPERATIONS

In the following paragraphs, the wage factor in the cost of construction will be discussed as an example to

directly results in a reduction in the number of hours spent per unit of work. Fig. 15 is a graph showing concrete pouring performance plotted against man-hours spent. As the five comparable construction sites extend over a period of four decades, the amount of rationalization is clearly recognized. If the skilled workers' wage based on collective labour agreements is introduced as a parameter and related to the number of hours spent as shown in Fig. 16, it will be seen that for moderate wage increases the labour costs have been constant or decreasing, while large wage increases have been absorbed to a very large degree by the rationalization. This has also been reflected by the prices, which have remained far behind the wage increase.

The eight power stations on the river Danube, constructed in the three decades between 1955 and 1985, are ideally suited to demonstrate the importance of rationalization at large run-of-river stations. As river barrages are much more complex than high concrete dams, it is necessary for any general considerations to reduce the work to the two main items, earthworks and concrete construction. For the sake of comparison, the

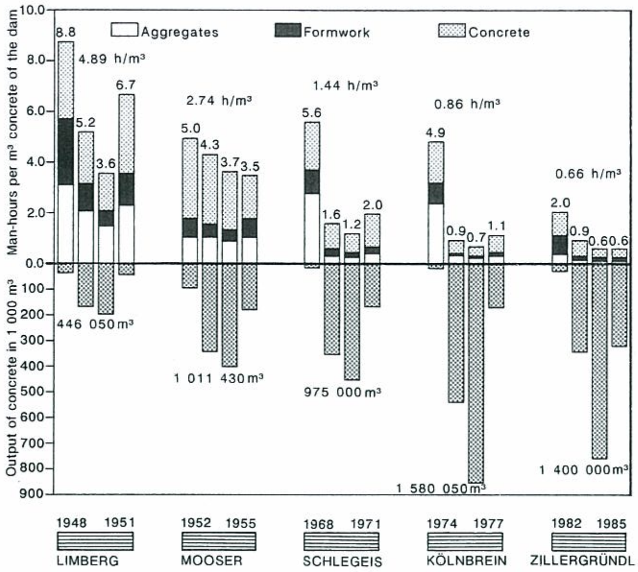
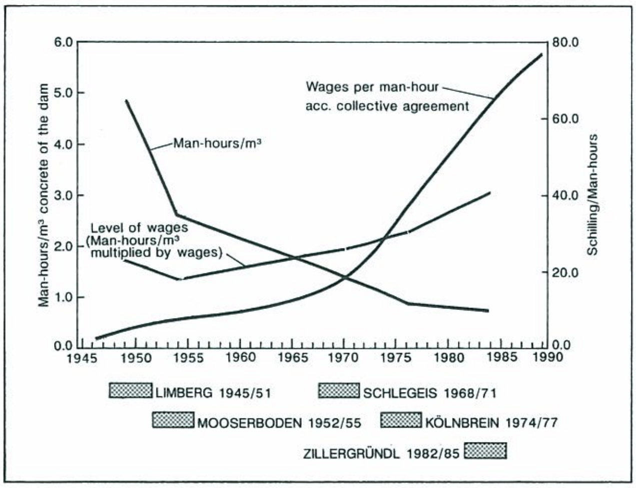


Figure 15 Graph showing concrete pouring performance plotted against man-hours spent at large-dam construction sites. Expenditure in the individual concrete-pouring years and total funds per construction site

Figure 16 Rationalization in the construction of large concrete dams as demonstrated by data from five comparable construction sites from the period between 1948 and 1985



construction cost of the respective main structure has been expressed in 1988 prices by use of appropriate indices. It should be noted in this context that cement and structural steel have always been provided by the owner. The large amount of data available shows that the ratio of expenses, both for manhours and construction cost, of 1 m³ of earthwork to 1 m³ of concrete has been 1: 20. All the cost items of these construction projects have been related to these two main work items. The amount of rationalization achieved can be seen from Figs. 17 and 18.

Analysis of the curves shown in the Figures leads to the following conclusion: Rationalization reached a climax between the construction sites of Ybbs-Persenbeug and Wallsee-Mitterkirchen, which was due to a large extent to the adoption of a new project idea providing for the construction of the project in a dry pit located across the neck of a river bend.

The rationalization curve flattens towards the end of the

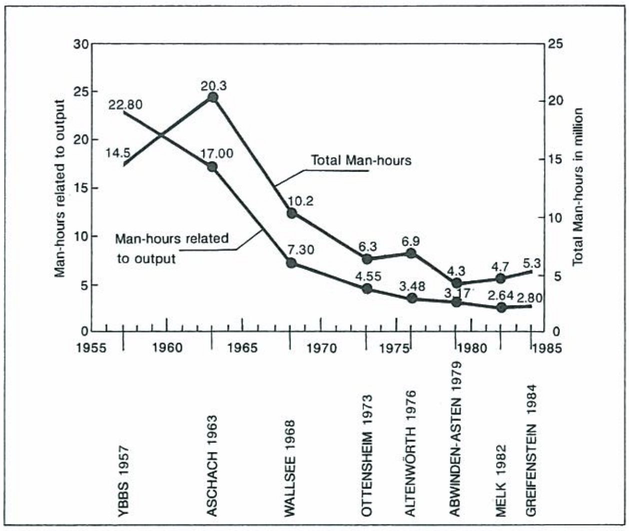


Figure 17 Development of man-hours in the construction of the main structures of the power projects on the Danube, both in absolute numbers and as parameters related to the main construction items

period under review. The slight increase in the number of man-hours spent per unit of work done at the Greifenstein power station is a result of the greatly reduced construction period.

The reduction in man-hours has for the greater part been reflected by the construction cost per unit of work done. Part of the savings in cost of labour have, however, been needed for covering increased cost of machinery.

Part of the savings in man-hours have been consumed by the increase in average gross wages being larger than the amount of rationalisation achieved, which is particularly true of the second part of the period under review.

10 CONCLUSION

The development of construction operations at the power project sites in Austria has led to a large measure of rationalization. This has implied substantial savings in the cost of constructing new projects. Another result of this development is that power projects that were formerly put aside as their construction was considered uneconomical are now becoming attractive in the light of the lower construction cost.

As mentioned earlier, other developments have led to a loss in public acceptance of power project construction. This makes it almost impossible at present to benefit from and continue to develop the experience gained in this field. But recent discussions regarding the ban on nuclear power have certainly initiated a reversal of opinion with respect to hydro power. At any rate, a large enough number of projects still exist in Austria both for run-of-river stations, as e.g. Freudenau on the Danube in Vienna, and for high-level sites, as e.g. a group of power schemes near Matrei in East Tyrol.

In any case, Austria's construction industry is ready to

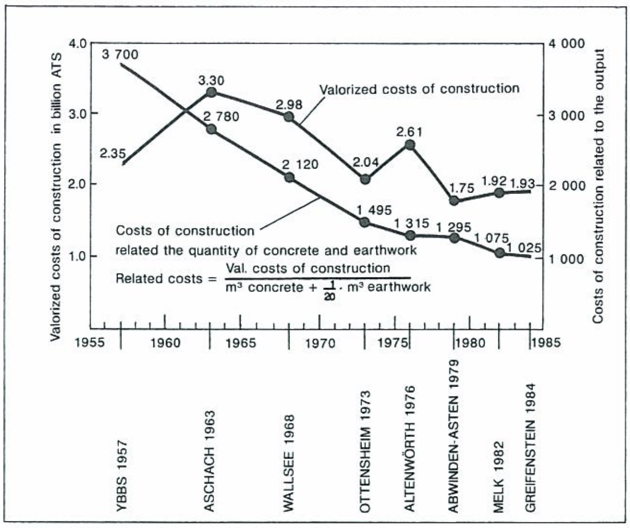


Figure 18 Development of construction cost for the main structures of the power projects on the Danube, expressed in 1988 prices, both in absolute values and as parameters for construction cost related to work items

continue to use and develop the experience gained in power project construction, and the Austrian construction firms offer their skill and experience for projects abroad.

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Chapter K

REGISTER OF DAMS IN AUSTRIA

CONTENTS

REGISTER OF DAMS IN AUSTRIA

By R.Widmann*

INTRODUCTION

The following Register of Dams in Austria shows the main data of all structures of more than 15 m in height, according to the ICOLD guidelines. Thus this Register is meant to update the former editions of "Large Dams in Austria" presented at

- the 8th Congress on Large Dams, Edinburgh 1964,
- the 45th ICOLD Executive Meeting, Salzburg 1977.

On the occasion of the 15th Congress of Large Dams in Lausanne in 1985 a further overview entitled "Hydro Power Schemes and Large Dams in Austria" was published.

A more detailed description of 26 dams of special interest is given in Chapter F (concrete dams) and G (embankment dams); these dams are marked with an asterisk *. The dams in the register appear in chronological order of the start of construction (first figure in accordance with the 1964 and 1977 editions); in each case the corresponding number in the World Register of Dams is added in brackets for easy cross reference.

The location of the dams can be found on the map at the end of this book.

LEGEND

The "Legend to the Register of Dams in Austria", in which all the symbols are explained, is printed on the next page. Only a few additional remarks are provided below.

H (Row 5, line 1, 2) The governing height of embankment dams from the design point of view is the height of the impervious zone above foundation, which may be somewhat smaller than the greatest height above the lower dam toe for dams with an impervious core.

W_A (Row 6, line 3) The hydrostatic load acting on the dam parallel to the valley is the most important parameter for characterizing the size of the dam and comparing dams of different types.

J (Row 8, line 1–3) The normal annual inflow into the reservoir is devided into natural inflow from the catchment area of the reservoir (row 7, line 1), and the collecting works (row 7, line 2), and inflows from pumping operations.

E (Row 11, line 1–3) From this figure one can see not only the importance of the reservoirs for the immediately connected power station, but also for all power stations downstream.

A,S (Row 12, line 1, 2) These figures are based on experiences at Austrian reservoirs with different geologies of their catchment areas.

T (Row 14, line 1) The time required for drawdown may be of importance for relieving the dam of hydrostatic load in the case of unforeseen events.

Q_v (Row 15, line 2) The volume of the flood wave may be of importance for evaluating retention volume in relation to surface area at top water level (row 9, line1).

Q_p (Row 15, line 3) The peak inflow of a design flood may be reduced at reservoirs with a large surface area and ungated spillways due to the retention volume between normal top storage level and storage level in case of a design flood.

F_F (Row 16, line 2) Negative values show the height of overflow above the dam crest.

F_p (Row 16, line 3) The observed probability of overspill at dams more than 10 years old is an indicator for the correct selection of the discharge capacity as well as for the improvement in flood protection of the downstream area.

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Legend to the Register of Dams in Austria

1	7 Chronological sequence (1/16) No. in the World Register of Dams
2	Name Name of dam; in brackets name of reservoir if different from name of dam Provinces K Kärnten (Carinthia) N Niederöstereich (Lower Austria) O Oberösterreich (Upper Austria) S Salzburg St Steiermark (Styria) T Tirol (Tyrol) V Vorarlberg
3	Owner Verbundkonzern EKW Ennskraftwerke AG - Steyr ÖDK Österr.Draukraftwerke AG - Klagenfurt TKW Tauernkraftwerke AG - Salzburg VIW Vorarlberger Jllwerke AG - Bregenz ÖBB Österreichische Bundesbahnen - Wien Landesgesellschaften EVN Energie-Versorgung Niederösterreich - Maria Enzersdorf KELAG Kärntner Elektrizität-AG - Klagenfurt OKA Oberösterreichische Kraftwerke AG - Linz SAFE Salzburger AG f. Elektrizitätswirtschaft - Salzburg S STW Salzburger Stadtwerke - Salzburg STEWEAG Steirische Wasserkraft-u. Elektrizitäts AG - Graz TIWAG Tiroler Wasserkraftwerke AG - Innsbruck VKW Vorarlberger Kraftwerke AG - Bregenz Contractor J.V. Joint Venture, Technical Management
4	Type of dam (according to the World Register) PG Gravity dam TE Earthfill dam VA Arch dam ER Rockfill dam Position and nature of impervious element 1[st] letter..... position f face i core 2[nd] letter..... nature a asphalt c concrete e earth G Geology R Rock S Soil Y Year of completion, /....Year of heightening or rehabilitation
5	Height of the dam H_F above foundation (at fill dams of the impermeable zone) m H_T at fill dams above the dam toe m L_C Crest length m

6	V_E	Volume of excavation (rock and overburden)	1 000 m³
	V_D	Volume of dam body	1 000 m³
	W_A	Water load parallel to valley above foundation	1 000 MN
7	C_R	Catchment area of reservoir	km²
	C_D	Catchment area of collecting works	km²
8		Normal annual inflow from the	
	J_R	Catchment area of the reservoir	hm³
	J_D	Catchment area of the diversions	hm³
	J_P	Pumping mode	hm³
9	SL	Surface at top water level	km²
	TL	Top operating water level	m a.s.l.
	ML	Minimum operating water level	m a.s.l.
10	V_T	Total volume	hm³
	V_A	Active storage capacity	hm³
		Type of reservoir RA Annual storage RW Weekly storage RD Daily storage RF Flood retention	
11		Energy potential of the reservoir	
	E_e	of the related power scheme	GWh
	E_d	including downstream power stations	GWh
	E_A	related to Austrian territory	GWh
12	A	Annual abrasion of C_R	mm/km²
	S	Annual sedimentation in the reservoir	% V_T
	M	Periodical measures necessary	yes/no
13	Q_D	Design flow of the power station	m³/sec
	GC	Generating capacity	MW
	Q_B	Design flow of the bottom outlet	m³/sec
14	T_E	Reservoir emptying time (Q_D+Q_B, without inflow)	h
	T_B	Method of energy-dissipation at bottom outlet S stilling basin G gate/valve	
	T_F	Type of flood discharge 1[st] letter: type 2[nd] letter: situation L(V) uncontrolled (controlled) spillway C Crest O through the dam body B Bypass	
15	N	Probability of design flood	Years
	Q_V	Volume of design flood wave	hm³
	Q_P	Peak inflow of spillage	m³/sec
16		Freeboard to dam crest	
	F_N	above normal top water level	m
	F_F	in case of design flood	m
	F_P	Probability of spillage	Years
	n	no data available	

REGISTER OF DAMS IN AUSTRIA

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
			Dam			Reservoir						Appurtenant Works			
No.	Name Province / nearest City River	Owner Engineering Contractor	Type G Y	H_F H_T L_C	V_E V_D W_A	C_R C_D –	J_R J_D J_P	SL TL ML	V_T V_A R	E_e E_d E_A	A S M	Q_D GC Q_B	T_E T_B T_F	N Q_V Q_P	F_N F_F F_P
1	Wienerbruck N / St. Pölten Lassing	EVN EVN EVN	PG R 1911	11 – 25	– 2.5 0.01	32 – –	33.6 – –	0.1 791.5 789.3	0.25 0.16 RW	0.06 0.1 0.1	0.038 0.39 no	2.5 3.0 13	8 – VB	500 – 115	1.8 0.8 >1
2 (1/1)	Erlaufklause N / Mariazell Erlauf	EVN EVN EVN	PG R 1911	37 – 88	– 22 0.2	45 – –	41.4 – –	0.23 779.4 771.4	1.7 1.4 RA	0.4 0.6 0.7	– – no	3.5 4.8 31	17 – LB+VB	500 – 130	6.4 5.0 >1
3 (1/2)	Gosau O / Hallstatt Gosaubach	OKA Stern & Hafferl Stern & Hafferl	TEic S 1911	21 17 50	1.5 13 0.03	34 – –	70 – 6	0.67 922 875	28 21 RA	17 27 37	– – no	5.5 5.9 –	– – VB	– – 40	1.8 0.8 n
4 (1/3)	Wiestal S / Hallein Almbach	SStW Müller-Georgini Municipality Salzburg	PG R 1913	28 – 66	– 11.5 0.45	175 – –	265 – –	– 554.6 544.1	– 7.5 RD	1.2 2.0 3.5	– – –	– 6.35 20	– – VC	5000 – 400	2.10 1.44 >1
5 (1/5)	Strubklamm S / Hallein Almbach	SStW Mayrhofer Pittel & Brausewetter	PG R 1924	36.5 – 86	– 9.14 0.55	100 – –	131 – –	– 668.0 658.0	– 2.5 RD	0.55 – 10.0	– – –	– – 30	– – LC	– – –	– – –
6a (1/7)	Spullersee – South V / Bludenz tr. Aflenz – III	ÖBB ÖBB Innerebner & Mayer	PG R 1925/65	39 – 298	14 66 1.22	11 7 –	14 9 –	0.57 1829.6 1790	16.9 15.7 RA	26 37 39	0.3 <0.2% no	6.3 36 15	276 GB LC	5000 0.7 62.8	0.4 0 0.1
6b (1/8)	Spullersee – North V / Bludenz tr. Aflenz – III	ÖBB ÖBB Innerebner & Mayer	PG R 1925/65	28 – 200	10 27 0.48	same Reservoir as 6a									
7 (1/6)	Langmann St / Voitsberg Teigitsch	STEWEAG STEWEAG Steir. Wasserbau – Syndikat	PG R 1925	28 – 85	– 12 0.15	170 – –	100 – –	0.05 633.8 624.3	0.4 0.32 RD	0.16 0.21 0.3	– – yes	16.5 30 160	0.6 SG LC+VO	>100 – 200	1.1 – >1
9 (1/12)	Vermunt V / Schruns Ill	VIW Lahmeyer Dyckerhoff & Widmann	PG R 1931	53 – 386	105 144 1.79	57 50 –	103 70 –	0.36 1743 1719	5.7 5.3 RW	8 14.2 14.5	– – no	26 156 37+14	26 – LC	5000 1.0 97.5	1.7 0.9 0.08
10 (1/11)	Pack St / Köflach tr. Teigitsch	STEWEAG STEWEAG Alpenland / Tiefbau	PG R 1931	33.4 – 183	20 39 0.55	63 – –	43 – –	0.6 866.3 843.6	5.6 5.4 RA	0.2 4.2 7.5	– – yes	3 0.6 7	155 SG VC	5000 6.75 250	1.0 –0.3 >1

The first Tauernmoos Gravity Dam (No. 8, 28 m heigh) was replaced by the new Tauernmoos Gravity Dam (see No. 46).

Register of Dams in Austria (cont'd)

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
			Dam			Reservoir						Appurtenant Works			
No.	Name Province / nearest City River	Owner Engineering Contractor	Type G Y	H_F H_T L_C	V_E V_D W_A	C_R C_D –	J_R J_D J_P	SL TL ML	V_T V_A R	E_e E_d E_A	A S M	Q_D GC Q_B	T_E T_B T_F	N Q_V Q_P	F_N F_F F_P
11 (1/13)	Enzingerboden S / Mittersill Stubach / Salzach	ÖBB ÖBB Universale	PG S 1940	29 – 68	– 11 0.12	45.8 6.7 –	91.2 13.4 –	0.06 1463.5 1461.0	0.25 0.20 RD	0.17 0.35 0.6	– – –	23 98 21.8	2 GB LC	– – 60	1.5 0.1 0.2
12 (1/17)	Gerlos T / Zell a. Ziller Gerlos / Ziller	TKW TIWAG Unionbau	VA R 1945/64	39 – 69	– 25.2 0.21	99 89 –	79 116 –	0.13 1191 1176	0.88 0.76 RW	1.07 1.5 1.8	0.40 6.8 yes	14.0 65 94	3 G LC	– – 216	2.37 0.60 >1
13a (1/22)	Silvretta V / Schruns Ill	VIW VIW-Lahmeyer A. Kunz – Beton Monierbau	PG R 1948	80 – 432	250 407 5.82	35 10 –	63 17 –	1.31 2030 1986	39.1 38.6 RA	22 104.2 –	– – no	14 29 26+28	212 – LC	5000 202 136	1.92 1.0 0.06
13b (1/23)	Biel V / Schruns Ill	VIW VIW-Lahmeyer A. Kunz – Beton Monierbau	TEic R 1948	25 – 733	68 393 0.85	same Reservoir as 13a									
14 (1/20)	Bürg S / Zell a. See Kapruner Ache	TKW TKW J. V., Rella	PG R 1947	19 – 73	24.2 11 0.06	27.9 – –	27 – –	0.07 847 842	0.24 0.21 RD	0.02 0.2 0.3	0.28 3.25 yes	1.0 0.5 26.0	3 G LC	– – 82	2.1 0.6 >1
15 (1/25)	Salza St / Liezen tr. Enns	STEWEAG STEWEAG J. V., Mayreder	VA R 1949	53 – 121	10 23 0.73	150 – –	145 – –	0.8 768.5 742.5	11.0 10.5 RA	2.1 8.0 13.8	– – yes	9 8 8	180 SG LC	>100 – 140	0 –1.1 1.1
16 (1/24)	Hollersbach S / Zell a. See Hollersbach / Salzach	SAFE SAFE + Interplan Hinteregger	TEie S 1949	14.4 23.1 87	2.5 16 0.02	67 2.4 –	126 0.9 –	0.04 879.5 875.0	0.165 0.135 RD	0.02 0.15 0.25	0.6 – yes	2.6 1.4 100	0.4 GB LC	5000 – 260	3.63 1.4 0.2
17 (2/3)	Hierzmann St. / Köflach Teigitsch	STEWEAG STEWEAG Ast – Universale	VA R 1950	58 – 172	16 43 1.18	160 – –	95 – –	0.52 711.3 678.3	7.6 7.1 RA	1.1 4.4 7.5	– – yes	16.5 11 20	58 SG LC	>100 – 180	1.4 0 >1
18 (2/4)	Ranna O / Rohrbach Ranna	OKA Beurle – Grengg – Reitz Ferro-Betonit	VA R 1950	45 – 126	10 32 0.60	158 8 –	110 3 2.2	0.30 493.0 473.0	2.4 2.1 RW	1.0 1.5 1.7	0.01 0.04 no	12 19 25	24 SG LC	5000 – 160	4.0 3.3 0.3
19 (2/6)	Limberg (Wasserfallboden) S / Zell a. See Kapruner Ache	TKW TKW J. V., Rella	VA R 1951	120 – 357	287 446 9.2	15 128 –	21 226 –	1.53 1672.0 1590.0	83 81 RA	160 226 297	0.38 0.03 no	36.5 220 93	240 S LB	– – 26	1.60 0.80 0.03

Register of Dams in Austria (cont'd)

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
			Dam			Reservoir						Appurtenant Works			
No.	Name Province / nearest City River	Owner Engineering Contractor	Type G Y	H_F H_T L_C	V_E V_D W_A	C_R C_D –	J_R J_D J_P	SL TL ML	V_T V_A R	E_e E_d E_A	A S M	Q_D GC Q_B	T_E T_B T_F	N Q_V Q_P	F_N F_F F_P
20 (2/5)	Bächental T / Jenbach Dürrach, Isar	TIWAG TIWAG Ast	VA R 1950	34 – 70	2.0 2.8 0.14	55 – –	70 – –	– 952 –	– – –	– – –	0.3 – yes	2.8 79 –	– – LC+VB	200 – 260	– – >1
21a (2/7)	Möll (Margaritze) K / Heiligenblut Möll	TKW TKW J. V., Porr	VA R 1952	93 – 164	9 35 0.79	44 20 –	97 25 –	0.21 2000 1980	3.8 3.2 RW	8.3 10.9 13.5	– 0.4 yes	20 13.4 83.4	15 GO –	– – –	2.15 0.50 –
21b (2/8)	Margaritze K / Heiligenblut Möll	TKW TKW J. V., Porr	PG R 1952	39 – 175	6 33 0.35	same Reservoir like 21a							– – LC	– – 210	– – 0.09
22 (2/11)	Dobra N / Zwettl Kamp	EVN Siemens J. V., Rella	VA R 1953	52 – 234	69 90 1.6	940 – –	222 – –	1.5 437 410	21 20 RA	2.7 3.5 –	– – no	30 16.2 80	70 BG LC S	900 – 500	2.5 0.2 0.6
23 (2/9)	Thurnberg N / Horn Kamp	EVN Siemens J. V., Mayreder	PG R 1952	25 – 48	28 23 –	1012 – –	235 – –	0.4 364 362	2.5 0.8 RW	0.03 – –	– – no	16.5 2.7 40	17 BG VG	700 – 500	2.0 1.0 >1
24 (2/10)	Weißsee S / Mittersill tr. Stubach / Salzach	ÖBB ÖBB J. V., Kunz	PG R 1952	39 – 235	25 64 –	5.42 5.18 –	15.0 11.0 –	0.50 2250 2197	16.0 15.7 RA	17 37 46	n n no	9 – 9	– GB LC	5000 1.1 –	1.5 0 1
25 (2/13)	Wiederschwing K / Villach Weißenbach	KELAG KELAG J. V., Mayreder	VA R 1953	30 – 74	3 8 0.02	153 – –	115 – –	0.10 675.5 663.0	1.0 0.95 RW	3.8 – 7.8	0.04 0.63 yes	7.0 9.1 24	8.5 G LC+VB	5000 13.8 160	3.8 1.54 >1
26a (2/16)	Mooser (Mooserboden) S / Zell a. See Kapruner Ache	TKW TKW J. V., Rella	PG R 1955	107 – 494	193 665 10.1	27 72 –	62 130 39	1.66 2036 1960	87 85 RA	237 307 362	0.14 0.01 no	36 112 62.5	411 G VC	5000 – 100	1 1 0.04
26b (2/17)	Drossen (Mooserboden) S / Zell a. See Kapruner Ache	TKW TKW J. V., Rella	VA R 1955	112 – 357	315 355 7.5	same Reservoir like Mooser									
27 (2/19)	Ottenstein N / Zwettl Kamp	EVN Siemens J. V., Rella	VA R 1956	69 – 240	83 124 3.0	889 – –	221 – –	4.3 495.0 476.5	73 51 RA	6 10 13	– – no	100 47 16	210 S VC+VO	5000 – 650	1 0.4 0.2

Register of Dams in Austria (cont'd)

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
			Dam			Reservoir						Appurtenant Works			
No.	Name Province / nearest City River	Owner Engineering Contractor	Type G Y	H_F H_T L_C	V_E V_D W_A	C_R C_D –	J_R J_D J_P	SL TL ML	V_T V_A R	E_e E_d E_A	A S M	Q_D GC Q_B	T_E T_B T_F	N Q_V Q_P	F_N F_F F_P
28* (2/22)	Rotgüldensee S / St. Michael tr. Mur	SAFE SAFE + Flögl / Linz J. V., Hinteregger	ERia R+S 1957/91	38 45 273	99.4 345 0.93	11.0 34.2 –	19.2 58.2 –	0.43 1733 1670	15.6 14.9 RA	20.4 8.0 61.7	– – no	14.0 68 18.5	15.5 GB LO	5000 0.82 104	3.0 2.0 0.43
29 (2/21)	Großer Mühldorfersee K / Spittal a. d. Drau tr. Möll	ÖDK ÖDK J. V., Universale	PG R 1957	46 – 433	47 153 2.04	3 5 –	4 14 –	0.24 2319 2255	7.9 7.8 RA	31.2 35.3 36.0	– – no	4.5 139 4	255 GO LC+VO	1000 0.8 17	1.13 0.39 10^{-4}
30 (3/1)	Kleiner Mühldorfersee K / Spittal a. d. Drau tr. Möll	ÖDK ÖDK J. V., Universale	PG R 1958	41 – 159	31 60 0.84	1 7 –	2 16 –	0.11 2379 2335	2.9 2.8 RA	11.1 12.6 12.8	– – no	4.5 139 16	38 GO LC+VO	1000 0.3 16	1.13 0.31 10^{-4}
31 (2/25)	Hochalmsee K / Spittal a. d. Drau tr. Möll	ÖDK ÖDK J. V., Universale	PG R 1958	24 – 237	10 29 0.33	2 6 –	3 15 –	0.15 2379 2330	4.2 4.1 RA	16.4 18.6 19.0	– – no	4.5 139 15	60 GO LC+VO	1000 0.6 15	1.0 0.31 10^{-4}
32 (3/3)	Radlsee K / Gmünd tr. Lieser	ÖDK ÖDK Oberranzmayer	ERic R 1958	16 212 38	22 0.08 –	1.7 5.9 –	2.6 9.4 –	0.12 2399 2354	2.5 2.5 RA	9.9 11.2 11.40	– – no	4.5 139 21	27 GO LB+VO	1000 0.5 21	1.55 0.67 10^{-4}
33 (3/2)	Lünersee V / Bludenz tr. Ill	VIW VIW J. V., Hinteregger	PG R 1958	30 – 380	15 41 0.56	9 3 –	12 3 –	1.55 1970 1897	94 78 RP	166 255 –	– – no	32 232 15+5	485 – LB	5000 2.0 57.8	1.6 0.85 –
34 (3/4)	Salzplatten S / Mittersill Stubach / Salzach	ÖBB ÖBB Kunz – Unionbau	PG R 1958	17 – 88	3.4 5.3 –	0.51 – –	1.40 – –	0.05 2298.5 2262.0	– 1.1 RA	1.3 4.0 4.2	– – no	– – 0.6	680 GB LC	500 0.6 –	0.2 0 –
35 (2/23)	Amer S / Mittersill Amerbach / Salzach	ÖBB ÖBB Kunz – Unionbau	PG R 1958	30 – 162	10.4 20.3 –	2.34 – –	6.2 – –	0.21 2279.5 2257.2	5.5 5.5 RA	6.0 16.0 16.5	– – no	– – 6.0	– GB LC	5000 4.2 –	1.5 0 –
36 (3/5)	Lutz V / Bludenz Lutz	VKW VKW Hinteregger	PG R 1959	19 – 40	2 4 0.04	180 – –	341 – –	0.02 585 582	0.05 0.03 RD	– – –	0.25 10 yes	16 8.6 100	0.3 – VC	5000 – 475	2.0 1.2 >1
37 (2/24)	Freibach K / Ferlach Freibach / Drau	KELAG KELAG J. V., Isola	TEie S, R 1958	49 41 150	38 235 0.33	44 – –	54.4 – –	0.39 729.2 705.0	5.5 5.1 RA	3.8 – 4.4	– – no	5.75 15 8.2	110 GS LC	1000 – 200	3.3 1.8 >1

* First stage 1957, height 18 m

Register of Dams in Austria (cont'd)

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
			Dam			Reservoir						Appurtenant Works			
No.	Name Province / nearest City River	Owner Engineering Contractor	Type G Y	H_F H_T L_C	V_E V_D W_A	C_R C_D –	J_R J_D J_P	SL TL ML	V_T V_A R	E_e E_d E_A	A S M	Q_D GC Q_B	T_E T_B T_F	N Q_V Q_P	F_N F_F F_P
38 (3/16)	Kops V / Schruns Zeinis / Ill	VIW VIW / Lombardi J. V., Hinteregger	VA R 1965	122 – 400	219 485 12.4	7 163 –	10 223 –	1.0 1809 1730	45 44 RA	74 124 –	– – no	37 247 21+41	170 – LC	5000 1.0 53.7	2.0 1.2 0
39 (3/14)	Gepatsch T / Landeck Faggenbach	TIWAG TIWAG J. V., Porr	ERie R 1965	152 153 600	1000 7100 26.7	107 172 –	125 178 –	2.61 1767 1665	139 138 RA	286 – 521	0.5 0.025 no	53 392 75	302 GB LB	5000 40 266	5.0 3.0 0
40 (3/11)	Dießbach S / Saalfelden tr. Saalach	SAFE SAFE Kunz	ERfa R 1963	29 36 204	15 165 0.43	11.9 10.0 –	15.48 – 10.0	0.25 1415 1390	5.06 4.92 RA	8.0 1.81 17.0	– – no	4.0 24.0 5.7	161 GO LB	(5000) 0.59 66.0	2.4 1.4 0.2
41 (3/21)	Raggal V / Bludenz Lutz	VKW VKW Hinteregger	PG R 1967	48 – 105	34 42 0.61	160 – –	295 – –	0.16 715 695	2.4 2 RW	0.6 0.3 1.5	0.25 1.7 yes	20 20 140	6 S VC	5000 – 430	1.65 1.25 >1
42 (3/18)	Durlaßboden T / Zell a. Ziller Gerlos	TKW TKW Porr – Rella	TEie R/S 1966	83 70 470	229 2520 8.15	45 30 –	64 31 –	1.88 1405 1360	53.5 52 RA	86 115 148	0.90 0.10 no	25.7 25 130	126 S LB	– – 200	4.0 2.35 0.05
43 (4/3)	Schlegeis T / Mayrhofen Zams / Zemm	TKW TKW J. V., Rella	VA R 1971	131 – 725	285 960 19.5	58 63 –	97 102 62	2.2 1782 1680	129 127 RA	327 406 465	0.16 0.01 no	52 230 153	216 S LB	5000 10.7 320	2.30 0.30 0.02
44 (3/23)	Eberlaste (Stillupp) T / Mayrhofen Stilluppe / Zemm	TKW TKW J. V., Oberranzmeyer	TEia S/R 1968	28 28 480	90 790 1.35	61 332 –	84 483 –	0.60 1120 1106	8.2 6.9 RW	7.5 11 15	0.25 0.5 yes	92 345 127	1.2 S LB	– – 450	4.0 1.3 0.04
45 (4/5)	Wurten K / Obervellach Wurten / Fragant	KELAG KELAG Negrelli	TEfa R/S 1971	34 51 282	144 265 0.90	21 87 –	38 114 –	0.22 1695 1675	2.8 2.7 RW	2.9 5.8 8.0	– – no	15.8 66 60	10.3 G LB+VB	5000 6.83 160	4.0 2.3 –
46 (4/10)	Tauernmoos S / Zell a. See Stubach / Salzach	ÖBB ÖBB J. V., Stuag	PG R 1973	53 – 1100	94 250 0.22	22 28 –	51 61 –	1.89 2023 1984.5	57 55 RA	60 131 161	0.86 0.5 no	18 81 33	460 GO LC	5000 8.6 108	1.5 0.5 –
47	Feldsee K / Obervellach Feldsee	KELAG KELAG J. V., Züblin	TEfa S/R 1980	16 17 250	n 65 0.18	1.6 – –	2.52 – –	0.12 2217 2196	1.6 1.6 RA	2.4 7.4 9.2	– – no	5.1 32 5.0	89 G LB	5000 0.12 25	3.0 2.0 –

Register of Dams in Austria (cont'd)

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
			Dam			Reservoir						Appurtenant Works			
No.	Name Province / nearest City River	Owner Engineering Contractor	Type G Y	H_F H_T L_C	V_E V_D W_A	C_R C_D –	J_R J_D J_P	SL TL ML	V_T V_A R	E_e E_d E_A	A S M	Q_D GC Q_B	T_E T_B T_F	N Q_V Q_P	F_N F_F F_P
48 (4/12)	Galgenbichl K / Gmünd Malta	ÖDK ÖDK Oberranzmeyer	TEfa R 1974	50 – 115	36 165 0.44	58 72 –	118 120 –	0.27 1704 1680	4.8 4.4 RW	11.4 13.7 27.9	– – no	80 730 30	12 GO LC+VB	1000 16 226	2.9 1.2 >1
49 (4/7)	Oscheniksee K / Obervellach tr. Fragant	KELAG KELAG J. V., Soravia	ERfa R 1972/79	61 116 530	45 2300 5.04	1.7 – –	3.07 – 136	0.43 2391 2245	34 33 RA	79 112 136	– – no	10.2 108 1.5	– – LB	5000 0.16 32	3.0 1.46 –
50 (4/17)	Klaus O / Steyr Steyr	EKW EKW Held & Franke	VA R 1975	55 – 188	14.5 39 0.55	539 – –	805 – –	0.9 463 457	12.6 7.8 RF	– – –	0.09 0.4 yes	50 18 400	8 – LC+VB	5000 – 1000	4 1 >1
51 (4/13)	Grossee K / Heiligenblut Zirknitz / Möll	KELAG KELAG Beyer – Strabag	ERfa R 1974/80	49 57 425	22 740 1.86	1.7 9.4 –	5.6 20.2 –	0.31 2417 2333	14.1 14.0 RA	21.9 54.2 65.9	– – no	5.1 32 6.5	335 GB LB	5000 0.16 32	3.0 1.5 –
52 (4/14)	Hochwurten K / Obervellach Wurten / Fragant	KELAG KELAG J. V., Soravia	TEfa S/R 1974/80	50 55 260	8 600 1.53	5.6 4.8 –	18.4 16.8 –	0.43 2416.8 2365	12.7 12.7 RA	19.9 49.2 59.9	– – no	5.1 32 11	219 2 GO LB	5000 0.55 70	3.2 1.8 –
53 (4/16)	Gösskar K / Gmünd Göss / Malta	ÖDK ÖDK J. V., Rella – Strabag	TEfa R/S 1975	55 – 260	430 540 1.93	11 119 –	22 216 –	0.12 1704 1680	2.0 1.8 RW	4.68 0.94 5.79	– – no	80 730 38	5 GO LB+VB	1000 3 96	3.65 2.37 –
54 (4/20)	Kölnbrein K / Gmünd Malta	ÖDK TKW / ÖDK J. V., Porr	VA R 1977	200 – 626	420 1580 54	51 79 –	105 – 133	2.55 1902 1750	205.0 190 RA	595 625 711	– – no	70 120 50	475 GO LB+VO	1000 14 188	0.70 0.70 –
55 (4/24)	Sölk St / Schladming Großsölk	STEWEAG STEWEAG – TKW Mayreder – Ast	VA R 1978	39 – 128	8 17 0.36	141 244 –	166 286 –	0.13 901.8 882	1.7 1.5 RD	0.7 1.1 2.4	– – yes	30 61 15	11 SG LC	5000 12.7 370	0 –1.1 >1
56 (5/4)	Längental T / Ötz Nederbach	TIWAG TIWAG J. V., Universale	TEfa S 1979	45 45 418	– 400 1.3	17 116 –	18 132 –	0.21 1901 1882	3.4 3.0 RW	8.9 10.6 12.5	0.2 0.1 no	49 491 20	14 GO LB	5000 1.5 79.5	3.0 1.23 –
57 (5/3)	Finstertal T / Ötz tr. Nederbach	TIWAG TIWAG J. V., Oberranzmeyer	ERia R 1980	96 150 652	– 4500 11.9	6 – –	7 – 54	1.05 2322 2220	60 60 RA	57 173 243	0.2 0.1 no	80 290 20	167 GB LB	5000 1.05 39.6	3.0 2.0 –

Register of Dams in Austria (cont'd)

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
			Dam			Reservoir						Appurtenant Works			
No.	Name Province / nearest City River	Owner Engineering Contractor	Type G Y	H_F H_T L_C	V_E V_D W_A	C_R C_D –	J_R J_D J_P	SL TL ML	V_T V_A R	E_e E_d E_A	A S M	Q_D GC Q_B	T_E T_B T_F	N Q_V Q_P	F_N F_F F_P
58 (4/23)	Bolgenach V / Bregenz Bolgenach	VKW VKW – VIW J. V., Jäger	TEie R 1978	92 102 240	150 1350 4.0	89 98 –	169 183 –	0.30 744.2 690	8.7 8.4 RW	5.4 5.8 7.5	0.19 0.09 no	32 74 100	20 S L/V/B	5000 21.6 450	4.2 2.5 >1
59 (5/7)	Bockhartsee S / Badgastein Bockhart / Gasteiner Ache	SAFE SAFE + SIEMENS – Linz Porr	ERic R 1983	31 69 240	70 228 0.38	5 58.4 –	9.5 – 4.7	0.41 1872.5 1812	14.8 14.2 RA	23.9 38.8 66.4	– – no	11.3 28.8 11.4	207 GB LB	5000 1.98 50	2.5 1.5 –
60 (5/23)	Zillergründl T / Mayrhofen Ziller	TKW TKW J. V., Maculan	VA R 1986	186 – 506	1700 1370 41	30 38 –	49 61 46	1.4 1850 1740	89.5 86.7 RA	241 266 330	– – –	65 360 45	267 S LB	5000 6.0 165	2.0 – –
61 (4/25)	Subersach V / Bregenz Subersach	VKW VKW – VIW Oberranzmeyer	PG R 1978	19 – 95	5 8 0.06	98 – –	183 – –	0.03 759 –	0.03 – –	– – –	0.19 30 yes	– – –	– – LC	1000 – 284	3.7 0.85 <1
62 (5/9)	Paal St / Murau tr. Mur	STEWEAG STEWEAG – TKW J. V., Strabag	VA R 1982	39 – 118	– 20 0.45	107.2 91.3	74 55	0.04 1158 1146	0.3 0.22 RD	1.9 3.4 6.0	– – yes	10 27 18	3 SG VC	>100 – 300	1.0 – 1
63 (5/13)	Zirmsee K / Heiligenblut Seebach / Möll	KELAG KELAG Strabag	ERfa R 1984	42 51 315	– 525 1.0	2.7 2.0 –	4.0 3.0 –	0.3 2529.5 2487	8.8 8.7 RA	13.0 29.5 35.5	– – no	5.1 32 5.0	484 G LB	5000 0.26 45	3.0 1.6 –
64 (5/5)	Naßfeld S / Badgastein Naßfelder Ache	SAFE SAFE + SIEMENS – Linz J. V., Hinteregger	PG R 1980	22 – 116.5	14 13 0.08	37.4 26.0 –	64.3 32.5 –	0.01 1572 1564	0.068 0.056 RD	0.06 0.10 0.22	– 10 no	11.5 46.0 80	0.25 GO LC	5000 – 200	1.0 1.0 –
65	Ginau S / St. Johann tr. Wagrainer Ache	OKA Flögl – Linz J. V., Mayreder	VA R 1987	33 – 88	3 10 0.18	9.4 108.2 –	7.4 110.6 –	0.02 824.5 813.0	0.071 0.057 RD	0.03 0.06 0.09	– 5 yes	8 16 16.7	2 SG LC	5000 – 90	2.3 0.65 –
66	Feistritzbach (Koralpe) St / Lavamünd Feistritz	KELAG KELAG J. V., Züblin	ERia R 1990	85 88 370	– 1600 3.8	29.8 37.0 –	22.4 27.9 –	0.87 1080 1053.5	22.2 16.2 RA	83.5 83.5 83.5	– – no	8.0 50 15.0	196 GS LB	5000 2.8 105	3.4 2.05 –

Appendix

WORLD REGISTER OF DAMS

FOLIOS CONCERNING AUSTRIA

From ICOLD'S World Register of Dams
1984 Full Edition, Updating 1988 and
Supplementary Data 1991

REGISTRE DES BARRAGES EN AUTRICHE
REGISTER OF DAMS IN AUSTRIA

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	
LIGNE *LINE* No.	NOM DU BARRAGE *NAME OF DAM*	ANNÉE D'ACHÈVEMENT *YEAR OF COMPLETION*	SITUATION – *LOCATION*: COURS D'EAU *RIVER*	VILLE LA PLUS PROCHE *NEAREST CITY*	ÉTAT PROVINCE OU DÉPARTEMENT *STATE PROVINCE OR COUNTRY*	TYPE	SITUATION ET TYPE D'ÉTANCHÉITÉ *POSITION AND NATURE OF SEALING ELEMENT*	FONDATION *FOUNDATION*	HAUTEUR AU-DESSUS DE LA PLUS BASSE FONDATION *HEIGHT ABOVE LOWEST FOUNDATION* (m)	LONGUEUR DE CRÊTE *LENGTH OF CREST* (m)	VOLUME DU BARRAGE *VOLUME CONTENT OF DAM* (10^3 m^3)	CAPACITÉ TOTALE DU RÉSERVOIR / SURFACE DU RÉSERVOIR *GROSS CAPACITY OF RESERVOIR / RESERVOIR AREA* (10^3 m^3) / (10^3 m^2)	DESTINATION *PURPOSE*	CAPACITÉ MAXIMALE DES ÉVACUATEURS *MAXIMUM DISCHARGE CAPACITY OF SPILLWAYS* (m^3/s)	TYPE DES ÉVACUATEURS *TYPE OF SPILLWAYS*	PROPRIÉTAIRE *OWNER*	BUREAU D'ÉTUDES *ENGINEERING BY*	CONSTRUCTEUR *CONSTRUCTION BY*	LIGNE *LINE* No.
1	ERLAUFKLAUSE	1911	Erlauf	Mariazell	Lower Austria	PG		R	37	88	22	1720 / 232	H	160	L/V	N E W A G	N E W A G	N E W A G	1
2	GOSAU	1911	Gosausee	Hallstatt	Upper Austria	TE	ic	R/S	17	50	23	24700 / 675	H	60	V	O K A	Stern & Hafferl	Stern & Hafferl	2
3	WIESTAL	1913	Almbach	Hallein	Salzburg	PG		R	28	66	12	800 / n	H	470	V	Municipality Salzburg	Müller - Georgini	Municipality Salzburg	3
4	LANGHALSEN	1924	Gr.Mühl	Rohrbach	Upper Austria	PG		R	17	117	10	800 / 220	H	400	V	O K A	G. Beurle	Spychinger & Hartmann	4
5	STRUBKLAMM	1924	Almbach	Hallein	Salzburg	PG		R	36	86	9	270 / n	H	300 / 30 } 330	L / V	Municipality Salzburg	Mayrhofer	Pittel & Brausewetter	5
6	LANGMANN	1925	Teigitsch	Voitsberg	Styria	PG		R	26	85	12	320 / 69	H	160 / 40 } 200	V / L	S T E W E A G	S T E W E A G	Steirisches Wasserbausyndikat	6
7	SPULLERSEE SOUTH	1925 / 1965 a)	tr. Alfenz	Bludenz	Vorarlberg	PG		R	39	298	66	13100 / 535	H	37	L	Federal Railways	Federal Railways	Innerebner & Mayer	7
8	Spullersee North	1925 / 1965 a)	tr. Alfenz	Bludenz	Vorarlberg	PG		R	28	200	27	"	H	-	-	Federal Railways	Federal Railways	Innerebner & Mayer	8
9	PERNEGG	1927	Mur	Bruck/Mur	Styria	PG / TE		R	25	161	24 / 7	1650 / 210	H	1500	V	S T E W E A G	S T E W E A G	n	9
10	MIXNITZ	1931	Mur	Bruck/Mur	Styria	PG / TE		R	12	143	8 / 15	550 / 180	H	1500	V	S T E W E A G	S T E W E A G	n	10
11	PACK	1931	tr,Teigitsch	Köflach	Styria	PG / TE		R	33	183 / 90	39 / 5	5600 / 580	H	190	V	S T E W E A G	S T E W E A G	Alpenland Tiefbau	11
12	VERMUNT	1931	Ill	Schruns	Vorarlberg	PG		R	53	386	144	5700 / 360	H	129 / 46 } 175	L / V	V I W	Lahmayer & Co	J.V. Universale-Dyckerhoff et alia	12
13	ENZINGERBODEN	1940	Stubache	Mittersill	Salzburg	PG		R	29	68	11	250 / 60	H	140	L	Federal Railways	Federal Railways	Universale	13
14	KIRCHBICHL	1941	Inn	Wörgl	Tyrol	PG		S	17	106	8	3500 / 7000	H	2000	V	T I W A G	Innerebner & Mayer	Innerebner & Mayer	14
15	SCHWABECK	1943	Drau	Völkermarkt	Carinthia	PG		R	34	141	160	26000 / 1600	H	3500	V	Ö D K	Alpenelektrowerke Grzywieński	Ast & Co	15
16	MÖTSCHLACH	1944	Mur	Bruck/Mur	Styria	PG		S	19	53	10	540 / 170	H	1200	V	S T E W E A G	S T E W E A G	n	16
17	GERLOS	1945 (1964)	Gerlosbach	Zell/Ziller	Tyrol	VA (+PG)		R	39	69	10 +22	880 / 125	H	245	L/V	T K W	Alpenelektrowerke - TKW	Unionbau	17
18	LAVAMÜND	1945	Drau	Völkermarkt	Carinthia	PG		R	19	144	52	5000 / 1200	H	3500	V	Ö D K	Alpenelektrowerke	Ast & Co	18
19	STANING	1946	Enns	Steyr	Upper/Lower Austria	PG		S	27	160	72	14100 / 2500	H	3500	V	E K W	Siemens	Holzmann	19
20	BÜRG	1947	Kapruner Ache	Zell/See	Salzburg	PG		R	19	73	11	236 / 105	H	108	L/V	T K W	T K W	J.V. Rella et alia	20
21	MÜHLRADING	1948	Enns	Steyr	Upper/Lower Austria	PG		S	21	200	75	4900 / 1100	H	3500	V	E K W	Siemens	Holzmann	21
22	SILVRETTA	1948	Ill	Schruns	Vorarlberg	PG		R	80	432	407	39100 / 1310	H	104 / 61 } 165	L / V	V I W	V I W - Lahmayer	Kunz - Beton Monierbau	22
23	Biel	1948	Ill	Schruns	Vorarlberg	TE	ic	R/S	25	730	375	"	H	-	-	V I W	V I W - Lahmayer	Kunz - Beton Monierbau	23
24	HOLLERSBACH	1949	Hollersbach	Mittersill	Salzburg	TE	ie	R/S	16	87	16	200 / n	H	200	L/V	S A F E	S A F E - Interplan	Hinteregger - Fischer	24
25	SALZA	1949	tr. Enns	Liezen	Styria	VA		R	53	121	23	11000 / 800	H	140 / 8 } 148	L / V	S T E W E A G	S T E W E A G	Mayreder-Teiml & Spitzy	25

NOTES : *FOOTNOTES* :

a) Dam of 1925 raised by 4 m in 1965

DoKW Österreichische Donaukraftwerke AG
EKW Ennskraftwerke AG
KELAG Kärntner Elektrizitäts AG
NEWAG Niederösterreichische Elektrizitätswerke AG
ÖDK Österreichische Draukraftwerke AG
OKA Oberösterreichische Kraftwerke AG

SAFE Salzburger Aktiengesellschaft f. Elektrizitätswirtschaft
STEWEAG Steirische Wasserkraft- und Elektrizitäts AG
TIWAG Tiroler Wasserkraftwerke AG
TKW Tauernkraftwerke AG
VIW Vorarlberger Illwerke AG
VKW Vorarlberger Kraftwerke AG

J.V. Joint Venture
n no data available

REGISTRE DES BARRAGES EN AUTRICHE
REGISTER OF DAMS IN AUSTRIA

FOLIO No. 2

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	
LIGNE *LINE* No.	NOM DU BARRAGE *NAME OF DAM*	ANNÉE D'ACHÈVEMENT *YEAR OF COMPLETION*	SITUATION – *LOCATION*: COURS D'EAU *RIVER*	SITUATION – *LOCATION*: VILLE LA PLUS PROCHE *NEAREST CITY*	SITUATION – *LOCATION*: ÉTAT PROVINCE OU DÉPARTEMENT *STATE PROVINCE OR COUNTRY*	TYPE	SITUATION ET TYPE D'ÉTANCHÉITÉ *POSITION AND NATURE OF SEALING ELEMENT*	FONDATION *FOUNDATION*	HAUTEUR AU-DESSUS DE LA PLUS BASSE FONDATION *HEIGHT ABOVE LOWEST FOUNDATION* (m)	LONGUEUR DE CRÊTE *LENGTH OF CREST* (m)	VOLUME DU BARRAGE *VOLUME CONTENT OF DAM* (10^3 m^3)	CAPACITÉ TOTALE DU RÉSERVOIR / SURFACE DU RÉSERVOIR *GROSS CAPACITY OF RESERVOIR / RESERVOIR AREA* (10^3 m^3) / (10^3 m^2)	DESTINATION *PURPOSE*	CAPACITÉ MAXIMALE DES ÉVACUATEURS *MAXIMUM DISCHARGE CAPACITY OF SPILLWAYS* (m^3/s)	TYPE DES ÉVACUATEURS *TYPE OF SPILLWAYS*	PROPRIÉTAIRE *OWNER*	BUREAU D'ÉTUDES *ENGINEERING BY*	CONSTRUCTEUR *CONSTRUCTION BY*	LIGNE *LINE* No.
1	TERNBERG	1949	Enns	Steyr	Upper Austria	PG		R	29	140	95	5900 / 700	H	2500	V	E K W	Stahlwerke Linz (VÖEST)	Ast & Co	1
2	GROSSRAMING	1950	Enns	Weyer	Upper Austria	PG		R	41	122	225	13600 / 1220	H	2420	V	E K W	Siemens	Kunz - Rella	2
3	HIERZMANN	1950	Teigitsch	Köflach	Styria	VA		R	58	172	43	7600 / 520	H	180 / 20) 200	L / V	S T E W E A G	S T E W E A G	Ast - Universale	3
4	RANNA	1950	Ranna	Rohrbach	Upper Austria	VA		R	45	126	32	2350 / 300	H	160	L	O K A	Beurle - Grengg	Ferro Betonit - Teiml & Spitzy	4
5	BÄCHENTAL	1950	Dürrach	Jenbach	Tyrol	VA		R	34	70	3	none	H	150	L	T I W A G	T I W A G	Ast & Co	5
6	LIMBERG	1951	Kapruner Ache	Zell/See	Salzburg	VA		R	120	357	446	86000 / 1530	H	90 / 26) 116	V / L	T K W	T K W	J.V. Rella et alia	6
7	MÖLL	1952	Möll	Heiligenblut	Carinthia	VA		R	93	164	35	3800 / 210	H	64)) 274	V	T K W	T K W	Porr	7
8	Margaritze	1952	Möll	Heiligenblut	Carinthia	PG		R	39	170	33	"	H	210)	L	T K W	T K W	Porr	8
9	THURNBERG	1952	Kamp	Horn	Lower Austria	PG / TE	fc	R/S	25	248	23) / 46) 69	2500 / 800	H	500	V	N E W A G	Siemens Bautechnik	Mayreder Kraus	9
10	WEISSEE	1952	tr. Stubache	Mittersill	Salzburg	PG		R	39	235	64	16000 / 500	H	23	L	Federal Railways	Federal Railways	Kunz	10
11	DOBRA	1953	Kamp	Zwettl	Lower Austria	VA		R	52	234	90	21000 / 1500	H	500 / 80	L / V	N E W A G	Siemens Bautechnik	Rella	11
12	ROSENAU	1953	Enns	Steyr	Upper Austria	PG		R	25	140	100	6800 / 1000	H	2575	V	E K W	E K W	Mayreder-Hamberger-Ast	12
13	WIEDERSCHWING	1953	Weissenbach	Villach	Carinthia	VA		R	30	74	8	1200 / n	H	108	L/V	K E L A G	K E L A G	Mayreder, Teiml & Spitzy	13
14	BRAUNAU-SIMBACH +)	1954	Inn	Braunau	Upper Austria/Bavaria	PG		S	22	240	140	38000 / 8500	H	6200	V	Österr.-Bayer. Kraftwerke AG	Ö B K	J.V. Ast & Co - Sager Wörner et alia	14
15	GSTATTERBODEN	1955	Enns	Hieflau	Styria	PG		S	18	45	9	840 / 170	H	1000	V	S T E W E A G	S T E W E A G	Mayreder, Teiml & Spitzy	15
16	MOOSER	1955	Kapruner Ache	Zell/See	Salzburg	PG		R	107	494	665	87000 / 1660	H	130)) 160	V	T K W	T K W	J.V. Rella et alia	16
17	Drossen	1955	Kapruner Ache	Zell/See	Salzburg	VA		R	112	357	355	"	H	30)	V	T K W	T K W	J.V. Rella et alia	17
18	JOCHENSTEIN +)	1956	Danube	Engelhartszell	Upper Austria/Bavaria	PG		R	20	383	440	50000 / 7500	H.N	11000	V	Donaukraftwerk Jochenstein	D K J	J.V. Philipp Holzmann - - Rella et alia	18
19	OTTENSTEIN	1956	Kamp	Zwettl	Lower Austria	VA		R	69	240	124	73000 / 4290	H	436	V	N E W A G	Siemens Bautechnik	J.V. Rella et alia	19
20	RUNSERAU	1956	Inn	Landeck	Tyrol	PG		R/S	16	52	14	900 / 3000	H	750	V	T I W A G	T I W A G	Rella - Hinteregger - Polensky - Unionbau	20
21	GROSSER MÜHLDORFERSEE	1957	tr. Möll	Spittal	Carinthia	PG		R	46	433	153	7700 / 210	H	13 / 4) 17	L / V	Ö D K	Ö D K	J.V. Universale et alia	21
22	ROTGÜLDENSEE	1957	tr. Mur	St. Michael	Salzburg	ER	ia	R/S	18	112	35	3000 / 290	H	61	L/V	S A F E	S A F E - Posch	Polensky Zöllner - Heinz - - Ferro Betonit	22
23	AMERSEE	1958	tr. Salzach	Mittersill	Salzburg	PG		R	30	162	20	5500 / 206	H	8	L	Federal Railways	Federal Railways	Kunz - Unionbau	23
24	FREIBACH	1958	tr. Drau	Ferlach	Carinthia	TE	ie	R/S	41	150	235	5500 / 390	H	200 / 8 } 208	L / V	K E L A G	K E L A G	Isola - Weyss & Freitag	24
25	HOCHALMSEE	1958	tr. Möll	Spittal	Carinthia	PG / TE	ic	R	24	237 / 120	29 / 5	4100 / 160	H	15	L/V	Ö D K	Ö D K	J.V. Universale et alia	25

NOTES : *FOOTNOTES*

+) Border Dam Austria/Fed.Rep.Germany, owned by bi-national power company.

DoKW Österreichische Donaukraftwerke AG
EKW Ennskraftwerke AG
KELAG Kärntner Elektrizitäts AG
NEWAG Niederösterreichische Elektrizitätswerke AG
ÖDK Österreichische Draukraftwerke AG
OKA Oberösterreichische Kraftwerke AG

SAFE Salzburger Aktiengesellschaft f. Elektrizitätswirtschaft
STEWEAG Steirische Wasserkraft- und Elektrizitäts AG
TIWAG Tiroler Wasserkraftwerke AG
TKW Tauernkraftwerke AG
VIW Vorarlberger Illwerke AG
VKW Vorarlberger Kraftwerke AG

J.V. Joint Venture
n no data available

REGISTRE DES BARRAGES EN AUTRICHE
REGISTER OF DAMS IN AUSTRIA

1 LIGNE / *LINE* No.	2 NOM DU BARRAGE / *NAME OF DAM*	3 ANNÉE D'ACHÈVEMENT / *YEAR OF COMPLETION*	4 SITUATION – *LOCATION*: COURS D'EAU / *RIVER*	5 VILLE LA PLUS PROCHE / *NEAREST CITY*	6 ÉTAT PROVINCE OU DÉPARTEMENT / *STATE PROVINCE OR COUNTRY*	7 TYPE	8 SITUATION ET TYPE D'ÉTANCHÉITÉ / *POSITION AND NATURE OF SEALING ELEMENT*	9 FONDATION / *FOUNDATION*	10 HAUTEUR AU-DESSUS DE LA PLUS BASSE FONDATION / *HEIGHT ABOVE LOWEST FOUNDATION* (m)	11 LONGUEUR DE CRÊTE / *LENGTH OF CREST* (m)	12 VOLUME DU BARRAGE / *VOLUME CONTENT OF DAM* (10^3 m^3)	13 CAPACITÉ TOTALE DU RÉSERVOIR / SURFACE DU RÉSERVOIR / *GROSS CAPACITY OF RESERVOIR / RESERVOIR AREA* (10^3 m^3) / (10^3 m^2)	14 DESTINATION / *PURPOSE*	15 CAPACITÉ MAXIMALE DES ÉVACUATEURS / *MAXIMUM DISCHARGE CAPACITY OF SPILLWAYS* (m^3/s)	16 TYPE DES ÉVACUATEURS / *TYPE OF SPILLWAYS*	17 PROPRIÉTAIRE / *OWNER*	18 BUREAU D'ÉTUDES / *ENGINEERING BY*	19 CONSTRUCTEUR / *CONSTRUCTION BY*	LIGNE / *LINE* No.
1	KLEINER MÜHLDORFERSEE	1958	tr. Möll	Spittal	Carinthia	PG		R	41	159	60	2800/110	H	16	L/V	Ö D K	Ö D K	J.V. Universale et alia	1
2	LÜNERSEE	1958	tr. Ill	Bludenz	Vorarlberg	PG		R	30	380	41	94000/1530	H	12)/20) 32	L/V	V I W	V I W	Kunz & Co	2
3	RADLSEE	1958	tr. Lieser	Gmünd	Carinthia	ER	ic	R	16	212	22	2600/120	H	21	L/V	Ö D K	Ö D K	J.V. Porr et alia	3
4	SALZPLATTEN	1958	tr. Salzach	Mittersill	Salzburg	PG		R	17	88	5	1100/50	H	4	L	Federal Railways	Federal Railways	Kunz - Unionbau	4
5	LUTZ	1959	Lutz	Bludenz	Vorarlberg	PG		R	19	40	4	129/60	H	570	V	V K W	V K W	Hinteregger	5
6	YBBS-PERSENBEUG	1959	Danube	Ybbs	Lower/Upper Austria	PG		R	25	437	690	74000/10000	H,N	11200	V	D o K W	D o K W	J.V. Porr-Rella et alia	6
7	ESSLING	1960	Enns	Hieflau	Styria	PG		R	19	44	10	950/180	H	1400	V	S T E W E A G	S T E W E A G	Mayreder - Teiml & Spitzy	7
8	EDLING	1962	Drau	Völkermarkt	Carinthia	PG		R	36	104	168	83000/10500	H	3500	V	Ö D K	Ö D K	J.V. Ast - Mayreder et alia	8
9	LOSENSTEIN	1962	Enns	Steyr	Upper Austria	PG		R	30	102	86	6600/730	H	2500	V	E K W	E K W	Unionbau	9
10	SCHÄRDING-NEUHAUS +)	1962	Inn	Schärding	Upper Austria/Bavaria	PG		S	25	241	168	38000/5900	H	6800	V	Österr. Bayerische Kraftwerke AG	Ö B K	J.V. Rella-Holzmann et alia	10
11	DIESSBACH	1963	tr. Saalach	Saalfelden	Salzburg	ER	fa	R	36	204	165	5000/250	H	61)/6) 67	L/V	S A F E	S A F E	J.V. Hinteregger et. alia	11
12	ASCHACH	1964	Danube	Eferding	Upper Austria	PG		R	34	398	1050	114000/9500	H,N	8920	V	D o K W	D o K W	J.V. Rella - Porr - Dyckerhoff et alia	12
13	GRALLA	1964	Mur	Leibnitz	Styria	PG		S	18	91	23	1300/320	H	1250	V	S T E W E A G	S T E W E A G	Porr - Hinteregger	13
14	GEPATSCH	1965	Faggenbach	Landeck	Tyrol	ER	ie	R/S	153	600	7100	140000/2600	H	250)/75) 325	L/V	T I W A G	T I W A G	J.V. Porr-Schaffir Muglin - Hochtief et alia	14
15	GROSSREIFLING	1965	Enns	Hieflau	Styria	PG		R/S	20	45	11	1350/220	H	1400	V	S T E W E A G	S T E W E A G	Rella - Mayreder - Teiml & Spitzy	15
16	KOPS	1965	tr. Ill	Schruns	Vorarlberg	VA/PG		R	122/43	400) 214) 614	485) 178) 663	44600/1000	H	42)/60) 102	L/V	V I W	V I W - Lombardi&Gellera	J.V. Porr-Losinger et alia	16
17	THURNSDORF	1965	Enns	Steyr	Upper/Lower Austria	PG		S	21	94	32	6400/n	H	3500	V	E K W	E K W	Mayreder - Hamberger- Unionbau	17
18	DURLASSBODEN	1966	Gerlosbach	Zell/Ziller	Tyrol	TE	ie	R/S	83	470	2520	53500/1880	H	200)/45) 245	V	T K W	T K W	J.V. Porr et alia	18
19	PASSAU-INGLING +)	1966	Inn	Passau	Upper Austria/Bavaria	PG		R	22	242	135	22000/3900	H	7400	V	Österr. Bayerische Kraftwerke AG	Ö B K	J.V. Rella-Holzmann et alia	19
20	GARSTEN	1967	Enns	Steyr	Upper Austria	PG		R/S	23	89	65	3700/600	H	2600	V	E K W	E K W	Mayreder - Hamberger - Unionbau	20
21	RAGGAL	1967	Lutz	Bludenz	Vorarlberg	PG		R	48	105	42	2400/160	H	615	V	V K W	V K W	Kunz & Co	21
22	WANDAU	1967	Enns	Hieflau	Styria	PG		R/S	21	45	11	1200/150	H	1100	V	S T E W E A G	S T E W E A G	Rella - Mayreder - Teiml & Spitzy	22
23	STILLUP (EBERLASTE)	1968	tr. Zemm	Mayrhofen	Tyrol	TE	ia	S	28	480	790	8200/600	H	450)/55) 505	L/V	T K W	T K W	J.V. Oberranzmeyer, Strabag et alia	23
24	FEISTRITZ	1968	Drau	Klagenfurt	Carinthia	PG		R/S	38	95	160	50000/4800	H	3100	V	Ö D K	Ö D K	Mayreder - Porr - Isola Soravia	24
25	WALLSEE	1968	Danube	Grein	Upper/Lower Austria	PG		R	32	470	905	53000/7800	H,N	8600/2600	V/L	D o K W	D o K W	J.V. Mayreder-Rella-Porr-Hofman Maculan et alia	25

NOTES :
FOOTNOTES :

+) Border Dam Austria/Fed.Rep.Germany, owned by bi-national power company.

DoKW Österreichische Donaukraftwerke AG
EKW Ennskraftwerke AG
KELAG Kärntner Elektrizitäts AG
NEWAG Niederösterreichische Elektrizitätswerke AG
ÖDK Österreichische Draukraftwerke AG
OKA Oberösterreichische Kraftwerke AG

SAFE Salzburger Aktiengesellschaft f. Elektrizitätswirtschaft
STEWEAG Steirische Wasserkraft- und Elektrizitäts AG
TIWAG Tiroler Wasserkraftwerke AG
TKW Tauernkraftwerke AG
VIW Vorarlberger Illwerke AG
VKW

J.V. Joint Venture
n no data available

REGISTRE DES BARRAGES EN AUTRICHE
REGISTER OF DAMS IN AUSTRIA

FOLIO No. 4

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	
LIGNE *LINE* No.	NOM DU BARRAGE *NAME OF DAM*	ANNÉE D'ACHÈVEMENT *YEAR OF COMPLETION*	SITUATION – *LOCATION*: COURS D'EAU *RIVER*	VILLE LA PLUS PROCHE *NEAREST CITY*	ÉTAT PROVINCE OU DÉPARTEMENT *STATE PROVINCE OR COUNTRY*	TYPE	SITUATION ET TYPE D'ÉTANCHÉITÉ *POSITION AND NATURE OF SEALING ELEMENT*	FONDATION *FOUNDATION*	HAUTEUR AU-DESSUS DE LA PLUS BASSE FONDATION *HEIGHT ABOVE LOWEST FOUNDATION* (m)	LONGUEUR DE CRÊTE *LENGTH OF CREST* (m)	VOLUME DU BARRAGE *VOLUME CONTENT OF DAM* (10^3 m³)	CAPACITÉ TOTALE DU RÉSERVOIR / SURFACE DU RÉSERVOIR *GROSS CAPACITY OF RESERVOIR / RESERVOIR AREA* (10^3 m³) / (10^3 m²)	DESTINATION *PURPOSE*	CAPACITÉ MAXIMALE DES ÉVACUATEURS *MAXIMUM DISCHARGE CAPACITY OF SPILLWAYS* (m³/s)	TYPE DES ÉVACUATEURS *TYPE OF SPILLWAYS*	PROPRIÉTAIRE *OWNER*	BUREAU D'ÉTUDES *ENGINEERING BY*	CONSTRUCTEUR *CONSTRUCTION BY*	LIGNE *LINE* No.
1	GMUNDEN	1969	Traun	Gmunden	Upper Austria	PG		R	21	103	35	5140 / 25700	H	1300	V	O K A	Siemens-Bautechnik	Hinteregger-Stern & Hafferl	1
2	WEYER	1969	Enns	Weyer	Upper Austria	PG		R	26	54	70	5400 / 700	H	2280	V	E K W	E K W	Mayreder-Hamberger-Union	2
3	SCHLEGEIS	1971	tr.Zemmbach	Mayrhofen	Tyrol	VA		R	131	725	960	129000 / 2200	H	324) / 146) 470	L / V	T K W	T K W	J.V. Rella et alia	3
4	URSTEIN	1971	Salzach	Hallein	Salzburg	PG		S	19	85	n	2200 / 560	H	2730	V	S A F E	S A F E - Flögl	Hinteregger-Porr-Heinz	4
5	WURTEN	1971	tr.Fragant	Obervellach	Carinthia	TE	fa	R/S	42	282	265	2800 / 220	H	160) / 60) 220	L / V	K E L A G	K E L A G	Negrelli	5
6	SCHÖNAU	1972	Enns	Weyer	Upper Austria	PG		R	25	95	60	3300 / 400	H	2240	V	E K W	E K W	Hinteregger-Ferro Betonit	6
7	OSCHENIKSEE	1972 1978b)	tr. Fragant	Obervellach	Carinthia	ER	fa	R/S	116	530	2250	33000 / 430	H	6	L	K E L A G	K E L A G	Soravia-Isola-Strabag	7
8	OTTENSHEIM	1973	Danube	Linz	Upper Austria	PG		R	31	408	800	34000 / 5000	H,N	5940 / 2980	V / L	D o K W	D o K W	J.V. Mayreder-Rella-Porr-Hofman Maculan et alia	8
9	ST. MARTIN	1973	Drau	Villach	Carinthia	PG		R/S	26	73	64	19000 / 2200	H	3100	V	Ö D K	Ö D K	J.V. Mayreder-Porr et alia	9
10	TAUERNMOOS a)	1973	Stubache	Mittersill	Salzburg	PG		R	53	1100	250	57000 / 1850	H	108) / 32) 140	L / V	Federal Railways	Federal Railways	J.V. Stuag et alia	10
11	GABERSDORF	1974	Mur	Leibnitz	Styria	PG		S	18	85	24	1600 / 430	H	1250	V	S T E W E A G	S T E W E A G	Hinteregger - Teiml & Spitzy-Porr-Universale	11
12	GALGENBICHL	1974	Malta	Gmünd	Carinthia	TE / PG	fa	R	50 / 15	115 / 85	65 / 7	4800 / 270	H	30) / 196) 226	V / L	Ö D K	Ö D K	Oberranzmeyer	12
13	GROSSEE	1974 1978c)	tr. Möll	Heiligenblut	Carinthia	ER	fa	R	57	445	740	1330 / 310	H	8	L	K E L A G	K E L A G	Beyer - Strabag	13
14	HOCHWURTEN	1974 1979c)	tr. Fragant	Obervellach	Carinthia	TE	fa	R/S	55	260	600	1270 / 425	H	21	L	K E L A G	K E L A G	Soravia - Isola- Strabag	14
15	FERLACH	1975	Drau	Ferlach	Carinthia	PG		R/S	40	97	93	33000 / 3900	H	3300	V	Ö D K	Ö D K	J.V. Mayreder-Porr et alia	15
16	GÖSSKAR	1975	tr. Malta	Gmünd	Carinthia	TE	fa	R/S	55	260	540	2000 / 120	H	96) / 38) 134	L / V	Ö D K	Ö D K	Rella - Strabag	16
17	KLAUS	1975	Steyr	Kirchdorf	Upper Austria	VA		R	55	188	39	12600 / 900	H,C	340) / 418) 758	L / V	E K W	E K W - T K W	Ferro Betonit-Held & Franke	17
18	LATSCHAU	1975	tr. Ill	Schruns	Vorarlberg	TE	fa	R/S	50	500	865	2300 / 100	H	75	L/V	V I W	V I W	J.V. Hilti-Jäger-Innerebner-Kunz	18
19	ALTENWÖRTH	1976	Danube	Tulln	Lower Austria	PG		R	37	457	1280	93000 / 11000	H,N	9300 / 1900	V / L	D o K W	D o K W	J.V. Mayreder-Rella-Porr-Hofman Maculan et alia	19
20	KÖLNBREIN	1977	Malta	Gmünd	Carinthia	VA		R	200	626	1580	205000 / 2550	H	138) / 50) 188	L / V	Ö D K	Ö D K - T K W	J.V. Porr et alia	20
21	OBERVOGAU	1977	Mur	Leibnitz	Styria	PG		R	17	93	23	1100 / 370	H	1250	V	S T E W E A G	S T E W E A G	Tiefbau AG	21
22	ROTLECH	1977	tr. Lech	Reutte	Tyrol	PG / ER	ic	R	32	80) / 48) 128	14 / 11	1260 / n	H	200	V	Elektrizitätswerk Reutte	Siemens - Bautechnik	Storf	22
23	BOLGENACH	1978	Bolgenach	Bregenz	Vorarlberg	TE	ie	R/S	92	240	1200	8350 / 240	H	530	V	V K W	V K W - V I W	Hilti Jehle-Jäger-Kunz Universale	23
24	SÖLK	1978	Grosse Sölk	Schladming	Styria	VA		R	39	128	17	1700 / 130	H	200	L	S T E W E A G	S T E W E A G - T K W	Mayreder - Ast	24
25	SUBERSACH	1978	Subersach	Bregenz	Vorarlberg	PG		R	15	95	8	150 / 30	H	285	L	V K W	V K W - V I W	Oberranzmeyer	25

NOTES :
FOOTNOTES

a) replaced and submerged earlier dam of same name but 20m lower, built from 1926 to 1929

b) stepwise construction as planned: first dam of 1972 raised in 1973, 1976 and 1979 by 7 + 20 + 16m

c) [illegible] 1978/1979

DoKW Österreichische Donaukraftwerke AG
EKW Ennskraftwerke AG
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NEWAG Niederösterreichische Elektrizitätswerke AG
ÖDK Österreichische Draukraftwerke AG
OKA Oberösterreichische Kraftwerke AG

SAFE Salzburger Aktiengesellschaft f. Elektrizitätswirtschaft
STEWEAG Steirische Wasserkraft- und Elektrizitäts AG
TIWAG Tiroler Wasserkraftwerke AG
TKW Tauernkraftwerke AG
VIW Vorarlberger Illwerke AG
VKW Vorarlberger Kraftwerke AG

J.V. Joint Venture
n no data available

REGISTRE DES BARRAGES EN AUTRICHE
REGISTER OF DAMS IN AUSTRIA

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	
LIGNE *LINE* No.	NOM DU BARRAGE *NAME OF DAM*	ANNÉE D'ACHÈVEMENT *YEAR OF COMPLETION*	SITUATION – *LOCATION* COURS D'EAU *RIVER*	VILLE LA PLUS PROCHE *NEAREST CITY*	ÉTAT PROVINCE OU DÉPARTEMENT *STATE PROVINCE OR COUNTRY*	TYPE	SITUATION ET TYPE D'ÉTANCHÉITÉ *POSITION AND NATURE OF SEALING ELEMENT*	FONDATION *FOUNDATION*	HAUTEUR AU-DESSUS DE LA PLUS BASSE FONDATION *HEIGHT ABOVE LOWEST FOUNDATION* (m)	LONGUEUR DE CRÊTE *LENGTH OF CREST* (m)	VOLUME DU BARRAGE *VOLUME CONTENT OF DAM* (10^3 m^3)	CAPACITÉ TOTALE DU RÉSERVOIR / SURFACE DU RÉSERVOIR *GROSS CAPACITY OF RESERVOIR / RESERVOIR AREA* (10^3 m^3) / (10^3 m^2)	DESTINATION *PURPOSE*	CAPACITÉ MAXIMALE DES ÉVACUATEURS *MAXIMUM DISCHARGE CAPACITY OF SPILLWAYS* (m^3/s)	TYPE DES ÉVACUATEURS *TYPE OF SPILLWAYS*	PROPRIÉTAIRE *OWNER*	BUREAU D'ÉTUDES *ENGINEERING BY*	CONSTRUCTEUR *CONSTRUCTION BY*	LIGNE *LINE* No.
1	ABWINDEN-ASTEN	1979	Danube	Mauthausen	Upper Austria	PG		R	31	395	850	46000/9500	H,N	8460/1040	V/L	D o K W	D o K W	J.V.Mayreder-Rella-Porr Hofman Maculan et alia	1
2	MARCHTRENK	1979	Traun	Wels	Upper Austria	PG		R	38	98	100	5800/720	H	2300	V	O K A	Siemens Bautechnik	J.V. Mayreder et alia	2
3	FINSTERTAL	1980	tr. Nederbach	Ötz	Tyrol	ER	ia	R/S	150	652	4500	60500/10400	H	20)/3) 23	V/L	T I W A G	T I W A G	J.V. Hochtief-Oberranzmeyer-Strabag et alia	3
4	LÄNGENTAL	1980	Nederbach	Ötz	Tyrol	TE	fa	R/S	37	418	400	3300/240	H	106)/20) 126	L/V	T I W A G	T I W A G	Mayreder-Universale-Illbau-Innerebner	4
5	NASSFELD	1980	Naßfeld. ache	Badgast.	Salzburg	PG		R	21	85	13	56/12	H	200/80 280	V	S A F E	SAFE-Siemens Bautechn.	Hinteregger-Porr-Universale	5
6	ANNABRÜCKE	1981	Drau	Klagenf.	Carinthia	PG		S	40	104	108	36000/3500	H	3300	V	Ö D K	Ö D K	J.V. Mayreder-Porr et alia	6
7	BOCKHARTSEE	1982	tr.Gasteiner Ache	Bad Gastein	Salzburg	ER	ic	R	33	240	228	14800/410	H	44)/12) 56	L/V	S A F E	SAFE - Siemens Bautechnik	Porr	7
8	BODENDORF	1982	Mur	Murau	Styria	PG		R	23	67	30	900/100	H	560	V	S T E W E A G	S T E W E A G	Hinteregger-Fritz	8
9	PAAL (Bodendorf)	1982	tr.Mur	Murau	Styria	VA/PG		R	39	118	20	300/35	H	300)/18) 318	V	S T E W E A G	S T E W E A G	Strabag-TS Bau-Stettin	9
10	MELK	1982	Danube	Melk	Lower Austria	PG		R/S	29	442	900	54000/7500	H,N	11170	V	D o K W	D o K W	J.V.Mayreder-Rella-Porr Hofman Maculan et alia	10
11	SPIELFELD	1982	Mur	Leibnitz	Styria	PG		S	19	99	27	1300/440	H	1660	V	S T E W E A G	S T E W E A G	Universale	11
12	TRAUN-PUCKING	1982	Traun	Linz	Upper Austria	PG		R	45	101	110	7600/830	H	2300	V	O K A	Siemens-Bautechnik	J.V. Mayreder et alia	12
13	WEINZÖDL	1982	Mur	Graz	Styria	PG		R	20	93	n	n	H	1800	V	Steiermärkische Elektrizitäts AG (STEG)	Suiselektra	Ast - Mayreder - VÖEST	13
14	ZIRMSEE	1983	tr.Möll	Heiligenblut	Carinthia	ER	fa	R	44	315	525	870/300	H	12	L	K E L A G	K E L A G	Strabag	14
15	BISCHOFSHOFEN	1984	Salzach	B'hofen	Salzburg	PG		S	21	82	36	1000/278	H	1356	V	S A F E + T K W	T K W	Hinteregger-Alpine	15
16	GREIFENSTEIN	1984	Danube	Stockerau	Lower Austria	PG		R/S	31	455	n	87000/10000	H,N	8640/2110	V/L	D o K W	D o K W	J.V.Mayreder-Rella-Porr Hofman Maculan et alia	16
17	VILLACH	1984	Drau	Villach	Carinthia	PG		R	27	88	52	3400/800	H	2500	V	Ö D K	Ö D K	J.V. Hamberger	17
18	KELLERBERG	1985	Drau	Villach	Carinthia	PG		S	27	88	45	3500/800	H	2500	V	Ö D K	Ö D K	Mayreder + Illbau	18
19	MELLACH	1985	Mur	Wildon	Styria	PG		S	20	87	26	1260/300	H	1650	V	S T E W E A G	S T E W E A G	Universale	19
20	ST. GEORGEN	1985	Mur	Murau	Styria	PG		R	23	56	25	450/85	H	560	V	S T E W E A G	S T E W E A G	Tiefbau	20
21	URREITING	1985	Salzach	St.Johann	Salzburg	PG		S	21	82	36	1100/273	H	1320	V	S A F E + T K W	T K W	Neue Reform Ast & Co. Hofman & Maculan,WS-Bau	21
22	VERWALL	1985	Rosanna	St.Anton	Tyrol	PG		R	31	57	12	250/24	H	190)/51) 241	L/V	E-Werk St. Anton	Siemens-Bautechnik	Jäger	22
23	ZILLERGRÜNDL	1986	Ziller	Mayrhofen	Tyrol	VA		R	186	506	1370	90000/1410	H	195)/45) 240	L/V	T K W	T K W	J.V. Hofman Maculan et alia	23
24	RABENSTEIN	c (1987)	Mur	Frohnleiten	Styria	PG		R	22	95	360	1500/300	H	1800	V	S T E G	Suiselektra	Hofman & Maculan	24
25	GINAU	c (1987)	tr.Wagrainer Ache	Wagrain	Salzburg	VA		R	33	88	11	85/10	H	90)/17) 107	L/V	O K A	O K A + FLÖGL	J.V. Hamberger, Hinteregger, Mayreder,Strabag	25

NOTES :
FOOTNOTES :

DoKW	Österreichische Donaukraftwerke AG	SAFE	Salzburger AG f. Elektrizitätswirtschaft	J.V.	Joint Venture
EKW	Ennskraftwerke AG	STEG	Steiermärkische Elektrizitäts AG	n	no data available
KELAG	Kärntner Elektrizitäts AG	STEWEAG	Steirische Wasserkraft- und Elektrizitäts AG		
NEWAG	Niederösterreichische Elektrizitätswerke AG	TIWAG	Tiroler Wasserkraftwerke AG		
ÖDK	Österreichische Draukraftwerke AG	TKW	Tauernkraftwerke AG		
OKA	Oberösterreichische Kraftwerke AG	VIW	Vorarlberger Illwerke AG		
		VKW	Vorarlberger Kraftwerke AG		

REGISTRE DES BARRAGES EN AUTRICHE
REGISTER OF DAMS IN AUSTRIA

FOLIO No. 6

1 LIGNE *LINE* No.	2 NOM DU BARRAGE *NAME OF DAM*	3 ANNÉE D'ACHÈVEMENT *YEAR OF COMPLETION*	4 SITUATION – *LOCATION*: COURS D'EAU *RIVER*	5 VILLE LA PLUS PROCHE *NEAREST CITY*	6 ÉTAT PROVINCE OU DÉPARTEMENT *STATE PROVINCE OR COUNTRY*	7 TYPE	8 SITUATION ET TYPE D'ÉTANCHÉITÉ *POSITION AND NATURE OF SEALING ELEMENT*	9 FONDATION *FOUNDATION*	10 HAUTEUR AU-DESSUS DE LA PLUS BASSE FONDATION *HEIGHT ABOVE LOWEST FOUNDATION* (m)	11 LONGUEUR DE CRÊTE *LENGTH OF CREST* (m)	12 VOLUME DU BARRAGE *VOLUME CONTENT OF DAM* (10^3 m^3)	13 CAPACITÉ TOTALE DU RÉSERVOIR / SURFACE DU RÉSERVOIR *GROSS CAPACITY OF RESERVOIR / RESERVOIR AREA* (10^3 m^3) / (10^3 m^2)	14 DESTINATION *PURPOSE*	15 CAPACITÉ MAXIMALE DES ÉVACUATEURS *MAXIMUM DISCHARGE CAPACITY OF SPILLWAYS* (m^3/s)	16 TYPE DES ÉVACUATEURS *TYPE OF SPILLWAYS*	17 PROPRIÉTAIRE *OWNER*	18 BUREAU D'ÉTUDES *ENGINEERING BY*	19 CONSTRUCTEUR *CONSTRUCTION BY*	LIGNE *LINE* No.
1	HALLEIN SOHLSTUFE	c (1987)	Salzach	Hallein	Salzburg	PG		S	19	145	55	310 / 230	H	2200	V	S A F E	S A F E + FLÖGL	Hinteregger-Alpine Polensky - Zöllner	1
2	LEBRING	c (1988)	Mur	Lebring	Styria	PG		R	24	98	630	1800 / 400	H	1800	V	S T E G	Suiselektra	J.V. Negrelli-Porr	2
3	PATERNION	c (1988)	Drau	Spittal	Carinthia	PG		S	27	88	45	3600 / 900	H	2500	V	Ö D K	Ö D K	Mayreder + Illbau	3
4	ST. VEIT	c (1988)	Salzach	Schwarzach	Salzburg	PG		S	21	82	36	520 / 155	H	1150	V	T K W	T K W	J.V.Hinteregger et alia	4
5	WALLNERAU	c (1989)	Salzach	Schwarzach	Salzburg	PG		S	28	65	30	425 / 70	H	1130	V	T K W	T K W	J.V.Hinteregger et alia	5
	SUPPLEMENTARY DATA 1991:																		
	FEISTRITZBACH	1990	Feistritzbach	Lavamünd	Carinthia	ER	ia	R	88	370	1600	22200 / 870	H	8	L	K E L A G	K E L A G	J.V.Züblin et alia	
	ROTGÜLDENSEE (new, replace 2/22)	1990	Mur	St.Michael	Salzburg	ER	ia	R	45	273	346	15600 / 430	H	104	L	S A F E	S A F E + FLÖGL	J.V.Hinteregger et alia	
	OBERAUDORF-EBBS	1992	Inn	Kufstein	Tyrol/ Bavaria	PG		S	35	88	150	4700 / 1200	H	2700	V	Ö B K	Ö B K	J.V.Porr-Züblin-Innerebner	

NOTES :
FOOTNOTES :

DoKW	Österreichische Donaukraftwerke AG
EKW	Ennskraftwerke AG
KELAG	Kärntner Elektrizitäts AG
NEWAG	Niederösterreichische Elektrizitätswerke AG
ÖDK	Österreichische Draukraftwerke AG
OKA	Oberösterreichische Kraftwerke AG
SAFE	Salzburger AG f. Elektrizitätswirtschaft
STEG	Steiermärkische Elektrizitäts AG
STEWEAG	Steirische Wasserkraft- und Elektrizitäts AG
TIWAG	Tiroler Wasserkraftwerke AG
TKW	Tauernkraftwerke AG
VIW	Vorarlberger Illwerke AG
VKW	Vorarlberger Kraftwerke AG
J.V.	Joint Venture
n	no data available

CONVERSION FACTORS

For changing International Standard Units to English-Pound

Denomination	SI	English
Length	1 m	3.281 ft
Area	1 m^2	10.76 ft^2
Volume	1 m^3	1.308 yd^3
	10^6 m^3 (1hm^3)	810.7 acre ft
Mass	1 kg	2.205 lb
Mass density	1000 kg/m^3	62.43 lb/ft^3
Unit weight	1 kN/m^3	6.243 lb/ft^3
Velocity	1 m/s	3.281 ft/s
Force	1 N	0.2248 lb
	1 kN	224.8 lb
	1 MN	112.4 US ton

Denomination	SI	English
Pressure/Stress	1 N/mm^2	145 psi
	1 MN/m^2	145 psi
	1 MPa	145 psi
Flow	1 m^3/s	35.31 ft^3/s
Energy/work	1 MJ	0.2778 kWh
Power	1 kW	1 kW
Cement content	100 kg/m^3	1.8 bags/yd^3
	55,6 kg/m^3	1 bag/yd^3

ABBREVIATIONS

a.s.l.	- above sea level
avg	- average
cal	- calorie
cm	- centimetre
dia.	- diameter
El.	- elevation
°C	- degrees Centigrade
g	- acceleration of gravity or gram
GN	- Giganewton (10^9 Newton)
GWh	- Gigawatt-hour (10^9-watt- hours)
ha	- hectare
hm^3	- cubic hectometre
hp	- horsepower
HQ 5000	- 5000 year flood
Hz	- Hertz
J	- Joule
kJ	- kilojoule
kg	- kilogram
kg/m^2	- kilogram per square metre
km	- kilometre
km^2	- square kilometre
km/h	- kilometre per hour
kN	- kilonewton
kPa	- kilopascal
kV	- kilovolt
kVA	- kilovolt-ampere
kW	- kilowatt
kWh	- kilowatt-hour
l/min	- liters per minute
l/s	- liters per second
l/s.km^2	- liters per second and square kilometre
log	- logarithm
m	- metre
m/s	- metre per second
m^2	- square metre
m^3	- cubic metre
m^3/s	- cubic metre per second
M	- Magnitude (Richter scale)
MCE	- Maximum Credible Earthquake
MN	- Meganewton (10^6 Newton)
MSK	- Medvedev - Sponheuer - Kárník - Scale 1964
mm	- millimetre
MM	- Modified Mercalli scale
MVA	- megavolt-ampere
MW	- megawatt
MWh	- megawatt-hour
N	- Newton
N/mm^2	- Newtons per square millimetre
Pa	- Pascal
pH	- potential Hydrogen
PMF	- Probable Maximum Flood
PMP	- Probable Maximum Precipitation
ppm	- parts per million
R,r	- radius
RCC	- roller compacted concrete
rpm	- revolutions per minute
s	- second
SI	- International System of Units
SIS	- Seismic Intensity Scale 1985
t	- ton
TWh	- Terawatt-hour (10^{12}-watt-hours)
%	- per cent
%o	- 0,1 percent
° d.H.	- degrees of hardness (German)

MMS

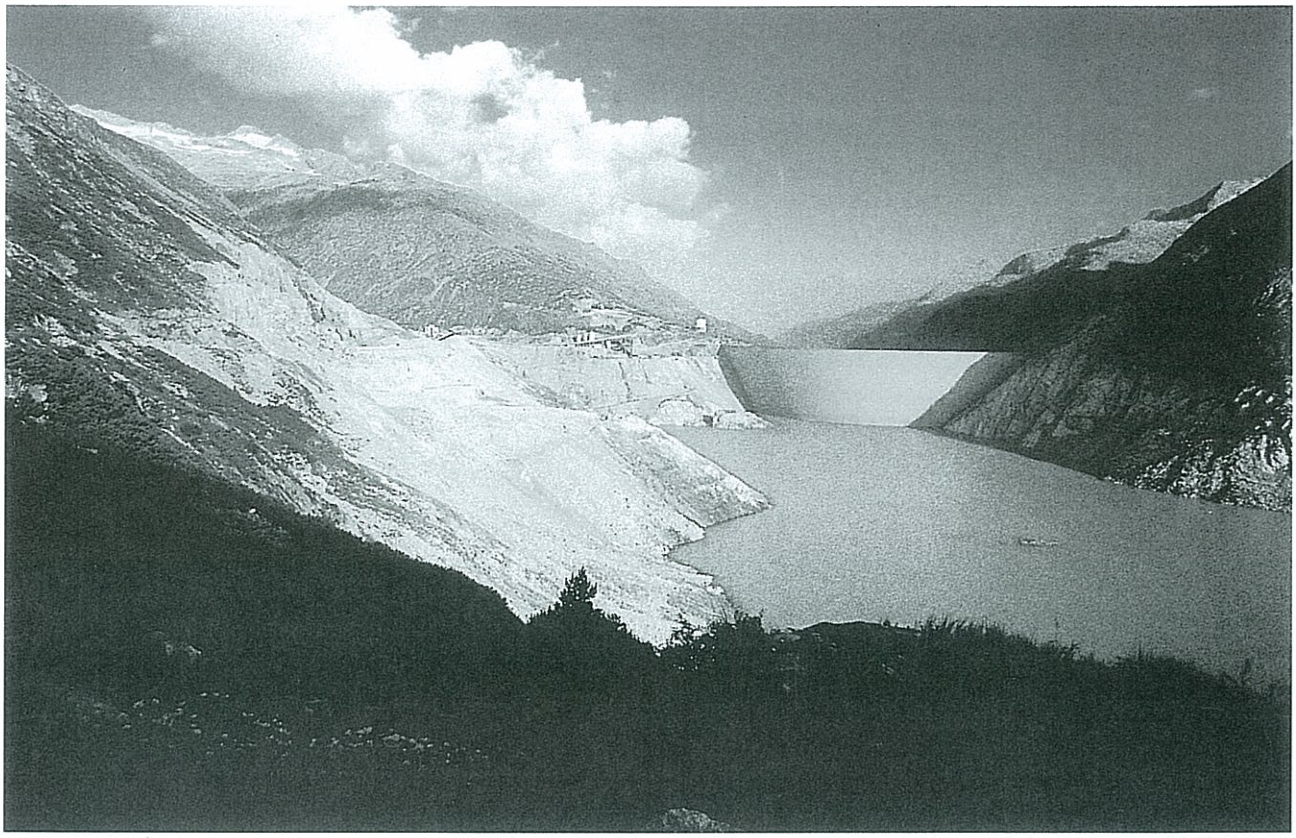
HOCH DAS GLAS UM S 0,90.

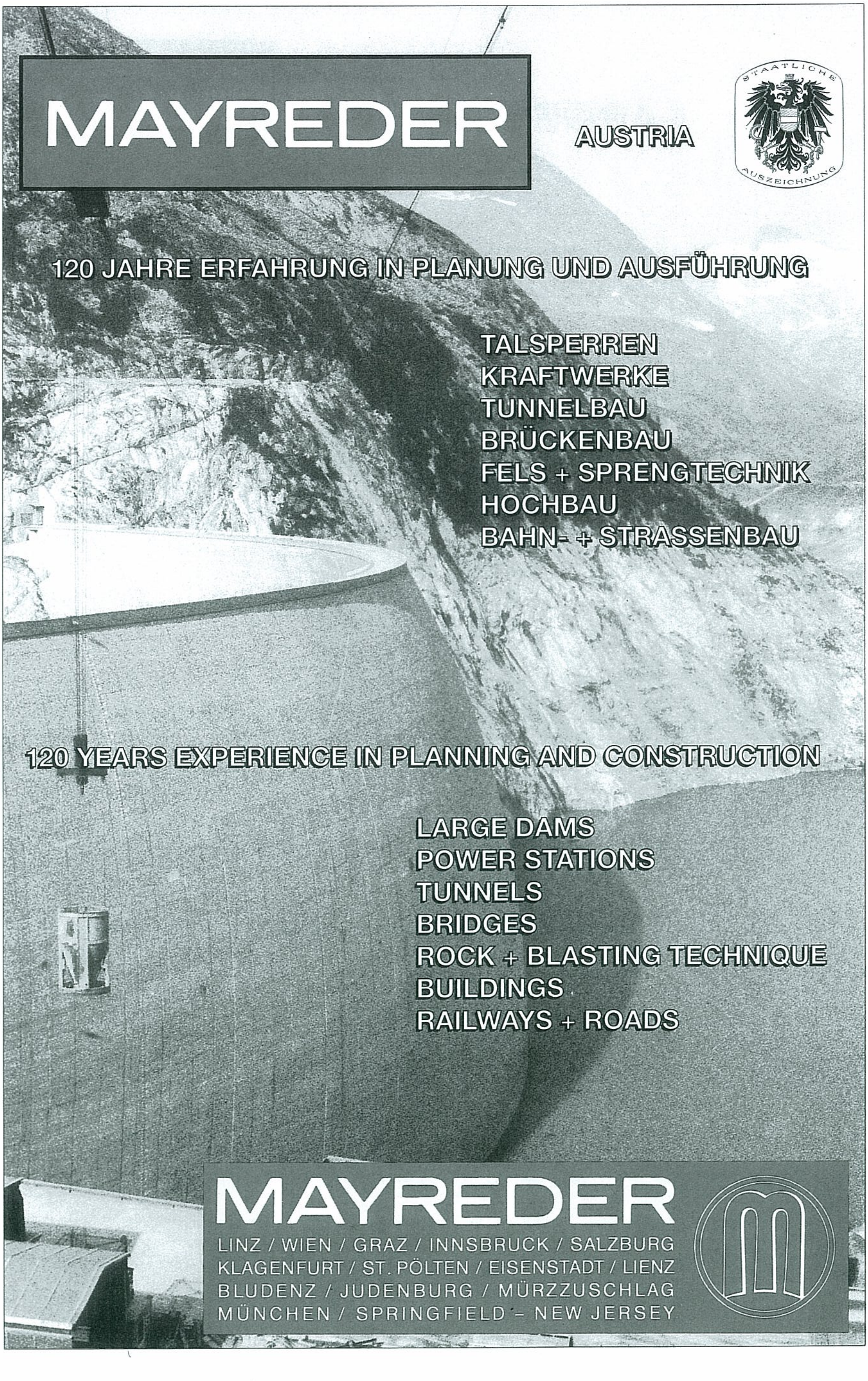
Wenn Weinkenner den trockenen Weißburgunder im stilechten, mundgeblasenen Glas goutieren, so hat das viel mit Energiesparen zu tun.*) Denn im modernen Elektro-Schmelzofen fallen pro Glas nur mehr S 0,90 an Stromkosten an – weniger als die Hälfte der Energiekosten im Vergleich zu früheren Ölöfen. Und anschließend kann jedes Glas, wie seit über 200 Jahren, in traditioneller Handarbeit gefertigt werden. Eines von vielen Beispielen, wie sinnvoller Einsatz von Strom den Energieverbrauch senkt: Denn Strom ist Energie zum Energiesparen. Haben Sie Fragen zu diesem Thema? Wir stehen Ihnen gerne zur Verfügung. Schreiben Sie einfach an den Verband der Elektrizitätswerke Österreichs, Brahmsplatz 3, 1041 Wien.

*) Wir gratulieren dem Gewinner des internationalen Wettbewerbes „Energie effizient" in der Kategorie 2 (Betriebe mit mehr als 100 Beschäftigten): Tiroler Glashütte GmbH, Claus Josef Riedel, 6330 Kufstein.

Ihr E-Werk

Hinweis des Wirtschaftsministers: Sparsamer Einsatz von Energie hilft unserer Umwelt.

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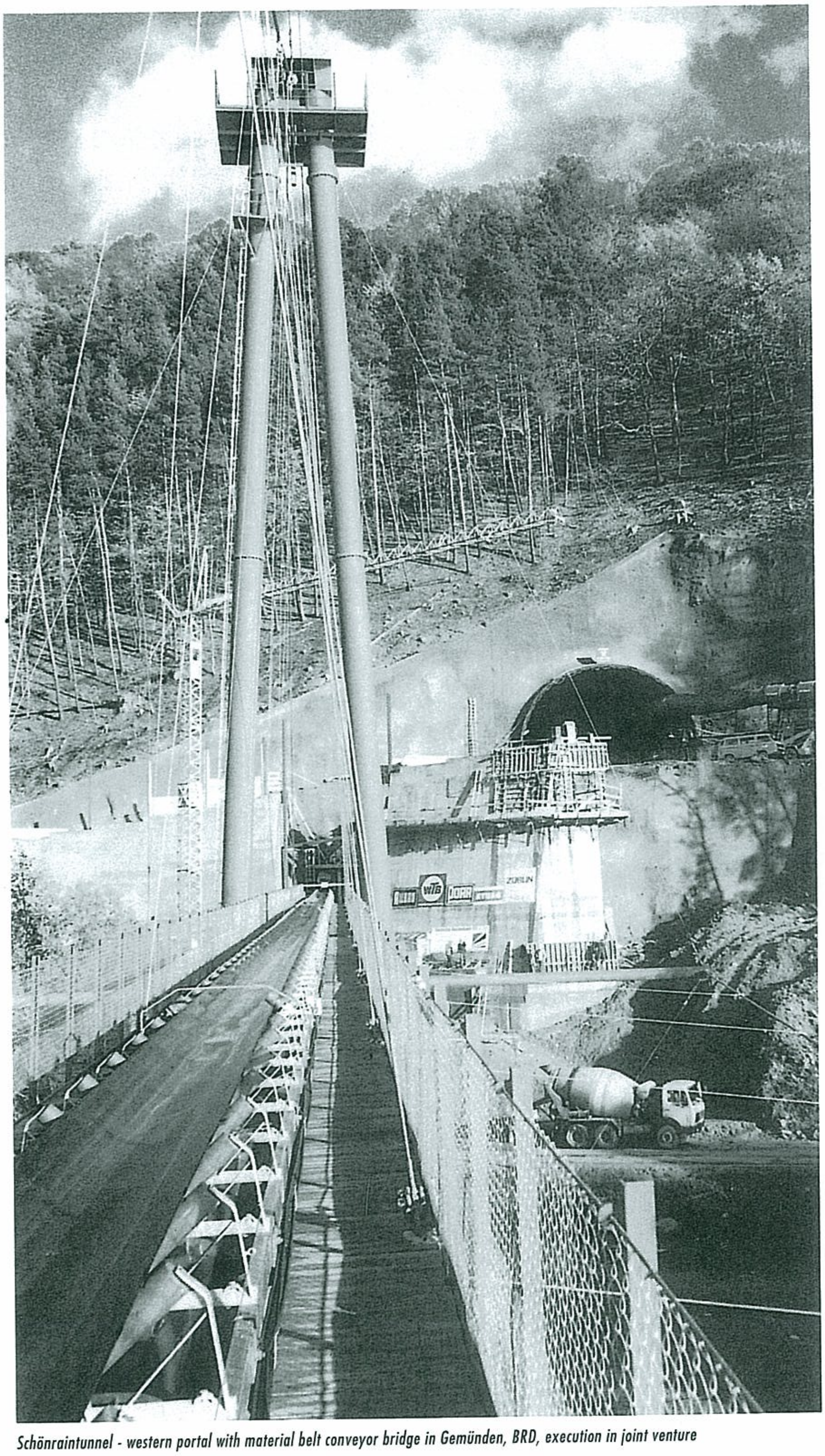

Schönraintunnel - western portal with material belt conveyor bridge in Gemünden, BRD, execution in joint venture

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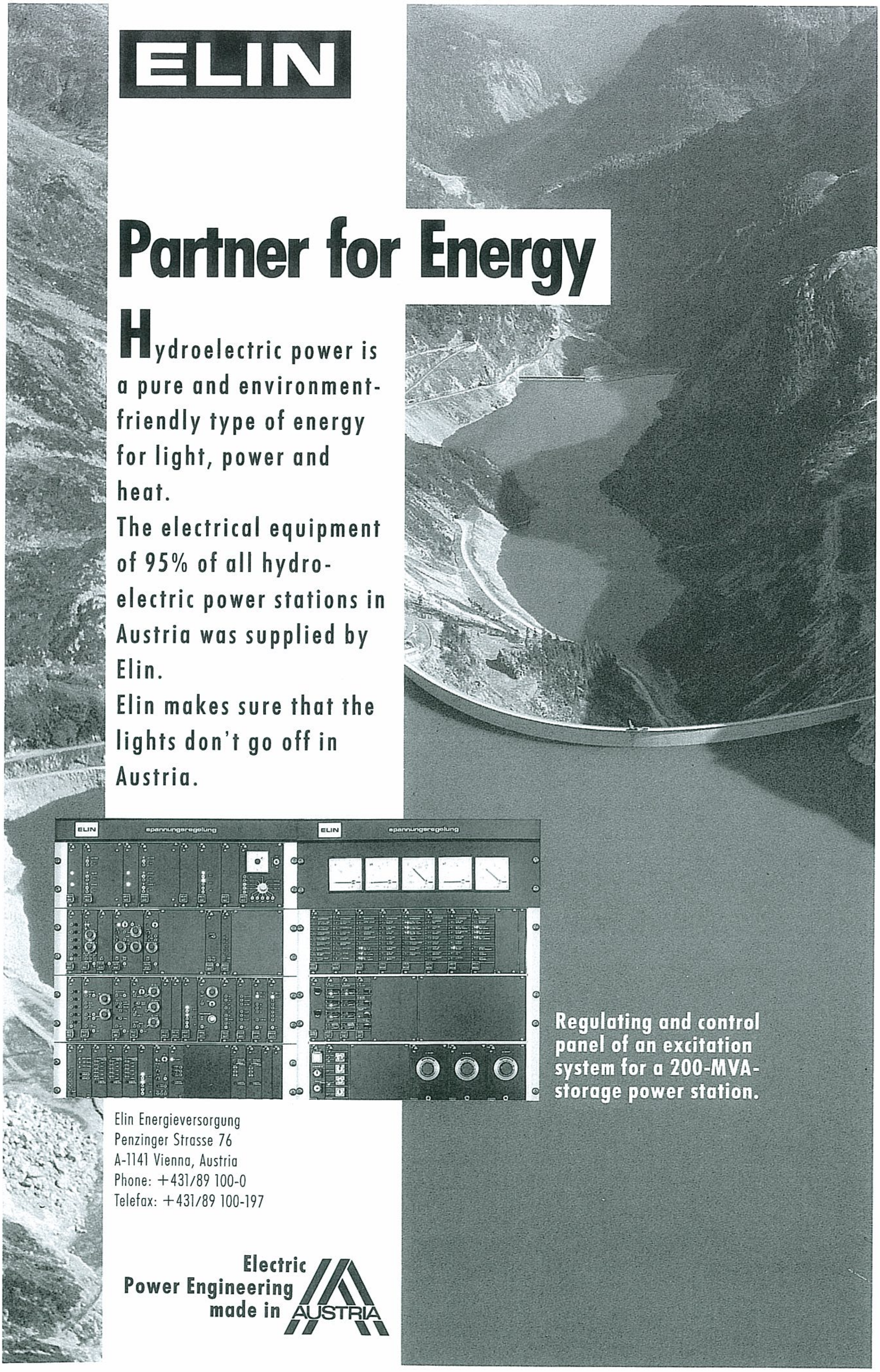
River power station Wald at Krimml.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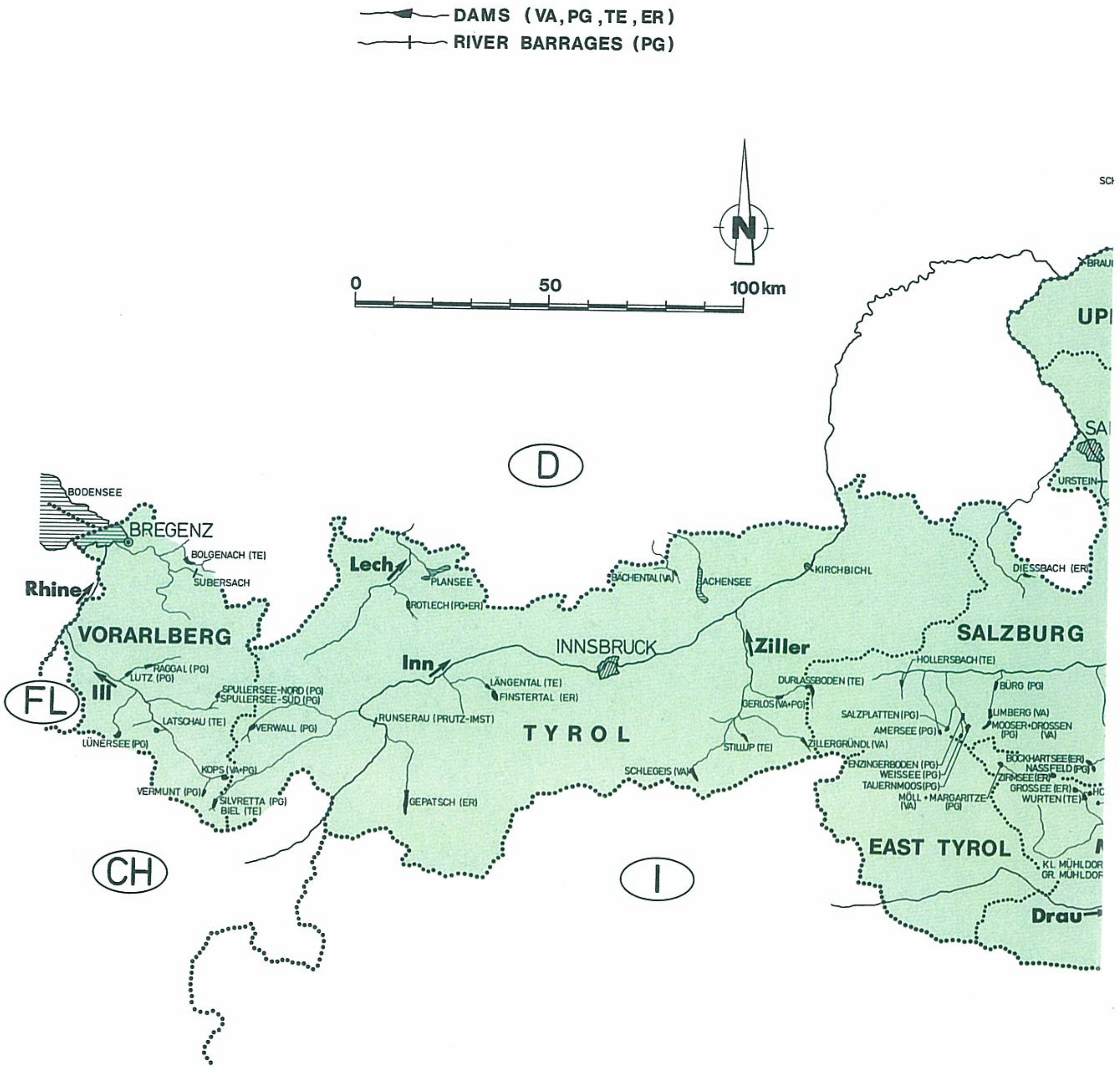

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