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IMPROVED DESIGN OF TUNNEL SUPPORTS: VOLUME 4 - TUNNELING PRACTICES IN AUSTRIA AND GERMANY

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16. Abstract
This five-volume report includes the results of an extensive research effort by the Massachusetts Institute of Technology to improve the design methodologies available to tunnel designers. The purpose of this report is to provide the tunneling profession with improved practical tools in the technical or design area, which provide more accurate representations of the ground-structure interaction in tunneling. The design methods range from simple analytical and empirical methods to sophisticated finite element techniques.

Volume 4 documents and evaluates extensive information gathered on tunnel construction practices in Austria and Germany, identifies differences compared to U.S. practices, and describes new developments. The objective was to assemble all available information about the economic, contractual, and technical aspects of tunneling in these countries. The cost information includes general cost data as well as specific costs for tunnels recently constructed, and discusses the reasons why tunneling there is often more economical and technically innovative than it is in the United States. The contractual information is based on a review of contractual standards and procedures in these countries and contractual arrangements for selected projects. The technical information includes general information on design philosophy and construction procedures, and detailed information on analytical and empirical methods and design aspects. A large number of transmountain tunnels in Austria and subway tunnel sections in Germany were visited, and many discussions were held with owner-authorities, design firms, and contractors.

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Austria; European Tunneling; Germany; Tunnel Contracts; Tunnel Costs; Tunnel Design; Tunnel Support; Tunneling; Tunnels and Tunneling; Underground Construction	Available to the public through the National Technical Information Service, Springfield, Virginia 22161

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PREFACE

This report is the fourth of five publications (the Executive Summary of this five-volume report was published in December, 1979) which include the results of an extensive research effort by the Massachusetts Institute of Technology (MIT) to improve the design methodologies available to tunnel designers. The contract, DOT-TSC-1489, was funded by the U.S. Department of Transportation (DOT) and was sponsored by the Urban Mass Transportation Administration's (UMTA) Office of Rail and Construction Technology. The contract was monitored by the Transportation Systems Center (TSC) Construction and Engineering Branch.

The objective of Volume 4 was to assemble all available information about the economic, contractual, and technical aspects of tunneling in Austria and Germany. The information includes general facts and figures about each aspect, as well as details about specific tunneling projects in these countries.

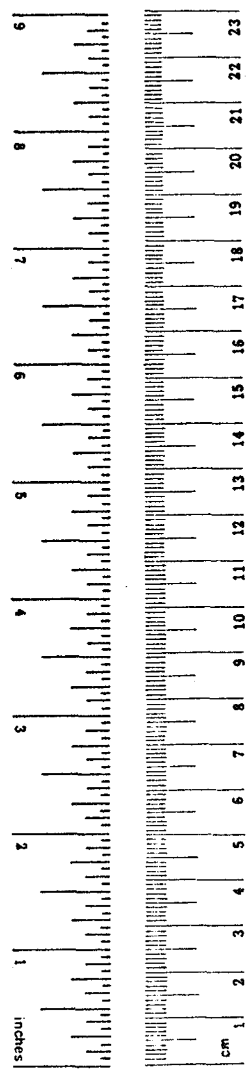
Outstanding and continuing coordination with our colleagues in Austria and Germany has enabled us to produce this collection of data relative to European tunnel construction, which will certainly be an invaluable tool to the U.S. tunneling community.

METRIC CONVERSION FACTORS

Approximate Conversions to Metric Measures

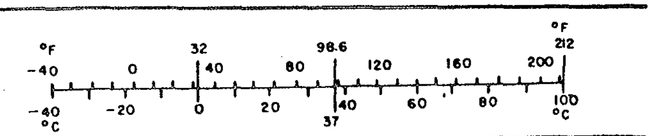
Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
		MASS (weight)		
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t
		VOLUME		
tsp	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

*1 in = 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.



Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
		AREA		
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
		MASS (weight)		
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
		VOLUME		
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



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1. INTRODUCTION

1.1 INTENT OF TRIP

One of the objectives of the research "Improved Design for Tunnel Supports" is to obtain detailed information on European tunneling practice in the technical, operational, contractual, and cost (economic) areas. This information will be needed for several purposes: (1) to show why European tunnel construction is frequently less expensive, faster and involves less litigation than in the U.S., (2) to show which characteristics of European tunneling practice could be applied and integrated with relative ease in U.S. practice and which characteristics could not, and (3) to provide detailed geotechnical and performance data from which an improved empirical design-construction approach can be developed. The same information will also be used to check analytical approaches. The necessary information was obtained by an extensive literature survey and by information gathering on the spot; the latter is necessary to get a feeling for the background which is usually not reported in the literature and to gain access to detailed data. In addition, contacts established during this trip were extensively used later to provide answers to new questions and supplemental information. The information-gathering trip concentrated on tunnels in Austria and Germany since highway and subway tunnel construction has been particularly active in these countries and since many innovations in tunneling have occurred there. The two countries also provide an ideal mix; shallow and deep lying tunnels, as well as a great variety of ground conditions. The information-gathering trip in Austria and Germany was conducted by Walter Steiner, Research Assistant, during the period of January 2, 1978 to February 3, 1978. W. Steiner's native language is German; thus detailed information could be easily collected and nuances in interviews properly interpreted.

1.2 STRUCTURE OF THIS VOLUME

Volume 4 consists of six sections and two appendixes. The sections in the main text present summaries of and, in particular, the evaluations of the corresponding information, while the appendices contain details like plans of design features, individual cost figures and records of interviews. Extensive referencing in the main text makes it possible for the reader to retrieve the detailed backup information from the appendices if so desired.

Section 2 (Trip Itinerary) gives an account of the agencies, firms, and contractors visited. The persons met and the topics discussed are also listed. In Section 3 the data on tunnel construction costs for subway and transmountain tunnels are presented. Data on the costs of subway tunnels were obtained in Germany; those on transmountain highway tunnels, in Austria. Section 4 presents contractual procedures as they are used in Germany and Austria. The pre-bidding, bidding, award, and execution phases in Austrian and German tunnel construction practices are presented. Section 5 discusses technical-operational aspects. Design procedures for subway tunnels in Germany and procedures for transmountain highway tunnels in Austria are described. Technical aspects of the official (the owner's) design are explained, followed by the requirements for alternate proposals which are routinely submitted, particularly in bidding for subway tunnels. Section 5 also summarizes detailed technical information regarding ground conditions, support placement, and observed performance of seven representative tunnels. In addition, unresolved technical problems are identified. Sections 3 through 5 are interrelated. For example, contractual aspects influence technical aspects which influence costs. On the other hand, costs, especially bid price, may influence contractual relations and technical procedures. Therefore, the areas of reciprocal influence and interrelations are pointed out. In particular, the great importance of interaction between owners, design engineers, and contractors during all stages of the tunnel construction will be shown. Section 6 concludes the main body of the report with an evaluation of the information and knowledge gained. Two appendices contain additional, detailed information. In Appendix A (Table 1.1) the information gathered during interviews with representatives of agencies, research institutes, design engineering firms and contractors is presented. Appendix B (Table 1.2) contains the information and data collected during the site visits.

TABLE 1.1 AGENCIES, CONTRACTORS, DESIGN ENGINEERS, AND RESEARCH INSTITUTES VISITED DURING INFORMATION-GATHERING TRIP

City	Name	Type	Data
Munich (Germany)	Subway Authority	G	Interview conducted
Innsbruck (Austria)	Beton- & Monierbau	C	Data on equipment and costs
	University: Prof. Seeber	ER	Research and case study reports
	University: Prof. Lessmann	CR	Interview conducted
	Arlberg Highway Authority	G	Data on Arlberg Tunnel
	ILF Engineers	E	Data on Arlberg Tunnel
Salzburg (Austria)	Tauern Highway Authority	G	Data on Tauern and Katschberg Tunnels
	Geoconsult	E	Papers on Tarbela Dam Tunnels and Documentation on Munich subway section
	Dr. Pacher	E	Data on Werfen Tunnels
	Laabmayr	E	Documentation on Munich subway
Cologne (Germany)	STUVA, Research Institute for Underground Construction	ER/CR	Research Report on Cost of Subway Tunnels
	Subway Authority	G	Data on open-cut construction
Vienna (Austria)	Porr	C	Data on equipment and costs
Graz (Austria)	Dept. of Public Works State of Steiermark	G	Data on Mitterberg Tunnel

Abbreviations:
- Government Agency : G
- Contractor : C
- Engineering Firm : E
- Engineering Research : ER
- Contractual Research : CR

TABLE 1.2 SUMMARY OF SITES VISITS

Place	Tunnel	Length (km)	Cross-Section (m^2)	Type	Support and Displacement Data
Munich	Line U8/1 Section 16	2 x 1.5 1 x 0.5	36 150	S S	Performance Data (settlement, convergence)
Essen	Section 24	2 x 0.3	36	S	Performance Data (settlement, convergences)
Essen	Section 17	0.46	65	S	No
Bochum	Sections A2 and A3/5	0.98	65 to 11 110	S	From Publications
Murzzuschlay	Ganzstein	2.2	75	H	General information on geology and support
Selzthal	Selzthal	1.01	75	H	Ground conditions, support, convergences
Flirsch	Gandertobel	0.32	150	H	
	Flirsch	0.82	80	H	
St. Anton	Arlberg East	8.9	90	H	
Langen	Arlberg West	5.1	90	H	Detailed information on ground conditions and support
Bregenz	Pfander	6.7	85	H	Detailed description of ground conditions and support

Abbreviations: S = Subway tunnel
H = Highway tunnel

The appendice do not only provide the detailed backup information for the main text, but also they represent a series of individual case histories, cost documentations and design or analysis approaches that stand by themselves.

2. TRIP ITINERARY

Mr. W. Steiner conducted the information gathering trip from January 2, 1978 to February 3, 1978. The most important site visits and interviews took place in western Austria and in Southern Germany. The trip itinerary was selected to accommodate working schedules in offices and on construction sites and also to gather first hand information in the home offices of designers and contractors before visiting a site. Table 2.1 shows the detailed trip ininerary and lists the authorities, companies, institutions, and persons met, as well as the sites visited. A brief summary of the information obtained during the trip and supplied later completes the tables. The locations of offices and sites visited are shown in Figures 2.1 and 2.2. The trip to Cologne, Essen and Bochum was initially not planned but it proved to be necessary to obtain information from the German Research Institute for Underground Transportation Facilities (STUVA*) in Cologne. Also, two Subway construction sites in Essen, and one completed in Bochum were visited.

*Studiengesellschaft für unterirdische Verkehrsanlagen

TABLE 2.1 SUMMARY OF TRIP

DATE	PLACE: AGENCY, FIRM	PERSON(S) MET	INFORMATION OBTAINED
Jan. 2 - 5 1978	Munich, Germany. U-Bahn Referat der Landeshauptstadt Munchen Hackenstrasse 12 D8-Munchen 2	Mr. A. Krischke, stv. Oberbaudirektor; (Director). Mr. Weber, Baurat, Head Design Department. Mr. Nowosad, Head Cost Monitoring Department	Interview on problems of subway construction. Documentation on construction of first subway line U3/6. Brochure on line U8/1, presently under construction. Article: Gebhardt, P. (1977) "Hydrogeologische Verhältnisse-Geotechnische Probleme und praktische Erfahrungen bei den Wasserhaltungen des Münchner U-Bahnbaus." Contractors Brochure by Kunz, Contractors of Munich. Documentation on Construction of U8/1 Section 9 and 18. Separate Print of "Rock Mechanics". Map of Subway System. Performance data from Section U8/1.14
		Mr. Weber, Head, Design Department Mr. Nixdorf, Head, Construction Supervision	Site visit to section 16 of Line U8/1 Section north of Hauptbahnhof station with bifurcation of Line U1 and Line U8
Jan.6 1978	Zurich, Switzerland ETH Federal Institute of Technology, Institute of Road and Rock Construction	Dr. Kovari, Head of Rock Engineering Department	Papers on Finite Element analysis of subways.

TABLE 2.1 SUMMARY OF TRIP (CONT.)

DATE	PLACE: AGENCY, FIRM	PERSON(S) MET	INFORMATION OBTAINED
Jan. 9 - 13 1978	Innsbruck, Austria Beton-und Monierbau Tunneling Department (Contractor) Zeughausgasse 3 A-6020-Innsbruck	Dr. Wagner Head, Design Offc. Mr. Blindow, Mgr. Innsbruck subsidiary Mr. Paulini, Mr. Schulter, Staff Engineers, Design Office Mr. Kluibensched Head, Tunneling Department Mr. Bublik, Head Internal Revision Department Mr. Westermayr, Estimator Mr. Decker, Mgr. Equipment Dept.	Paper on Pfaffenstein-Tunnel Design calculations for Section 5, Munich subway, to test MIT's closed form elastic solutions Cost data of Essen Section 24 Wage Rates Detailed list of Equipment for Pfandertunnel and arrangement of equipment in tunnel. Performance Data for Essen Section 24. Cost and Crew Data on Werfen and Pfander Tunnels
Jan.11 1978	Innsbruck Technical University, Institute for Hydraulic and Tunnel Construction Techniker Str. 13 A-6020 Innsbruck	Prof. Seeber Head of the Institute Mr. Keller, Research Associate	Interview Paper: Seeber, "Die Sicherheit im Tunnelbau" Research Reports on tunnel supports
Jan. 12/13 1978	Innsbruck Technical University, Institute for Construction Management Technikerstr. 13 A-6020-Innsbruck	Prof. Lessmann, Prof. of Construction Management, Project Manager, Bilfinger & Berger Dr. Becker, Research Associate	Interview Previously-obtained data on Sendlingertorplatzstation, Section 9 and Section 18.2.

TABLE 2.1 SUMMARY OF TRIP (CONT.)

DATE	PLACE: AGENCY, FIRM	PERSON(S) MET	INFORMATION OBTAINED
Jan. 12, 13 1978	Innsbruck, Ingenieur-gemeinschaft Lasser-Feizlmayr Framsweg 16 A-6020-Innsbruck (Design Engineers)	Dr. M. John, Senior Engineer, Tunnel and Rock Engineering Dept.	Interview Papers: John, M., "Adjustment of programs of measurements based on the results of current evaluation." Symposium FMRM, Zurich Judtmann, "Anpassung an besondere gebirgsverhaltrisse", Salzburg Colloquium 1977 John, "Arlberg Schachte," Salzburg Colloquium 1977 Pfander-Brochure Reprint Literature Review from South Africa Geotechnical data on Arlberg tunnels Bid schedule of Arlberg & Pfander tunnels
Jan. 13 1978	Innsbruck: Arlbergstrassentunnel-gesellschaft ASTAG (Arlberg Highway Authority) Heiliggeiststrasse 21 A-6020 Innsbruck	Mr. Posch, General Manager	Permission to visit Arlberg tunnel General brochures Data on Arlberg tunnel

TABLE 2.1 SUMMARY OF TRIP (CONT.)

DATE	PLACE: AGENCY, FIRM	PERSON(S) MET	INFORMATION OBTAINED
Jan. 16 -19 1978	Salzburg Laabmayr Consulting Engineers Schallmooser Haupt-strasse 22a A5020-Salzburg	Mr. Laabmayr Consulting Engineer (Owner)	Interview
	Geoconsult Consulting Engineers Sterneckstrasse 55 A-5020 Salzburg	Mr. Golser, Partner Mr. Mussger, Project Engineer	Interview Papers: Golser, Hackl, Jostl, <u>Munich Subway</u> Golser, Tarbela Dam Tunnels, Salzburg,
	Salzburg Tauernautobahn AG (Tauern Highway Authority) Alpenstrasse 94 A-5020, Salzburg	Mr. Köllensperger General Manager	Interview Data of Tauern and Katschberg Tunnels
	Salzburg, Dr. Pacher Consulting Engineer Franz-Josefstrasse 3 A-5020 Salzburg	Dr. Pacher, Consulting Engineer (Owner)	Data from Werfen Tunnels (Helberberg, Brentenberg, Zetzenberg) Publication on Badgasteiner Bundesstrasse Paper by Weber on Rock classification
Jan.20 1978	Cologne, Germany STUVA, Research Institute on Underground Construction Mathias-Brüggen Str. 41, D-5 Köln 31	Dr. Haack, Head of Dept. on Construction Techniques Dr. Klawa, Head, Dept. of Construction Execution & Supervision	Interview & visit of Institute Preprint of research report on tunnel construction costs. Other publications on German tunnel construction

TABLE 2.1 SUMMARY OF TRIP (CONT.)

DATE	PLACE: AGENCY, FIRM	PERSON(S) MET	INFORMATION OBTAINED
Jan. 20 1978	Cologne Cologne Subway Authority Schildergasse 32 D-5 Köln 1	Mr. Behrendt Head, Cologne Subway Construction Dept. and Chairman of Subcommittee on subway construction costs of German Federation of Cities	Interview Brochures with cost data of some subway sections in Cologne (Open Cut).
Jan. 21 1978	Essen Section 24, Subway Beton-und Monierbau	Mr. Wachtlechner, Site Engineer for Beton-& Monierbau Dr. Wagner, Head, Technical Dept. B & M	Site visit of Section 24 subway station with two open cut and adjacent twin single track tunnels in NATM Performance Data
	Essen Section 17a	Mr. Kondmann Deputy Site Mgr.	Site visit of double track subway tunnel driven with blade shield
Jan. 23 1978	Vienna, Austria Porr AG Contractor Rennweg 12 A-1031 WIEN	Mr. Köhler, Principal Manager Mr. Pöchhacker, Exec. Vice President Mr. Zotter, Chief Estimator	Interview Detailed Data on equipment for Arlberg tunnel. Base for estimates of Arlberg tunnel excavation, by ground class. Data on wage rates, procedure to estimate wage surcharges. Publications by Porr: No. 61/62: Gleinalm-Pilot tunnel 69/70: Vienna Subway 57/78: Gasteiner Bundesstrasse (Klamm tunnel)

TABLE 2.1 SUMMARY OF TRIP (CONT.)

DATE	PLACE: AGENCY, FIRM	PERSON(S) MET	INFORMATION OBTAINED
Jan.25 1978	Mürzzuschlag, Steiermark ARGE GANZSTEINTUNNEL	Mr. Muller, Site Manager for B&M	Interview General information Site visit
Jan. 25/26 1978	GRAZ Steiermärkische Landesregierung (local govt., DPW)	Dr. Gobiet Head of the Tunnel Dept.	Interview Data from Mitterberg tunnel
Jan. 27/28 1978	Selzthal-Tunnel Steiermärkische Lanolesregierung	Mr.Sieberer Site Engineer for DPW	Site visit Convergence measurements and ground description of Selzthal tunnel
Jan.30 1978	Innsbruck Beton-und Monierbau	see Jan. 12 - 16	Collected more data on site organization
Jan.31	St. Anton Arge Arlberg Ost.	Mr. Treichl, Site Manager Mr. Schefzik, Deputy Site Mgr.	Site visit to Flirsch, Gandertobel & Arlberg East Tunnels Interview Data on crew sizes, and equipment at the Flirsch and Gandertobel tunnels. Publication on the construction of the eastern section of the Arlberg tunnel.
Jan.31 1978	St. Anton ASTAG (Owner) Geologists' Field Off.	Dr. F. Kunz, geologist	Interview on geological and related problems

TABLE 2.1 SUMMARY OF TRIP (CONT.)

DATE	PLACE: AGENCY, FIRM	PERSON(S) MET	INFORMATION OBTAINED
Feb. 1 - 2 1978	Langen a.A. Arlberg West Contractors	Mr.Mayrhauser, Site Manager Mr. Obst, Site Engineer	Interviews Site visit Convergence measurements, collected data of 10 measurements, cross-sections photographs
	Langen a.A. ASTAG (owner) Geologist Field Offc.	Dr. J. Kaiser, geologist	Interviews on geologic problems
Feb. 3 1978	Bregenz Pfänder-tunnel	Mr. Rucker, Site Manager of Contractor joint venture	Interview Data on site organization, crew size
		Mr. Wogrin, Site Engineer of ILF* (representing owner)	Interview Data on support and geology (copies of tunnel face sketches) Bid schedule Publication on Tunnel.
		*ILF = Ingenieurgemeinschaft Lässer - Feizlmayr	

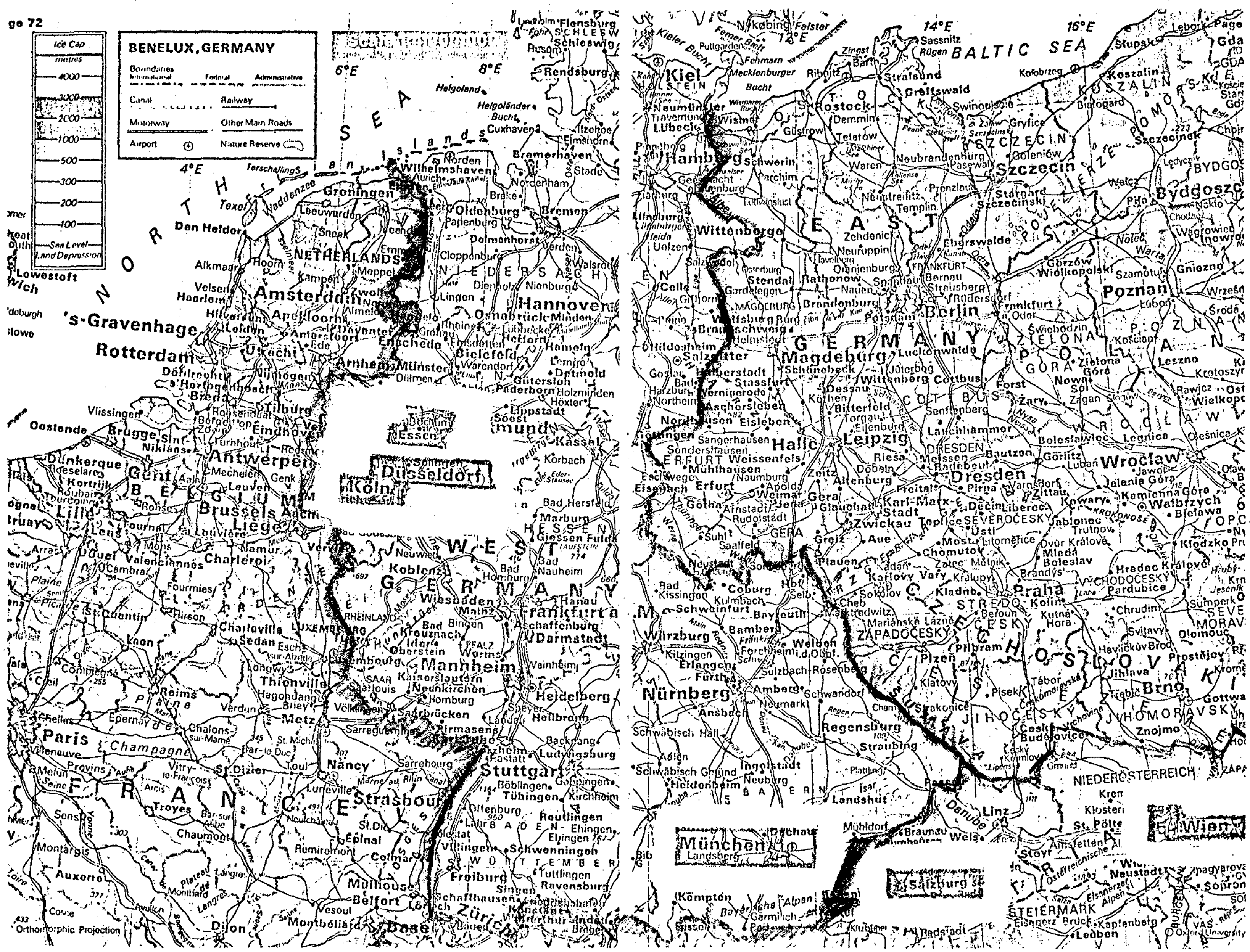


FIGURE 2.1 MAP OF WEST GERMANY WITH STATIONS DURING TRIP

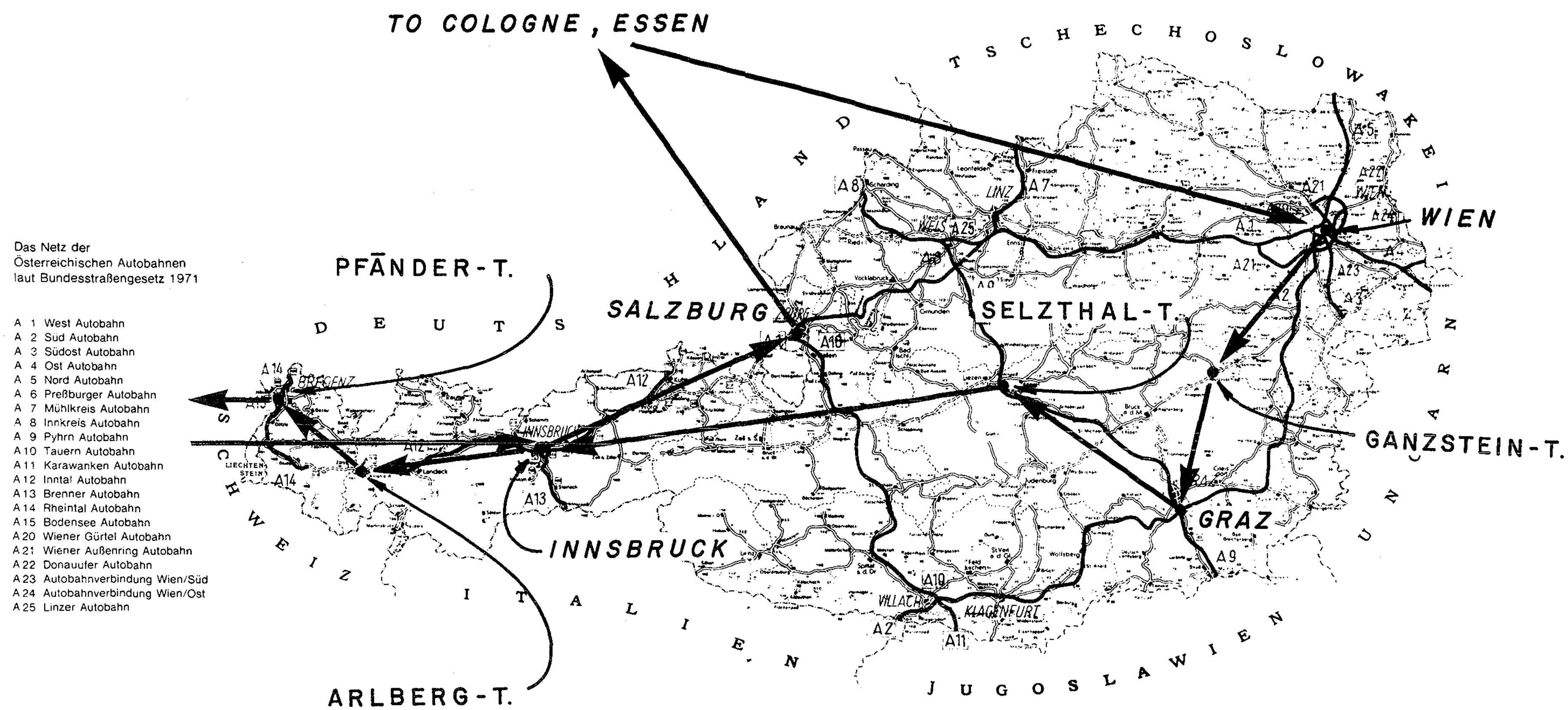


FIGURE 2.2 TRIP ITINERARY IN AUSTRIA

3. COST OF TUNNEL CONSTRUCTION

In this section both cost data and parameters affecting tunnel construction costs are presented (Sections 3.1 through 3.3). Cost data were collected by W. Steiner through interviews and from documentation on individual tunnel projects. Parameters affecting tunnel construction cost and their relative importance were obtained from a study conducted by STUVA (Studiengesellschaft für unterirdische Verkehrsanlagen, Köln, Germany, 1975) and from an evaluation of interviews and project documentation. The review of these parameters is particularly important with regard to cost comparisons between the U.S. and Europe, and most importantly, for a determination of areas where costs could be reduced.

3.1 RESEARCH ON TUNNEL CONSTRUCTION COSTS BY STUVA

STUVA (Studiengesellschaft für unterirdische Verkehrsanlagen - Research Institute for Underground Transportation Facilities), in collaboration with two German contractors, P. Holzmann, AG and Wayss and Freytag, AG, has performed a study on tunnel construction costs. The study is aimed at examining parameters that affect construction costs and is based on the idealized conditions described in Table 3.1. It should be noted that in Table 3.1 the New Austrain Tunneling Method (NATM) term is used in a generic sense rather than

TABLE 3.1 VARIABLES CONSIDERED IN THE STUVA STUDY

Ground Conditions	Construction Procedures	Construction Material	Geometrical Variables		
			Tunnel & Station Size	Length of Construction Section	Spacing of Stations
Two types are considered, 1. A medium stiff clay, similar to the ground conditions encountered in Frankfurt (but without limestone layers); and 2. A sand with clay lenses, similar to the ground conditions encountered in Hamburg.	1. <u>Open-Cut</u> 2. <u>Mined</u> a. shield driven, using either manual or fully mechanical shields. b. NATM(1), using either hydraulic crawler excavators or a roadheader (partial face, TBM(2)).	1. <u>Open-Cut</u> a. cast-in-place concrete b. precast elements c. wall-invert method(3) 2. <u>Shield Driven Tunnels</u> a. segmented cast-iron b. segmented precast concrete 3. <u>NATM Tunnels</u> a. shotcrete only b. shotcrete with steel sets and wire fabric	Station size (length, cross-section) and tunnel size varied. For example, for mined circular tunnels, the considered diameters are 3, 4.5, 6 and 7.5m.	9.75, 1.5, 3 and 6 km	0.75 km

(1) NATM = New Austrian Tunneling Method
(2) TBM = Tunnel Boring Machine
(3) Wall-invert method = placement of braced walls, underwater excavation and tremie concrete followed by construction of tunnel (see Appendix A12)

relating to specific details of the NATM. The common features in all "NATM" applications are the use of shotcrete, the flexibility in support dimensioning and use of additional support material (bolts, wirefabric, steelsets) and the flexibility concerning excavation procedures (partial face, full face).

The parameters listed in Table 3.1 have been combined in nearly all possible combinations. As intuitively expected, particular methods are well suited in some ground conditions but not in others. For instance, open-cut excavation by the wall-invert method (laterally braced walls, underwater excavation and tremie concrete) is economically feasible in case of highly pervious ground and high ground water level, but not if the ground water level lies below the tunnel.

As an example of the results obtained from the STUVA Study, Figure 3.1 shows the influence of section length and tunnel arrangement (two single track vs. one double track tunnel) on construction costs. Costs per route meter decrease with increasing section length. If running tunnels are considered, one double track tunnel would cost less, but if station costs are included, the solution with two single track running tunnels is less expensive. This is primarily due to the necessary widening of the double track running tunnels near the stations to accommodate a central platform (side platforms are even more expensive due to more entrances and

- Shield tunneling in sand with clay lenses "Hamburg ground", no ground water
- Excavation of running tunnels with mechanical shield
- Tunnel liner = cast iron
- Station = open-cut, spaced every 750 m
- 1 ϕ 7.5 = one double-track, diam.=7.5 m
- 2 ϕ 4.5 = two single-track tunnels, diam.=4.5m

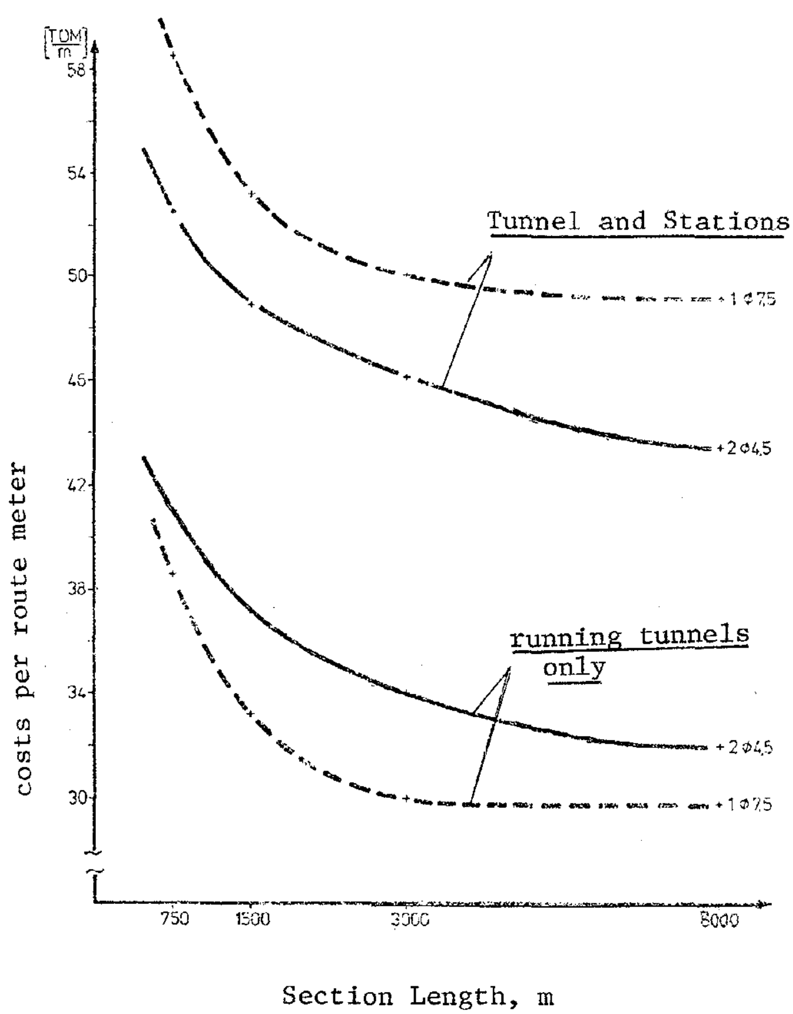


FIGURE 3.1 COST COMPARISON OF SUBWAY TUNNELS AS A FUNCTION OF CONSTRUCTION CONTRACT SECTION LENGTH (FROM GIRNAU, 1975)

escalators). The report by STUVA contains many more results. A complete translation of these findings by STUVA is not possible in the context of this report. We recommend that the entire STUVA Report be translated or extensively summarized and thus made accessible to the U.S. tunneling community. A summary in English of the STUVA Research has been recently given by Girnau (Tunnels & Tunneling, 1978).

3.2 COST DATA COLLECTED BY W. STEINER

A first review of tunnel construction costs can be performed on a route mile basis. Although such a review is rough and aggregate, it provides an interesting first assessment, especially if route costs are evaluated together with the major variables affecting construction costs. Some of these principal variables are: ground conditions, tunnel cross-section, and surface constraints (A detailed discussion of variables influencing construction will follow in Section 3.3). It should also be mentioned that in Germany and Austria detailed costs are rarely published, even after completion of construction. Bids are not accessible to outsiders since standards and ordinances often do not allow their publication.

Construction costs of subway tunnels in Germany and transmountain tunnels in Austria are presented in Tables 3.2 and 3.3, respectively (More detailed data on wage rates from contractors will be presented in Section 5 on technical-operational aspects). The costs are given in German Marks,

TABLE 3.2 CONSTRUCTION COSTS OF SUBWAYS IN GERMANY ($1 U.S. = 2.00 DM)

CITY	COST PER ROUTE METER		COST PER CUBIC METER		CROSS SECTION	TYPE	GROUND CONDITIONS	OTHER REMARKS	SOURCE
	DM	$US	DM	$US	sq. m (excavated)				
MUNICH 1970-77	25,000	12,500	347	174	2 x 36	2 single track tunnels, each 36m² CS. Cost for running tunnels only. Costs for mined tunnels same as for open-cut tunnels	stiff clay	average costs. Either shield with concrete single shell precast liner, or NATM, or open-cut	Interview (Munich)
	20,000 to 40,000	10,000 to 20,000	278 556	139 278			stiff clay various hydrologic conditions, some required grouting and ground water lowering	range of costs	
MUNICH 1970-77	60,000 to 70,000	30,000 to 35,000	N.A.	N.A.	variable	average cost for stations and running tunnels in city center stations: length 120m spacing 600m some 4-track stations, some tunneling under buildings	stiff clay	cost includes access stairs and pedestrian underpasses	
MUNICH 1970-77	30,000	15,000	N.A.	N.A.	variable	average costs in outer districts of city, including stations	stiff clay		
FRANKFURT	23,000	11,500	320	160	2 x 36	2 single track tunnels	stiff clay with limestone inter layers		Behrendt (oral communication)
DORTMUND	12,000	6,000			N.A.	NATM	N.A.		Behrendt (oral communication)
BOCHUM 1974-75	18,000	9,000	240	120	75 m²	double track tunnel, curved in plan and elevation	sandy clay and sandstone, clay shale (mixed face, little groundwater)		Behrendt (oral communication)
COLOGNE OST 1 1974-76	30,000	15,000			variable	open-cut with wall-invert method, slurry trench walls tremie concrete, 2-track tunnel with temp. ramp at one end.	probably pervious gravel & sand	underwater excavation & tremie concrete	Brochure, City of Cologne, Section OST 1
COLOGNE OST 3 1976-77	31,500 including new sewer and track work	15,750			cross-section of box tunnel, approx. 65m²	double track tunnel, box cross section	no groundwater lowering required	includes also related work (sewer & temporary railroad bridges	Brochure, City of Cologne
	21,900	10,950	336	168				cost of tunnel only	
ESSEN Section 24 1977-78	8,700	4,350	120	60	2 x 36	2 single track tunnel. Cost for initial support only	sandy clay, groundwater lowered with external wells		communication with Betonund Monierbau
NURNBERG (Lorenz-kirche) Station 1975	55,000	27,500	390*	195*	2 x 71	twin tunnel station with 5 cross-cut linking the 2 tunnels length of station = 100 m	sandstone		Bauernfeind (1977)

*The total costs have only been related to the cross-section of the station tunnel, the volume of the five cross-cuts has not been included.

TABLE 3.3 CONSTRUCTION COSTS OF TRANSMOUNTAIN TUNNELS IN AUSTRIA (15 AUSTRIAN SCHILLINGS = $1.00)

TUNNEL	COST PER ROUTE METER		COST PER CUBIC METER OF EXCAVATION		CROSS-SECTION	TYPE	GROUND CONDITIONS	REMARKS	SOURCE
	Austrian Schilling, AS	$US	AS	$US	(excavated) sq. meters				
TAUERN 1970-74	265,000	17,700	(2,530)*	(169)	105 m^2	highway tunnel, 2 lanes, with ventilation channels	phyllite with high overburden	cost includes caverns for ventilation & portals	Golser and Tauern auto bahn
	230,000	15,300	2,190	146				tunnel only	
KATSCHBERG 1970-74	177,000	11,800	(1,924)	(128)	88-95	highway tunnel, 2 lanes, with ventilation channels in crown	gneiss and schists	cost includes ventilation cavern and entrances	Golser and Tauern autobahn
	134,000	9,000	1,457	97.1	average= 92			tunnel only	
GLEINALM -78	76,000 to 79,000	5,070 to 5,300	904	60.3	84	highway tunnel, 2 lanes, ventilation channel in crown	granite-gneiss	bid price	Golser and Tauern autobahn
	90,000	6,000	1,071	71.4				final costs	
SELZTHAL 1976-78	110,000	7,400	1,300	86.3	85	highway tunnel, 2 lanes, no separate air channels (blowing, longtitudinal ventilation)	phyllite with high horizontal stresses		budget from sign on site
ARLBERG 1974-78	203,000	13,530	1,971	131.4	103 (includes invert)	highway tunnel, 2 lanes with ventilation channels in crown. Final cost includes shafts, caverns and entrances	gneiss to mica schists with shear zones striking parallel to tunnel	bid price	brochure ASTAG
	277,610	18,507	2,695	180				final costs	
PFANDER 1977-81	13,350	890	1,271	85	10.5	pilot tunnel only	shales, sandstone conglomerates	pilot tunnel with TBM excavation	paper on Pfander tunnel
	100,000	6,667	1,250	83.3	75 to 85 average = 80	highway tunnel, 2 lanes with ventilation channels in crown. Cross-section and cost do not include pilot tunnel. Cost does not include caverns and shafts		main tunnel with drill & blast excavation	interview with John, Judtmann
	120,000	8,000	(1,500)	(100)		highway tunnel. costs include shafts and ventilation chambers			
	133,350	8,890	(1,482)	(99)	85 to 95 average = 90 m^2	highway tunnel. costs include pilot tunnel, shafts & ventilation chambers			
EISENHOWER (second bore)		37,570		279 based on total volume of excavation	average 135 m^2, in multiple drift zone cross-section is 235 m^2	highway tunnel traffic space + ventilation channels = 90 m^2	granite gneiss to fault zone	cost per cubic meter determined, based on total excavation volume	brochure of Colorado DOT

*Costs in parentheses represent total cost normalized to the volume of tunnel only excluding cavern.

Austrian Schillings and in U.S. dollars based on the exchange rates shown in the tables. The reported numbers are the actually incurred costs at the time, or over the time, period indicated; they have not been converted to reflect escalation. This has been done since the exchange rates varied considerably over the time period, and a combined correction for varying exchange rate and escalation would be confusing. Escalation in Germany and Austria during the period 70 - 77 as reflected by consumer price index and labor cost index are shown in Table 3.4. However, tunnel construction cost actually decreased over most of this period as shown in Figure 3.2 and as will be further discussed in Section 3.3. The tables are self-explanatory. Most subway running tunnels (Germany) have costs in the range of $10,000 to $20,000 per route meter (approx. $3,500 to $6,500 per route ft.). For inner city sections including stations, the average cost per route meter is in the $20,000 to $40,000 range. The range of route meter costs for the Austrian transmountain highway tunnels is $9,000 to $17,000, i.e., generally lower than for subways. Reasons for these and more detailed differences will be discussed below.

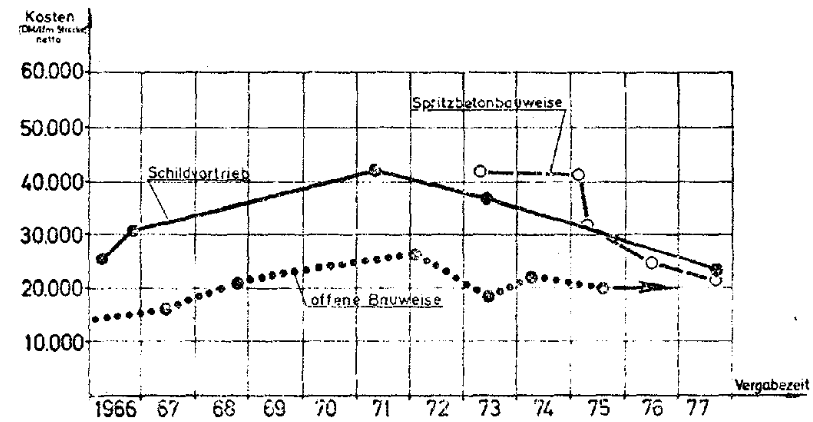
3.3 DISCUSSION OF FACTORS INFLUENCING TUNNEL CONSTRUCTION COSTS

Construction costs in Tables 3.2 and 3.3 scatter over a wide range. Based on information gathered during interviews and site visits, and on the research report by STUVA, we have

TABLE 3.4 CONSUMER PRICE AND WAGE INDICES FOR AUSTRIA AND GERMANY (FROM IMF)

YEARS	CONSUMER PRICE INDEX		WAGE INDEX	
	AUSTRIA	GERMANY	AUSTRIA	GERMANY
1970	100	100	100	100
1971	104.7	105.3	113.6	111
1972	111.3	111.1	126.7	120.9
1973	119.7	118.8	143.0	133.5
1974	131.1	127.1	165.6	147.1
1975	142.2	134.7	187.7	158.7
1976	152.6	140.8	204.7	168.8
1977	~160	~145	-	180.75

COST PER METER OF LINE
(MARKS)



TIME OF AWARD OF CONTRACT
(CALENDAR YEAR)

Spritzbetonbauweise:	Shotcrete Support (NATM)
Schildvortrieb:	Shield Tunnel
Offene Bauweise:	Open-cut Construction

FIGURE 3.2 VARIATION OF TUNNEL CONSTRUCTION COSTS WITH TIME

compiled Tables 3.5 and 3.6 which list factors influencing construction costs for subway and transmountain tunnels. The discussions on these influencing factors will follow the sequence in Tables 3.3 and 3.4 separately for subway tunnels (Section 3.3.1) and for transmountain highway tunnels (Section 3.3.2)

3.3.1 Factors Influencing Subway Construction Costs

The influence of geometrical aspects on construction costs of subway tunnels is discussed in detail in the STUVA report (Klawa et. al., 1976). Geometrical variables that influence the costs are the depth and the size of tunnel. However, since the cross-sections of the running tunnels of the actually built subways are essentially identical, it appears that primary differences in construction cost are caused by variations of ground conditions. For example, ground conditions in the Ruhr District (Bochum, Dortmund, Essen) are particularly favorable; there is little groundwater, which can be easily lowered with a few wells. The ground encountered most often is a sandy clay, or mixed face with rock in the invert. In these ground conditions tunneling is less costly than in Munich or Frankfurt. This is evident if one compares the costs of the Munich and Essen Tunnels in Table 3.2. In Munich, ground water is present and precautions to prevent running and ravelling have to be taken. Gebhardt (1977) quotes ground water control costs (dewatering and/or

TABLE 3.5 FACTORS INFLUENCING SUBWAY CONSTRUCTION COSTS

Geometrical Aspects:

- Cross-Section of Running Tunnels:
  - Two single track tunnels,
  - One dual track tunnel,
  - Third rail vs. catenary.
- Size of Stations:
  - Spacing of stations
  - Cross-section
  - Length.
- Secondary Construction:
  - Number and size of entrances per station
  - Depth
  - Type; stairs, elevator, escalator

Ground Conditions:

- Type of Ground: Soil - clay
  - silt
  - sand

  Rock - intact
  - jointed
- Hydrologic Conditions
- Accuracy of Prediction of Ground Conditions

Construction:

- Type of Construction
  - Open cut
  - Mined - shield
    - NATM
    - others
- Availability of Construction Material:
  - Concrete aggregates (primarily transport)
  - Cement
  - Steel (World market situation)
- Accessibility of Site
  - Surface constraints

Design Aspects:

- Approved Structural Design

Contractual Aspects:

- Fixed-price contract
- Unit price contract
- Changed condition clause
- Price escalation clause

Economic Conditions:

- Excess Capacity of Contractors

TABLE 3.6 FACTORS INFLUENCING COST OF TRANS-MOUNTAIN TUNNELS

Cross-Section of Tunnel:
- Traffic Space:
  - Width of lanes
  - Emergency walkways
- Ventilation Requirements:
  - Type of ventilation
  - Cross-section of channels
- Invert, necessary or not

Ground Conditions:
- Type of Rock
- Discontinuities:
  - Persistence
  - Orientation
- State of Stress:
  - Overburden
  - Horizontal stress
- Groundwater

Type of Support and Construction Procedures:
- Type of Support:
  - Shotcrete
  - Light steel sets
  - Rockbolts
  - Heavy steel sets
- Excavation Procedure:
  - Heading and Bench
  - Pilot tunnel
  - Full face
  - Multiple drifts
- Training of Crews
- Equipment:
  - Rental, depreciation
  - Adaptability to changed conditions
  - Availability, reliability
  - Substitution of equipment of outdated machinery with newer one
- Contractural Procedure:
  - Firm fixed price contract
  - Unit price contract
  - Changed condition clause
  - Price escalation clause
- Economic Conditions
  - Excess capacity of contractors

grouting) totalling 5 to 21% of the construction costs. The influence of ground water control is also evident by comparing the costs of two open-cut sections of the Cologne Subway (Table 3.2). Section Ost 1 (Betzdorfer Strasse) lies in a highly pervious gravel with high ground water level. The wall invert method was used for construction, the excavation is under water, the diaphragm walls are internally braced and tremie concrete is placed. The final structure is made of reinforced, impervious concrete (see Appendix A-12). Section Ost 3 was in an open-cut supported by soldierpile walls with timber lagging and did not require dewatering. Comparing construction costs of the tunnels only (i.e., by neglecting the costs of the railroad relocations for Section 3), the costs of Section 1 (with ground water) are 35% higher.

Construction material may cause some of the cost differences, as has been mentioned by Mr. Behrendt (Chairman of the subcommittee on Subway Construction Costs, German Federation of Cities). For instance, cost of concrete aggregate is primarily influenced by transportation. In some German cities, concrete aggregate has to be brought in over long distances (e.g., in Stuttgart approximately 100 to 150 kms), whereas in other cities the aggregate is at the door step (e.g., Frankfurt).

The influence of site accessibility and surface constraints is illustrated by the cost of sections in the inner city of Munich which is roughly twice that of sections in the suburbs (Table 3.2). At least part of this difference stems from surface constraints; some may stem from different passenger capacity requirements for the stations.

Cost can be affected by excess capacity of contractors. In Munich, tunnel construction costs remained essentially the same since 1972 for open-cut construction. For mined tunnels, costs actually dropped and they are at the present time (according to a statement by a representative from the Munich Subway Authority during the interview) comparable to those for open-cut construction of running tunnels. This is in spite of the fact that wages have constantly risen since 1972 (the wage index has risen by 49.5% from 1972 to 1977, Source: Financial Statistics IMF). However, one other factor is probably more important. In 1972 the Olympic Games were held in Munich. The first subway lines U3/6 had to be ready for the Olympic Games. Not only had the subway to be completed on time, but additional large scale construction work was necessary for the facilities of the Olympic Games, and the construction industry was working at full capacity. After 1972 the demand rapidly dropped, a trend that was further accelerated by the 1973/74 recession. At the present time no change in this trend can be seen.

3.3.2 Factors Influencing the Costs of Transmountain Tunnels (Table 3.6)

The cross-sections of the tunnels considered range from 80 to 105 square meters (Table 3.3). The cross-section is determined by traffic space and ventilation requirements. Traffic space is identical in these highway tunnels (approximately 4.7 x 8.0 m). The difference in cross-sectional area is caused by variable ventilation requirements and ground conditions which require either a flat or an arched invert. The largest ventilation channels are those in long trans-mountain tunnels with cross-sectional areas of 26 square meters (Tauern and Arlberg Tunnels). Shorter tunnels and tunnels in better ground conditions (no invert arch) have smaller cross-sections (Gleinalm, Selzthal). However, the average costs for different tunnels in Table 3.3 do not conclusively show that cross-section has an influence on costs. The variation of the cross-section is only 25 to 30% compared to a variation in cost by a factor of three.

On the other hand, ground conditions have a significant effect on tunnel costs. A comparison between the cost of the Katschberg and Tauern Tunnels, both on the Tauern Highway and built roughly at the same time, is striking. The Katschberg Tunnel (cost = 9000 \$/m) is in more favorable ground conditions, primarily NATM Ground Classes I to III (Weiss, 1975). The northern section of the tunnel (two thirds of the length) was excavated full face, whereas in the southern section, heading and bench excavation alternated with full face excavation. The ground conditions of the Tauern Tunnel (cost = 15350 \$/m) are NATM Class V or higher and required a heading with two benches.

The ground conditions for the Selzthal Tunnel are comparable to those of the Tauern Tunnel, although the overburden is less (100 m instead of 700 m). The cost (bid) are considerably less (7400 \$/m) than those for the Tauern (15350 \$/m) (One has to conclude that the contractor submitted an unreasonably low bid as evident to some extent by claims; the final cost will thus be higher).

The cost differences between the Tauern and Arlberg Tunnels represent the effect of changing economic conditions. Average bid price for the Arlberg Tunnel was approximately 23% less than that for the Tauern Tunnel. The actual costs were 4.8% more. However, the ground conditions for the Arlberg Tunnel are much less favorable. A somewhat depressed economy and free contractor capacity seem to cause this difference.

When comparing costs of subway tunnels with transmountain tunnels, one concludes that subway tunnels are generally more expensive (Tables 3.2 and 3.3). The construction procedure for subway tunnels has to be aimed at minimizing surface effects (surface displacements). This means that the round length and often heading size have to be reduced to minimize the risk of damage. A second factor contributing to higher subway tunnel construction costs may be the fact that the interior liner has to carry the full present overburden load and anticipated future loads resulting in heavily reinforced interior liners compared to the unreinforced interior liners of transmountain tunnels.

3.4 SUMMARY COMPARISON OF TUNNEL CONSTRUCTION COSTS IN THE U.S. GERMANY AND AUSTRIA

This comparison simply states some facts. The underlying causes will be discussed in detail in Section 5 and have been pointed out in the summary of this volume

In Table 3.7 we have summarized cost data presented by Birkmeyer and Richardson in 1975. The costs in their report were brought to a January 1974 level for all projects; they have been updated in Table 3.7 to January 1978 levels based on the ENR Construction Cost Index (Escalation factor from January 1974 to January 1978 is 1.3773). The costs are presented as total section costs, cost per meter (route meter) of section, and cost per cubic meter of net tunnel volume. The net tunnel volume is the space occupied by the tunnel structure; refilled space (as frequently occurs in open-cut construction) is not included.

By comparing construction costs for subway tunnels in Germany (Table 3.2) with those for the U.S. one notes that:

1. The costs per route meter for running tunnels tend to be 30 to 80% higher in the U.S. than in Europe.
2. For stations, the costs are considerably higher per meter of station (two to four times) in the U.S. (comparing similar track arrangements).

TABLE 3.7 CONSTRUCTION COSTS OF SUBWAYS IN U.S., ADAPTED FROM BIRKMYER AND RICHARDSON, 1974

Station, City Section	Total Costs Mill $	Costs Per Route Meter		Cost Per Cubic Meter of Excavation $/m³		Length	Net Volume or Cross-Section	Ground Conditions	Other Remarks
	1974	1974	1978	1974	1978	ft/meter	yd³/m³		
Oakland BART K0016	17.678	69212	95327	207	286	838 ft 255 m	111475 yd³ 85229 m³	sand-clay high water table	3-track station open cut
Farragut Station WMATA C0021	16.419	88459	121836	281	388	770 ft 235 m	76313 yd³ 58345 m³		open cut 2-track station
Farragut Line WMATA C0021	20.761	28800	39667	349	480	2365 ft 721 m	77896 yd³ 59556 m³	sandy silty clay bottom in rock	open cut open cut
Muniline BART S0031	12.190	13156	18120	157.8	217.4	3040 ft 927 m	101000 yd³ 77220 m³	clayey sand or medium fine silty sand with rock(sandstone outcrops, water in sandstone	
Oakland Line BART K0016	2.508	38088	52459	285.2	393	216 ft 66 m	11500 yd³ 8792 m³	sand - clay high water table	open cut
Oakland BART K0016	10.074	30463	41957	338.5	466.20	1085 ft 331 m	3 x 30 m²	sandy clay sand strata high water table	3 parallel single track tunnels
Market Street BART K0016	30.092	19817.9	27296	330.30	454.92	5119 average ft 1560 m	2 x 30 m²	water bearing sands and silts (compressed air, shield)	2 parallel tunnels
Potomac WMATA C0041	5.099	12211.3	16819	203.5	280	1370 ft 418 m	2 x 30 m² cross-section	sandy silty clay overlying rock (partial mixed face)	twin tunnel shield open face (breasted when necessary; part of large section, data based or bid may not be representative)

Updated cost to 1978 level based on ENR Construction Cost Index, (Escalation Factor, EF = 1.3773, from January 1974 to January 1978).

3. Costs per cubic meter tend to be higher in the U.S. than in Germany (50 to 100%).

The difference in costs per cubic meter tunnel volume, as defined earlier in this section, stems from different design practices and lower production rates. However, the large difference in station cost (per meter) primarily stems from the large excavations for the subway stations in the U.S. The single arch span for a double track station has a cross-sectional area which is often twice to three times that of the twin parallel station tunnels in Europe.

Similar cost differences can be observed in transmountain highway tunnel construction. The cost of the Eisenhower Tunnel is about $2\frac{1}{2}$ times that of the Arlberg Tunnel (see Table 3.3); This is in spite of the fact that the conditions were in many aspects more favorable in the Eisenhower Tunnel (For a detailed comparison of these cases see Einstein, (1977).

4. CONTRACTUAL ASPECTS

4.1 INTRODUCTION

This section presents contractual aspects influencing tunnel construction practice in Austria and Germany. The section is divided into subsections dealing with general and prebid aspects (4.2), the bidding process (4.3 and 4.4), the execution and litigation phases (4.5), and a synthesis of contractual problems (4.6). In each section, the situation in Austria and Germany will be discussed separately, followed by a summary.

4.2 REGULATIONS, ORDINANCES AND STANDARDS FOR THE BASIS FOR BIDDING AND CONSTRUCTION

4.2.1 Introduction

Standards and ordinances reflect the experience gained with time in construction practice. New experience gained is incorporated during revisions. Many of these regulations apply to all types of construction work (building, heavy, and highway construction) and are not only limited to tunnel construction. However, there are regulations which apply primarily to one particular area. The contract regulations are listed in Table 4.1. Separate detailed discussions for Austria and Germany are given in Sections 4.2.2 and 4.2.3, respectively.

TABLE 4.1 REGULATIONS AND ORDINANCES FOR CONSTRUCTION CONTRACTS

<u>Germany</u>:

VOB:	Verdingungsordnung für Bauleistungen "Ordinance for the bidding and award of contracts".	
CITY REGULATIONS:	e.g., General Conditions by the City of Munich.	
DIN - Standards:	- for contracts (also compiled in VOB) - for technical regulations	

<u>Austria</u>

ÖNORM B 2110:	Allgemeine Vertragsbestimmungen "General Contract Conditions"
ÖNORM B 2111	Verfahren für die Umrechnung Veränderlicher Preise "Procedure for Determining Price Escalations"

4.2.2 The Basis for Bidding and Construction Contracts In Germany

Standards and Ordinances

In Germany, construction has to adhere to the DIN-Standards (DIN = Deutsche Industrie Norm = German Industrial Standard). They may be compared to the Uniform Building Code (UBC) in the U.S., or the ASTM Standards, but are much more detailed than the UBC. DIN-Standards, which are quite stringent, deal with technical and contractual matters and are not limited to construction but apply to other industries and trades. The DIN Standards dealing with contractual aspects of construction are compiled in the VOB (Verdingungsordnung fuer Bauleistungen, - Ordinance for the award and execution of construction work). The VOB will be discussed in more detail later in this section. One important aspect of the technical DIN Standards is the requirement that the static computations are checked and approved by a licensed inspection engineer (Pruefingenieur). This requirement has a significant influence on the implementation of new methods and will be discussed further in Section 5.

Contractual Regulations

The following is a discussion of the VOB Standards, construction-contract standards divided into three parts. Part A deals with the general conditions for bidding and award of construction work. Part B deals with general conditions for the execution of construction work. Parts A and B are thus important to understand German Construction Contract Procedures.

Part C, which forms the bulk of the standards, deals with general and detailed technical descriptions of different types of construction work (this part simplifies the design work by giving the designer the opportunity to simply refer to the VOB). Part C will not be discussed here, since it does not apply to the general discussion of contractual aspects.

Parts A and B of the VOB have been summarized to some extent in the NCTT-Report "Better Contracting for Underground Construction." This summary has been short and doesn't provide all the details, which we feel are necessary for an understanding of the contractual procedures, as well as the current development of bidding practice in Germany. Therefore, we recommend the translation of all of Parts A and B of the VOB.

Although the provisions of the VOB (Parts A and B) represent standard practice, form part of the construction contract, and are referred to by the courts in case of litigation, they do not have to be adhered to. However, a change from the VOB has to be made explicitly in the contract documents. Changes are mostly made based on experience, as is the case in Munich where such experience in subway construction has been accumulated for more than a decade. This led the Munich subway authority to develop its own general conditions for subway construction contracts, which differ to some extent from the provisions set by the VOB and are adapted to the specific problems of underground construction. Specific aspects of the VOB, along with any deviations, are mentioned in the later part of this section.

In summary, the construction process in Germany is regulated on a detailed level. Changes from the existing regulations are possible and frequently made but they have to be explicitly stated.

4.2.3 The Basis of Bidding and Construction Work in Austria

In Austria, contractual and technical aspects are regulated by some standards. The major difference from Germany lies in the fact that they are much less stringent. The standards do not regulate construction in the contractual and technical areas on a detailed level. In particular, there is no requirement on the technical side that the static computations have to be approved by a licensed inspection engineer (however, the design has to be performed by a licensed engineer). Actually, in case no established Austrian technical standard is available, the owner often chooses to follow the appropriate technical German (DIN) Standards.

For construction contracts, standards ONORM B2110 and B2111 are important. B2110 describes the general contract conditions which ought to be followed. Standard B2111 describes several procedures for price escalation determination. The ministry of building construction developed some supplementary rules to the standards. These deal primarily with the duration of a contract for which no price escalation will be granted. At present, this limit is set at one year. However, the ministry would like to increase the limit further. The ministray also publishes the official wage and price escalation indices, which have to be applied for some of the price escalation procedures.

A list of unreliable contractors is published, to whom no public contracts can be awarded while they are on this list (at present, only some smaller contractors and supply firms, e.g., concrete plants, are listed). For a period of time Washington D.C. Metropolitan Area Transit Authority (WMATA) followed a similar procedure.

In summary, the contractual standards in Austria are not as stringent and detailed as in Germany, they set guidelines for contracts, and they may be substituted by other conditions if the owner considers this to be necessary. However, these changes have to be explicitly stated in the contract documents.

4.3 THE BIDDING AND THE AWARD OF CONTRACTS IN GERMANY

4.3.1 Introduction

Sections 4.3 and 4.4 deal with procedures employed in Germany and Austria, respectively, during the bidding process and leading to the award of a contract. The preparation of the right of way and the exploration of the underground are briefly described followed by a description of the bidding process.

It is important to note that in Austria and Germany the contractor does not simply base his bid on the official design prepared by the owner. He is allowed, and is sometimes asked, to submit alternate proposals involving other designs, which may apply to parts of the project or its entirety. Although it is well known to practitioners, it should be emphasized that alternate proposals are different from value engineering proposals. An alternate proposal enters into competition with all proposals during bidding, contrary to the value engineering proposal, where the contractor has first to be the lowest bidder for the official proposal, and only then will his value engineering proposal be considered.

4.3.2 Preparation of Design and Bid Documents

Large parts of this section apply to the procedure that is used by the city of Munich. More details can be found in Appendix A-1 (Munich).

Lengthy preparatory work is performed before the actual bidding process starts. This includes the securing of the right of way by means of easements and purchases. The procedure is such that all legal issues are settled before construction starts.

The subsoil exploration program is very detailed, consisting primarily of borings spaced at 40 to 60m along the right of way, reaching at least below the foundation of the tunnel (see also Appendix A-1). The official design for the bid documents is developed for the thereby obtained ground conditions.

Prequalification

The VOB (Part A, Article 25.2) states that the contractor, to whom a contract is awarded, should have the necessary technical expertise, capacity, and reliability and also have the necessary technical and economical resources to complete a contract. This is an extreme prequalification clause and the owner can set new criteria for each project.

4.3.3 The Official Design

The official design is developed by the staff of the subway authority and includes the bid schedule. The official proposal is fully developed. The static computations have been performed and are approved by the licensed inspection engineer (see Section 5). The quantities in the bid schedule are determined from this design. Table 4.2 shows an excerpt from the bid schedule for Section 5 of Line U5/9 of the Munich subway. The bid items are described in detail (Usually in more detail than in the U.S.). Note especially Item 2.1.4, which specifies a surcharge for extraordinary water inflow. An estimated length over which such conditions will be encountered is quoted as well as the measurement criterion (water inflow exceeding the limit of 3 to 10 liters per second). This item will only be paid for the length of tunnel where these conditions are actually encountered. Since a price for these conditions has been set in advance, the possibility of litigation is greatly reduced. This and other items considering extraordinary conditions can be further differentiated (e.g., several water inflow levels).

Evaluation of the Bid

Since the detailed bid schedule is lengthy, it is convenient to handle bids by electronic data processing, EDP. Actually, already the bid schedule and the description of the bid items are prepared by the owner with EDP.

Changed Condition Clauses

VOB, as well as the general conditions of the City of Munich, include a changed conditions clause for the official design. The practical application of the changed condition clause will be described in the section on the execution of construction work (Section 4.5).

Section 5/9-5, Theresienwiese Station, Bid No.

Ordinal number Standard Bid Schedule Number*
Description of Item
Quantity Unit

2.1 Station Tunnel, Theresienwiese Station

2.1.1 Standard Bid Schedule, No. 397.900.71*
Start of tunnel from access shaft, cutting through of diaphram walls
Start-up of entire cross-section
Cross-sectional area 172 m^2
1 unit

2.1.2 Standard Bid Schedule No. 397.902.11
Breaching of support wall for in shaft at end of tunnel. Support: soldier piles and wood lagging, for complete cross-sectional arch, 172 m^2
1 unit

2.1.3 Standard Bid Schedule No. 397.90652.11
Excavation and initial support of platform tunnel with steelsets, liner plates, wire fabric, rock bolts and shotcrete of the double-track station
Cross-section according to Plan No. 519.5.3016
Total cross-sectional area, 172 m^2
The excavated material becomes the property of the contractor and has to be removed.
122.0 m

2.1.4 Standard Bid Schedule No. 397.9.10.01
Payment for all extraordinary work and difficulties during excavation and support for a water inflow above 3 to 10 ℓ/S in the section being excavated. 50.0 m

2.1.5 Standard Bid Schedule No.397.992.51.19.01
Concrete (cast-in-place) of inner liner including formwork for double track station in regular impervious concrete.
Reinforced concrete quality BN 250.
Thickness of liner more than 50 to 70 cm^2
Payment by meter of tunnel
also includes the columns of 150/80 cm cross-sectional area
122.0 m^2

2.1.6 Standard Bid Schedule No. 013.660.19.12.09
Gasket (for waterproofing) with reinforced edges, manufactured in synthetic rubber. Make:
If not specified by owner, the bidder has to specify his choice.
Width 320 mm.
660.0 m

2.1.7 Standard Bid Schedule No. 013.661.13.01
Welding of gaskets in corners
96 units

2.1.8 Standard Bid Schedule No. 013.099.00.10.22.13
Concrete (cast-in-place) for station platform, slab regular concrete, quality grade BN 250.
Slab thickness more than 18 to 25 cm.
370.0 m^3

*Standard bid schedule number is used to identify items for Electronic Data Processing

Escalation Clauses

Escalation clauses are included in contracts of projects that will last for more than one year after bid submission. Both the VOB and the general conditions of the city of Munich provide a wage escalation clause. Material price change clauses have only recently been included in contracts. However, only price fluctuations for steel (both price increases and decreases) have so far been included.

Material and labor fluctuations are only considered if they exceed 0.2% of the total bid price. Only the amount exceeding this limit of 0.2% is paid or deducted.

Declaration of the Bidder

When submitting a bid, a contractor has to submit a statement that he read and studied all the documents and that the failure to do so will not result in any additional reimbursements. With this statement, the owner wants to prevent the contractor from raising claims on grounds that the bid documents were incomplete and misleading. In case the contractor has problems with interpretation during bidding, he has to inquire of the owner.

4.3.4 Alternate Proposal

Alternate proposals are routine practice in German subway construction. By submitting alternate proposals, a contractor can optimize the use of his equipment, procedures and crews. Most European contractors have the capability of performing the design in-house, although they may sometimes use outside consultants. The official bid documents often state that alternate proposals are expected.

Requirements

From a contractual point of view, an alternate proposal has to fulfill several requirements:

- It has to be technically sound and feasible so that it can be compared with the official design on the same technical level. In other words, the level of technical details of an alternate proposal has to correspond to that of the official design.

- The contractor has to develop a bid schedule to the same level of detail as the official design (compare Table 4.2).

- Quantities and unit prices of the alternate proposal have to be guaranteed (practically resulting in a firm fixed price contract). How changed condition clauses apply in such a case will be discussed below.

- A construction time schedule has to be developed and guaranteed.

- The declaration of the bidder (as for the official proposal) has to be submitted.

Changed Condition Clauses

VOB does not permit changed condition clauses for alternate proposals. However, such clauses are included in the general conditions of the city of Munich. The changed condition clause primarily applies in cases where the subsoil conditions deviate from the predicted ones.

If an alternate procedure fails during construction, the owner may require the contractor to switch back to the official design without increasing the price beyond that of the alternate proposal.

Price Escalation Clause

The same procedure as for the official proposal applies for alternate proposals.

Technical Negotiations With Owner

If a contractor chooses to develop an alternate proposal, the bid documents advise him to contact the subway authority and to discuss the technical feasibility of the alternate proposal prior to the full development and submittal of a bid. The subway authority can then point out what would be technically required. These technical negotiations may be continued after bid opening and before awarding the contract to further evaluate the technical feasibility of the alternate proposal.

4.3.5 Development of Bidding Practice

Most of the contracts let for the Munich subway are based on alternate proposals; only one section of an estimated total of 50 sections was let based on the official proposal. The development of a detailed official design has thus become practically unnecessary, since nobody bids on it. The work put into an official proposal would be essentially lost and detailed official designs are no longer developed. Only the alignment geometry, cross-sections and so-called construction recommendations -- e.g., methods of ground water control (dewatering from tunnel or grouting around the tunnel) -- are specified

VOB provisions (Part A, Article g.10 to g.14) apply to these design-construction bids. The appropriate bid documents include only a 'bid-program' describing the extent of the project and providing boundary conditions, but no detailed official design. Although this type of bidding is new in tunnel construction practice, it has been successfully used for a long period of time in bridge construction, particularly in conjunction with long highway bridges.

4.3.6 Award of Contracts

The low bidder is very often considered the best bidder, and will be awarded the contract, provided all other requirements (technical and schedule) are fulfilled. VOB sets guidelines for the award of contracts. The bids have to be complete and unequivocal (e.g., unit prices for all items have to be stated). Bids which can not fulfill these requirements are automatically excluded. At bid opening, the total bid prices, subsets of bid items (e.g., "excavation, concrete work") whether an alternate proposal has been submitted and disclaimed by contractors, but no further details, are given to the representatives of all the bidding contractors (bid opening is not open to the public). After bid opening, negotiations between owner and contractor can only consider technical issues, like feasibility of the proposed construction procedure, the time schedule, or the materials proposed. However, no negotiations concerning the bid price are permitted. A contractor who wants to renegotiate the price must be excluded (VOB B, Article 24.3).

The contract is awarded based on the verified total bid price usually to the low bidder as mentioned above. The VOB states that a low bid is not the only criterion on which the selection of contractors is based. (Part A, Article 25.2.2). For example in one case in Munich, the guarantee of keeping the time-schedule was more important than the price, since the particular section was on the critical path for completing the subway line. The low bidder apparently could not guarantee sufficient reserve resources that could be mobilized in case of unexpected delays. The contract was thus awarded to a contractor who could potentially provide the necessary resources.

VOB also states (Part A, Article 25.2.2) that the contract should not be awarded to a contractor who submitted an unreasonably low bid.

4.4 THE BIDDING AND AWARD OF CONTRACTS IN AUSTRIA

4.4.1 Preparation of Design and Bid Documents

The construction of a transmountain tunnel in Austria is often initiated by the Department of Public Works, DPW, of one of the states. However, the federal government has, in most cases, to approve the project because it is usually subsidized and has thus to fulfill federal standards. The design capacity of the DPW is often limited, and design is thus frequently performed by consulting engineers. Sometimes when the state and the federal government cannot finance the highway, a highway authority is formed after the federal government has created the legal basis. The highway authority deals primarily with financial matters and the general supervision of work, but usually not with the design, which is again performed by consulting engineers.

The design and contract procedures for transmountain tunnels in Austria is substantially different from the procedures used for subway tunnels in Germany. This is primarily caused by the larger uncertainties with respect to ground conditions as compared to shallow subway tunnels. Exploration is rarely as dense for deep tunnels as for shallow tunnels. The information gathered by a few deep borings does not sufficiently lower the level of uncertainty to enable contractors to properly estimate their risks. In some cases pilot tunnels are driven, but the extrapolation of support requirements from the pilot drift to main cross-section is not yet satisfactorily solved. The largely uncertain conditions in prediction and extrapolation of ground support and excavation procedures may invite claims, but the NATM, which is practically exclusively used, is contractually structured to minimize such claims. The key to understanding the NATM in the contractual (and technical) sense is the "ground classification," which will be shortly introduced, in Section 4.4.2 (Official Design). The detailed procedure of determining ground classes will be described in Sections 4.5.3 and 5.4.3.

Prequalification

The bid documents state that a contractor who wants to submit a bid for a tunnel construction project must show that he has the necessary expertise in building tunnels with the NATM. Prequalification requires that a contractor has experienced supervisory personnel. In case the contractor does not fulfill this requirement, he still may enter bidding in a joint venture. However, at least one partner in the joint venture has to fulfill the prequalification requirement. A list of previously completed tunnel construction work by the bidder (length, volume, bid price) and lists of the relevant experience of the supervisory personnel may satisfy the owner.

If a contractor does not perform satisfactorily, he may be blacklisted by agencies. The federal ministry for construction keeps this list of unreliable contractors to whom a contract must not be awarded as long as they are on this list.

4.4.2 The Official Design

Included in the bid documents is an official design prepared by the consulting engineer in close collaboration with the owner and specialists (e.g., geologists). Tunnel design and construction center around the NATM ground classification, which relates ground conditions to excavation and support procedures in a qualitative manner. Ground conditions are described in a qualitative, behavioral manner (for a detailed description of these criteria see Einstein, et al. 1977, and also Sections 4.5.3 and 5.4.3), including petrographic and geologic terms as well as a characterization of the geotechnical behavior (e.g., squeezing). For each category of ground conditions or "ground class," average support requirements are specified: thickness and strength of shotcrete; wire-fabric; number of bolts (per unit area or length of tunnel), their capacity, length and type (prestressed or not), type and the spacing of steelsets. The thickness of an unreinforced interior liner is also specified; the theoretical thickness of this liner is, in most cases, the same for all ground classes and is usually 25 to 30 cm (the actual thickness is often twice the theoretical one due to overbreak).

The design engineer prepares a detailed bid schedule and description of bid items. For each bid item, the contractor has to quote the total unit price as well as the fractional unit price for labor, equipment and material. This is necessary for the assessment of price escalation surcharges, which are based on the escalation of material prices and wages.

Payment provisions require a bid schedule which is even more detailed than the German one. One of the primary differences is that, in Austria, site installation and site supervision are paid separately and are not included in other items. These separate site installation items are often further subdivided (like temporary housing, cafeteria facilities, concrete plant, repair shop, and site offices for contractor and construction supervision). The unit price is often quoted as installation and removal (single payment) plus monthly payments (rent).

Construction bid items are subdivided into categories for excavation, initial support, and final support (or final concrete work). Excavation is paid by ground class per cubic meter of theoretical excavation, i.e., to the theoretical line of excavation (Figure 4.1). The theoretical line of excavation is the line (Figure 4.1) to which one should excavate to achieve exactly the required thicknesses of shotcrete and the interior liner. This theoretical line of excavation is

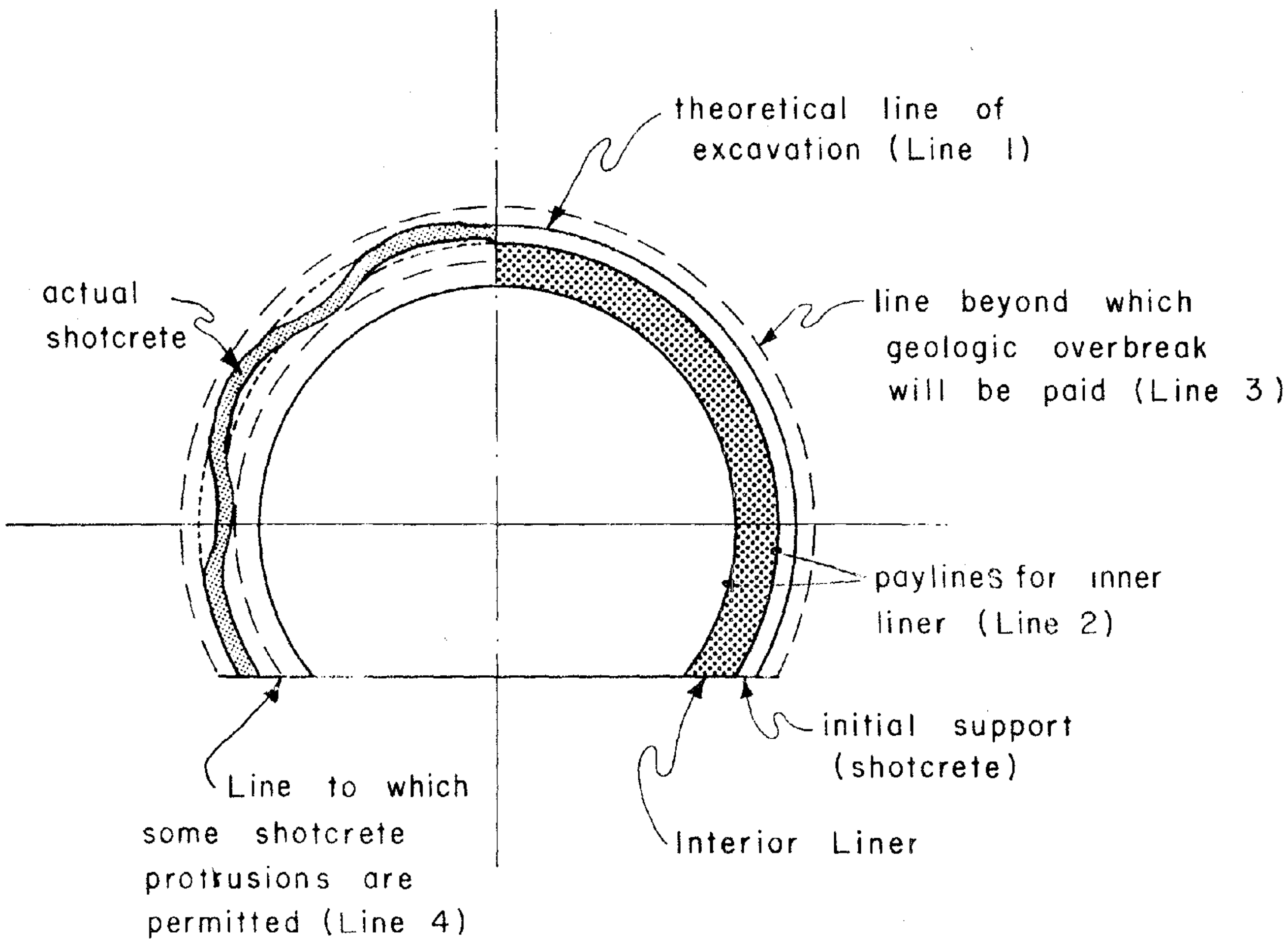


FIGURE 4.1 PAY LINES IN AUSTRIAN TUNNEL CONSTRUCTION PRACTICE

not identical either to the A- or B-line in U.S. practice. Most provisions allow that initial support (shotcrete) protrudes some limited extent (to Line 4) into the inner liner (in one case, 3 cm are allowed). Also, as will be discussed in Section 4.5.3, overbreak (overbreak due to geologic conditions, provided the contractor has exercised the required care) will only be paid if it reaches beyond line 3, which is specified at some distance (e.g., 30 cms) from the theoretical line of excavation (Line 3). The contractor has to estimate and state in his bid this technical overbreak (between lines 1 and 3), i.e., overbreak primarily due to the inclination of the blast holes to the tunnel axis. The cost for such overbreak has to be included in the excavation item. Overbreak beyond the technical overbreak, and given that the contractor employs proper blasting procedures, is considered geologic overbreak and is usually paid as discussed above. The unit price for geologic overbreak may be different from the price for the excavation. In one interview, it was stated that geologic overbreak should be paid as a percentage of the excavation unit price. For the Pfander Tunnel (Appendix B-8) the contract provisions required that geologic overbreak had to be included in the excavation unit price, since a pilot-tunnel was available which allowed the contractor to estimate the overbreak. The same excavation unit price applies for different cross-sections in the same ground class, e.g., widened sections for break-down lanes. The contract provisions state that any difficulties resulting from changing cross-sections have to be included into this unit price.

Contract provisions often require the contractor to include all "difficulties associated with support" placement in the excavation unit price. As explained earlier in this section, average support requirements for each "ground class" are specified; however, the actually placed support may be different and this may change the excavation conditions anticipated by the contractor (as will be discussed later in this section -- this is one of the major contractual areas). By asking the contractor to include support placement cost in the excavation unit price, the owner hopes to give the contractor an incentive for careful work, since support requirements are believed to be at least partly dependent on careful work. In a recent modification of this pricing procedure (first applied at the Pfander Tunnel), the unit price for the excavation includes the cost of the excavation only but not the support placement.

Support is paid by item placed: shotcrete per square meter of different specified nominal thicknesses; wirefabric per square meter or weight (the overlap has to be included); steelsets per lineal meter of set or weight (the overlap and fixtures have to be included); and bolts per piece of a certain length and capacity (the anchor plates, grout and resin have to be included). Separate items are backfill shotcrete and concrete for geologic overbreak. In the Tauern, Katschberg, and Arlberg Tunnels, payment provisions included a Bonus-Malus clause: Support exceeding the standard design quantities for each ground class was only paid at 75% of the quoted unit price, while support not placed because it was not necessary was still paid at 25% of the unit price. The intention of this procedure was to give the contractor an additional incentive for careful work. However, these contract provisions have two disadvantageous effects: first, the contractor tends to press for a higher "ground class" (representing less favorable ground conditions) to qualify for a bonus, thus distorting the ground classification; second, if consistently more support than designed has to be placed, the malus prevales and the contractor is paid unfairly (this naturally depends also on the contractor's bid unbalancing). At the Arlberg west, these provisions led to considerable disputes, since the placed support greatly exceeded the design quantities; in some sections five times the designed quantities of rock bolts were placed (See Appendix B-7).

These types of payment provisions were subsequently abandoned in the Pfander Tunnel, where support was paid per unit placed, including the placement costs and difficulties in excavation caused by placing the support (as mentioned before, the excavation unit price for the Pfander Tunnel only includes excavation costs). As a consequence, disputes on the site were greatly reduced. During W. Steiner's visit to the Pfander Tunnel, the representative of the owner stated that no significant claims had been submitted by the contractor. The representative of the contractor called the contract 'tough but fair' and acknowledged that "disputes" can hardly be justified.

Changed Conditions

The bid documents do not include a changed condition clause as such. Such a provision is not necessary since the ground classes supposedly represent the actually encountered conditions. Also, the support actually placed can be further varied within each ground class. In addition, the ground classification system contains a class (usually Class VII) requiring special procedures. For this ground class only, wage rates have to be quoted in advance, since the work will be either reimbursed on a cost plus fee basis or will be determined by a new detailed estimate for this particular work. The owner and contractor will negotiate the procedure and the price.

Escalation Clauses

Contracts of a duration longer than one year include a price escalation clause. Two procedures are available, a simplified and an accurate one. The simplified procedure grants an increase based on official wage and material price indices which are applied to the total wage and material costs. The second procedure uses a weighted average of the price changes for major quantities (separately for labor and material). A price escalation is granted every time the escalation exceeds 2% of the projected project price; however, the contractor has to notify the owner within 6 weeks after this 2% limit has been exceeded. A new change is only granted when the increase again exceeds 2%. Incremental escalation below a total of 2% has to be carried by the contractor.

Design by Contractor

Some items are not designed by the owner, and the contractor is asked to perform this design as well as to have it approved by the owner. One such item is the excavation by blasting scheme. Another example is the design of the reinforcement for the slab separating the ventilation channels from the traffic space. The owner (design engineer) leaves the design to the contractor in cases where the contractor is believed to have more technical expertise and where the contractor would almost certainly propose an alternate scheme. Interestingly, most official designs and bid documents specifically exclude the use of heavy steel sets as initial support, since it proved to be uneconomical in Austria.

4.4.3 Alternate Proposals

As already mentioned in the section on official design, the contractor is required to do some design. It is difficult to draw a definite line between official and alternate proposals in Austria, since the official proposals are very flexible as indicated. However, alternate proposals can be submitted and they primarily deal with excavation and support procedures. For example for the Selzthal Tunnel (Appendix B-6), the contractor submitted an alternate proposal to excavate the tunnel by means of the Bernold System and a forepoling shield in the crown. However, after 28 m of tunnel excavation, the roof-forepoling shield sagged and had to be abandoned. Whenever the alternate methods fails, the contractor has to return to the official procedure according to the NATM. However, in Austria, as in Germany, no price increase is granted if this is necessary (This was the case of the Selzthal Tunnel).

Other alternate proposals deal primarily with the type of support, e.g., whether channel type or I-type light steel sets are used and the specific type of rock bolts.

4.4.4 Award of Contracts

In Austria, as a first rule, the low bidder is the best bidder. The regulations ONORM A 2050, however, state that a low bidder must not be awarded the contract if he is not considered to be technically responsive or to a bidder whose bid is unreasonably low. The task of proving that a bidder is not technically responsive may not be too difficult. However, to prove that a bid is unreasonably low may be more difficult. This task is facilitated by the detailed bids. In Austria, not only the total bid price is compared when evaluating bids, but also the main categories of bid items are compared separately, as well as the total of labor and materials components. Although it may be easy to eliminate a clear cut case of an unreasonably low bid, this may be difficult for marginal cases.

Negotiations (after bid opening) between owner and contractor are usually on issues concerning the type of excavation procedure and the time schedule. For example, at the Arlberg Tunnel (Appendix B-7) the bid schedule was prepared for five separate sections (3 tunnel sections and 2 shaft sections). However, during the negotiations of the bid, the boundaries of the sections were changed and the contract was finally let to two joint ventures (each conducting one shaft plus tunnel section). The boundary in the tunnel between the two sections was shifted to accommodate the time schedule. Actually, the section boundary shifted once more during construction, based on the actual advances. As we learned recently, one of the joint ventures bidding for the Arlberg Tunnel had low bids for all parts of the project, but the owner did not want to award the construction of the entire tunnel to a single contractor. Negotiations with the second lowest bidder were initiated by the owner and dealt also with price issues in this case.

4.5 CONTRACTUAL ASPECTS DURING AND AFTER CONSTRUCTION (CHANGED CONDITIONS, LITIGATION)

4.5.1 Introduction

Aspects involving the contract execution during and after the completion of the work are discussed in this section. Of primary interest are cases where changed conditions are encountered. Both in Austria and Germany, the contractor encountering a changed condition has to notify the owner immediately and cannot submit a claim later. However, the procedures of supervision, handling of changed condition claims, and litigation are different in Germany and Austria and will be discussed separately.

4.5.2 Execution of Contracts in Germany

General

Procedures as they are primarily used during subway construction in Munich are discussed. To a large extent this represents the current tunnel construction practice in Germany. The experience in Munich is particularly interesting since subway tunnel construction has been very extensive over the last decade (the most extensive in Germany).

Supervision

One of the major components of successful tunnel construction is continuous and competent supervision both by the owner and the contractor. The owner is particularly well qualified in the city of Munich. The subway design department originated from a core of engineers from private industry and from the subway department of Hamburg and Berlin, where subways had previously been built. Thus, experience with subway construction procedures was transferred and was available from the beginning. A continuous and competent construction supervision allows the owner to observe continuously and accurately the ground conditions and construction procedures and to have records available in case of litigation. However, the supervision by the contractor is also important since the contractor has to insure proper construction procedures. For example, in Munich, the contractor has to name the responsible site personnel before the contract is signed. The city retains the right to request removal of contractor's personnel (including the site manager).

Changed Conditions

As mentioned before, VOB differentiates between official and alternate proposals. Usually only the official proposal can include changed condition clauses. In Munich alternate proposals also contain a changed condition clause. Whenever the contractor encounters a changed condition he has to notify the owner immediately in writing (VOB B Article 2.6) and at the same time has to submit a supplemental bid stating quantity, unit price, and the total price of the additional work. No work can be started prior to the approval of the additional work by the owner. The supplemental bid has to be based on the same wage and material rates as the original bid. The city of Munich has the capability to verify whether these rates agree, since the general contract conditions require that the contractor deposits a copy of his detailed estimate in a sealed envelope (Note that the German bid prices, in contrast to the Austrian ones, are not listed in cost fractions).

The public owner either approves or disallows the proposed additional work within 2 months in writing (VOB, B 18.2). During W. Steiner's visit, it was mentioned that they rarely exceed 5 to 10% of the total bid price. Another factor which helps to reduce changed conditions claims is the detailed bid schedule listing items for extraordinary conditions, including the application criteria.

Litigation in Court/Arbitration

Problems encountered can first be discussed by the supervisory personnel on site and if no solution can be found it will reach higher levels of the owner and contractor. In case of the Munich Subway, over a period of 12 years only two cases had to be decided in court (from 1965 to 1977).

Arbitration is possible if approved prior to construction in separate documents (VOB A, Article 10.5). In case of public owners, and in particular the subway authority of Munich, arbitration is rare, if not excluded.

VOB regulates the administrative handling of disputes involving public agencies. The contractor shall notify the agency superior to the agency he has been dealing with (VOB, Part B, Article 18). This superior agency has to let the contractor give an oral presentation of the problem. An answer to the contractor shall be prepared by the superior agency within 2 months (if possible). The contractor has the right to dispute this ruling in writing within 2 months; if he does not dispute the decision within this time limit, the dispute is settled.

One section of the above-mentioned article deals with disputes on the quality of materials. The following is a translation of the relevant article (VOB, Part B, Article 18.3): "In case of disputes on the quality of materials, and if approved testing procedures (DIN Standards) are avialable, either party can, after notification of the other party, have the material tested by a government approved laboratory. The costs of the investigation have to be borne by the losing party."

4.5.3 The Execution of Tunnel Construction Contracts in Austria

This section deals with the construction of transmountain tunnels in Austria (the Vienna subway has not been studied). The procedure of the NATM is briefly recalled; it involves several steps listed below:

(1) Exploration. Existing geologic data is analyzed (Maps, Records of existing tunnel). Borings are rarely used due to the limited amount of information they provide. However, in some cases pilot tunnels are constructed.

(2) Preliminary Design. Relations between typical ground conditions, excavation procedures, and support requirements are developed for typical "ground classes". Ground conditions are behaviorally described mainly in a qualitative manner. The level of description depends on the information available from the exploration phase. For each similar class of ground conditions, an excavation procedure (full face vs heading and bench, round length) is assigned as well as typical support requirements. Table 4.3 gives an abbreviated summary of typical ground classes; a detailed description of these classes can be found in Einstein, et al. (1977). The bid is based on these ground classes.

(3) Construction. After each round of excavation, the ground class and the required support are determined on the spot by a representative of the owner and the contractor. The ground conditions are mapped (maps of the face and the circumference of the tunnel). The two representatives sign a form, documenting the agreed upon ground class. In case of a disagreement, a usually preassigned mediator will decide later on the ground class, based on the information provided

TABLE 4.3 ABBREVIATED SUMMARY OF GROUND CLASSIFICATION USED IN AUSTRIA

Ground class I

GROUND CONDITIONS

Intact rock of high strength relative to the major principal stress. Water does not influence stability of rock mass.

EXCAVATION METHOD

Full face excavation
Roundlength = 3 to 4m (limited by equipment)

SUPPORT

Wire mesh : 1.78kg/m^2 in crown, with
Rockbolts : capacity = 15 tons
length = 1 to 3m
spacing > 2m, or
Shotcrete : 50mm in crown

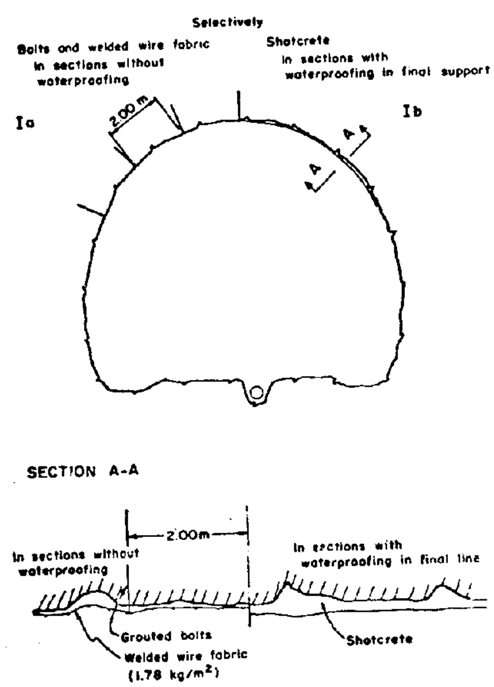


TABLE 4.3 ABBREVIATED SUMMARY OF GROUND CLASSIFICATION USED IN AUSTRIA (CONT.)

Ground class II

GROUND CONDITIONS

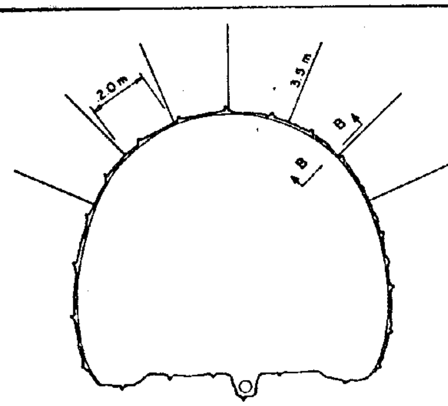
Rock more jointed and fractured than in I, but is still unweathered. Groundwater does not chemically alter rock; water pressure may cause loosening.

EXCAVATION METHOD

Full face excavation
Maximum roundlength = 3m

SUPPORT

Wire mesh : 1.78kg/m^2 (in crown)
Shotcrete : 5 - 10cm
Prestresses grouted bolts :
capacity = 15 tons
length = 3 to 3.5m
one per 4 - 6m^2



SECTION B-B

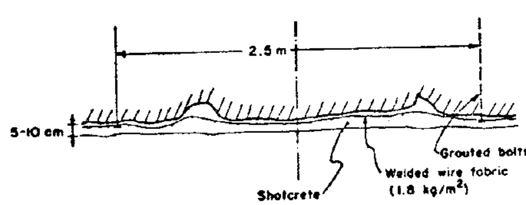


TABLE 4.3 ABBREVIATED SUMMARY OF GROUND CLASSIFICATION USED IN AUSTRIA (CONT.)

Ground class III

GROUND CONDITIONS

Heavily jointed rock in several directions. Joints have little shearing resistance. Water does not alter the rock.

EXCAVATION METHOD

Full face or heading and bench
Maximum round length = 1.5m for full face
= 3m for heading and bench

SUPPORT

Wire mesh : 3.12 kg/m^2
Shotcrete : 10 cm.
Prestressed grouted bolts :
25-ton capacity
length = 4m
one per $3m^2$

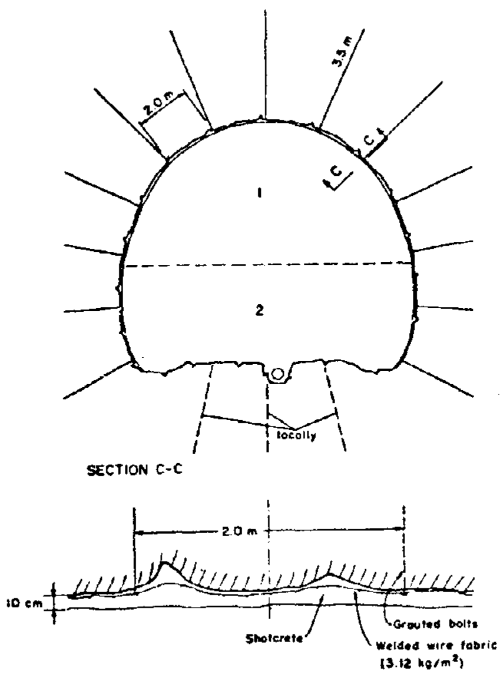


TABLE 4.3 ABBREVIATED SUMMARY OF GROUND CLASSIFICATION USED IN AUSTRIA (CONT.)

Ground class IV

GROUND CONDITIONS

Squeezing ground conditions completely disturbed or broken ground, chemically altered. Water reduces stability of ground.

EXCAVATION METHOD

Heading and bench (numbers on figure)
Roundlength = 1.0 to 1.5m

SUPPORT

Shotcrete : 15 cm
Wire mesh : 3.1 kg/m^2
Steelsets : 21 kg/m, spaced 1.0 to 1.5m
Grouted bolts :
capacity = 25 tons
length = 4-6 m
one per 2 m^2

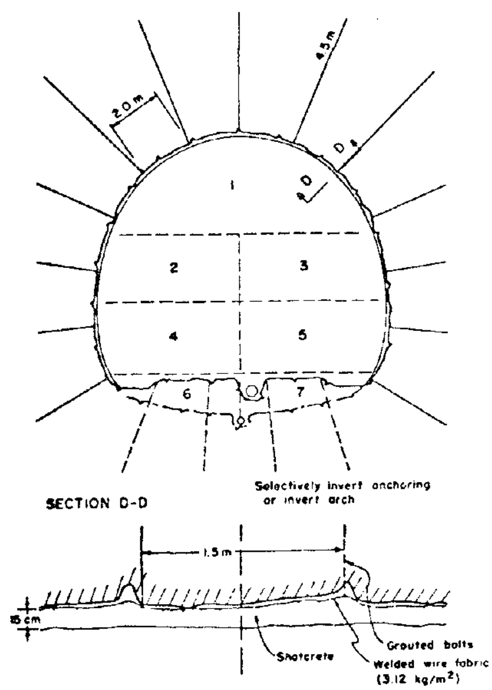


TABLE 4.3 ABBREVIATED SUMMARY OF GROUND CLASSIFICATION USED IN AUSTRIA (CONT.)

Ground class V

GROUND CONDITIONS

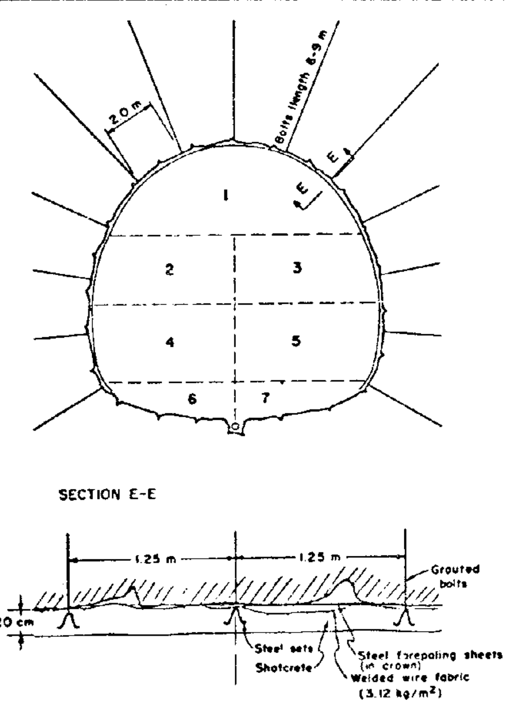
Heavily squeezing ground. Very heavily fractured rock and soils of loose blocky debris. Ground near cavity moves and then squeezes after excavation.

EXCAVATION METHOD

Heading and bench (numbers on figure)
(no blasting)

SUPPORT

Shotcrete : 20 cm
Wire mesh : 3.1 kg/m^2
Steelsets : 21 kg/cm, spaced 0.7 to 1.5 m
Bolts :
capacity = 25 tons
length = 6 m or longer
one per 1.5 m^2



by the two representatives, as well as the monitored performance (see below). Until the mediator has made his decision, the opinion of the owner prevails; however, the contractor is free to place more support which he considers necessary for the safety of the crew*, but he risks not being reimbursed for the additionally placed support.

(4) <u>Monitoring.</u> The performance of the tunnel and support is monitored. Convergence is typically monitored every 10 to 50 m depending on ground conditions. Principal monitoring cross-sections (with convergence measurements, extensometers, stress cells, instrumented bolts, load cells) are typically placed every 500 m. If the performance is judged not to be satisfactory, additional support will be placed. Performance criteria are usually the rate of convergence and total convergence.

(5) <u>Final Liner.</u> The final liner is placed once the monitored performance reaches an appropriate level, usually a certain rate of convergence (Appendix B-7, Arlberg). The final liner is

*The contractor is responsible for the safety of the crew.

usually non-reinforced concrete of approximately 30 cm theoretical thickness.

The key to success and negligible litigation in NATM applications lies in the just described ground classification procedure. Application of this procedure is straightforward, but it requires experienced personnel, since the relations between ground conditions and support requirements are primarily qualitative and the interpretation requires substantial experience. However, several problem areas still exist and were identified during the interviews by W. Steiner; they are: (a) learning period, (b) unreasonably low bids, (c) incomplete contract documents, (d) overexcavation, (e) geologic overbreak. These areas are discussed below:

(a) Learning Period. In a particular tunnel, some startup time is required until the classification procedure works. Disputes between the field representatives the owner and contractor are most frequent during the first few hundred meters of a tunnel. Obviously, in this phase the mediator is particularly important; it is thus imperative that the mediator is named before the project starts.

(b) Unreasonably Low Bid. It appears that some of the encountered problems are at least somewhat related to the profit margin of the contractor for the particular project, i.e., whether the bid was reasonable or if it was too low.

These general statements can be illustrated by the Selzthal Tunnel case (Appendix B-6). In the Selzthal Tunnel, geologic conditions are similar to the Tauern Tunnel and only the overburden is different (Selzthal = 150 m, Tauern = 700 m). However, the bid price per meter for the Selzthal Tunnel (\$7,400/m) is less than half that for the Tauern Tunnel (\$15,300/m). The difference seems to be caused by the present economic situation (excess capacity) in the Austrian construction industry.

(c) Incomplete Contract Documents. The case of the Mitterberg Tunnel involves both contract documents which were not unequivocal and an unreasonably low bid price. The dispute arose over the classification criteria, which were primarily of petrographical nature. Since different rocks were encountered in the same cross-section, the petrographic classification criteria were not sufficient to clearly determine the appropriate ground class in the particular section. The dispute centers around this gap in the classification. However, one has to mention that a pilot tunnel was available during bidding and also that the principal contractor went into bankruptcy. Thus, one has to conclude that in this case the bid might have been unreasonably low, magnifying the problems due to the incomplete contract documents.

(d) Overexcavation. Another contractual problem occurred during the construction of the Arlberg Tunnel. In squeezing conditions, overexcavation is necessary to prevent removal of tights at a later stage, but reliable prediction of this overexcavation is difficult. The contractor is paid by cubic meter of theoretical liner, i.e., basically he is paid per lineal meter of tunnel. Although the theoretical liner thickness is 30 cm, the contractor has to base his estimate on the true volume of concrete, which depends on overbreak and overexcavation. The true liner thickness is often 50 to 60 cm. Overexcavation is important since it may either lead to greater concrete volume if too large or to tights if not large enough (the latter is particularly cumbersome and costly if many bolts have to be cut-off and anchor plates replaced). At the present time, the contractor has to carry the risk associated with overexcavation. The owner claims that he awarded the work to a contractor with the necessary expertise in handling these ground conditions. However, at the Arlberg Tunnel, the monitored convergences were larger than anticipated (Appendix B-7 Arlberg, Figure B-7.18). These disputes led to the inclusion of a clause into the newest draft of the Austrian Standard on ground classification (ONORM B 2203) stating that the designer has to specify the anticipated overexcavation. This of course does not completely solve the problem, since the prediction of this overexcavation may prove to be difficult.

(e) Geologic Overbreak. The problem of geologic overbreak has received considerable attention; however, it is handled differently in different contracts. For example, at the Pfander Tunnel, the contractor had to also include the geologic overbreak into the excavation unit price. In other cases, the geologic overbreak will be paid as discussed in Section 4.4.2 and shown in Figure 4.2.

The geologic overbreak leads to another recently encountered problem, the collapse of already supported roofs (this problem will be also discussed in Section 5 on technical and operational issues). Roof collapses also have contractual implications, particularly with respect to responsibility and liability. Several cases of roof collapse have occurred recently (Arlberg and Selzthal Tunnels in Austria; Pfaffenstein Tunnel in Germany). Of particular interest is the roof collapse between station 280 and 300 m of the Arlberg western section (Appendix B-7). The problem centered around the question whether steel sets are necessary or not in ground class III. From a petrographic point of view, the conditions encountered are considered to be "Ground Class III" by both parties. However, two large discontinuities intersected in this section, freeing the rock mass. The support was by shotcrete and 4 m bolts only (no steel sets). During the mucking in the top heading, a sudden roof collapse buried two frontend-loaders and their operators. The contractor wanted to place steel sets also in these ground conditions - however, not at his expense. The decision not to place steel sets seems to have

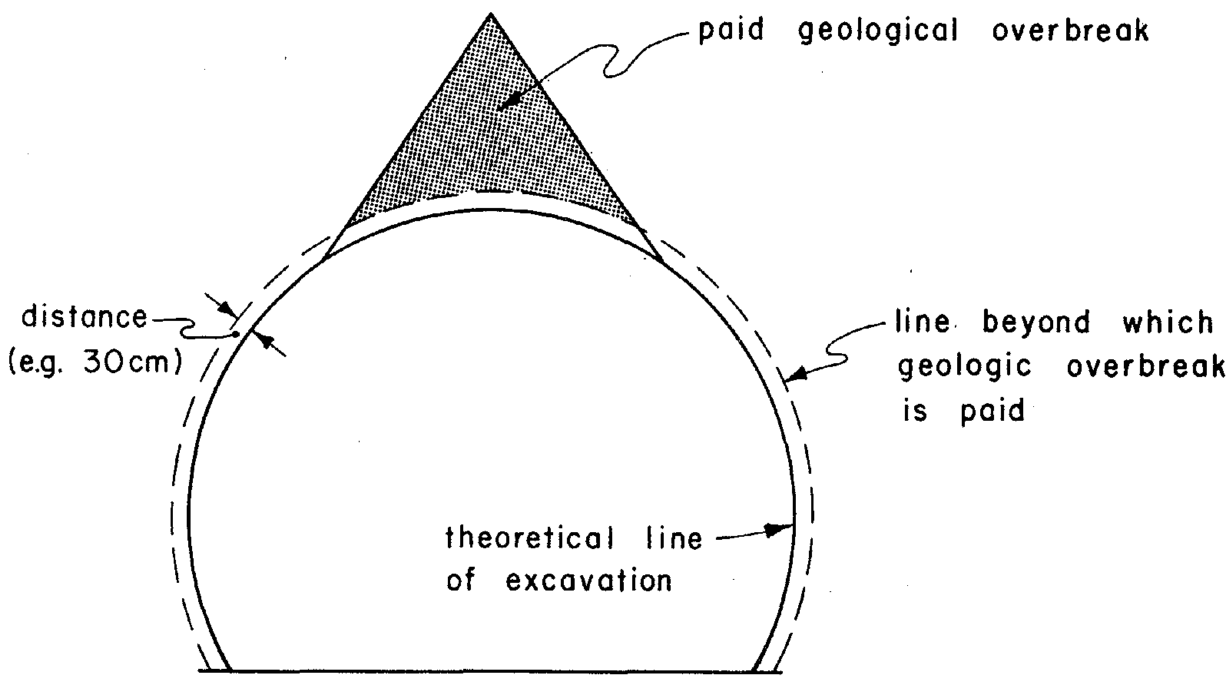


FIGURE 4.2 PAYMENT PROCEDURE FOR GEOLOGIC OVERBREAK

been made by several experts on site only the day before the collapse. After the collapse, Ground Class III, and thus the contract, was modified to include steel sets.

Changed Condition Clause

Tunneling with the contract provisions in Austria does not require an explicit changed conditions clause, since the procedure of "ground classification" considers changing conditions. The range of the "ground classes" is such that all ground conditions should be included. Even for completely unexpected conditions, a ground class is provided (Class VII) which in a sense could be considered a "changed condition class". The two parties have to agree that ground conditions fall into this class. Also, the construction procedure and the estimated costs have to be agreed upon by the owner (or by his representative) and the contractor. This "changed condition class" is only necessary for conditions that have not been anticipated by the design engineer.

Conclusions

Successful execution and completion of tunnel construction contracts requires carefully prepared bid documents, including design and a geologic prediction by the owner and design engineer. The contractor has to study these documents carefully and make a reasonable estimate for his bid. Problems seem to develop primarily in cases where contracts were awarded to an unreasonably low bidder. In the "ground classification" procedure, the contractor is paid for the actual conditions, and basically no explicit changed condition clause is necessary.

4.6 SYNTHESIS OF CONTRACTUAL PROBLEMS

From a technical point of view, there are substantial differences between the construction of subway and transmountain tunnels. However, the contractual procedures do not differ as much even considering that subway tunnels discussed here are build in Germany and mountain tunnels are built in Austria.

The key to success, relatively low cost, and small amount of litigation lies in the complete, thorough preparation of the bid and contract documents. Bid schedules and supplementary descriptions of the bid items are very detailed. Each item is clearly described and items applying only under particular conditions are provided with clear criteria when the item applies. A price has been fixed prior to construction and only the quantity has to be approved on site.

Changed condition clauses are provided in the contracts; however, the detailed bid schedules with very detailed items make changed conditions clauses nearly redundant.

Many of the contractual disputes were traced to an unreasonably low bid price. Legally, it is possible to exclude bids which are considered to be unreasonably low; however, the owner has first to prove that this is the case, a task which is not always easy.

Alternate proposals are the key to the progress achieved in European Tunneling Practice; they give contractors the opportunity to optimize the use of their equipment, procedures, and crews. The feedback of experience gained on one site is faster than could be achieved by designers. The fact that almost all sections of the Munich Subway were let for alternate proposals led to a change in design and bidding practice. Instead of developing an official design, the subway authority calls for design-construction bids. The contractor has to design the section and prepare the bid for this design. Most European contractors have the capability of performing the design in-house; occasionally, they may also hire a consultant for this purpose.

Regulations and contractual procedures which are used in Germany and Austria incorporate most of the recommendations of the NCTT-Report (National Academy of Sciences, 1974). The procedures in Germany are summarized by NCTT; however, important details are not mentioned (e.g., the provisions for design-construction bids). We thus recommend that some of these regulations are translated, in particular:

- Contract regulations in Germany VOB, Parts A and B;
- Austrian Standards on Tunneling ONORM B 2203. (Once the finalized standard is published.)

5. TECHNICAL-OPERATIONAL ASPECTS

5.1 INTRODUCTION

It would be basically possible to study subway and transmountain tunnels both in Germany and Austria, but we limited ourselves to studying subway tunnels in Germany and transmountain tunnels in Austria since these are the prevailing applications in the two countries, respectively.

Although several tunnel construction methods were studied, emphasis was placed on applications of the NATM, which differs most strongly from U.S. tunnel construction methods.

In this section, aspects of design, construction, and site organization are discussed. These technical-operational aspects are strongly related to the previously discussed contractual procedures. It should be emphasized again that contractual procedures, as they are used in Europe, favor the implementation of innovative ideas developed and advanced by contractors. Such ideas are usually presented to the owners in the form of an alternate bid proposal.

The key to many technical innovations developed in European construction practice is the direct competition of official (the owner's) and alternate proposals during bidding. The contractor's primary motivation to submit an alternate proposal is economic, attempting to gain a competitive edge by submitting an alternate proposal. By combining available equipment and his expertise with certain methods in a specific design-construction procedure, he hopes to have a lower bid price and a higher profit margin.

Submitting an alternate proposal requires design capabilities. European contractors usually have in-house design capabilities, although in some cases they may also hire consultants.

Philosophy, historic developments, technical requirements, and problems related to the official design and alternate proposal are discussed in detail in two sections of this chapter (5.2 and 5.3). Section 5.4 describes design methods and ground classification procedures both in Germany and Austria, concentrating on subway tunnels in Germany (5.4.2) and dealing with design of transmountain tunnels in Austria (5.4.3). Problems of site organization are presented in Section 5.5; they include: type of equipment, number of points of attack, advance rates, crew sizes, shift arrangements, and wage rates. As the situation differs between Germany and Austria, the problems will be discussed in separate subsections (5.5.1 and 5.5.2). A section on safety problems (5.6) in tunnel construction precedes the conclusions (5.7) of this chapter.

5.2 THE OFFICIAL DESIGN

The official design is part of the bid schedule. It is practically proven, which does not mean, however, that it is outdated, since the official design undergoes changes based on experience gained. For a better understanding, a brief historic overview is useful.

Tunnel construction procedures advanced during periods when many tunnels were constructed. One period of intensive tunnel construction in Europe is linked to the railroad construction and, in particular, to the large transalpine tunnels. This period started in the middle of the last century and lasted into the beginning of this century. During this period of classical tunnel construction, timbering was used for the initial support and masonry for the final liner. Particularly in Europe, different methods evolved and received generic names. For example, cut-and-cover tunnel construction methods for subways were called Berlin or Hamburg methods, depending on their particular characteristics. (For details, compare Mandel and Wagner, 1968).

Mined tunnels in the railroad tunnels period were constructed by the Austrian ('old'), Belgian, German or English method. The Belgian and the German methods will be described in some more detail below. The old Austrian and the English methods used timber supports and masonry lining with particular excavation sequences. The old Austrian method is not used in practice any more since it does not allow the use of large equipment. For a detailed description of the methods, the reader is referred to Szechy (1966). However, some of the methods were adapted to new construction materials and equipment. In particular, timber support was replaced by steelsets and shotcrete, and masonry, by concrete. With the advent of a new tunnel construction boom, some of the old methods were revived. For example, the Belgian method was adapted by replacing masonry with concrete. In the Belgian method (Figure 5.1), the top heading is excavated and supported close to the face with a thick rigid liner, which is widened at the springlines (haunches) to achieve sufficient bearing capacity. The Belgian method was initially used for the Massenberg Tunnel near Leoben. However, after 70 m of heading excavation, the roof collapsed because the footings were probably too narrow (Figure 5.2, left side). The support procedure was changed (Figure 5.2, right side) with only a thin shotcrete liner (20 cm) and rockbolts; note that instead of an arch resting on a footing, there is now a closed ring. This was one of the first successful applications of the New Austrian Tunneling Method, NATM; however, it did not have this name at the time. More details on the Massenberg Tunnel can be found in Rabcewicz (1965) and Einstein et al. (1977).

The German method, or core construction method, has also been revived in several instances. Of particular interest is the subway construction in Munich. The official design and the actually used procedure for the Marienplatz Station of Line U3/6 followed the German method. The Sendlingertoplatz Station of Line U8/1 was also designed in this manner, but was built by an alternate method.

The Marienplatz Station is a mined, twin-tunnel station some 30 m below the Munich City Hall. Figure 5.3 shows cross-sections with the excavation procedure. First, two base

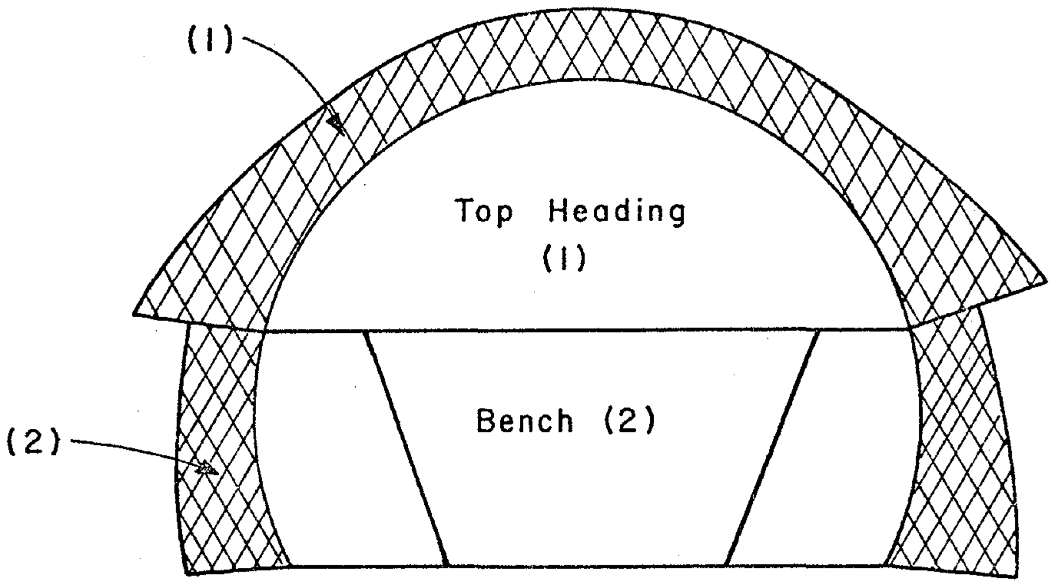


FIGURE 5.1 SCHEMATIC CROSS-SECTION OF A TUNNEL BUILT ACCORDING TO THE BELGIAN METHOD

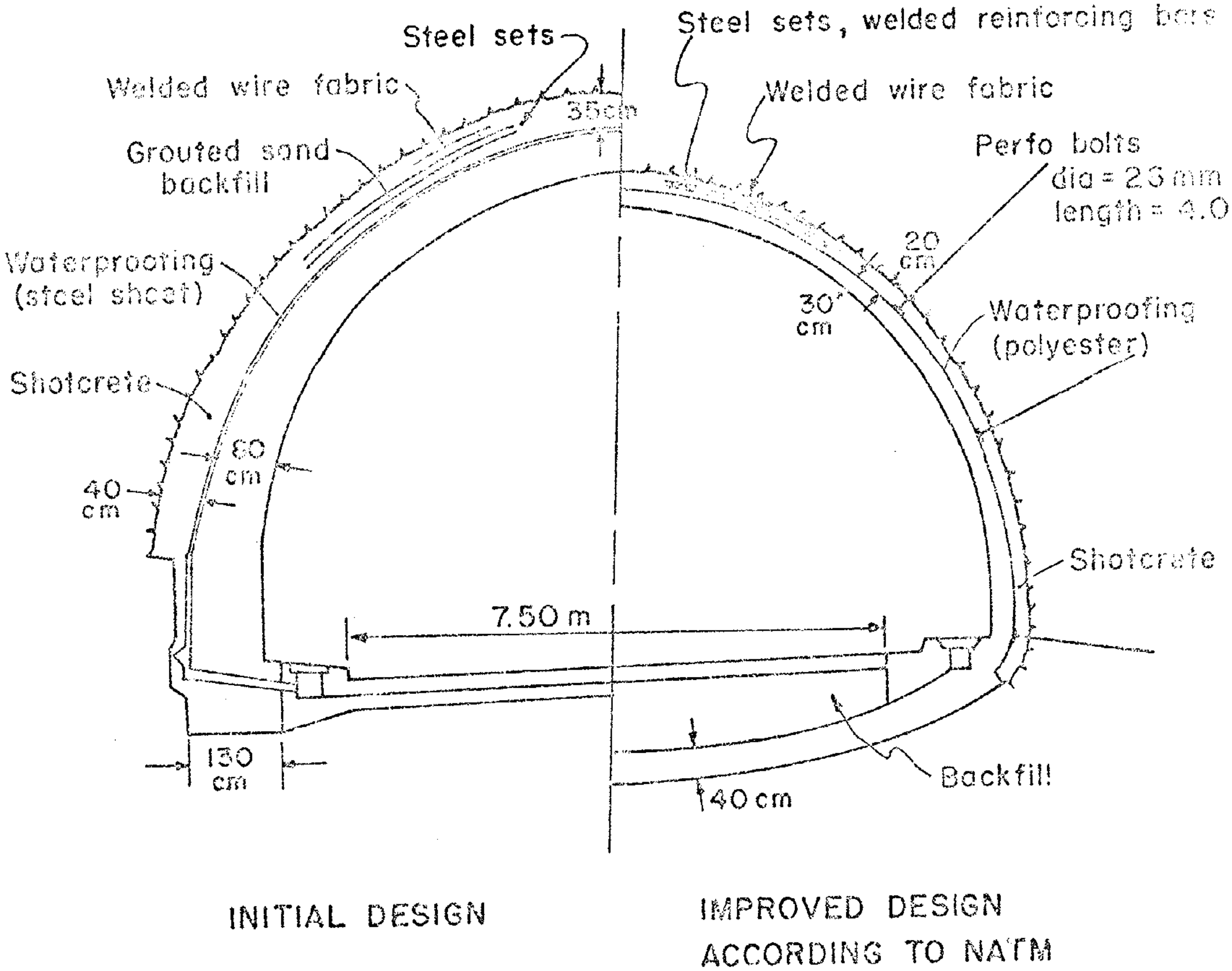


FIGURE 5.2 COMPARISON OF SUPPORT (BELGIAN METHOD VS. NATM) FOR THE MASSENBERG-TUNNEL NEAR LEOBEN, STEIERMARK, AUSTRIA (FROM RABCEWICZ, 1965)

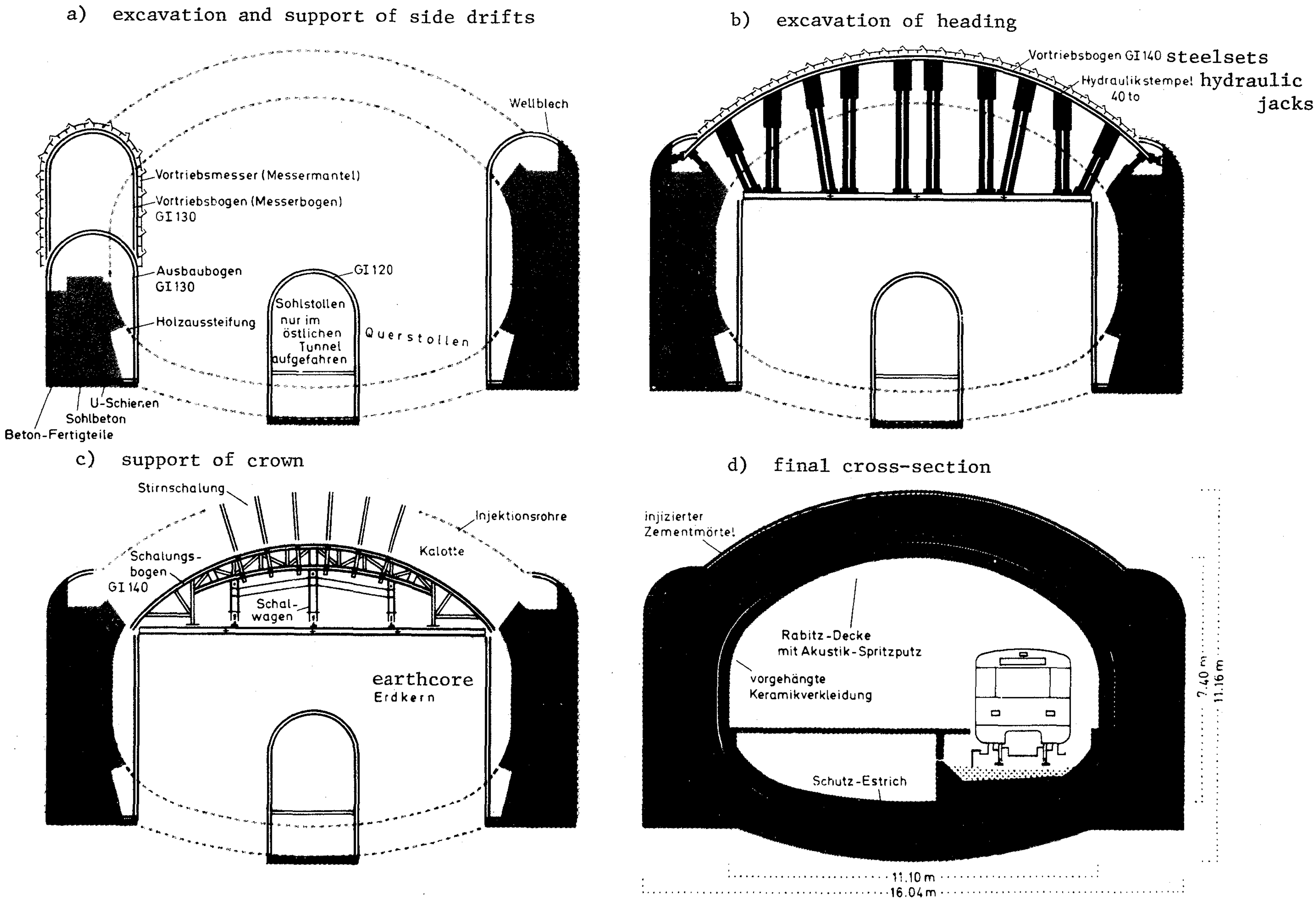


FIGURE 5.3 CROSS-SECTION OF MARIENPLATZ STATION BUILT WITH GERMAN OR CORE CONSTRUCTION METHOD (FROM MUNICH, 1971)

sidedrifts were driven (Figure 5.3a), then the concrete was placed, the two upper side drifts were excavated and the linear poured. The curved crown was excavated under a blade shield (Figure 5.3b), which was supported by hydraulic jacks on the remaining core; the crown support was then poured in sections (Figure 5.3c). Finally, the remaining core was removed and the invert placed. The linear is very thick (approximately 2 to 2.5 m).

This method applied at the Marienplatz Station was primarily based on textbook experience. The German requirement to have static computations approved by a licensed inspection engineer may favor "textbook" construction methods. Since the method was successful at the Marienplatz Station, it was adapted for the design of the Sendlingertorplatz Station. However, the construction contract was let based on an alternate proposal, using the NATM (compare, Appendix A-1, Munich). For this alternate proposal, the contractor had to prove the technical feasibility with finite element analyses and a test section. This shows a basic willingness of the owner to accept radically new approaches; this particular section actually provided the breakthrough for the NATM in Munich. The experience gained from the construction of the Seidlingertorplatz Station was incorporated into the official design for other sections. Nevertheless, most of these sections were bid with alternate proposals employing new methods proposed by different contractors. This led to the previously mentioned elimination of complete official designs in Munich (Section 4, Section 4.3) and their replacement by alignment requirements and recommendations on which contractors base their design-construction proposals and bids. Even without this final step, recent experience is easily incorporated, and what was an alternate method may quickly become the official design.

5.3 ALTERNATE PROPOSALS

Alternate proposals have already been discussed in Section 5.2 on official design, and in Section 4. The prerequisites for alternate proposals are briefly recalled:

(1) alternate proposals must be permitted by the owner,

(2) the contractor must have the motivation to submit an alternate proposal (economical, technical), and

(3) the contractor must have the design capability to submit an alternate proposal.

These three prerequisites are fulfilled for most bids solicited in Austria and Germany. Standards and most bid documents expressly allow alternate proposals. The contractor has primarily an economic incentive in submitting an alternate proposal, hoping to become the lowest bidder with his proposal. The contractor first estimates the costs for the official design, and then decides whether to submit an alternate proposal. (Some, but not all, owners require bids on the official design in addition to alternate bids.) The price of the official design may only reflect his first estimate and does not necessarily reflect the lowest bid he could submit on the official design.

As mentioned before, an alternate proposal allows the contractor to optimize his equipment, procedures, and his trained crews. The equipment and techniques develop with time, and hence the contractor who is constantly dealing with new equipment is able to best judge the feasibility of such equipment. Different contractors prefer different procedures. For example, one contractor favors the excavation of the ground with hydraulic shovel excavators; another uses small roadheaders (partial face TBM). The excavation-support procedure is different for the two types of excavation. A hydraulic shovel requires a large height, stands on the invert, and excavation is by a short heading and benching (the excavator reaches to the face of the heading) or an inclined face (the procedure is similar to the one used in Essen, Section 2.4, Appendix B.2). The roadheader, in contrast, has a long reach and alternates between heading and bench; thus the ramp construction method was developed (Appendix B-1, Munich, Section 16).

Contractors often have subsidiaries in different cities, and may thus work on various tunnel construction jobs. This enables them to gather substantial experience and to apply this knowledge in other locations. Designers, however, encounter difficulties in transferring newly developed methods from one place to another. There is always a lag from the beginning of a design process to the actual construction, and during this time there may have been new developments that cannot be incorporated.

5.3.1 Technical Requirements for Alternate Proposals

The alternate proposal has to be technically feasible. The Standards (VOB Part A) or the bid documents state that the proposed alternate method must be technically equivalent to the official design (naturally, it must be technically superior). In addition, the bid documents require that an alternate proposal is: "...technically fully developed and that it can be checked and compared...," which means that alternate proposals must be studied and designed to the same level of detail (in particular the static computation) as the official design. Even if the contractor only changes the excavation procedure and uses the official design for the liner, he still may be required to do design work showing the adequacy of each excavation-support step (see Section 5.4.2).

The feasibility may be demonstrated in several ways; among them are:

1 - test section prior to the start of tunneling;

2 - application of the same method of another site with comparable ground conditions; and

3 - finite element analyses.

Thus the major problem in submitting an alternate proposal is to have a design that will be approved.

The owner sets very high standards for the approval of alternate proposals, with economic considerations ranking second to the technical feasibility of the proposal. The economic gains due to an alternate proposal are not shared equally and explicitly between contractors and owners as in value engineering. The contractor has to estimate his risk and profit margin when preparing the alternate proposal and will not obtain additional benefits when submitting an alternate proposal. Only the owner benefits from the difference between lowest bid on the official proposal and the successful alternate bid.

In summary, alternate proposals make it possible to have the contractor develop his own ideas and optimize his design-construction procedures, resulting in successful bids and in economic advantages for the owner and contractor. Both parties benefit technically and contractually from this contractual procedure. Feedback of new experience into new construction is faster on the contractor's level than on the owner's level. Favorable experience with alternate proposals led some owners to abandon detailed official designs and to call for design-construction bids.

5.4 DESIGN AND RELATED PROBLEMS

5.4.1 Introduction

Differences in design philosophy exist between subway and transmountain tunnels. Reasons for these differences are:

- the level of exploration (uncertainty) is different for subway (shallow) and transmountain (deep) tunnels;
- technical problems are different; in shallow tunnels the major concern is surface deformations; in deep tunnels the stability of the opening is important;

- in Germany (subway tunnels), the design has to be approved by a licensed inspection engineer, while this requirement does not exist in Austria.

Hence, the design procedures used in these two countries will be discussed in two parts. Section 5.4.2 deals with the design of shallow subway tunnels in Germany, while Section 5.4.3 deals with the design of transmountain tunnels in Austria (the Vienna subway has not been studied).

5.4.2 Design of Shallow Subway Tunnels in Germany

It shall be recalled that:

(1) An approved static computation is required. Approval is given by a licensed inspection engineer. The licensed inspection engineer is not a government agency, but may be a highly recognized engineering firm or often a university professor who performs this task with the aid of his assistants. The owner awards the contract for verification of the static computation to this inspection engineer.

(2) The most important problems in shallow tunneling are surface effects.

(3) Ground conditions are fairly well known due to the shallowness of the tunnel and the dense exploration.

Static computation procedures as used in the design of subway tunnels will now be discussed, followed by comments on soil parameter selection and in situ measurements.

Static Analyses. The primary concern in any static analysis is to determine the support (liner) loads. Before the advent of finite element analyses, FEM, only frame analyses subject to ground loads were used.

Frame Analyses. Ground loads are assumed and redistributed on an embedded ring (Figure 5.4). The ground loads correspond to the initial stresses in the ground, i.e., the vertical stresses in the crown correspond to the overburden stress at that elevation and the lateral earth pressure is the earth pressure at rest. However, in some cases the horizontal stress is assumed equal to active earth pressure. The liner is divided into articulated segments. The spacing of the articulated joints is chosen such that the segments between two joints can be approximated by a straight beam. Ground loads are assumed to be distributed over the segments (e.g., Segment A, Figure 5.4) or often simply to be nodal forces. Ground reaction is represented by linear radial springs (sometimes non-linear springs are introduced). The joints in the liner are selected in order to represent actual conditions, i.e., in case of a continuous liner (shotcrete), the joints transmit bending moments, whereas in case of actual joints (precast elements) no bending moments will be transmitted. The analysis is usually performed with a computerized frame analysis program (e.g., STRESS).
As previously mentioned, the lateral earth pressure is varied between at rest and active (a discussion of earth pressure assumptions will follow later in this section). In addition,

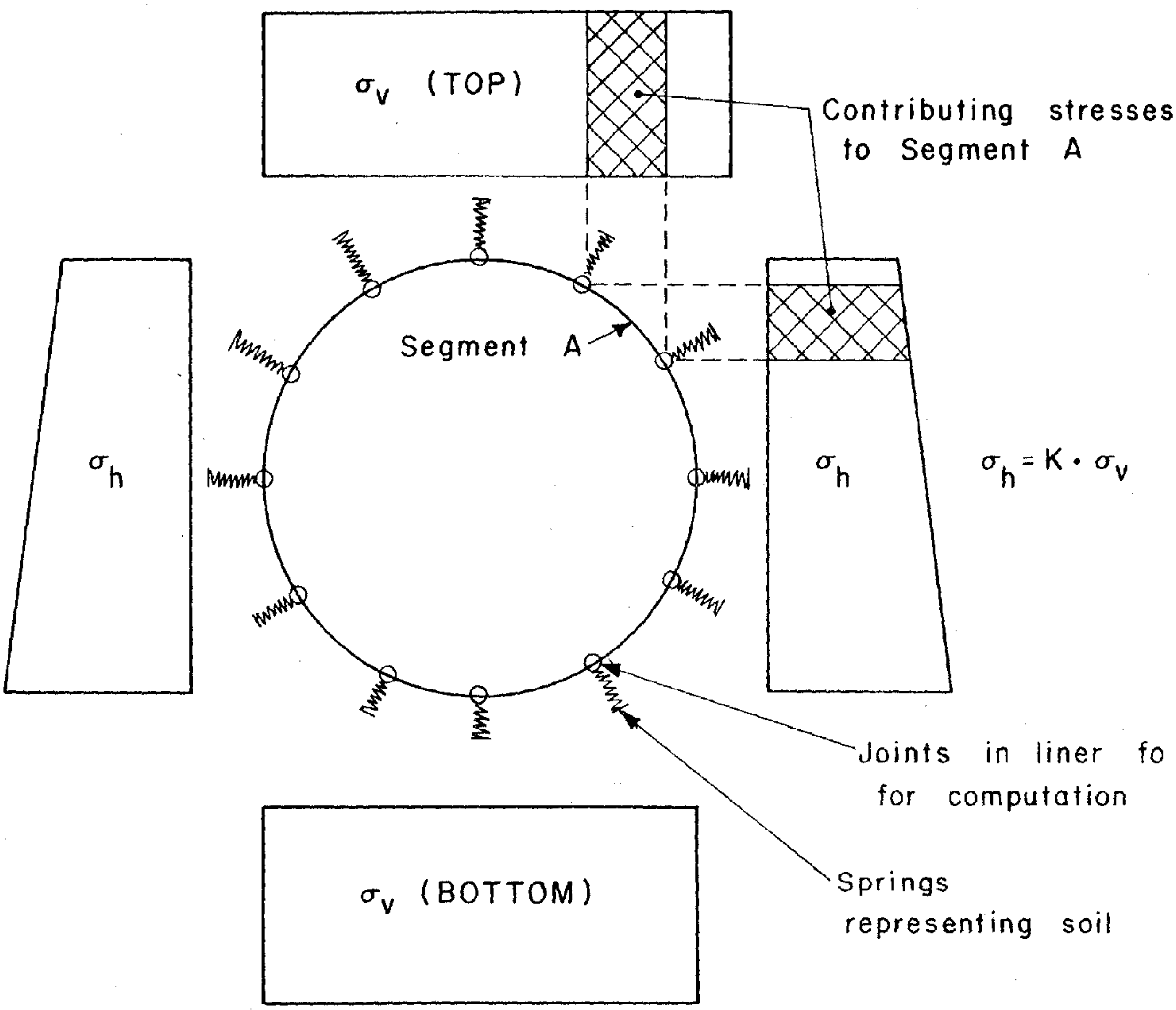


FIGURE 5.4 ANALYSIS PROCEDURE FOR EMBEDDED RING ON WINKLER FOUNDATION

the degree of embedment will be varied; e.g., often a sector of 90 degrees in the crown is assumed to be subject to loading without ground reaction. This procedure (Windels, 1966) is believed to model the incomplete embedment of a segmented liner. By varying the degree of embedment, the liner loads, in particular the liner bending moment, will vary. However, the selection of the proper design parameters often depends on the preferences of the licensed inspection engineer (Pruefingenieur), who has to approve the static computations. These frame analyses cannot predict surface displacements due to tunneling. Also, the displacements of the liner computed in the frame analysis are not considered in practice.

Finite Element Analysis. Plane strain finite element analyses are used if the surface displacements and/or more complicated boundary conditions (buildings, parallel tunnels) have to be considered. However, plane strain finite element analyses cannot consider the deformations in the ground ahead of the face. Since stresses are redistributed ahead of the face, large displacement of the ground and lower liner loads result in reality than in an "instantaneous" excavation as simulated by a plane strain finite element analysis.

Hoffman (1975) proposes a procedure to determine maximum displacements and maximum liner loads. To obtain the maximum displacements, an unlined excavation is considered (Figure 5.5)

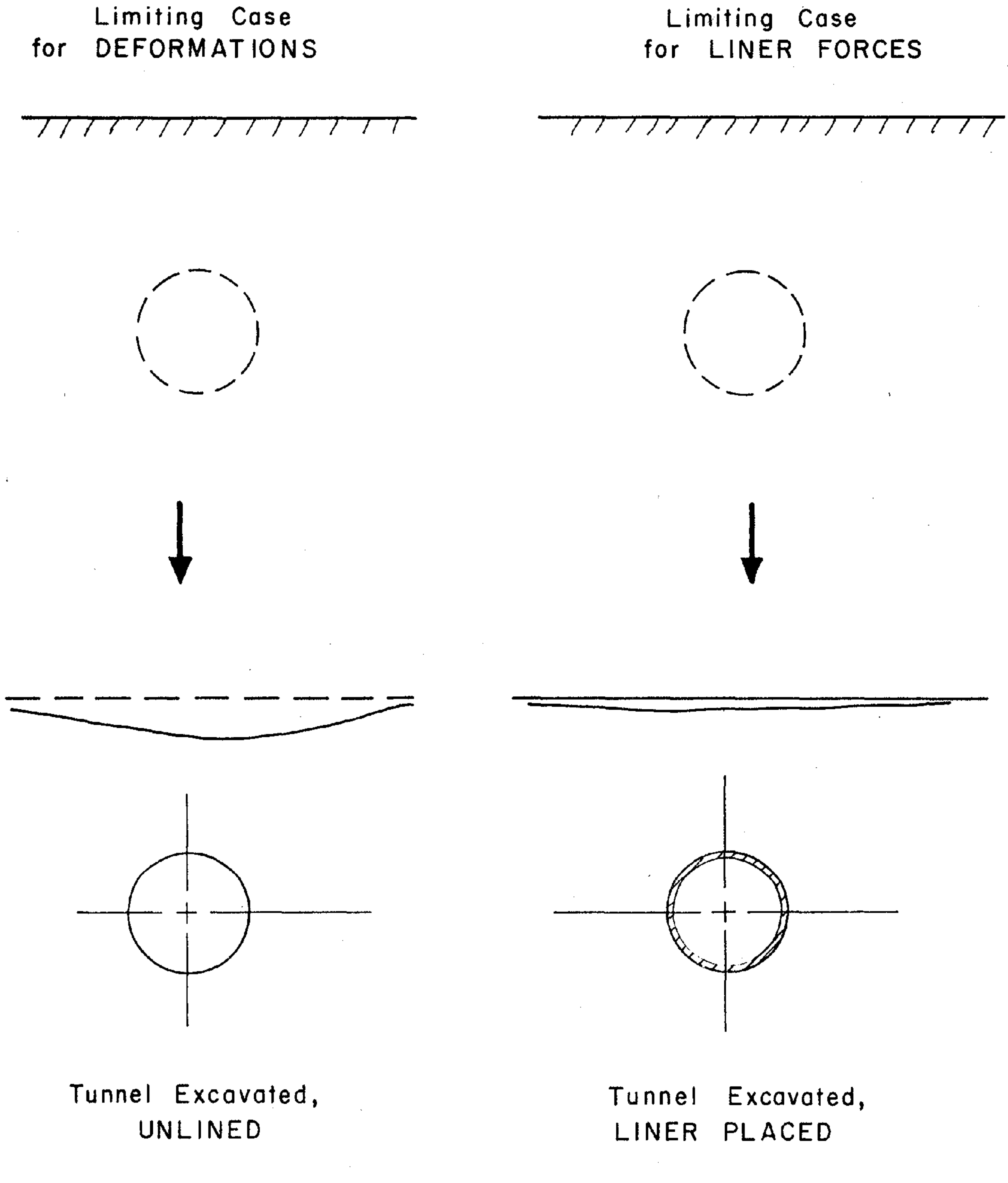


FIGURE 5.5 PLANE-STRAIN FINITE ELEMENT ANALYSIS FOR SHALLOW TUNNELS (AFTER HOFMANN, 1975)

in the finite element analyses. The maximum possible liner loads are determined by considering an instantaneously placed liner, i.e., excavation and liner placement occurs simultaneously. A comparison of the analysis performed for Section A2 in Bochum shows that the displacements (at the surface, and at tunnel level) obtained by this procedure agree fairly well with the computed ones; however, the actual contact stresses and linear loads are approximately five times smaller than the computed ones. Swoboda (1978) has developed a different procedure (Figure 5.6), primarily for the design of tunnels in Munich constructed by the ramp-construction method. The procedure allows the simulation of different excavation stages (shown in Figure 5.6). In case of multiple tunnels, the procedure can be expanded to include additional steps. No data is yet available supporting this analysis procedure, since the tunnels are presently under construction. Note that one analogous procedure is incorporated in the Simplified Analysis Method, Volume 1 of this report series.

The preceding discussion has primarily dealt with the static design of the initial support. An interior final liner of impervious concrete is poured in Munich (no waterproofing is necessary). This liner is substantially reinforced. A static analysis of this interior liner is also required. Since equilibrium between ground and support has already been achieved with the initial support, the interior liner would take only water loads and loads of future buildings, but no

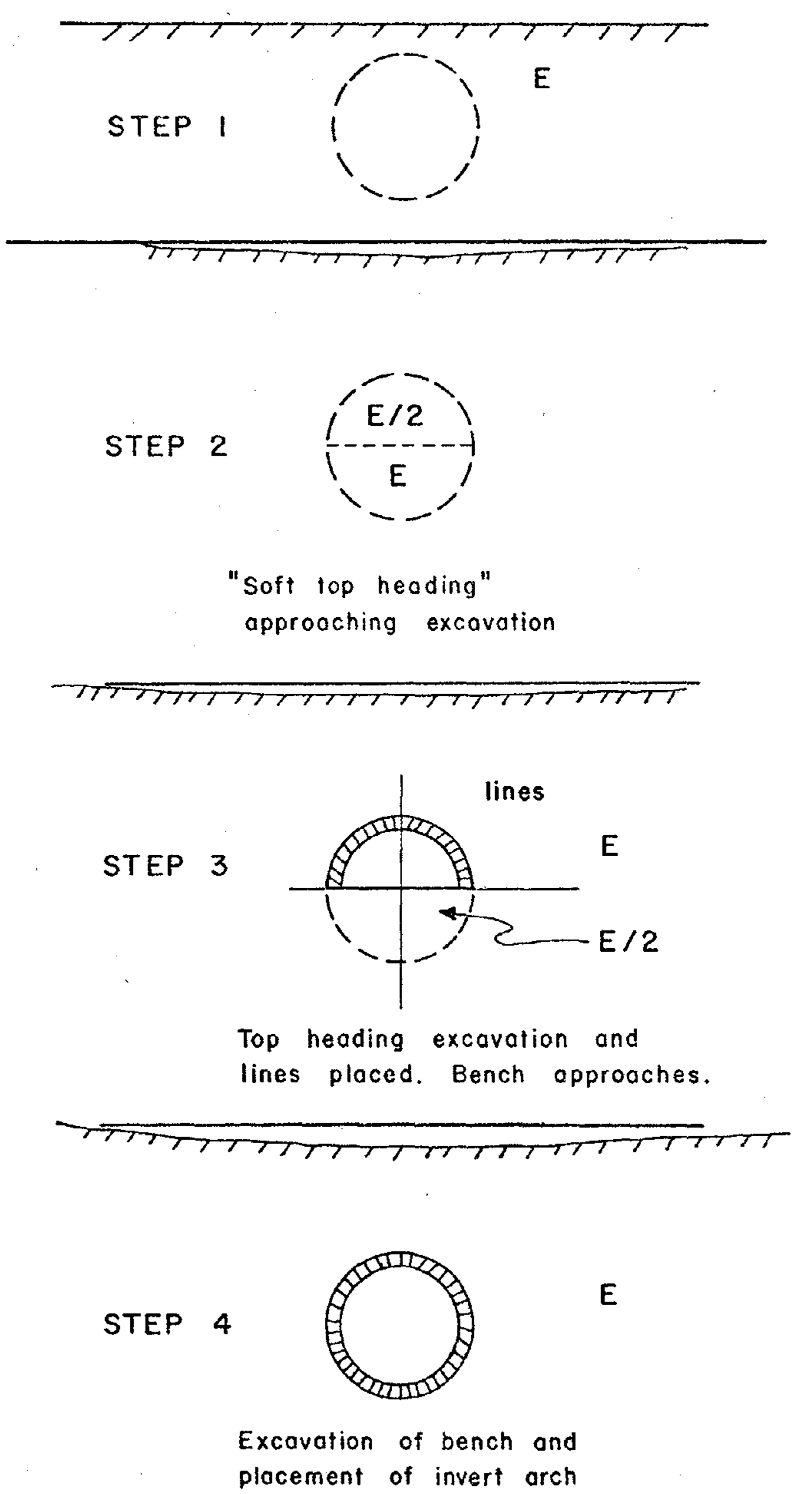


FIGURE 5.6 PLANE STRAIN FINITE ANALYSIS PROCEDURE FOR TUNNEL EXCAVATION ACCORDING TO SWOBODA

ground loads. Sometimes the ground loads are "redistributed "; i.e., it is assumed that portions of the ground load are transmitted to the interior liner, e.g., by assuming that a percentage of the ground load is taken by the interior liner (for frame analyses), or by assuming long term deterioration of soil properties (in finite element analyses). Liner loads can be computed in such a manner; however, supporting measurements are lacking. The procedure, which is probably very conservative, is justified at the present time since the long term behavior is not known and since placement of additional buildings on top of the tunnel may be possible.

Soil Properties

The bid documents in the city of Munich contain a table of soil properties for design. Drained strength parameters and drained stress-strain characteristics as well as modulus of elasticity for loading and unloading are given. The details of the tests are not described. However, they are presumably triaxial tests isotropically consolidated and then axially loaded to some level to obtain the loading modulus; the unloading modulus is obtained by unloading from this level. Strength properties are determined by loading the specimen to failure. These properties can be used in elastic and elasto-plastic finite element analyses. In the case of the Lorenzkirche Station in Nurnberg, nonlinear finite element analyses were performed, since special tests were run to determine the complete stress-strain characteristics (Bauernfeind, 1977).

Horizontal earth pressure is important in designing tunnels (until recently in Munich the ratio of earth pressure at rest, K_o, was determined with the empirical relation $K_o=1-\sin\phi$). Since the friction angle ϕ is approximately 30 degrees, a $K_o=0.5$ results. Recently, in one tunnel section, the covergence of various cross-sections was monitored (oral communication in Munich, no data available), and based on these observations, it was decided to increase the ratio of earth pressure at rest to $K_o=0.8$.

Performance Monitoring

With the advent of new tunneling methods, new monitoring techniques evolved. In particular, the deformations due to tunnel driving are more carefully monitored. The most complete performance monitoring is in the design of cross-sections for which a FEM Analysis was performed.* In such sections, deformations at the surface and at different depths as well as liner loads and contact stresses between ground and liner are monitored (see example in Appendix B-1). The monitored displacements reasonably agree with the computed ones; however, liner loads are considerably lower (e.g., in Bochum by a factor of 5).

Although performance is monitored, feedback into the

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*Only typical cross-sections are analyzed. The results from these are assumed to be valid over the length of tunnel for which these design cross-sections are representative.

construction process is not yet as direct as it could be in a fully developed observational procedure. A good example of possible support adaptation based on surface effects is the tunnel in section 2A in Essen (Appendix B-2). In a section where the steelsets (in crown only) were spaced at 1.25 m, surface settlement was 4 cm, whereas in a section underneath a building, the steelsets (crown and invert) were spaced at 0.85 m and the surface settlement was only 2 cm. Another example of observational feedback occurred in Munich, where, as mentioned in Section 5.4.2 on soil properties, convergence monitoring in tunnels of various cross-sections led to an increase of the ratio of earth pressure at rest. At present, performance monitoring is intended to substantiate the predicted performance. For an integrated observational method, detailed procedures have to be developed further and feedback of monitored performance into construction has to be improved.

Summary

The procedures used for the design of subway tunnels are primarily structural analysis methods. Only liner loads which had to satisfy the requirements of an approved structural analysis were of primary concern until recently. However, surface deformations are considered in finite element analyses, which only recently became generally accepted.

Field measurements are primarily taken to substantiate the analyses; feedback of field measurements into construction is delayed, and the procedure is thus not a true observational approach (Peck, 1969).

5.4.3 Design of Tunnels and Ground Classification in Austria

The design procedures for Austrian transmountain tunnels are different from those for shallow subway tunnels in Germany due to the following reasons:

(1) The standards do not require a static analysis.

(2) The exploration is primarily by mapping, since borings are, in most cases, too expensive due to the great depth.

(3) Compared to shallow subway tunnels, ground conditions for deep transmountain tunnels are largely unknown.

In the following lines, the present design procedures in Austria are recalled, followed by a brief historic overview of ground classification and a discussion of tunnel design problems. A summary and evaluation of collected data concludes this section.

The design procedure for transmountain tunnels involves several steps:

(1) Exploration

Existing geologic data is analyzed (map, possible construction records of existing tunnels). Borings are rarely used due to the great cost and limited information they provide. However, sometimes pilot tunnels are constructed prior to bidding.

(2) Preliminary Design

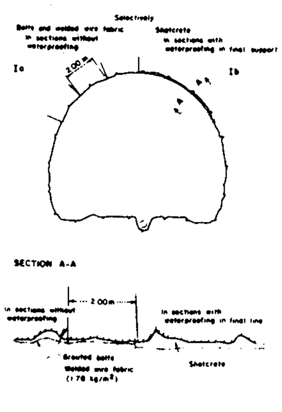
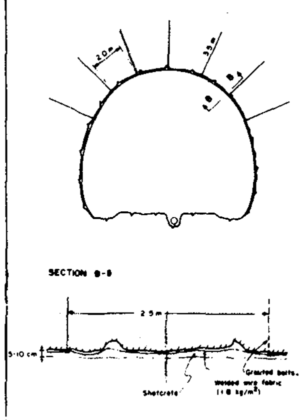
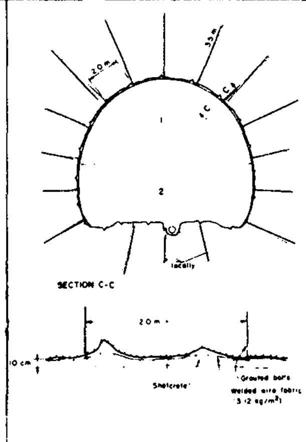
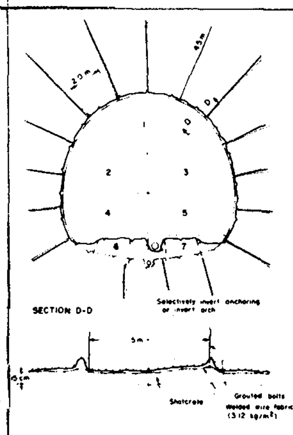
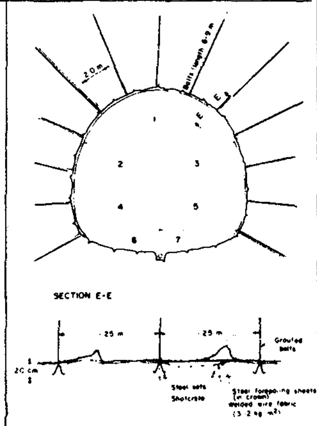
Relations between typical ground conditions, excavation procedures, and support requirements are developed and formulated in the form of "Ground Classes." Ground conditions are described qualitatively the level of detail depending on the information available from the exploration phase. To each range of ground conditions or Ground Class, an excavation procedure (full face vs. heading and benching, round length) is assigned, as well as typical support requirements. Table 5.1 summarizes the typical Ground Classes (a detailed description can be found in Einstein et al., 1977)

(3) Construction

After each round, representatives of the owner and the contractor decide on the spot on the ground class and the required support and excavation procedures. The ground conditions are mapped (maps of the face and the circumference of the tunnel). The two representatives sign a form, documenting the agreed-upon ground class. In case of disagreement, the owner's opinion prevails, but the case is submitted to a mediator, usually preassigned, who decides later on the ground class. (The contractor may place more support than the owner wants if he thinks safety requires this. Payment of the additional support will, however, depend on the mediator's decision.

(4) Monitoring

The performance of the tunnel and support is

Class	I	II	III	IV	V
Ground Conditions (abbreviated)	solid rock (standfest)	rock is over-breaking	over-breaking to lightly squeezing ground	squeezing ground	heavily squeezing ground
Excavation Procedure Round Length (RL)	full face RL = 4 m (limited by equipment)	full face RL = 3 m	full face RL = 1.5 m or heading & bench RL = 3.0 m	heading and benching (1 to 2) RL = 1.0 to 1.5 m	heading and bench (no blasting) RL = 0.5 to 1.0 m scatface (3 cm)
Support	Bolts locally Capacity = 15 tons length = 1 to 3 m with wire fabric or Shotcrete in crown	Shotcrete = 5 to 10 cm Welded wire fabric ($1.8\ kg/m^2$) in crown Prestressed grouted bolts: Cap. = 15 tons L = 3 to 4 m, one per $4 - 6\ m^2$	Shotcrete = 10 cm Welded wire fabric $w = 3.12\ kg/m^2$ Prestressed grouted bolts: L = 4 m, one per $3\ m^2$	Shotcrete = 15 cm WM = $3.1\ kg/m^2$ SS = 21 kg/m spaced 1.0 to 1.5m Grouted Bolts: L = 4 to 6 m 1 per $2m^2$ Cap. = 25 tons	SC = 20 cm WM = $3.1\ kg/m^2$ SS = 21 kg/cm Spaced 0.7 to 1.5 m Bolts Cap. = 25 tons L = 6 m andlonger 1 per $1.5\ m^2$
Figures	 SECTION A-A	 SECTION B-B	 SECTION C-C	 SECTION D-D	 SECTION E-E

monitored. Convergence is typically monitored every 10 to 50 m depending on the ground conditions. Principal monitoring cross-sections (with convergence measurements, extensometers, and stress cells) are typically placed every 500 m. If the performance is judged unsatisfactory, additional support will be placed. Performance criteria are usually the total deformation over a certain time and the deformation rate.

(5) Final Liner

The final liner is placed once the monitored performance reaches an appropriate level. The final liner is non-reinforced concrete of approximately 30 cm thickness. Depending on the rate of residual deformation of the initially supported opening, the specified strength of the concrete, and possibly the thickness of the interior liner, is modified (Appendix B-7). The contribution of the initial support is taken into consideration in the design of the final liner.

History of Ground Classification

A short historical review of the classification procedures should provide some insight into its concept.

The earlier attempts to classify ground conditions were made by Wilhelm Ritter (1879), Bierbaumer (1913), and Kommerell (1940). These classifications give the weight of a fictitious body over the tunnel as a function of ground conditions. The relations were improved by Stini (1950) and similarly by Terzaghi (1946). Both classification procedures provide roof loads as a function of the ground conditions and opening size. The roof loads are in turn used to design the support. Rabcewicz (1957) published a classification which directly relates ground conditions and support requirements. In the following year, Lauffer (1958) published the widely known stand-up time charts that reflect the experience of TIWAG (Tiroler Wasser-kraftwerke AG = Hydroelectric Power Company of Tyrol, Inc.) during the construction of pressure-tunnels of the Prutz-Imst Power scheme in Austria. Data (support and ground conditions) were available for a pilot tunnel with a cross-section area of 10 m^2 and the main tunnel (in the same alignment) with a cross-sectional area of 25 to 30 m^2. Lauffer's relations are largely qualitative and based on limited experience; extrapolations should thus be made with care.

The direct qualitative relations between ground conditions and support (and excavation procedures) were further developed, in particular by Rabcewicz, Pacher and Golser (1974), reflecting experience gained during the construction of the Tauern and Katschberg Tunnels. Work by M. John (1976) includes the additional experience gained during the initial phases of construction of the Arlberg Tunnel. Table 5.1 is an abbreviated summary (John, 1976) of the ground classes for the bid documents of the Arlberg Tunnel. (The actual classification was made much more detailed, particularly the description of the ground conditions, see Einstein et al., 1977). The present state-of-the-art of the Ground Classification procedure is included in the draft of an Austrian standard on tunnel construction work (ONORM B 2203). The final version of this standard has not yet been published and the draft will certainly undergo changes.

Correlations between ground conditions, excavation procedure, and support requirements are qualitative. The major classification criteria are behaviorial, i.e., they describe in a qualitative way how the ground behaves at the face, the crown, and the springline. These classification criteria may suit a well-trained, experienced tunneller (engineer, geologist); however, great care must be applied when these qualitative criteria are used by inexperienced personnel, and when they are transferred to another geological setting, particularly to conditions where no experience with the NATM exists.

In Austrian practice, the pure behaviorial classification is supplemented by a geologic-petrographic classification for each particular project; in some instances, classification focuses only on these geologic-petrographic criteria. However, a simple petrographic classification may cause problems, since the ground often varies in the same cross-section (as described in Section 4.5.3, in some cases, quite substantial disputes arose over these issues).

The NATM ground classification does not explicitly consider the following parameters which are, however, implicitly anticipated:

(1) initial state of stress (overburden, horizontal stresses)

(2) strength of the rock mass (intact rock, discontinuities)

(3) size of the opening.

The actual relation between ground conditions, support requirements, and opening size are estimated by means of Rabcewicz's shear body analysis and relevant assumed ground strength parameters (John, 1977 and 1978). The selection of ground strength parameters is largely judgmental.

Data Collected

One of the objectives of this information gathering trip was to obtain data on ground conditions and placed support. Data were collected for seven tunnels and are summarized in Table 5.2. Detailed data are compiled in the appropriate appendices. Detailed data of 2.3 km of the western section of the Arlberg Tunnel was obtained from Ingenieurgemeinschaft Lasser-Feizlmayr Consulting Engineers, Innsbruck, with the permission of the Arlberg Strassen tunnel AG, Innsbruck. Data from the Tauern (6.4 km length) and the Katschberg Tunnels (5.4 km length) were obtained from the owner, the Tauern Autobahn AG (Tauern Highway Authority), Salzburg.

The data from these three tunnels include detailed geologic descriptions, actually placed supports (bolts, shotcrete, steel sets), and the monitored performance (convergence, stress measurements in final liner).

Convergence monitoring was used to adapt the support and to decide when to place the final liner. Interesting details of the design-construction techniques that were developed in the

TABLE 5.2 SUMMARY OF DATA COLLECTED

Case	Length	Cross-Section	Geology	Problems Encountered	Detailed Data (see appendix)
Werfentunnels Brentenberg Zetzenberg Helbersberg	615 + 568 m 540 + 568 m 836 + 803 m (2 parallel tunnels)	80 m^2 80 m^2 80 m^2	Dolomites Dolomites Werfen-schist	None None None	Pacher A-9
Mitterberg	2.2 km	80 m^2	Gneiss to mica, schists with 2 persistent joint sets.	Large overbreaks and problems with stabilizing the roof. Predictions of support requirements from pilot drift has not been satisfactory.	Graz. A-14
Selzthal	1.01 km	80 m^2	Schists to weathered phyllites	Large crown settlements stopped with invert closure. Slope above tunnel is unstable, influence on tunnel not yet determined. Possibility of high horizontal stresses, since tunnel is near base of slope.	Selzthal B-6
Arlberg	14 km Data for 2.3 km	90 m^2	Mica schists to gneiss and phyllites. Discontinuities striking subparallel to the tunnel.	Large convergences of the tunnel (70cm) developed after excavation. Sometimes heavy support requirements (600m/m of bolts) in some sections. Accidents due to unfavorable ground conditions : 1) Roof-fall after placement of support which proved to be insufficient (2 fatalities). 2) Popping rock in cross-cut due to discontinuity pattern (2 fatalities).	Arlberg B-7
Pfaender	6.7 km (Data for 1.5 km)	80 m^2	Shales, sandstones, conglomerates	In tunnel no problems, except insufficient support in pilot tunnel (additional support had to be placed to secure ventilation through pilot tunnel.) In northern shaft, the TBM was removed and shaft sinking continues by drill and blast.	Pfander B-8
Tauern	6.4 km	90 m^2	Different Types of Phyllites: - graphitic phyllites - sericitic phyllites - quartzitic phyllites - serpentine - anhydrite - dolomite - marble - limestone	Convergences of up to 25 cm were encountered. In section roof settlements of 120 cm. Large convergences encountered resulted in change of ground classification with the addition of a special class with longer and denser bolts.	A-10
Katschberg	5.4 km	90 m^2	Gneisses, amphibolites and mica schist in Northern part. Phyllites and schist in southern	No problems in northern section: No large convergences observed in southern section.	A-10

Arlberg and Tauern Tunnels are described later in this section under the subheading "overexcavation" and "final liner" as well as in Appendices A-10 and B-7. A detailed analysis of the data will be given in Volume 5 of this series of reports.

Problems in Tunnel Construction

A summary of problems encountered in tunnel construction is shown in Table 5.3 and will be discussed below.

Overbreak is a problem in blocky rocks. From a technical point of view, reliable prediction of overbreak is desirable, as large overbreak results in large amounts of concrete for the interior liner. However, even a pilot tunnel, as was the case in the Mitterberg Tunnel (Appendix A-14), cannot always provide an accurate prediction, since relating the behavior of a small to a large tunnel is unresolved.

Closely related to overbreak are rockfalls, which from a technical point of view may be considered to be large scale overbreak. Rockfalls are particularly dangerous when they occur suddenly with little warning. Two types of rockfalls can be differentiated; one type occurs as the face caves toward the tunnel opening with a collapsed zone that often extends above the final line of excavation (Figure 5.7). In such a case, the crew will usually see that something may happen and an escape way will not likely be cut off. More dangerous is the second type of rock fall, roof collapses caused by insufficient support (see Figure 5.8). Since support has been placed, the crew feels safe and warning signs are difficult to see. In addition, these failures may occur in a sudden manner

TABLE 5.3 SUMMARY OF PROBLEMS IN TUNNEL CONSTRUCTION

Groundclass	General geotechnical description of ground	Problems encountered, not solved
I	Intact rock, few widely spaced joints relative to opening size	None
II	Blocky rock	Overbreak caused by discontinuities Roof falls, failures at the face and often propagating into the roof over the face
III	Blocky-seamy rock	Prediction of behavior of main tunnel based on behavior of pilot tunnel
IV	Crushed rock	For deep tunnels, overexcavation has to be specified to accommodate convergences
V	Disintegrated, altered rock	Loads on final liners

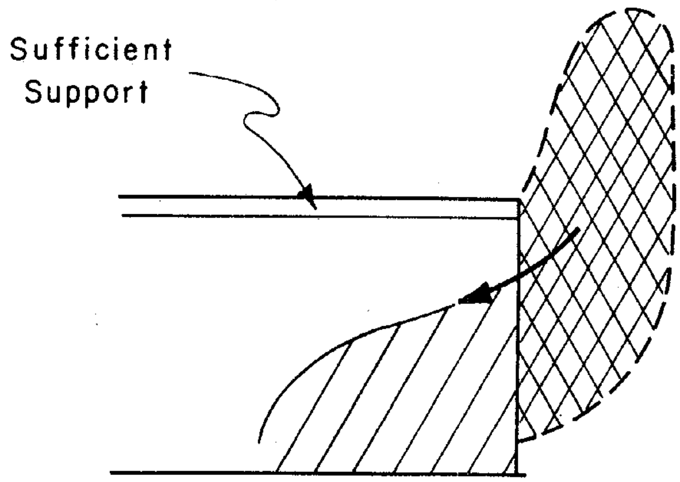


FIGURE 5.7 COLLAPSE AT THE FACE OF A TUNNEL WITH SUFFICIENT LATERAL SUPPORT

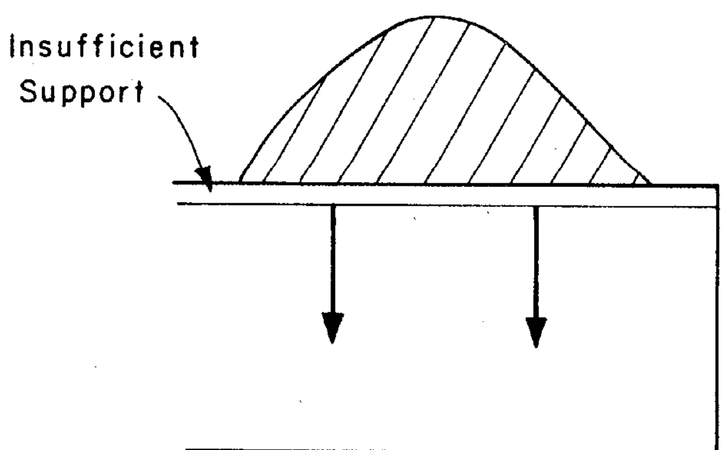


FIGURE 5.8 ROOF COLLAPSE IN A TUNNEL WITH INSUFFICIENT SUPPORT

without advance warning. An escape is often impossible since the route is cut off by fallen rock. Such a sudden roof fall, in an already supported tunnel, occurred between stations 280 and 300 m at the Arlgerg Western Section (Appendix B-7), causing two fatalities. The ground was classified as Class III requiring no steelsets. Due to this accident, steelsets were subsequently incorporated in Ground Class III.

Overexcavation is necessary to accomodate convergence in squeezing ground and requires a reliable prediction. However, a reliable prediction is only possible once experience has been gained in the particular tunnel and geology conditions. For example, in the Arlberg Western Section (Appendix B-7), it became possible to predict the total convergence after approximately 1 kilometer of tunneling. Specifically, it was established that the final convergence, and thus the necessary overexcavtation, will be 50 cm on the average if: 1 - the convergence after two days does not exceed 4 to 6 cm in sections with no major shear zones in the profile of the tullen, and 2 - the convergence after two days does not exceed 8 to 10 cm with major shear zones intersecting the tunnel. If the cited 2-day convergence values were exceeded, additional support had to be placed to keep the final convergence within the 50 cm limit. The earlier the support was placed, the more effective it was, i.e., additional support placed considerably later (1 to 2 months) did not effectively reduce deformations.

The final liner has a theoretical thickness of 30 cm; however, the actual thickness is 50 to 60 cm in most cases. Due to overbreak and conservative overexcavation, it is less expensive to place more concrete than to re-excavate tights. The liner is cast-in-place unreinforced concrete that takes only very little load even in squeezing ground. In the Arlberg Tunnel, the performance was monitored by stress cells and convergence measurements. W. Steiner was told during his visit that only minimal stresses were recorded in the liner (tangential stresses of a few kg/cm^2 - Max = 14 kg/cm^2). In the Tauern Tunnel, larger stresses are monitored (tangential stresses of 60 kg/cm^2). Some fluctuations and an increase in stresses are observed at the Tauern; however, these fluctuations are believed to be caused by seasonal temperature changes. The problem is presently being studied by the Tauern Highway Authority.

Other problems which are not yet fully understood are:

- Effect of rock bolts (density vs. length)
- Effect of cleft and pore water pressures; also, the effect of water introduced by flushing when drilling bolt holes
- Effect of a pilot tunnel (drainage, stress relief)
- Extrapolation from pilot tunnel to main tunnel
- Effect of workmanship

The effect of substituting long rock bolts by denser patterns of shorter bolts and vice versa is not fully understood. Apparently, there are no satisfactory analytical models availble. Experience in squeezing rock indicates that longer bolts are often more effective.

The effect of cleft and pore water pressures has been recognized; e.g., Pacher claims that the presence of cleft and pore water results in a reduction of the ground quality.

Expressed in terms of ground classes, ground conditions with water result in an increase of one to two classes (see Table 5.1) compared to ground with no cleft or pore water present.

The problem of water introduced into the ground during drilling of boltholes cannot be neglected (and was particularly stressed by Treichl, Appendix B-7), especially when large quantities of bolts are placed in bad rock. Wet drilling is required due to health regulations (silicosis) and bit cooling requirements.

Extrapolation from a pilot tunnel to the main tunnel is difficult. It was previously discussed that overbreak could not be predicted in the Mitterberg Tunnel. In the Pfander Tunnel, the problem of extrapolation was to some extent circumvented by contract provisions (Section 4) that basically provided for payment as executed. In the Arlberg Eastern Section, some of the problems may be illustrated by the comparison of convergence measurements in the pilot tunnel and the main tunnel. In one instance, the convergence was larger in the pilot tunnel than the main tunnel. During the interviews no positive, definite answer was received on this subject.

In many interviews, the effect of workmanship was pointed out. One geologist even claims that the mood of the crew is important, that there are some differences whether the work is performed in the middle of a shift or at the end and also if the time-off approaches (end and beginning of the "decade", see Section 5.6).

5.5 SITE ORGANIZATION

Due to differences in site organization between shallow subway tunnels and deep lying transmountain tunnels, a separate discussion is required.

5.5.1 Site Organization for Shallow Subway Tunnels (Germany) Equipment

Shallow subway tunnels can be built in an open-cut or mined. At the present time, open-cut and mined running tunnels have comparable costs. However, the share of mined tunnels has increased since a mined tunnel causes less surface disruption. As a matter of fact, by taking the cost of surface disruption into account, a mined tunnel may have economic advantages. Aspects of open-cut construction will not be discussed in greater detail; rather the difference between various types of mined tunnels will be stressed. In particular, shield tunnels are compared to NATM* type tunnels (Table 5.4). A shield tunnel requires a constant cross-section, while cross-sections can be easily varied with the NATM. The NATM is thus advantageous where the cross-sections change frequently, whereas a shield has advantages in long tunnels with a constant cross-section.

Lining in shield tunnels consists at the present time mostly of precast segmented single shell elements (with waterproofing gaskets); however, in one case, a cast-in-place liner without

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*Recall that in shallow tunnels, the NATM is used in a generic sense rather than relating to specific details of the NATM.

TABLE 5.4 COMPARISON OF SHIELD AND NATM TUNNEL CONSTRUCTION METHODS

	Shield	NATM
Equipment	Shield, mucking equipment, Grout pump (tail void)	Hydro. excavator wall or partial face TBM Shotcrete pump. Mucking equipment (dumper)
Cross-section	Constant	Can vary
Lining, most economical at present time	Single shell, pre-cast elements	Shotcrete plus interior cost-in-place, impervious liner
Advance rates per point of attack Single track tunnels (average) (maximum)	14 m/day (20 m/d)	3.5 m/day 6 m/day
Total advance	Depends on possible points of attack	
Points of attack	As many as there are shields, i.e., 1 to 2. Usually no intermediate points of attack.	several, e.g., 4 up to 10 for large sections. Sometimes intermediate access points are necessary to keep schedule.
Investment	Large	Small
Economics of scale (length)	Yes	No
Worst ground conditions (that can be handled without improvements)	Running and flowing	Slowly ravelling ground
Ground improvements	Not necessary (air pressure can be used)	Grouting may be necessary, but requires fairly high permeability to be economically feasible
Air pressure	Yes	No Fogging due to evaporation from shotcreting

initial support was used following a blade shield (Appendix B-3, Essen). For NATM tunnels, initial support is by shotcrete, followed by an interior liner of reinforced "impervious" concrete. Shields with precast element liners require a large investment for the element plant and the shield, while investments for the NATM are lower.

Advance rates in shield tunnels are higher by a factor of 3 to 4. However, one has to consider the erection time of a shield, which is considerably longer than the startup for a NATM tunnel. Also, with more points of attack, NATM tunneling may achieve the same total advance rate as a shield. However, if additional access shafts are required to provide additional points of attack, the investment for the NATM increases and may exceed that for a shield. Blindow (1977) stated that for sections of 1400 m and longer, shield tunneling is more advantageous.

The NATM cannot be used in all ground conditions since it requires that the ground has some stand-up time (approximately corresponding to slowly ravelling ground). When ground improvement is economically possible, it may be used successfully in worse ground conditions. Air pressure cannot be used with shotcrete, however, since water and air from the shotcrete pump would fog the pressurized tunnel.

There are thus different ranges of application for NATM and shield construction methods, and in some instances a combination of the methods may be useful. Tunnel construction in the Munich Subway can serve as an example for the various methods. Mined tunnels of the first subway line (3/6), built 1966-1971, were excavated by shields, and the support consisted of an outer liner of precast segmented elements, an interior cast-in-place liner, and waterproofing in between. (Munche, 1972). On subway line U8/1, presently under construction (with some sections already completed), both shield and NATM were and are applied. Sections 9 and 16 have variable cross-sections, and in these cases the NATM proved particularly favorable. However, Section 7.1, which includes the crossing of the Isar River, was built with a shield (two single track tunnels) and precast segmented single shell elements. The best example of combining the advantages of both shield and NATM is Section 5 of Line U5/9 (Theresienwiesen, see Appendix A-2), a contract let at the end of 1977. The two pairs of running tunnels from the Theresienwiesenstation eastbound to the Hauptbahnhofstation and westbound to Messehallenstation will be driven by shields (the eastbound tunnels, probably with air pressure). The Theresienwiesenstation, a tunnel for a wye (westbound) and a tunnel for a connecting line at the western end of this section are constructied by the NATM. Access is from shafts at both ends of the Theresienwiesenstation.

In contrast to Munich, the ground conditions in Hamburg do not allow NATM tunneling, since the ground has not sufficient stand-up time and is too impervious to grout. The Hamburg mined tunnels are almost exclusively excavated with shields.

In summary, the selection of equipment, and especially construction method, is governed by ground conditions, geometric conditions, the size of the job, and the prevailing economic conditions and has to be performed individually for each new job.

Personnel and Crewsize

NATM crew sizes are small (Table 5.5). In a single track tunnel, there are only 3 to 4 men working in one heading. It is interesting to note the considerable progress in work rates that took place in NATM applications in subway construction; Section 25 of the Frankfurt subway (built in 1969) required 150 man-hours/meter (man-hours per meter of single track tunnel), while in Essen, Section 24 (1977-1978), the rate dropped to 35 man-hours/meter. Data for one shield tunnel shows that (Appendix B-3) the crew size is comparable to the NATM.

Shift Arrangements. There is usually a day and a night shift of 10 hours duration with a one hour break between (although the break may be staggered so that work is continuous).

Wage Rates. A typical wage rate and fringe benefits for a tunnel construction site in Germany amounts to 31.50 DM/hour = 15.75 dollars/hour (1 US dollar = 2.0 DM) for the contractor (Table 5.6).

The wage rates are higher than those quoted by Engineering News Record in its world wide statistics for common heavy labor. In ENR of March 23, 1978, a basic wage of 9.27 DM/h = 4.41 dollars/h for Bonn, Germany* was quoted, which does not include any social payments. (A contractor has, however, to base his

*In ENR of June 22, 1978, the social payments were included for Bonn, Germany, and the rate for common labor (heavy) jumped to 17.25 DM/h = 8.21 dollars/h, nearly twice the basic wage rate.

TABLE 5.5 CREWS FOR SUBWAY TUNNELS

City	Section	Type of Tunnel	Excavation	Total crew	Shifts	Points of Attack	Rates Average	Men per shift per heading
Munich	16	single and 3-track tunnel	NATM	220	2	10	3.5m/day	11
Essen	24	2 single track running tunnels	NATM	20 (not including adjacent open cut)	2	2 (shifting from one to the other tunnel)	3.5m/day to 7.5m/day	5(10 for both tunnels,alternating between tunnels)
Essen	17	double track	blade shield	≃ 35	2	1	2.5m/day	≃17

TABLE 5.6 AVERAGE WAGE RATE IN GERMANY

	DM/hr.	$/hr.
Base pay	10.84	5.42
Premiums/overtime Fringe benefits	5.47	2.74
Total average pay per hour	16.31	8.16
Overhead & social payments	12.10	6.05
Total	28.41	14.21
Wages of Foreman	3.08	1.54
Total for estimation	31.49	15.75

Note: 1 U.S. $ ≃ 2.00 DM

estimate on the total rates as shown in Table 5.6). A tunneler has an average gross weekly pay of 800 DM or 380 dollars, based on a 50 hour work week.

5.5.2 Site Organization in Transmountain Tunnels (Austria) Equipment

Some criteria for the selection of equipment for transmountain tunnels are listed in Table 5.7. The listed items will be discussed shortly.

Excavation Procedures. Until now, all large highway tunnels in Austria have been excavated by drilling and blasting (with smooth blasting techniques). For the Pfander Tunnel, a set of bid documents for TBM were also prepared and contractors had to submit bids for both drill and blast and TMB. However, all bids but one for excavation with the TBM were higher than the parallel drill and blast bids (the one exception was that of the fifth lowest bidder). Two TBM's would have been necessary to keep the time schedule, while the drilling and blasting excavation in one direction is sufficient. This is an additional factor, since in this case material can only be deposited on the southern side of the tunnel.

For certain ground conditions, full face excavation is possible; however, less favorable ground conditions require excavation by heading and benching. It is possible to change the method of excavation every time new ground conditions are encountered, or alternatively, it is possible to have the heading and bench excavation continuing through better ground conditions.

TABLE 5.7 SELECTION OF EQUIPMENT

AREA	SELECTION CRITERIA
Excavation	Drill & Blast vs. TBM Full Face vs. Heading & Bench Pneumatic vs. electro-hydr. drills
Energy supply	Compressors at portal vs. in tunnel (Loss of air pressure)
Haulage Mucking	Track vs. Tire
Adaptability of equipment	Can be adapted to changed conditions (smaller heading)
Redundance of equipment	2 medium size machines instead of a single large one
Reliability	Proven elsewhere. Reputation of contractor.

The latter solution seems to be advantageous, since the work cycle is not interrupted once it has been established.
A continuous heading and benching method was thus selected for the Pfander Tunnel. According to Mr. Rucker, Site Manager of the Pfander Contractors, adopting this scheme in the Pfander Tunnel resulted in high advance rates which compare favorably with those of the Frejus Tunnel excavated primarily by full face excavation, Table 5.8, (Rucker, pers. comm., 1978). The advance rates for the Pfander Tunnel are approximately twice those of the Frejus Tunnel.

A comparison between pneumatic and electrohydraulic jumbos seems to tend in favor of the latter. The contractor for the Pfander Tunnel estimated a savings of 30 Austrian Schillings, AS (2.00 dollars) per cubic meter of excavation with electrohydraulic jumbos, a total savings of approximately 15 mill AS = 1,000,000 Dollars.

The energy supply (air, pressurized water, electricity) has changed. Previously, compressed air and pressurized water were produced outside the tunnel near the portal and brought into the tunnel with supply lines with substantial losses, particularly for compressed air. In the Arlberg East and also the Pfander, compressors are placed on flatbed trailers and follow the excavation by a few hundred meters; thus only low pressure water and electric lines extend to the portal. Also, the new electrohydraulic drills reduce the demand for air (only shotcreting needs compressed air), while the demand for electricity

TABLE 5.8 RATES OF EXCAVATION

CASE	RATES	RATES
Pfander (Austria) (Heading and one Bench)	max. average	24 m/day 400 m/month 3.1 km/year (≃10 m/day)
Frejus Tunnel (Full Face) (Rucker, personal communication, 1978)		3 km/2 years (≃5 m/day)
Arlberg West	max. min. average	11 m/day (Class III) 1 m/day (Class V) 6.8 m/day

increased. The heat generated by the compressor evidently causes no problems.

Adaptability and Redundance of the equipment are two related aspects. The equipment must be adaptable; i.e., in case a small size heading is necessary, the equipment has to fit into it. This makes it simultaneously possible to achieve redundance by employing two medium-size pieces instead of one large piece of equipment (e.g., one large drill Jumbo). The medium-size equipment works in parallel in large size headings. In this manner, a breakdown only leads to a reduction but not a complete stop of production. Equipment reliability is achieved by using proven equipment or by using new types of equipment only from a manufacturer with a good reputation.

Crewsize

Table 5.9 presents a summary of crewsizes in some Austrian tunnels presently under construction. Each point of attack, i.e., heading or bench, requires a crew of approximately 12 to 15 crew members. The 15 crew members perform a variety of tasks. In case of the Flirsch and Gandertobel tunnels, the 15 crew members perform the task of several trades; if each crew member would only perform a single specific task, at least 44 crew members would be necessary (Appendix B-7). (Thus on a job with stringent union work rules, a total of at least 44 men would be required to perform the same work.)

Tunnel construction crews in Austria are well trained and experienced. Their basic training is not limited to a single task; they have to understand and be able to perform many of the

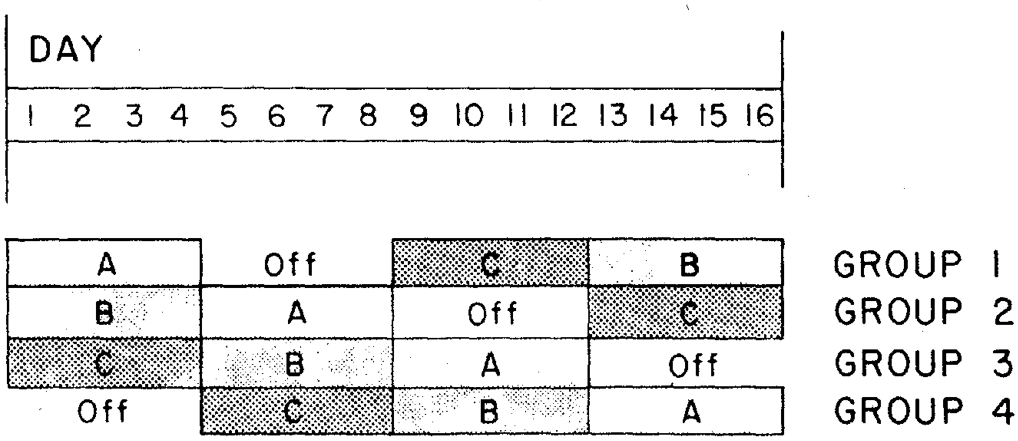
TABLE 5.9 CREW SIZES PER SHIFT
(SEE TEXT FOR SHIFT ARRANGEMENT)

Case	Cross-section	Type of Excavation	Excavation	Concrete	Total
Gandertobel (App. B - 7)	150 m^2	sloping face	15	NA	15
Flirsch (App. B - 7)	80 m^2	heading 50 m	15	NA	15
Arlberg West	90 m^2	heading & 2 benches	20	Concrete: N.A. Machine Shop = 20	40
Pfander (App. B-8)	75 m^2 without pilot tunnel	heading & benching invert	25 8 33	profile control 3 concrete 8 slab of vent channels 3 wall 3 18	51 + machine shop 25 per shift

tasks required in the heading (drilling, shotcreting, loading of explosives). Although unionized, work rules do not restrict a single crew member to a certain task. Contractors provide the workmen with formal training in utilizing equipment (drill rig, excavator). One contractor has introduced his own operator's licenses for his personnel in order to be able to select people with the necessary experience for a given task.

There are some government regulations, particularly in connection with blasting. The loading of explosives can be performed by several crew members; however, one foreman is responsible for checking the circuits and the ignition of the blast.

The shift arrangement varies from site to site and also for different tasks on the same site. The excavation and initial support generally require the most stringent shift arrangement, since all other tasks depend on them. The shift arrangement reflects the fact that the sites are remote and the driving time to the home of the crew members generally exceeds many hours. (Distances reaching 300 to 600 kms over undivided highways). Thus a schedule arranged along the lines of regular work weeks would not allow the workers to return home. On most sites, the men work for 10 to 12 days and then have 4 to 5 days off. This work period is called a decade. Excavation is three eight-hour shifts per day. Continuous work on the project is achieved by a so-called 4/3 operation (Figure 5.9). This means that 3 shifts are working and the fourth shift is off. A continuous 4/3 schedule has implications on housing since



A = Shift from 10 pm to 6 am

B = Shift from 6 am to 2 pm

C = Shift from 2 pm to 10 pm

Off = Not Working

FIGURE 5.9 SHIFT ARRANGEMENT FOR 4/3 OPERATION accomodations for all four shifts must be available. Also, maintenance of the equipment requires speical provisions in such a continuous operation.

Concreting is usually scheduled differently, with only two shifts of 10 hours duration. On a particular job, the shift arrangement will be adapted based on the experience during construction; i.e., when the work is behind schedule, a 3/3 operation might be changed to a 4/3 operation. The decision has to be made by the site manager, considering the particular conditions.

Average Wage Rates. The average wage rate used by contractors in their estimates (average for all men on one site) was quoted to be 200 to 220 Austrian Shillings per hour (13.33 to 14.67 US dollars). The base pay and social payments of a worker is on the order of only 40 to 50 Austrian Shillings per hour. Due to bonuses and fringe benefits, the actual pay for a worker is higher, on the average between 100 to 110 Austrian Schillings per hour (6.6 to 7.3 US dollars/hour), the same as for Germany. With 220 work hours per month, a worker has a take-home pay of 22500 AS = 1500 dollars per month. Wages are thus comparable to the U.S.

Summary

A large part of the success of the Austrian Tunnel Construction Practice stems from equipment and personnel policies. Equipment must be reliable and adaptable. Reliability is guaranteed by either using proven equipment, or in case of newly developed equipment, emphasis is placed on the reputation of the manufacturer (i.e., the reliability of his previous models). To prevent complete shutdown of the operation due to the breakdown of key equipment, redundancy is provided by having two or more smaller units (e.g., drill rigs) rather than one single large one.

Personnel is well trained and frequently follows the contractor from site to site. Although unionized, work rules do not limit the tasks an individual worker can perform. Thus, only small crews are necessary, which explains the lower labor cost in spite of comparable pay rates in Austria as compared to the U.S.

5.6 SAFETY OF TUNNEL CONSTRUCTION

Detailed statistics of construction accidents are kept in Europe, but these statistics lump all accidents together and no detailed tunnel construction accident statistics are available. Attempts are under way to improve this situation. Tiefbau-Berufsgenossenschaft, Munchen, the mandatory insurance agency for heavy construction work in Germany, has published regulations that apply to heavy and underground construction work. These regulations are similar to OSHA; they are very detailed and often include details of construction procedures. Some safety statistics collected during interviews by Mr. Steiner are reported here.

In general, accidents can be divided into two categories: those related to ground conditions and others. In the particularly unfavorable geologic conditions of the Arlberg Tunnel, most of the accidents are attributed to the ground conditions. However, most other accidents are not related to ground conditions and are often traffic or blasting related.

Specifically during construction of the Tauern Tunnel (6.4 km) and the northern section of the Katschberg Tunnel (3.83 km), a total of 10.2 km of tunnel, no fatal accidents were recorded. In the eastern section of the Arlberg Tunnel (length = 8.9 km) three million man hours with no fatal accidents and two severely injured persons were recorded. At the Arlberg west (length = 5.1 km), 5 fatalities were recorded (compare Appendix B-7). Four of these fatalities were attributed to rock conditions. Two fatalities occurred when a section of the roof buried two excavator operators. Two other fatalities were recorded in one of the cross-cuts of the Arlberg Ventilation Cavern where popping rock was encountered. The fifth fatality was a traffic accident, a locomotive hitting a concrete car. The safety record and the accident prevention measures in the Arlberg Tunnel are discussed in detail by Stix (1978). Construction work was monitored (noise level, concentration of toxic and non-toxic gases) and the crew was under constant medical supervision. If persons showed signs of sickness, they were removed from harmful environments.

The following statistics have been summarised by Stix (1978); the amount of severe and minor injuries is 40% and 60% of all accidents, respectively. 65% of the accidents occurred in the tunnel, 15% in the shops, 15% on other construction work (surface construction work on this site), and 6% of the accidents occured on travel between the workers' homes and the site* Stix states that many injuries could be avoided, or reduced, if the safety gear would be properly worn.

In some cases, exceptions from standard safety rules have been granted for construction procedures; however, in such cases modified safety rules were formulated and applied. This was usually done if adhering to the standard rules would have led to additional safety problems. A good example is the use of lift platforms** for work above ground. If the work space in the heading is crowded with equipment, adding lift platforms would cause additional safety problems. Instead, front end loaders that are in the heading anyhow can be used, with special safety rules. In particular, the shovel is equipped with a bar on the rear side and a flat surface is formed by placing fine grained soil in the bottom of the shovel. Using frond end loaders for this purpose was only permitted for lift heights smaller than two meters. For larger lift heights, regular platforms with a safety railing had to be used. The management (supervisory) personnel has to enforce the safety rules by explanations or warnings. Theoretically, crew members can be legally fined for safety violations; enforcement is, however, difficult and tedious. Instead, premium pay has been cut for crew members who do not follow safety regulations,

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*Travel between site, home and back, is usually only every 10 to 12 days; however it is often over great distances.

**Which are required by the rules

up to 100 AS (≈ 70 US$) per violation. In the Ganzstein Tunnel, (Appendix B-5 , total length = 2.2 km , not yet completed), a blasting accident was recorded with one fatality and one maimed.

5.7 CONCLUSIONS

The key to progress in tunneling practice made in Austria and Germany lies primarily in the integration of design and construction. This occurs through alternate proposals, which usually involve redesign to optimally fit the contractors construction concepts. Another possibility is that the owner advertises design-construction bids, providing only the boundary conditions; the detailed design is done by the contractor. Technical discussions may take place between contractors and owners during bidding and prior to the award of contracts. The interaction between owner and contractor continues during the construction; however, this interaction relies on the contract documents and technical facts, and it is by no means a soft relationship.

The progress of tunneling methods had its roots primarily in Austria and spread to Germany. German design practice, where the static design has to be approved by a licensed inspection engineer, impedes the introduction of new methods to some extent; however, the finite element method seems to have brought about a breakthrough, since it allows better modeling of ground structure interaction and shows that some of the empirically developed tunneling methods like the NATM are statically sound. In transmountain tunnels (Austria) with essentially unknown ground conditions, the observational procedure makes it possible to adapt the support and excavation procedures to the encountered conditions.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 CONCLUSIONS

The goal of the information gathering trip was to study tunnel construction practice in Austria and Germany, to identify differences compared to U.S. practice, and to describe new developments.

The primary conclusions are that, compared to the U.S., in Austria and Germany:

a) Tunnel construction costs are lower.

b) Support quantities are smaller for similar ground conditions.

c) Crew sizes are smaller, but labor costs per worker are higher.

d) There is less litigation since tunnel construction contracts implicitly and explicitly include changed condition clauses and price escalation clauses.

e) The submittal of alternate proposals is strongly encouraged, in some cases going to the extreme that no official detailed design exists. This results in strong technical competition between contractors and leads to many innovations.

f) Design and construction are, to a large extent, integrated (as a consequence of item e).

Tunnel construction costs in the U.S. are higher by 30 to 80% per route meter for running tunnels and 100 to 300% for stations, while unit volume costs are 50 to 100% higher. The main reasons are less conservative supports, smaller crew sizes, and higher advance rates under complex conditions.

Support quantities are lower because design construction procedures make an adaptation to the encountered ground conditions possible. This is facilitated by performance monitoring, through which the actual behavior is continuously compared to the predicted one. The technology regarding excavation equipment and support material and placement makes an adaptation through changes in excavation procedures, support dimensions, and materials easily possible.

Crew sizes are considerably smaller in Europe than in the U.S. Although labor is unionized, no work rules exist that restrict union members to a particular task. The take-home wages in Europe are comparable to those in the U.S. However, in Europe the social payments and fringe benefits and thus the total labor costs are higher than in the U.S.

Litigation is limited due to:

(1) Uniform and complete bid and contract documentation that includes all available information on ground conditions;

(2) The bid schedule is very detailed; this forces the contractor to consider details in advance and to specify a price; it also makes it possible to take the largely uncertain nature of the underground conditions into account; and

(3) Bid and contract documents contain detailed

procedures according to which support and excavation will be determined (ground classification). Such procedures frequently contain mediation-arbitration as a standard feature. Also, changed conditions clauses are explicitly provided in all contracts to take care of unanticipated conditions. The changed conditions clauses contain detailed procedures on price calculations under such circumstances.

Alternate proposals and integrated design-construction make it possible for the contractor to optimize his resources (crew, equipment). Alternate proposals bring innovative developments to construction. The contractor has frequently his own design staff which prepares the alternate proposals (often in collaboration with consultants). The submission of alternate proposals practically results in innovations with every tunnel construction project. This pace of innovations is faster than through designer originated innovations, since immediate feedback is possible on the contractor's level. European owners willingly accept alternate methods proposed by contractors.

It should be noted that, although very innovative, tunnel design in Europe is to a larger extent empirical than in the U.S. This works well if experienced owners, engineers and contractors are involved, but may be problematic if one party lacks the experience.

6.2 RECOMMENDATIONS REGARDING FURTHER INFORMATION TRANSFER

Contractual practice has been formalized in Austria and Germany; for a full appreciation of the procedures used, we recommend the translation of the appropriate ordinances and standards. Many of the recommendations of the NCTT-Report "Better Contracting in Underground Construction" are already fulfilled in these standards, and they contain also new developments. In particular we recommend to translate:

(1) Verdingungsverordnung fur Bauleistungen (Ordinance for the award and execution of construction contracts), Parts A and B. Beuth-Verlag, Berlin.

(2) ONORM B2203, Austrian Standard B2203, for underground construction work.

Valuable research on cost parameters in subway tunnel construction cost has been performed by STUVA. This research considers the influence of ground conditions, type of tunnel cross-section, station size, and spacing. The report has been revised and will be published as a book; we recommend also to translate this report or to extensively summarize it:

Research Report by STUVA (Studiengesellschaft fur unterirdische Verkehrsanlangen) on "Parameters Influencing Tunnel Construction Costs," Final Report to be published as Vol. No. 22 of Forschung & Praxis, Alba-Verlag, Dusseldorf.

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APPENDIX A

INTERVIEWS AND INFORMATION GATHERED FROM GOVERNMENT AGENCIES, RESEARCH INSTITUTIONS, ENGINEERING FIRMS AND CONTRACTORS

APPENDIX:	A-1
NAME:	U-Bahn Referat der Landeshauptstadt, München (Subway Authority of the State Capital, Munich)
ADDRESS:	Hackenstrasse 12 D-8000 München 2 Germany, Federal Republic
DATE OF MEETING:	2nd January to 5th January, 1978
PERSONS MET:	Mr. Krischke, Oberbaudirektor, Head of the Subway Construction Department Mr. Weber, Baudirektor, Head of the Design Office; Mr. Nowosad, Head of the Cost Supervision Department.

1) Introduction

The city of Munich started with the construction of a rapid transit system in the year 1965, although already in the years 1938-1941 a short section was built which was incorporated in the present system. The first stage of construction consisted of the subway line U3/6 (Figure A-1.1) and the "S-Bahn" line linking the central railroad station and the east railroad statioı These lines had to be completed by early summer 1972 to be ready for the Olympic Games. In addition, two more subway lines

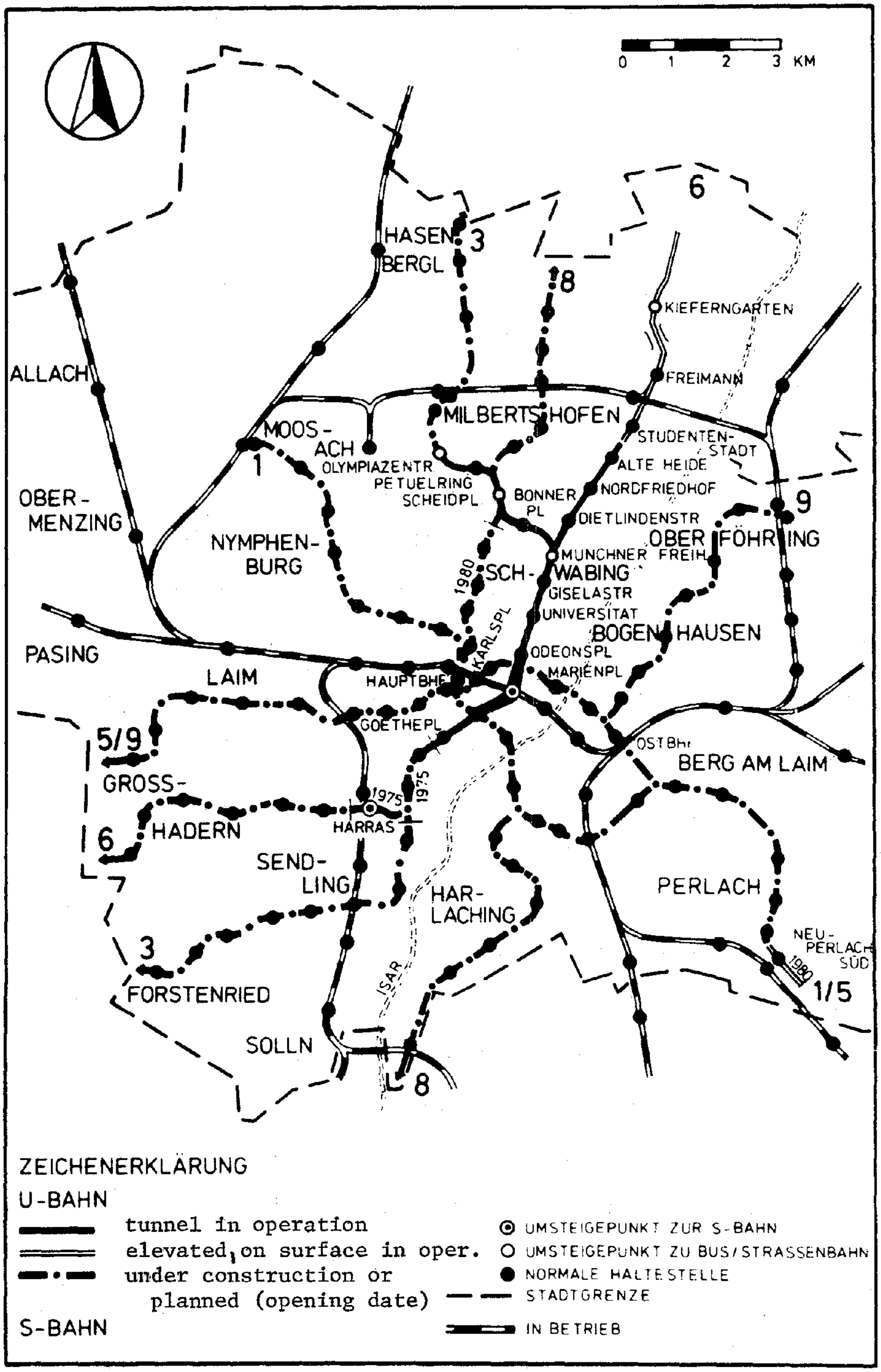


FIGURE A-1.1 MAP OF THE MUNICH SUBWAY SYSTEM (FROM BLENNEMANN, 1975)

U1/8 and U5/9 are at this time in the design and construction stage. U1/8, from Neu Perlach to Scheidplatz, is slated to be in operation by 1980. Major construction work is already completed or in progress. On Line U5/9, the first contracts have been let and construction work has started.

2) Review of the Legal Process Prior to Bidding

2.1 General Remarks

Planning of subways is performed in-house by the Subway Authority (U-Bahn Referat). Before construction starts, a lengthy process of obtaining permits and publicizing the planned construction takes place. This period of planning and preparation takes several years; it ensures that after construction work has started no work interruptions are necessary to fight legal battles.

2.2 Permits to Build a Line (Streckengenehmigung)

This step involves the various governmental agencies and major private organizations but not directly the individual citizens. The needs for the particular line are evaluated, the mode of financing is determined, and the agencies which may be affected by the construction may present their opinions.

For this purpose, the Subway Authority has prepared design plans for the running line section (scale of 1:1000) and the stations (scale 1:250). Agencies involved in this process are the City of Munich and its public works and utility departments (water, gas, electric, public roads), the agencies of the state government (the State of Bavaria: the planning board, the water control authority, Department of State Roads), and the agencies of the federal government (Federal Railroads, Federal Post & Telephone, Federal Department of Transportation). Once, and only if, an agreement can be reached, a permit document is issued (Genehmigungsurkunde) by the local government of Oberbayern (Upper Bavaria).

2.3 Public Advertisement of Plans (Planfeststellungsbeschluss)

This public advertisement usually concerns only a section of a subway line and shows how and where the individual citizen and property owner may be affected. The plans contain all the necessary information, including the proposed method of construction, but only in a general sense, for instance whether open-cut or mined construction is planned. Plans are on a scale of 1:1000 for the running line sections and 1:250 for stations and contain plan views, longitudinal sections, cross-sections, and subsurface conditions. They also show which parts of public and private property will be affected, including the areas which may be only used for construction purposes, e.g., where tiebacks might be located. The type and extent of groundwater lowering are indicated, as are zones where the subsoil will be grouted.

The plans can be inspected for a period of two weeks after the advertisement. The individual citizen may formulate objections against the proposed construction, which includes the right to object against a specific type of construction, but the objections have to be "reasonable."

2.4 Construction Decree (Bescheid) by the Local Government of Oberbayern (Upper Bavaria)

Objections raised against the project will be solved, by direct negotiations if possible, and the construction decree (permit) will be issued by the local government. If the negotiations are not successful, the local government of Oberbayern decides on the remaining objections in the decree. In any case, the subway authority attempts to obtain a decree with immediate effect (Bescheid mit sofortiger Vollziehbarheit), which means that construction can start immediately after the decree is issued. To obtain a decree with immediate effect, the subway authority attempts to solve all problems by negotiations. Once this decree has been granted, virtually no additional objections can be raised. (A decree with immediate effect requires "unequivocal" supporting documents. What constitutes "unequivocal" was determined by a court decision by precedence).

Property is acquired by regular transactions or by expropriation. An easement has to be obtained by the Subway Authority if a tunnel is passing underneath a property. The property owner is reimbursed for the easement by the amount that the property's value is reduced relative to non-affected property.

Businesses adjacent to construction sites will be reimbursed for their estimated losses and will also be awarded low-interest loans.

3) Bidding and Contractual Practice

3.1 Regulations and Bid Documents

All bids and construction contracts have to be in agreement with the VOB (Verdingungsordnung für Bauleistungen = Ordinance for the award and execution of construction contracts). The VOB has been described in some detail in Chapter 4 of this report.

Of particular interest is Article 9 of VOB, Part A. This article states that the construction work and the bid items have to be described by the owner in a complete and exhaustive way and that all available information has to be furnished to all bidders or made accessible to them. This, e.g., includes information on ground conditions and underground utilities. Note that this regulation corresponds to the recommendation given in the NCTT report "Better Contracting for Underground Construction " that all subsoil data should be made accessible.

As a consequence of Article 9 of VOB, the bid documents are very detailed. The bid schedule for a single subway section is several hundred pages long. In addition, the City of Munich has developed its own contract regulations which supplement or change VOB. These general contract conditions reflect the experience gained by the City of Munich during more than a decade of subway construction.

Of particular interest are the required documents a bidder has to submit if he bids on the official design or when submitting an alternate proposal. Table A-1.1 is a translation of such a listing of required documents, as included in the bid documents for Section 5, Theresienwiesen, of Line U5/9.

In addition, the bid documents contain detailed technical specifications and plans describing the work. These documents include:

- a bid schedule
- a detailed description of bid items
- blueprints, scale 1:250 for the running tunnels
- blueprints, scale 1:100 for the stations including longitudinal, transverse sections and details
- traffic plans for surface traffic, notably re-routing of traffic
- plans of existing underground utilities conduits (Spartenpläne)
- subsurface conditions

All data available are given to the contractor. The subsurface conditions are determined by means of borings reaching 3 to 5 m below the lowest point of construction. The spacing of the borings varies. For open-cut construction, exploratory borings are spaced 80 to 100 m. For mined tunnels, the spacing is reduced to 40 to 60 m.

TABLE A-1.1 REQUIRED DOCUMENTS FOR BID SUBMISSION FOR SECTION 5, SUBWAY LINE U5/9 IN GERMANY

REQUIRED DOCUMENTS FOR BID SUBMISSION*

1. Documents required when bidding (Official Design)

The bid has to include the following fully completed, and as far as necessary, signed documents:

a) Bid Declaration.
b) Bid Schedule.
c) All Appendices to the Bid Schedule (Tables and Listing). If unit prices in the tables do not agree with unit prices in the bid schedule, the tables prevail.
d) Time Schedule.
e) Payment Schedule for the estimated monthly payments by the owner.
f) Site Installation Plan.

2. Alternate Proposals

These are only considered if they include the following:

a) Bid Schedule.
b) Explanatory Technical Report.
c) Design drawings of the Alternate Proposal.
d) Time Schedule (adapted from official design).
e) Site Installation Plan.
f) Payment Schedule for the estimated monthly payments by the owner.
g) Static Computations (as far as necessary).
h) List of Quantities considering the reductions and increases in quantities compared to the items of the official proposal.

Alternate or partial alternate proposals have to include cost increases or decreases on the bid items of the official proposal that are caused by the alternate method. Also any additional work caused by the alternate proposal, regardless of who has to perform it, has to be included in the bid.

*Translated from Bid Schedule, Theresien wiesen, Section 5, Subway Line U5/9 (1977)

Based on these detailed specifications, the contractor will submit his bid. Bids for alternate methods (alternate proposals) for the entire construction, as well as bids containing alternate methods for only parts of the job, are permitted. These bids must fulfill the conditions mentioned earlier and listed in Table A-1.1.

In general, most contracts awarded are based on alternate methods.

The procedure of bid opening, awarding the contract and bid evaluation is described in detail in VOB, Part A. Basically, the bids are opened in a closed session where only representatives of contractors who submitted bids may be present. During bid opening, the total bid price as well as sub-totals for groups of items are announced (e.g., "excavation total", "concrete total"). Furthermore, it is announced who submitted alternate proposals. Disclaimers by the contractors are read as well. However, no further details are announced.

The bids are then evaluated by the owner's staff. The arithmetic accuracy for all bids and the technical and economic feasibility of alternate proposals are checked.

Alternate bids must contain a complete technical description on the level of detail of the official specifications. In case design errors are detected by the subway authority, it alerts the contractor of this fact; however, no price increase will be allowed. Also, the subway authority does not assume any liability for undetected errors; this remains with the contractor.

The price of alternate bids has to be guaranteed by the contractor (however, while changed condition clauses are not permissible in alternate proposal contracts, according to VOB, the City of Munich includes such a clause).

3.2 Award of Contracts

Awarding the contract is handled according to VOB, Part A, where a detailed description is given. Negotiations between contractors and owners may precede the award. However, these negotiations do not involve prices; they are concerned only with technical questions and with the capability (technical, capacity) of the contractor to perform the task.

Negotiations between owners and contractors may also serve to substantiate the reasonability of the bid. The City of Munich may require the contractor to deposit detailed computations and bases for his estimated unit prices in a sealed envelope with the city once he is awarded the contract. With this procedure the owner can, in case of disputes, check the basis for the bid.

A contract does not have to be awarded to the low bidder. A low bidder may be excluded on technical grounds, e.g., he cannot guarantee the timely completion of the work. Bids with unreasonably low prices can be excluded also.

3.3 Changed Conditions

Changed conditions clauses are described in VOB, Part B, Article 2, and in the additional contract conditions of the City of Munich. If work not listed in the official specifications should become necessary, the contractor has to immediately notify the owner in writing of this changed condition and at the same time has to submit a bid for the additional work. The owner then decides whether this supplemental work is necessary and approves unit prices, quantities and total price of this work. The price of this supplemental work has to be based on the unit prices of the initial bid. New unit prices can only be negotiated for deviations in quantities by more than ± 10% according to VOB (the City of Munich has changed this to ± 20% in its general contract conditions).

Supplemental work will generally be awarded to the same contractor unless the original work is substantially exceeded, which means by considerably more than 20%. In the City of Munich, the amount of supplemental work awarded varies in the range of 5 to 10% of the total cost of a subway section. Most of this additional work seems to stem from additional measures to control the ground water. A detailed description of ground water control measures can be found in section 5 of this appendix. The additional work primarily involves wells and grouting of the subsoil.

3.4 Insurance

The City of Munich provides a "wrap-up type" combined liability-construction insurance for subway construction work. The coverage for liability regarding damage to persons is limited to 5 million DM, but no more than 1 million DM per person. For damage to fixed property the maximum coverage is 5 million DM; for damage to moveable property it is 100,000 DM, and 1 million DM for water damage. Deductibles of 2,500 DM for damages to buildings are part of the contract. For damage to underground utility lines, the deductible is on the order of 20%, with a minimum of 100 DM and a maximum of 5,000 DM.

3.5 Law Suits

During the entire history of the subway construction, only a very few claims and disputes could not be resolved by direct negotiations. W. Steiner was told that two cases out of a total of 40 to 50 (estimated) subway construction sections had to be solved in court.

4) Cost of Subway Tunnels

At the time of the visit, only approximate numbers for the construction cost of subway tunnels were given.

The approximate costs as quoted during the interview in Munich are listed in Table A-1.2. At present, the competition amongst the contractors is very strong and prices have thus to be considered low. Gebhart (1977) quotes costs of ground water control, which are listed in Table A-1.3. The development of the new ground water control measures (as described in Section 5) reduced these costs by as much as 5-10%. The relative costs of a ground water control of 5% are for the case of a mined tunnel (see Section 5) with a sufficient

TABLE A-1.2 APPROXIMATE VALUES OF SUBWAY CONSTRUCTION COSTS IN THE CITY OF MUNICH

		2 single track tube, NATM or shield		
		Range of costs of running tunnels only	Costs in city center including stations (with pedestrian underpasses)	Costs in outer districts of city including stations
Construction Costs in Millions	DM/km	20 to 40	60 to 70	30
	U.S.$/Mile	16 to 32	48 to 56	24

Note

U.S.$ = 2.00 DM

TABLE A-1.3 CONSTRUCTION COSTS OF SUBWAY TUNNEL AND COST OF GROUNDWATER CONTROL (FROM GEBHARDT, 1977)

ITEM		COST PER SECTION (Millions) DM	U.S.$
Total Construction Costs (absolute Costs)	Maximum	66	33
	Minimum	16	8
Cost of Ground-water Control	Maximum	12	6
	% of total cost	≃ 15%	≃ 15%
	Minimum	0.5	0.25
	% of total cost	≃ 5%	≃ 5%

NOTE
1.00 $US = 2.00 DM

cover of impervious tertiary clay (Figure A-1.2). The relatively high costs of 15% (Table A-1.3) are for the case of a tunnel lyin completely or partially in the quarternary deposits; in this case either considerable pumping was necessary or the ground around the tunnel had to be grouted. The relative cost of dewatering in the case of open-cut tunnels is about 4.5 to 8% of the total construction cost.

5) Technical Problems

5.1 Subsurface Conditions

Figure A-1.2 shows a general block view of the geology of the City of Munich. The underground of Munich can be divided into two main strata, the quarternary and the tertiary deposits. Figure A-1.3 shows geologic sections along Line U5/9, where construction started only recently. The top quarternary stratum consists mainly of gravel and sand and is very permeable. Subsoil properties are summarized in Table A-1.4. Underlying the quarternary deposits are tertiary deposits of sands and marls. These deposits are rich in mica, which is called "Flinzel". The marl is thus called "Flinzmergel" (flinzmarl), and the sand, "Flinzsand". The tertiary deposits are often highly calcarious with high unconfined compressive stengths as quoted in Table A-1.4. The hydrologic conditions vary; there is a free water table in the quarternary and artesian tables are found in the tertiary strata, notably in the Flinzsand lenses (Figure A-1.3).

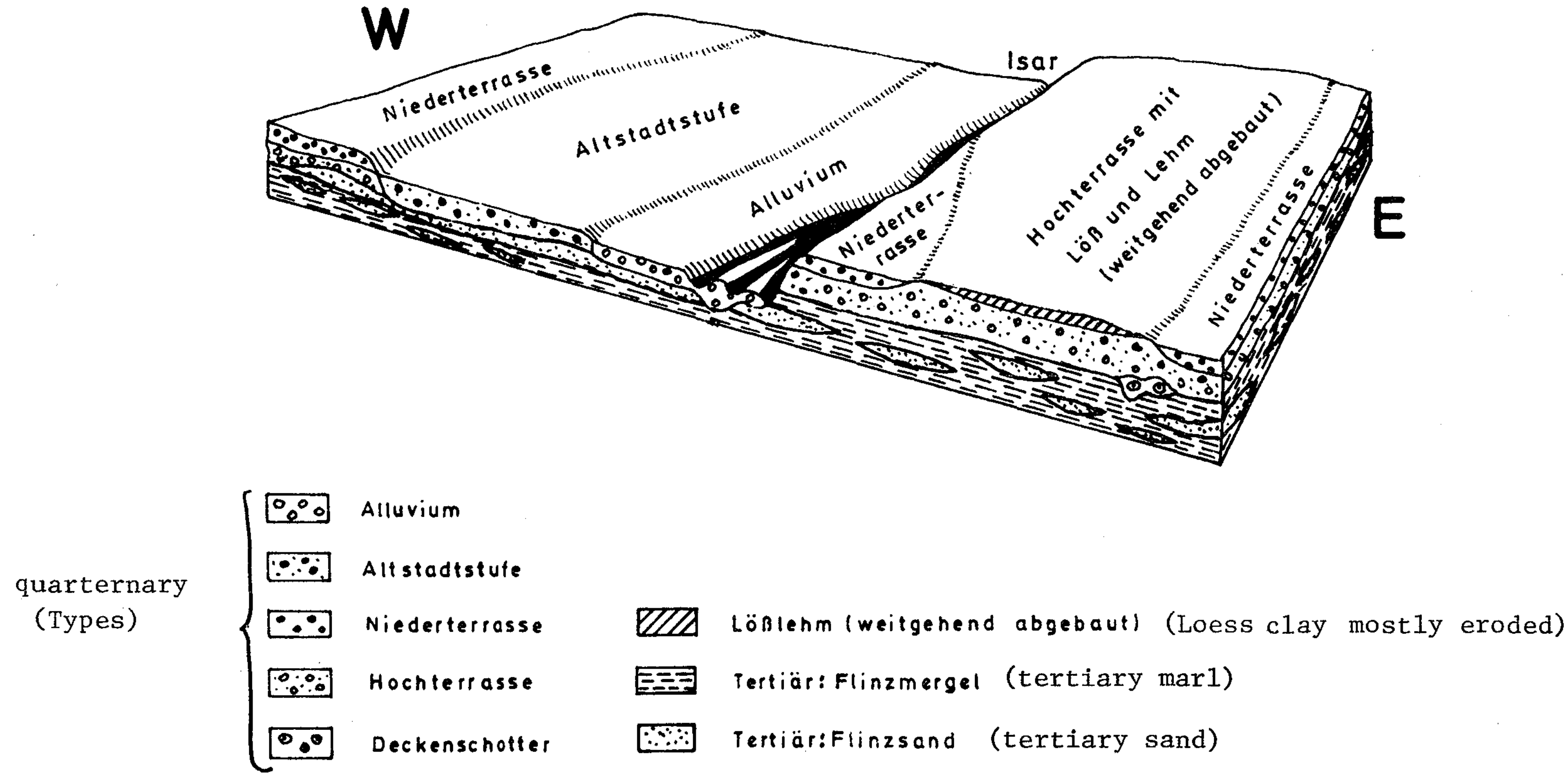
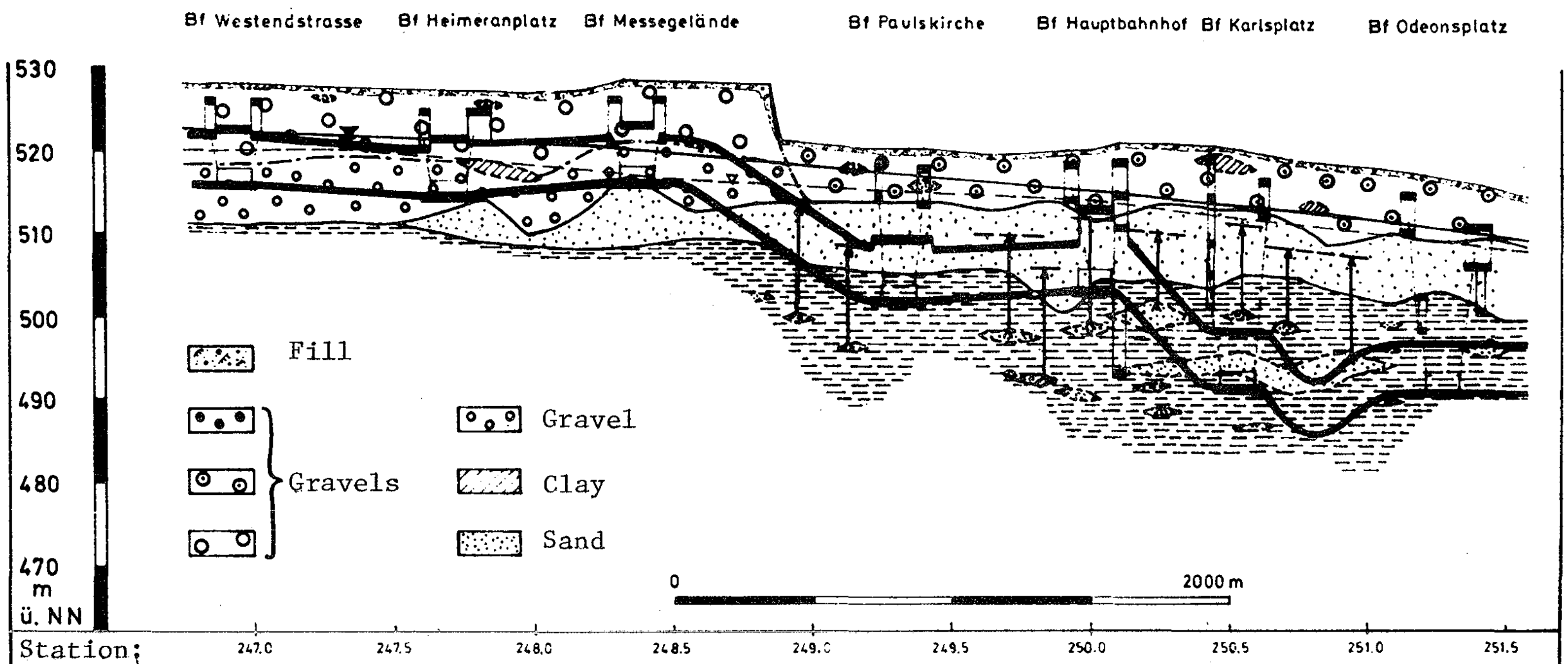


FIGURE A-1.2 BLOCK VIEW OF THE GEOLOGY IN THE CITY OF MUNICH (FROM GEBHARDT, 1977, AFTER MUNICHSDORFER, 1922)



Section:	12	11	10	5			1	3	6
Construction Method	OC	SC / OC	UR	SH	SC	SH	UR	SC	

OC=Open-cut   UR=Under the Roof   SC=Mined Shotcrete(NATM)   SH=Shield

Notes
Schematic is 30 times superelevated
Legend is continued in Fig. A-1.3b

FIGURE A-1.3a HYDROGEOLOGIC SECTION ALONG SUBWAY LINE U5/9 IN MUNICH
(FROM GEBHARDT, 1977)

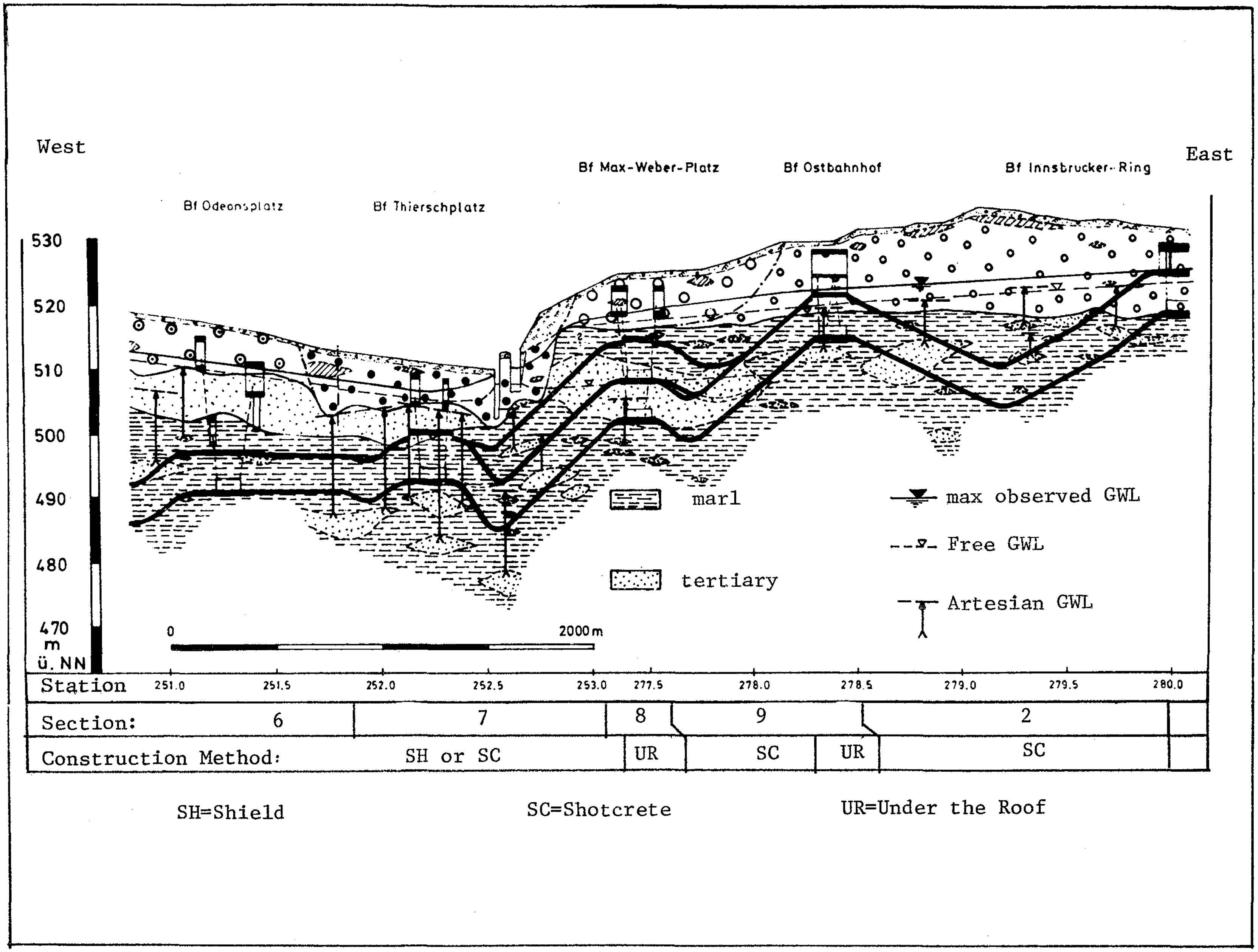


FIGURE A-1.3b HYDROGEOLOGIC SECTION ALONG SUBWAY LINE U5/9 IN MUNICH (FROM GEBHARDT, 1977)

TABLE A-1.4 SOIL PROPERTIES OF THE MUNICH SUBSOIL [FROM BID SCHEDULE, THERESIENWIESE, GOLSER ET AL. (19799), GEBHARDT (1977)]

GEOLOGIC UNIT	TYPE OF SOIL	UNIT WEIGHT	BUOYANT UNIT WEIGHT	PERMEABILITY	EARTH PRESSURE	FRICTION ANGLE	COHESION	MODULUS OF ELASTICITY		UNCONFINED COMPRESSIVE STRENGTH
		t/m^3	t/m^3	m/sec	at rest K_o	$\bar{\phi}^{\circ}$	$\bar{c}(\frac{kg}{cm^2})$	loading kg/cm^2	unloading kg/cm^2	kg/cm^2
quartenary	gravel	2.2	1.3	10^{-4} to 7×10^{-2}	0.5 to 0.65	37.5	0	1300	2000	---
tertiary	sand	2.1	1.1	10^{-7} to 3×10^{-4}	0.5 to 0.65	37.5	0	1000	2500	0.6 to 150
	marl	2.1	1.1	10^{-10} to 10^{-8}	0.5	25	0.4	1500	2000	
	clay marl	2.1	1.1	10^{-11} to 10^{-10}	0.5	18	0.8	1000	1500	

5.2 Ground Water Control

In Munich, methods were developed which are adapted to the particular underground conditions where pervious and impervious layers alternate and are often connected. There is one ground water table in the overlying pervious quarternary soil (Figure) A-1.3); however, there are other ground water tables in the tertiary. The tunnels may lie in the tertiary or in quarternary soil. In the tertiary, the water table in sand lenses is lowered to prevent runs into the tunnel. In the quarternary, ground water control is necessary with grouting or dewatering.

Whenever possible, tunnels are located in the tertiary marl with a minimum overburden of 1.5 m of tertiary marl over the tunnel. If this overburden is smaller, either grouting or dewatering in the vicinity of the tunnel may be required. Usually, in the pervious quarternary gravels, a zone of 2 m minimum thickness around the circumference of the tunnel is grouted. Grouting can be performed from the surface or from the face. Grouting from the surface ahead of the tunnel is often preferred since it does not interfere with the advance of tunnel. In one case (Figure A-1.4), grouting for shield tunnels proceded from a pilot tunnel above the ground water table.

The other method is to dewater or reduce the pore pressure in the vicinity of the tunnel. This solution is better than grouting, if sand (tertiary Flinzsand), which is less pervious than quarternary gravel, is overlying the marl. The sand is

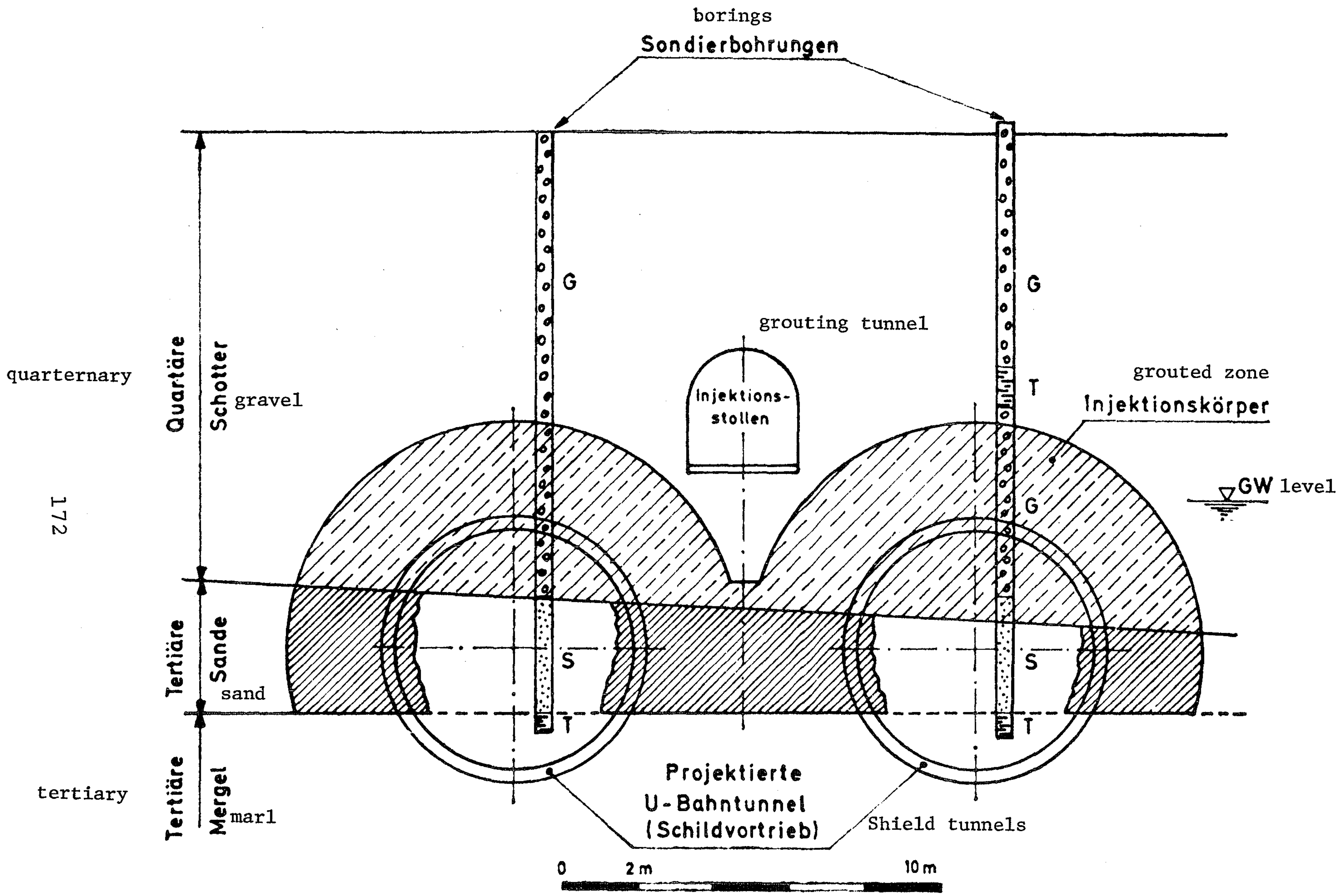


FIGURE A-1.4 GROUTED ZONES AROUND TUNNELS PENETRATING INTO GRAVEL AND SAND (AM HARRAS, LINE U6, SHIELD TUNNELING) (FROM GEBHARDT) dewatered with wells drilled from the surface or from a bottom drift of the tunnel. Dewatering in some layers requires a grouted seal (Figure A-1.5a). The wells from the tunnel usually extend only into the sand (Figure A-1.5b) to achieve a local pressure relief.

The decision as to which method ought to be used depends on several factors: whether wells can be placed from the surface (is the area built over?), the overburden, and the particular ground conditions. The decision to either grout or to lower the ground water is based on economical and scheduling aspects as well as on aspects of ground water pollution control. Every grouting operation contaminates ground water and thus possibly the city's water supply.

Of particular interest is the technique of passing under a local depression in the marl surface, as illustrated in Figure A-1.6 and Figure A-1.7. The top heading of a tunnel supported with steelsets and shotcrete has been lowered and becomes now a bottom heading in order to have a sufficient cover. Thus, the tunnel excavation need not to stop due to this local problem, which will be treated at a later stage. The tunnel will be excavated to its full cross-section once additional ground water control measures (earlier described) have been taken. Clearly, this technique can only be realized with NATM type construction.

WELL DRIVEN FROM TUNNEL

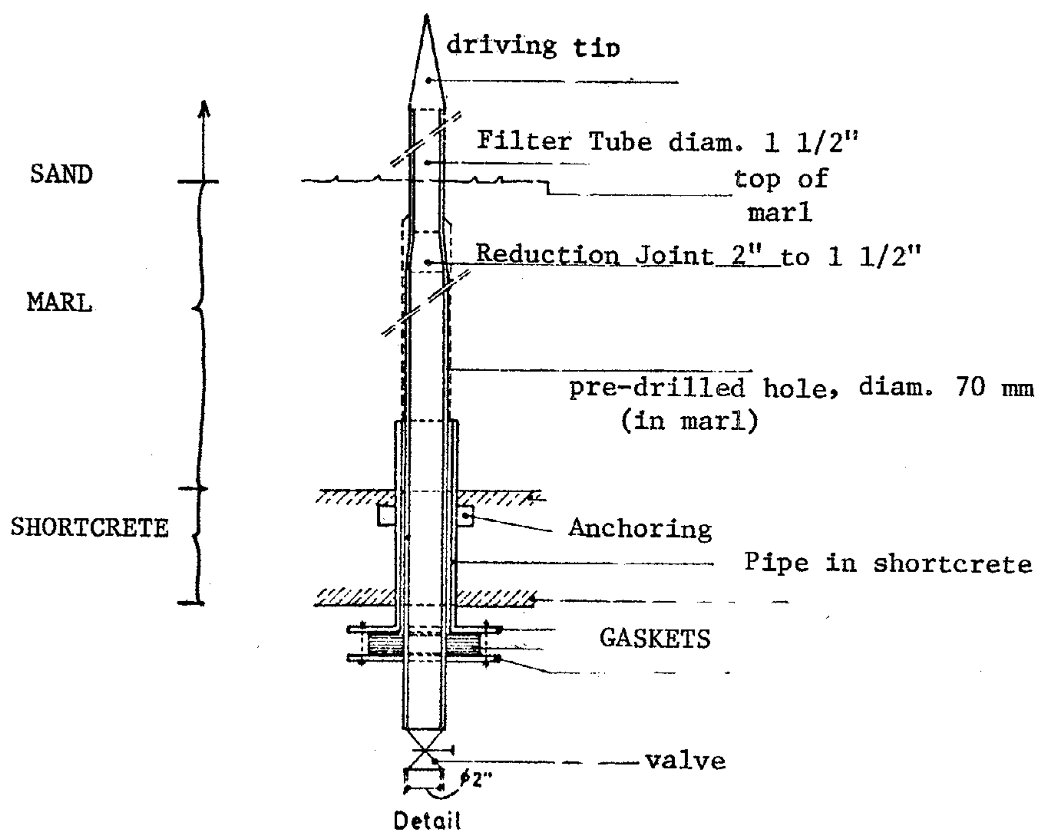


FIGURE A-1.5 SCHEMATIC VIEWS OF WELLS USED FOR LOCALIZED DEWATERING (FROM GEBHARDT, 1978)

WELL DRILLED FROM GROUND SURFACE

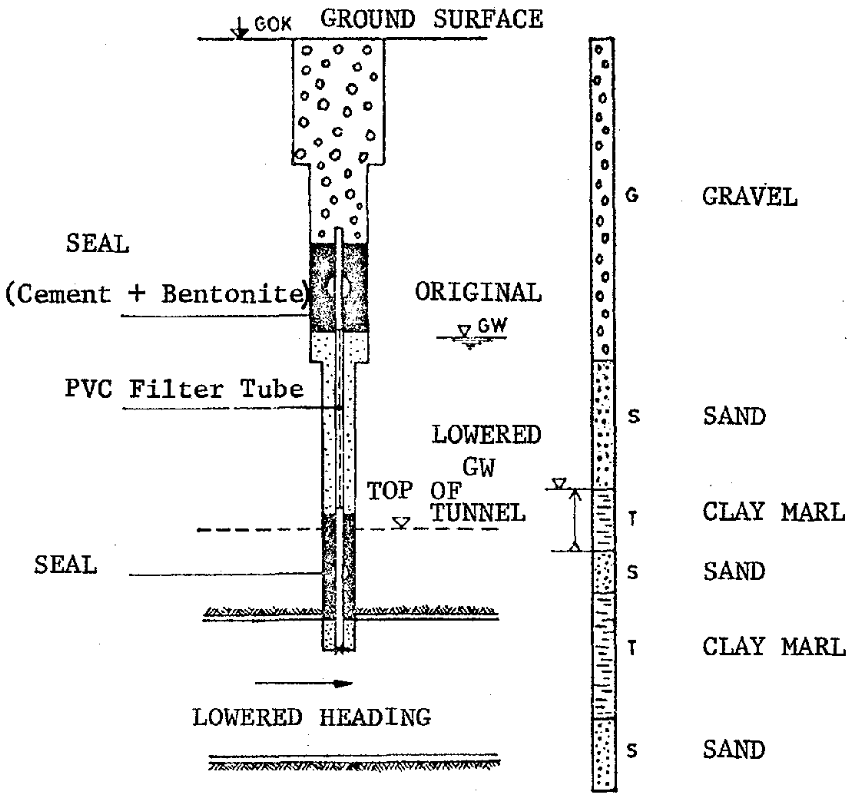


FIGURE A-1.5 SCHEMATIC VIEWS OF WELLS USED FOR LOCALIZED DEWATERING (FROM GEBHARDT, 1978) (CONT.)

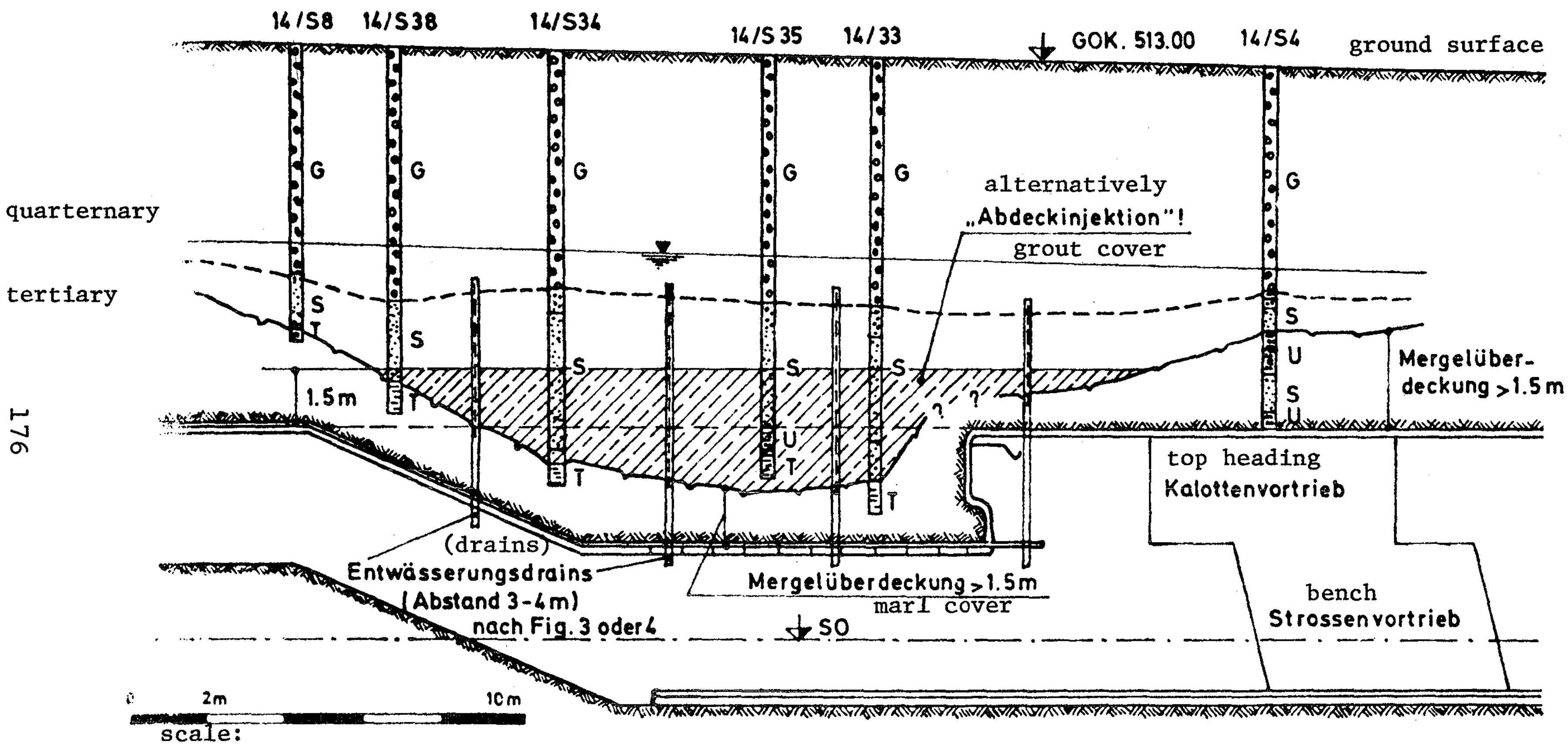


FIGURE A-1.6 PASSING UNDER A LOCAL DEPRESSION (LONGITUDINAL SECTION) IN THE MARL SURFACE AND LOCALIZED DEWATERING (FROM GEBHARDT, 1977)

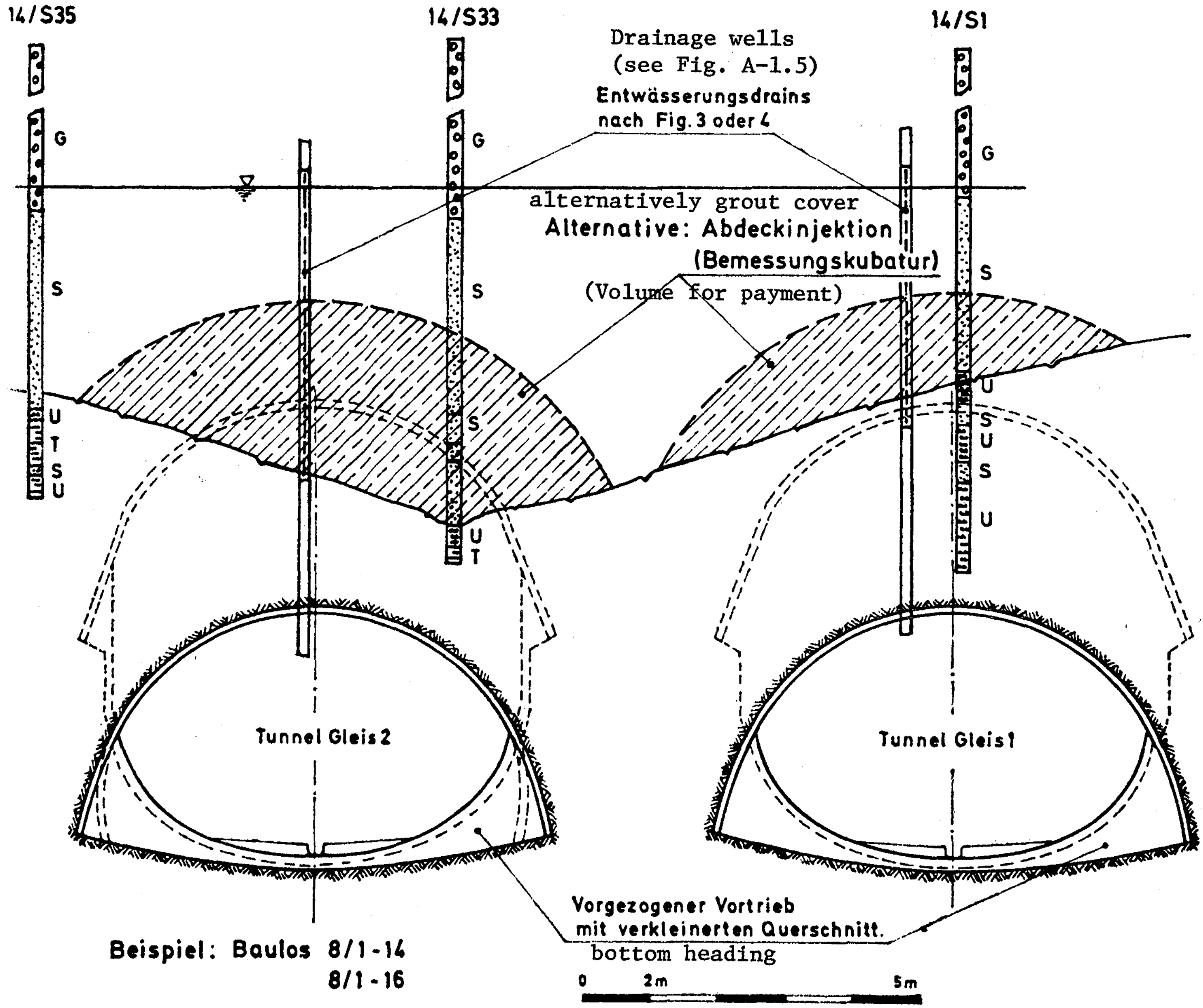


FIGURE A-1.7 PASSING UNDER A LOCAL DEPRESSION IN THE MARL SURFACE, CROSS SECTION (FROM GEBHARDT, 1977)

5.3 Comparison of Tunnel Supports

Data was obtained from Professor Lessmann for the official and alternate designs of section 9 (Sendlingertorplatz) where the NATM was applied for the first time in Munich. Section 9 consists of several short tunnels of different cross-section (Figure A-1.8). Cross-sections of the two-track tunnels are shown in Figure A-1.9. A comparison of the total tunnel support for official and alternate proposals is given in Table A-1.5. The NATM led to a considerable reduction in dimensions and quantities. We do not know the actual costs; however, Laabmayr (1976) quotes cost savings in the order of 35% for the alternate proposal.

Golser (1977) reports on section 8.1 of Line U8/1, which was put up for bid with steel support and concrete liner. Although we lack detailed support quantities, a rough comparison is still possible (Figure A-1.10, Table A-1.6). The total thickness of concrete has been reduced from 60 cm (concrete arch) to 50 cm (15 cm shotcrete and 35 cm concrete). The placed steel for the alternate proposal is also less, since forepoling plates and stellsets were only used in the crown (Figure A-1.10), in contrast to a complete initial steel support according to the official design.

5.4 Performance Monitoring

In sections built according to the NATM, monitoring measurements include: surface displacements; settlements at

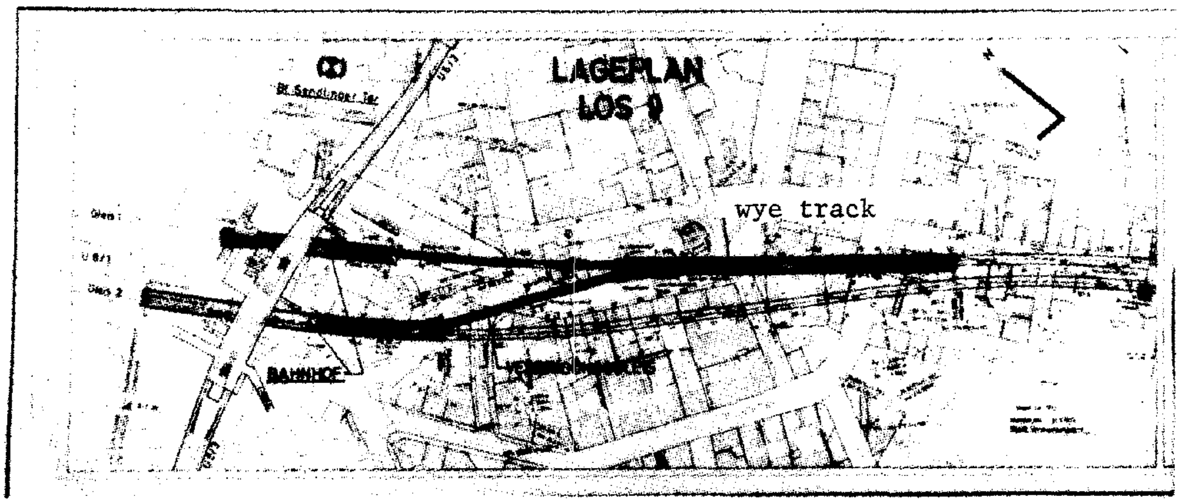


FIGURE A-1.8 PLAN VIEW OF SECTION 9, SENDLINGERTORPLATZ, MUNICH, (FROM LAABMAYR, 1976)

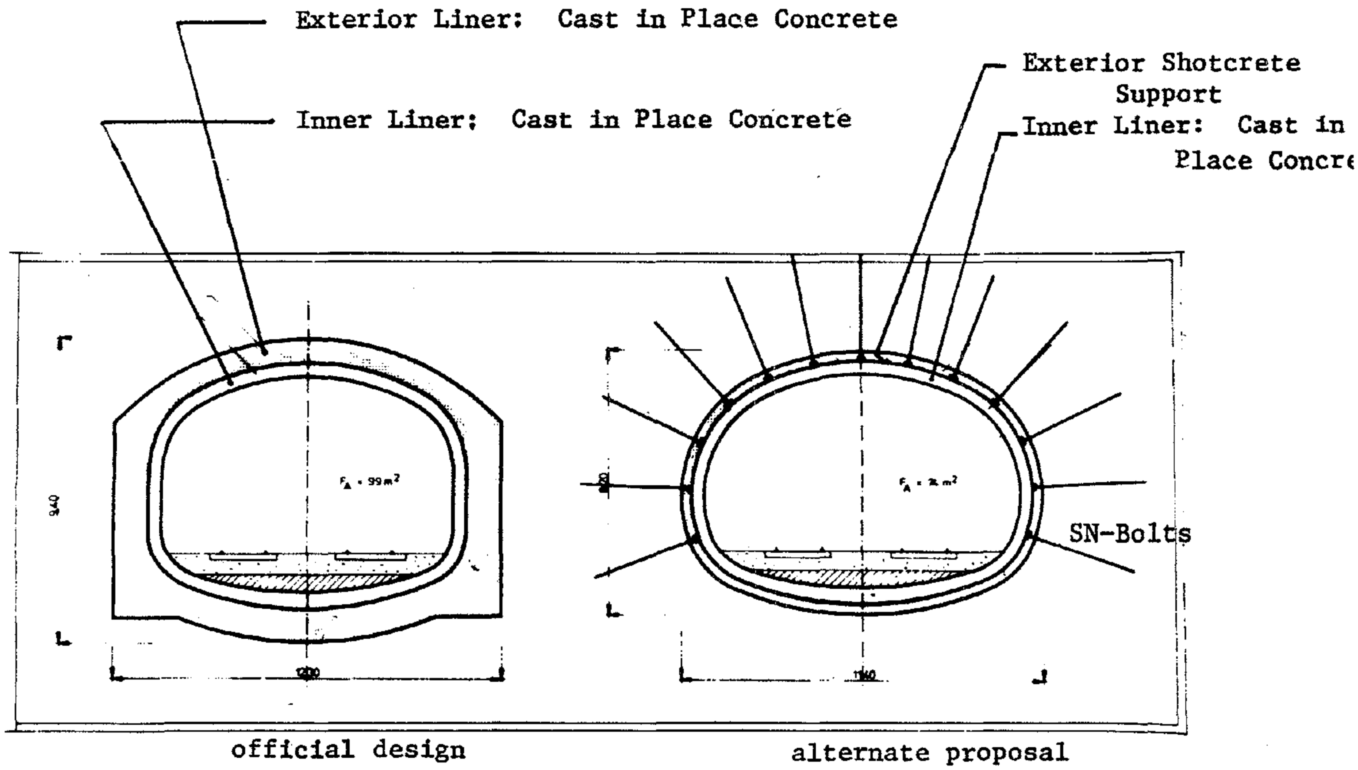


FIGURE A-1.9 CROSS-SECTIONS, SENDLINGERTORPLATZ: OFFICIAL VERSUS ALTERNATE PROPOSAL FRO DOUBLE-TRACK TUNNELS (FROM LAABMAYR, 1976)

(FROM LESSMAN, 1977)

ITEM	OFFICIAL DESIGN	ALTERNATE PROPOSAL, NATM (does not include the minor quantities required in tunnel junctions)	PERCENT REDUCTION (from official design)
EXCAVATION	66,591 m^3	45,497 m^3	32%
STEEL	4,289 tons	1,424 tons	70%
CONCRETE & SHOTCRETE	32,031 m^3	15,902 m^3	50%

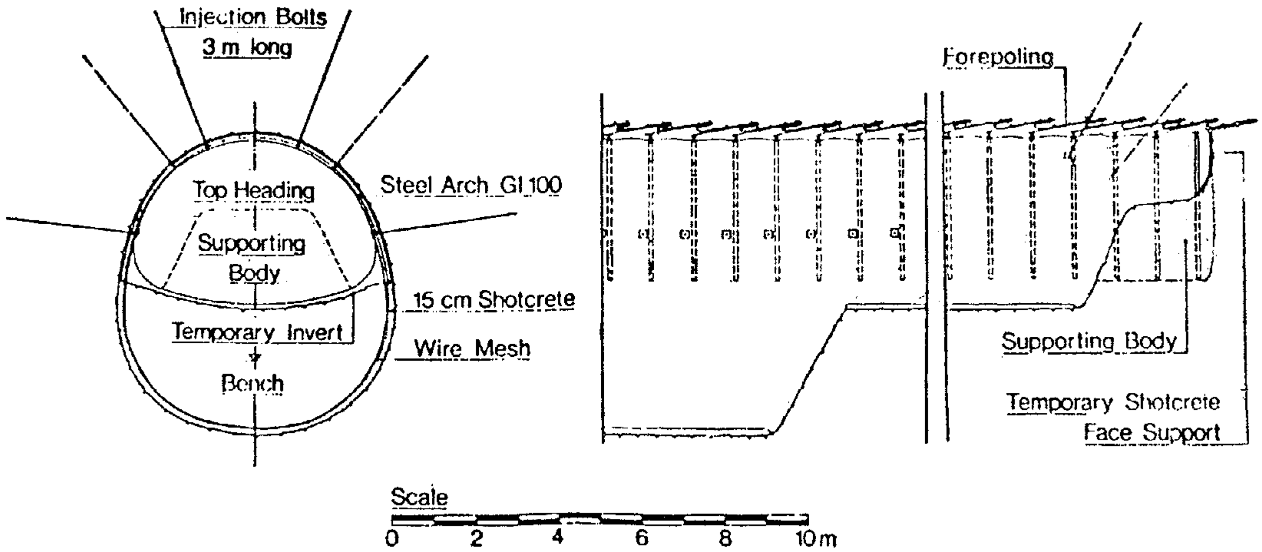


FIGURE A-1.10 CROSS-SECTION AND LONGITUDINAL SECTION OF ALTERNATE PROPOSAL, MUNICH SUBWAY SECTION 8.1 (FROM GOLSER ET AL., 1977)

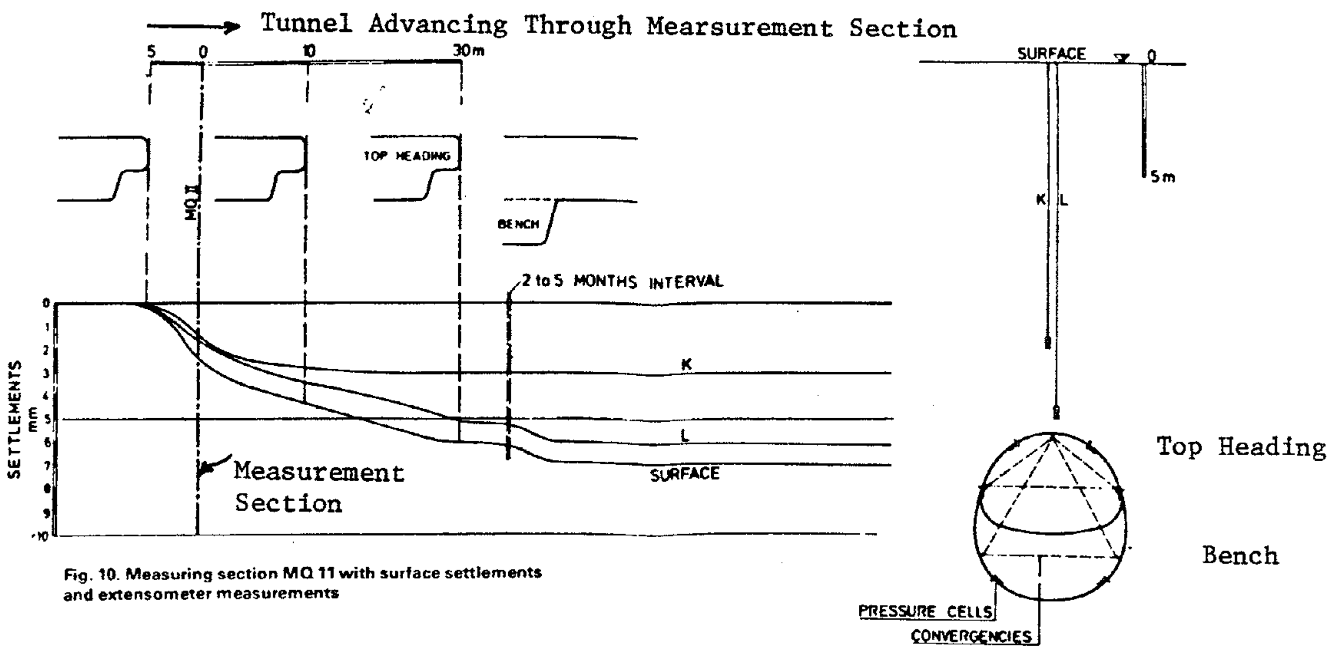


Fig. 10. Measuring section MQ 11 with surface settlements and extensometer measurements

FIGURE A-1.11 DEVELOPMENT OF SETTLEMENTS IN A MONITORING CROSS-SECTION OF THE MUNICH SUBWAY (FROM GOLSER ET AL., 1977)

TABLE A-1.6 SUPPORT DATA, GIESINGER HANGAUFFAHRT
(AFTER GOLSER ET AL. 1977)

SUPPORT TYPE	DIMENSIONS OF INITIAL LINER	DIMENSIONS OF FINIAL LINER
OFFICIAL DESIGN (steel-sets, horseshoes)	steelsets with steel lagging and forepoling, dimensions unknown	60 cm concrete arch reinforcement not known 70 cm baseplate
ALTERNATE METHOD NATM	15 cm shotcrete and steel-sets GI 100 (≃ W 4 x 13) temporary invert	final inner liner 35 cm reinforcement ≃ 440 kg per m tunnel

depths measured with extensometers; convergence in the tunnel; stresses in the liner; and contact stresses between liner and ground. During construction of previous sections not built according to the NATM, monitoring had not been as extensive. During application of the NATM, experience was gained and the measurement procedures were refined. The subway authority is now specifying measurement types and procedures. Section 16 of Line U8/1 is most advanced in this respect. The procedure is presented in depth in Appendix B-1.

For section 8.1 of Line U8/1, Golser et al. (1977) cite the following surface settlements:

- maximum = 15 mm
- average = 10 mm

They further estimated that 20% of the settlement can be attributed to groundwater lowering, 50% occur when the heading is excavated and the remaining 30% are due to the excavation of the bench. Figure A-1.11 shows the development of the surface settlement with advancing excavation in one control section.

In general, the measured settlements vary in the range from 2 to 4 cm (20 to 40 mm); the average is closer to 2 cm. In Munich, no damage has been recorded for surface settlements which are less than 4 cm (40 mm). Settlements larger than 2 to 3 cm are considered alarming. If this occurs, the problem is analyzed more closely. Details of the excavation procedure and the geologic conditions as well as other pertinent factors are studied and remedial action is sought.

In one case, settlements on the order of 6 to 7 cm developed, resulting in surface damage. A detailed account of the causes and the damage has not been published, since this case is in court and is currently being tried.

5.5 Design Aspects

The original alternate bids proposing the NATM based their design on the Rabcewicz shear body analysis (Rabcewicz et al. 1977). This was considered insufficient by the subway authority and the licensed inspection engineers*. A plane strain finite element analysis was performed by Dr. Kovari of ETH Zurich, which was accepted by the authority and the licensed inspecting engineer (Kovari, 1975) for the subway tunnels in the City of Munich. However, substantial discrepancies still exist between predictions and measurements, particularly concerning the liner loads. This is illustrated by the analysis performed by Golser et al. (1977). Figure A-1.12 shows computed thrusts and moments for variable liner stiffness. Although not mentioned in Golser et al., it is believed that to vary the flexural stiffness EI only the modulus of elasticity was varied; thus, the compressive stiffness of the liner (EA) was also

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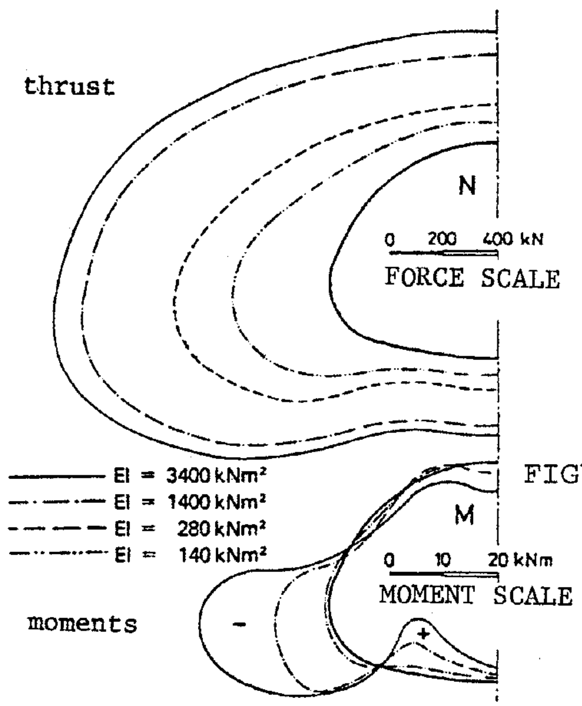
*The German standards (DIN) required that static analysis be approved by a licensed inspection engineer (Prüfingenieur)



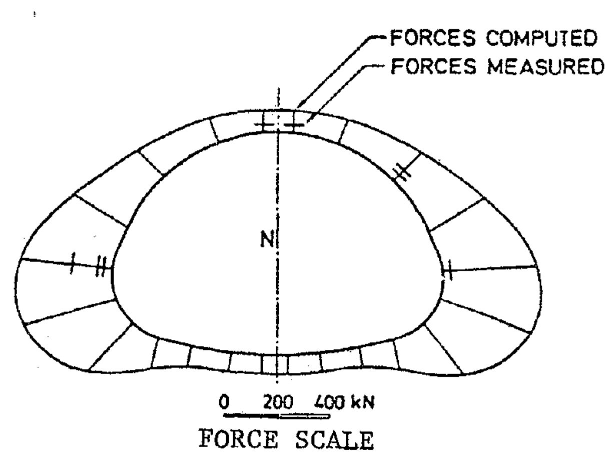


FIGURE A-1.13 PREDICTED VS. MEASURED THRUST FORCES IN THE LINER (SECTION 8.1, MUNICH, FROM GOLSER ET AL., 1977)

FIGURE A-1.12 VARIATION OF THRUST FORCES AND MOMENTS (TOP HEADING, SECTION 8.1) IN LINER WITH STIFFNESS OF LINER (FROM GLOSER ET AL., 1977) changed, resulting in considerable change of liner thrust as well as bending moments. Not unexpectedly, moments and thrusts decrease for more flexible liners. But even with the lowest stiffness, the computed forces are still five to ten times larger than the measured ones (Figure A-1.13). This divergence is due to the fact that the three-dimensional behavior before support placement has not been considered in the analysis.

5.6 Other Types of Analyses

For the analysis of single tubes, a frame analysis with a computer program of the STRESS-type is also accepted by the subway authority (see Section 5).

5.7 Ground Parameters

Each bid schedule contains a table, prepared by the subway authority, with design ground properties, similar to Table A-1.13 shown earlier. The method (tests) by which the parameters are determined is not described; they are probably isotropically consolidated drained triaxial tests. Loading and unloading moduli and drained strength parameters are obtained. The horizontal earth pressure at rest (K_o) is given, but until recently, it was simply assumed to be:

$$K_o = 1 - \sin \phi ,$$

thus for $\phi = 30°$ the earth pressure ratio reduces to:

$$K_o = 0.5.$$

Field measurements performed in test sections led to the conclusion that the horizontal earth pressure at rest is higher and on the order of $K_o = 0.8$. No detailed information on these field measurements is available at present.

6) Construction Aspects

6.1 Shift Arrangements

Construction work for the Munich Subway progresses in two ten-hour shifts for five days a week. A permit for night work from 8 p.m. to 7 a.m. will only be granted by the subway authority if laws and ordinances on environmental control are not violated. This means in particular that only during daylight hours can muck be removed from the tunnel or supplies brought to sites; therefore, there have to be temporary muck storage areas to allow for night excavation.

Care is taken to limit noise. Problems in this respect occurred during the construction of the subway station in front of the main railroad station. This four-track station was built by means of the under-the-roof construction method (Deckelbauweise). Excavation of the diaphragm walls continued uninterruptedly day and night to avoid collapse of the trench. Bands of calcareous marl in the tertiary deposits had to be chiseled through, causing considerable noise and resulting in numerous protests since the area is a hotel district.

Subsequently, the excavation schedule had to be changed. This in turn required a change of the specifications for the slurry and the excavation procedure to prevent collapse of the trenches.

6.2 Rates of Advance

Average rates of advance for shield tunnels were quoted to be on the order of 12 to 14 m per day. Tunnels driven with shotcrete and steelset support (NATM) averaged 3 to 4 m per day. However, the number of points of attack is not the same; shield tunnels have only one or two points of attack, while tunnels driven with the NATM have more, normally 3 to 4. In one large section (Section 16) there were 10. Given these differences, the same length of tunnel per day is usually built by either shield or NATM. A shield requires considerable installation time for each point of attack, while the NATM excavation can start essentially once the access shaft has been completed; thus, total construction time is usually less.

APPENDIX:	A-2
NAME:	Beton-und Monierbau GmbH Subsidiary Innsbruck Tunneling Department
ADDRESS:	Zeughausgasse 3 A-6020, Innsbruck, Austria
DATE OF MEETING:	9th to 13th January and 30th January, 1978
PERSONS MET:	Mr. Blindow, General Manager of the Instruck Subsidiary Dr. Wagner, Head of the Design Department Mr. Kluibenschedl, Head of the Tunneling Department Mr. Bublik, Manager of the Internal Review Department Mr. Decker, Manager of the Equipment Department Mr. Westermayr, Estimator Mr. Paulini, Project Engineer, Design Department Mr. Schulter, Project Engineer, Design Department

1) Introduction

Beton-und Monierbau (B & M) is, since its merger with a Dutch contractor, the largest European contractor. The headquarters are located in Dusseldorf (Germany) with subsidiaries mainly in Germany and Austria. The tunneling department has been consolidated in the Innsbruck office. The firm has published a reference volume (in German, French, English and Spanish) describing their experience with underground work. B & M has pioneered the application of the NATM in its first application in the Schwaikheim Tunnel and also its first application in subway construction in Frankfurt. The firm has a wealth of information on wages, type of equipment used, advance rates, site organization and new technical development which it is willing to make available. A large amount of data was collected during the period from January 9 to January 13, 1978, and on January 30, 1978. More data was sent later by mail.

In this appendix, topics discussed at the home office of Beton-und Monierbau, B&M, are presented. The information on specific sites built by B&M and visited by W. Steiner has been incorporated in the respective appendices. This primarily concerns the following sites: Essen, Section 24 (Appendix B-2); Ganzstein Tunnel (Appendix B-5); Pfänder Tunnel (Appendix B-8). In section 2 of this appendix, cost problems from a contractor's point of view are discussed; in section 3, contractual aspects; and in section 4, technical and organizational issues.

2) Cost Issues

Cost aspects that will be discussed here are the labor costs (2.1), equipment costs (2.2), interest rates (2.3), and also whether price escalation (2.4) is granted.

2.1 Labor Costs

Data on wage rates and labor costs have been obtained for section 24 in Essen (Appendix B-2) and for the Pfänder Tunnel (Appendix B-8). For example, the average labor costs including the apportioned foreman's wage are 31.49 DM/man-hour (15.36 $/h) in Essen as of December 1977 (Appendix B-2). For the Pfänder Tunnel the average wage rate is 220 AS/hour (= 14.67 $/h).

2.2 Equipment Costs

Equipment costs are based on monthly rental and repair costs listed in the "Baugeräteliste" (Equipment Handbook). The same handbook is used in Austria and Germany. Due to the present economic conditions, the rate of depreciation has been cut and monthly rentals are approximately in the order of 50% of the values quoted in the handbook. (This fact has also been confirmed during the interviews with Porr Contractors).

2.3 Interest Rates

Interest rates are important for the contractor since he has to invest in equipment and pay his employees at the end of each month, while payment is usually made one to two months after the costs occurred. In addition, the owner deducts a guarantee retainage of usually 10% (VOB) from each monthly payment. This retainage will only be paid after satisfactory completion of the project. However, it is now possible to substitute the retainage by a guarantee bond. It is important to notice that this bond is not the bid bond as required in the U.S.; in Germany, bid bonds do not exist and usually there are no performance bonds either, although the owner is allowed to require a performance bond.

The rates of interest for borrowed money is on the order of 9 1/2 to 10% per year in Austria and Germany. For the abovementioned guarantee bonds, the rate of interest is on the order of 0.5 to 0.7 percent per year.

According to Mr. Westermayr, the total interest costs composed of all aforementioned components amount to 0.5 to 1.0% of the bid price.

2.4 Price Escalation

In Austria and Germany, escalation clauses are provided for contracts which last longer than one year. More details are given in Section 4 of this report.

As an example, for the Werfen Tunnel whose construction time was about three years, the price increases amounted to approximately 20% of the total costs. Such price increases would be difficult to estimate in a firm-fix price contract, which are thus not used in projects of longer duration. We were told that with a firm-fix price contract, the bid price for the Pfänder Tunnel, a project of three to four years duration, would have been at least 20 to 30% higher than with the actual contract that includes price escalation clauses.

3) Contractual Aspects

3.1 Contract Disputes

A distinction, which relates to dispute settlement procedures, is made between technical contract disputes and payment disputes. Technical contract disputes may arise over issues which can only be settled by technical experts; for these cases arbitration is thus preferred. On the other hand disputes over payments may be settled better by courts. Thus, depending on the type of work, the contract includes or excludes arbitration. Contracts between general and subcontractor often, but not exclusively, involve payment problems; thus an arbitration clause is seldom included. However, a contract between owner and contractor involves technically disputable matter, and arbitration is thus frequently included.

3.2 Arbitration

If a contract includes the option of binding arbitration, the following procedure applies (according to ONORM B2110): owner and contractor designate one expert arbitrator each and these two expert arbitrators select a third one who is considered neutral. The experts are chosen from a list of arbitrators prepared by the organization of the construction industry. A decision by these arbitrators is in most cases faster and cheaper than trials. A decision by arbitration can be made within a few months. The decision is binding and neither party can ask for a court trial. (Payment of arbitrators is by published rates per meeting.)

However, a dispute between contractor and owner may be settled before restoring to binding arbitration through direct negotiations. Each party, contractor and owner, may hire a technical expert who assists the contractor or the owner during direct negotiations where technical solutions are sought as well as the payment terms are renegotiated. The technical experts are consulting engineers or university professors. A solution is often found, because the next step would be binding arbitration which would result in further delay and probably cost both parties more than a negotiated solution.

For example, at the Ganzstein Tunnel, the contract had to be renegotiated because the predicted and encountered ground conditions at the eastern portal were entirely different and much worse than anticipated. A solution was found through negotiation.

4) Technical Aspects

In this section, some of the newest developments in tunnel construction for subways are discussed, followed by a discussion on the selection of excavation procedures for transmountain tunnels.

4.1 Subway Tunnels

For section 5 of line U5/9 of the Munich Subway, a combination of shield tunnels and NATM tunnels will be used. Figure A-2.1 shows a schematic plan view of section 5, which contains the Theresienwiesen Station in the middle and the running tunnels to the northeast to the Hauptbahnhof Station and to the southwest to Messehallen Station. Excavation starts from two shafts on both sides of the Theresienwiesen Station. The station tunnels will be built according to NATM similar to the method used in Bochum (Appendix B-4). The section where the switches are located is built by the open-cut under-the-roof method. West of the switch area, a wye-track is located, requiring an additional tunnel of approximately 150 m length; this and the connecting track (V) at the western end of the section will be built according to the NATM. The running tunnels, which, as mentioned above, are shield driven and supported by single shell precast segmented concrete liners, are driven past the wye-tunnel and the connecting tunnel after the final concrete liner has been placed in these tunnels. The contractor has proposed to drive the tunnels to the northeast under air pressure to prevent running and ravelling and to avoid the use of grouting, which is hindered by existing buildings. One building houses the printing presses for one of Munich's newspapers, which are very sensitive to settlement; no further

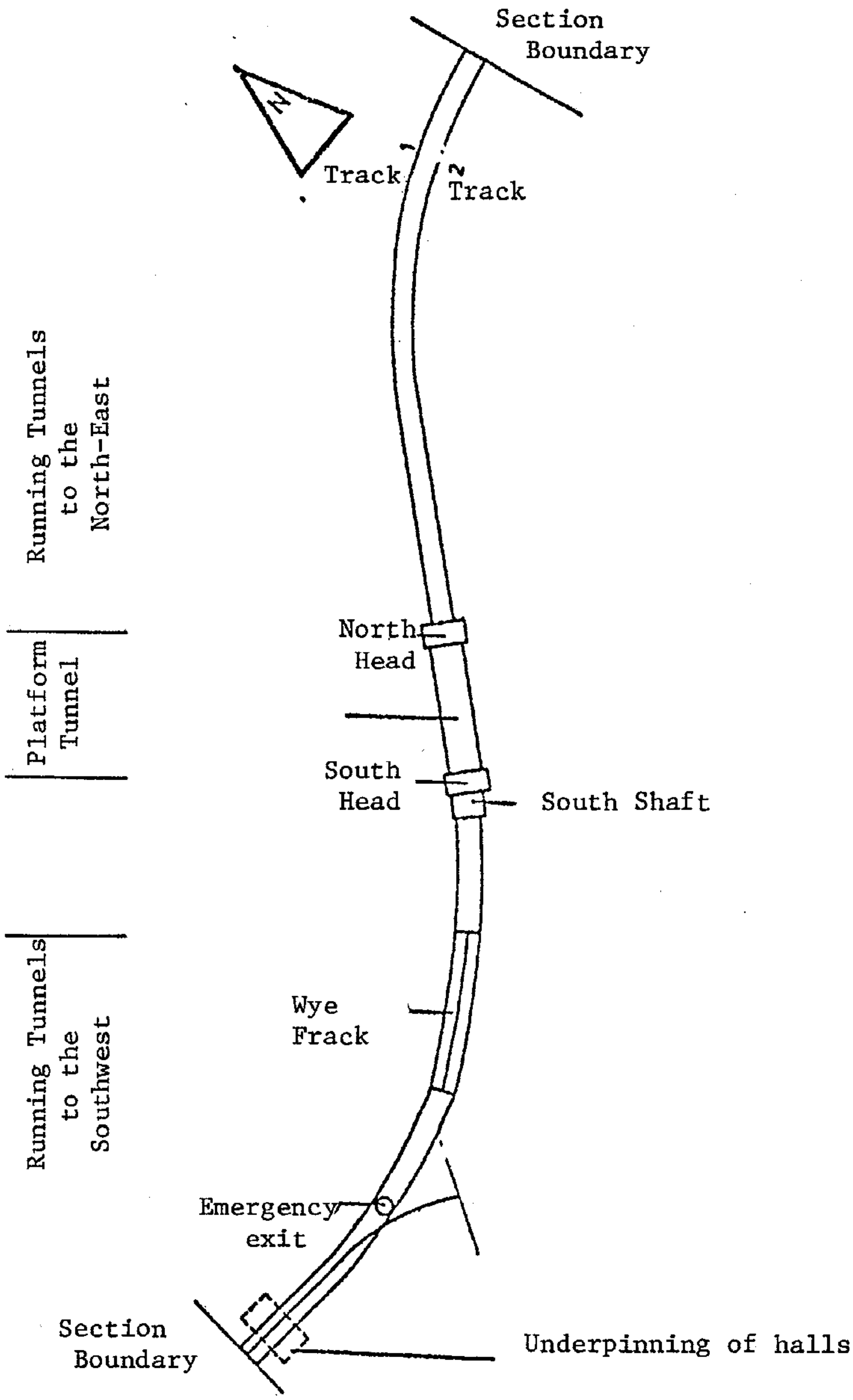
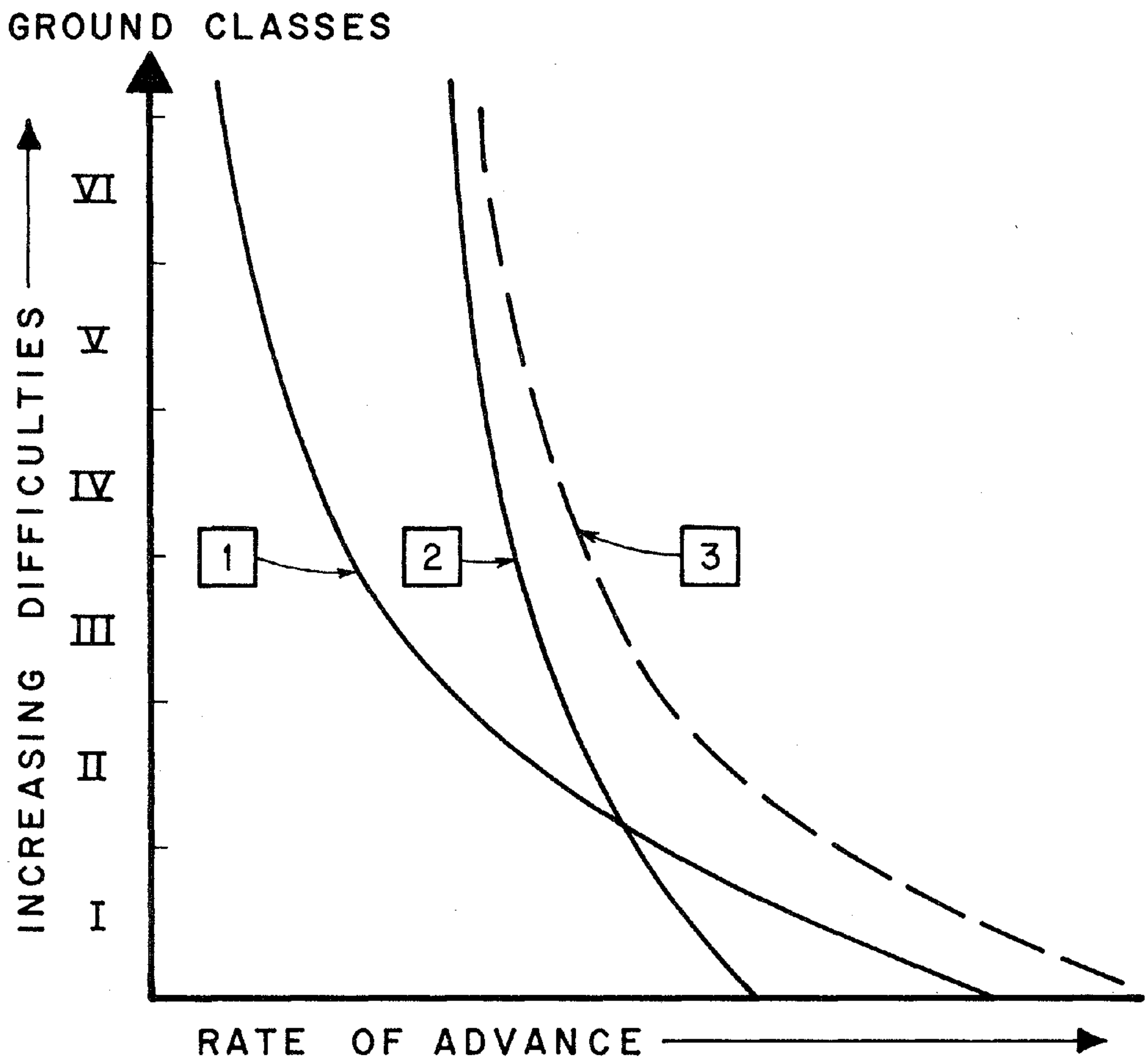


FIGURE A-2.1 SCHEMATIC OF SECTION 5, LINE U5/9, MUNICH (FROM BETON-UND MONIERBAU, 1978) details have been obtained .

The decision on using shield tunnels or NATM depends on the length of sections with a constant cross-section, as was discussed in Section 5 of the main body of this volume. For tunnels of variable cross-sections (shape, area) and short tunnels, the NATM is more economical. According to Mr. Blindow, shield tunneling can only compete with the NATM for section lengths of more than 1400 m length per shield.

4.2 Selection of Excavation Methods for Transmountain Tunnel

The type of equipment and the site organization considerably influence the rate of advance and the success of a job. The adaptability to changed conditions is a major factor in this respect, which can best be illustrated by a qualitative diagram (Figure A-2.2). The advance drops considerably when full face has to be changed to a heading and benching method (curve 1). However, a continuous heading and bench method may result in somewhat lower advance rates for the good ground classes, but considerable higher advance rates for the bad ground classes. At the Pfänder Tunnel, the advance rates in better ground classes seem to exceed those for a full face excavation; there a heading and benching procedure has been chosen because 60% of the excavation would have required it. With the frequent changes from full-face to heading and bench excavation would have resulted in lower average advance rates (see Appendix B-8). In this context, it is also interesting to mention the effect of ground class on the advance rates; in



1 Excavation method primarily designed for full-face excavation. Change to heading and benching is time consuming.

2 Excavation procedure primarily adapted to heading and benching.

3 Qualitative Curve for Pfänder Tunnel.

FIGURE A-2.2 REATES OF ADVANCE FOR DIFFERENT EXCAVATION PROCEDURES the Werfen Tunnels, the advance rates where 10 m, 7.5 m, 4.5 m, 1.5 m, 0.9 m per day in classes I-V, respectively.

APPENDIX: A-3

NAME: Institut fur Konstruktiven Wasserbau und Tunnelbau (Institute for Hydraulic Construction and Tunneling) University of Innsbruck

ADDRESS: Technikerstrasse 13
A-6020, Innsbruck
Austria

DATE OF MEETING: January 11th, 1978

PERSONS MET: Professor Dr. G. Seeber
Mr. Keller

1) Introduction

At the Technical University of Innsbruck, research funded by the Austrian government and directed by Professor Seeber is carried out on tunnel support and deformation in tunnels; a paper has been published at the Salzburg Geomechanics Conference in 1978 and one will be published in the proceedings of the International Conference on Rock Mechanics in 1979. Professor Seeber started an educational program in tunnel design three years ago; prior to that time, it was not taught in Innsbruck. Before coming to the university, Professor Seeber was employed by TIWAG (Hydro-Power Company of Tyrol). He was one of the collaborators of Dr. Lauffer (author of the stand-up time charts). Professor Seeber was a consultant to ASTAG (Arlberg Highway Tunnel Authority) during the construction of the Arlberg Highway Tunnel. The information obtained by Professor Seeber thus deals with experience gained at the Arlberg and some of his remarks on ground classification.

2) Experience Gained from the Arlberg Roof Falls

In the western section of the Arlberg Tunnel (see Appendix B-7), an unexpected roof collapse in ground class III (support by shotcrete, wire mesh and rock bolts of 4 m length, but no steelsets) occurred. The support was improved by adding steelsets and using longer bolts; essentially the ground was reclassified as ground class IV. Evidently, insufficient bolting (bolts too short) is the cause of the problem.

Prediction of the Final Tunnel Convergence and Load on final liner

A reliable prediction of tunnel convergence is required in order to select the necessary overexcavation. If the over-excavation is too small, the tunnel has to be re-excavated, which is nearly impossible with the density of the rock bolts as used at the Arlberg; if over excavation is too large, the quantity of concrete for the final liner will increase unnecessarily. A second problem is the prediction of the stresses in the final liner. Seeber (1976) uses a quasi-elastic approach along with the procedure of characteristic curves.

3) General Comments on Ground Classification

3.1 Stand-Up Time

In answering the questions about the relevance of Bieniawski's interpretation of Lauffer charts, Seeber agrees that this chart is erroneous (Bieniawski, 1975). As a former collaborator of Lauffer, he is familiar with the development of Lauffer's chart, which was developed during the construction of the Prutz-Imstpower scheme. The span stand-up time relation reflects the experience gained during the excavation of the pilot tunnel (10 m^2) and the main tunnel (25 to 30 m^2).

For class 'e' the shotcrete had to be applied immediately after each round, whereas for class 'c' one could wait 1 to 2 days. The chart should not be over-interpreted, as it is based on limited data. Furthermore, Lauffer only considers span and ground conditions but not the method of excavation. Seeber considers the method of excavation and support procedure to be important for the classification.

3.2 Assignment of Ground Class

The determination of ground classes is a major point of dispute between the owner and the contractor, and the one with more endurance wins. From his experience with TIWAG, Seeber concludes that in cases where the owner had a weak representative, the tunnels were built more expensively.

APPENDIX A-4

NAME: Institut fur Bauverfahren und Bauwirtschaft (Institute for Construction Management, University of Innsbruck)

ADDRESS: Technikerstrasse 13
A-6020 Innsbruck
Austria

DATE OF MEETING: 12 January, 1978

PERSONS MET: Professor Lessmann, Head of the Institute
Dr. Becker, Research Associate

1) Introduction

Professor Lessmann and Dr. Becker are part-time professor and research associate, respectively, in project management at the Technical Faculty of Innsbruck University. For the remainder of their time they are employed by Bilfinger and Berger, Contractors, Munich Subsidiary. Bilfinger and Berger was a partner in the joint venture that built section 9 of Line U8/1 (Sendlingertorplatz) in Munich, the first application of the NATM in Munich.

Professor Lessmann seemed to have been informed by the Munich Subway Authority about our project and requests. He was concerned that "negative effects might arise from our research, and that we might render a disservice to tunnel construction by concluding that subways are too expensive to build." W. Steiner explained our intention to show that tunnels can be built less expensively than is presently the case.

Part of the discussion with Professor Lessmann dealt with contractual procedures used for tunnel construction, information that is discussed in detail in the main text. Lessmann stressed the contractor's point of view, which will be presented in section 2 below. During the discussion, Lessmann also presented his ideas on site organization for tunnel construction (section 3).

2) <u>Contractual Aspects</u>

In subway tunnels (Germany), the subsoil conditions are well-known and explored. The contractor is thus able to assess the risk. Contrarily, in alpine tunnels (Austria) the ground conditions are never known with the same accuracy and reliability. The contracts in Austria and Germany are thus different, the difference reflecting different tunneling conditions rather than just different national influences. Nevertheless, geologic risk is carried by the owner in Germany if the contractor bids the official design. However, if a contractor submits an alternate bid, he carries the geologic risk.

3) <u>Ideas on Subway Construction</u>

Lessmann's prerequisites for subway tunnel construction are:

a) the method selected has to be adapted to the ground conditions,

b) the method has to be economic.

In particular, the NATM cannot be used under air-pressure, since the water and compressed air necessary for the shotcrete pump would lead to fogging of the pressurized work area. However, Lessmann envisions a cheap, non-toxic grouting method which would allow simple groundwater control to prevent running ground and at the same time reduce the cost of ground water control. However, at the present time, air pressure is necessary in certain grounds which are difficult to dewater and thus prevents application of the NATM.

With respect to the NATM, Lessmann likes the possibility of several points of attack and the use of standard heavy construction equipment. For example, he favors hydraulic excavators over partial face TBM's (road-headers); a breakdown of a road-header may continue over a prolonged period of time when parts are not available, whereas a hydraulic excavator can be replaced easily since it is often available from other jobs. Also, an excavator may be used again on a completely different job, which does not have to be a tunnel; the depreciation costs of the excavator are thus lower than for the partial face machine.

Lessmann points out that Bilfinger & Berger, contractors, have a strict internal cost control system, and each site is reviewed every four weeks. In particular, isolated high advance rates as well as erratic advance and production rates are not appreciated; a site has to show a continuously "high" average rate. Lessmann mentions that operation planning follows procedures developed by BWI (Betriebswirtschafticches Institut =

Institute for Project Management) of the University of Economics and Management, St. Gallen, Switzerland. No detailed description of the procedure is available; in brief, it is based on a comparison of actual performance with set goals. During the duration of the project, the goals may be changed, based on the actual performance.

APPENDIX:	A-5
NAME:	Arlberg Strassen Tunnel AG
	ASTAG
	Arlberg Highway Authority
ADDRESS:	Heiliggeiststrasse 21
	A-6020 Innsbruck
	Austria
DATE OF MEETING:	13th January, 1978
PERSON MET:	Mr. Posch, General Manager

The meeting with Mr. Posch was organized by Dr. John of ILF (Ingenieurgemeinschaft Lässer-Feizlmayr). The written request for a visit and data collection at the Arlberg tunnel had not reached ASTAG, the owner of the Arlberg tunnel. Nevertheless, Mr. Posch gave W. Steiner the permission to visit the sites. Later ASTAG responded favorably to a formal request for geologic and geotechnical data from the Arlberg tunnel. Data for 2 km of the Arlberg tunnel (the most difficult zone) have been obtained in the meantime from ILF (Design Engineer).

APPENDIX: A-6

NAME: Ingenieurgemeinschaft Lässer-Feizlmayr (Design Engineer)

ADDRESS: Framsweg 13
A-6020, Innsbruck Arzl
Austria

DATE OF MEETING: 12th and 13th January, 1978

PERSONS MET: Dr. M. John, Senior Project Engineer

1) Introduction

ILF (Ingenieurgemeinschaft Lässer-Feizlmayr) is the design and construction supervising engineer for the Arlberg, Pfänder and Dalaas tunnels. The discussions with Dr. M. John, Project Engineer, centered around problems related to the design and construction of these tunnels. The results of these discussions have been largely incorporated in the respective appendices (B-7, B-8). Dr. John has provided us with preprints of several papers on tunnel construction and detailed data from the Arlberg tunnel. The discussions with him gave us considerable insight into the design philosophy for the Arlberg and Pfänder tunnels and greatly facilitated this research.

2) Ground Classification and Contract Documents

Ground classification and contract conditions cannot be separated. Complete and comprehensive contract documents are important in tunnel construction. Ambiguities in the documents have to be avoided. Dr. John considers it useful to develop contract and bid documents in collaboration with a lawyer. However, this procedure is time-consuming and might not always work in practice. Also, he does not favor a bonus-malus clause for support payments as is used at the Arlberg tunnel; it is a source of disputes and leads to a distortion of the ground classification (see Appendix B-7). For the Pfänder and Dalaas tunnels, the bonus-malus provision has been abandoned. Payment provisions have also been changed in that excavation and support are treated as entirely separate pay items. The difficulties related to support placement have to be included in the support item and no longer in the excavation. As experience at the Pfänder shows, these changes reduced disputes considerably.

Dr. John favors a simple ground classification system as it is used in Austria (see Sections 4 and 5). A procedure like Bieniawski's* or Barton's, where first several parameters have to be determined which will then yield the ground class, may lead to disputes over each parameter which has to be determined. At the present time, Dr. John considers a qualitative behavioral classification, as used in Austria, the best practical solution.

For each ground class the support is designed assuming average ground strength properties representative of the particular ground class and using Rabcewisz's shear body analysis.

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*Bieniawski has developed the ground classification system after Dr. John had already left CSIR.

Clearly, the major issue is a proper assumption of the ground strength properties; this is done judgementally, based on experience.

APPENDIX:	A-7
NAME:	Geoconsult, Consulting Engineer
ADDRESS:	Sterneckstrasse 55 A-5020, Salzburg Austria
DATE OF MEETING:	16th January, 1978
PERSONS MET:	Mr. Golser, Partner Mr. Mussger, Project Engineer

1) <u>Introduction</u>

The interviews at Geoconsult dealt with different topics, and much of the information gained during these interviews will be of value for future research. The discussion in this appendix is strictly limited to the information gathered; some sections may thus appear to be rather sketchy. The following topics were discussed: cost of tunnel construction in Austria, ground classification, roof collapses and problems related to the construction of the Tarbela Dam diversion tunnels.

2) <u>Construction Cost of Austrian Tunnels</u>

Average costs for the Tauern, Katschberg and Gleinalm tunnels are shown in Table A-7.1. These costs include construction planning and supervision and the cost of financing during construction.

TABLE A-7.1 TUNNEL COSTS IN AUSTRIA
($1 = 15AS)

Case	Price per meter of tunnel AS/m	$/m
TAUERN TUNNEL, cross-section = 105 m^2		
with cavern and portals	265,000	17,700
without	230,000 = 2200AS/m^3	15,333
KATSCHBERG TUNNEL		
including cavern and portals	177,000	11,800
without cavern and portals	134,000	8,933
GLEINALM		
bid	76,000 to 79,000	5,067 to 5,267
complete	90,000	6,000

3) Ground Classification

The ground classification procedure used in Austria has been described in Section 5. In an attempt to reduce disputes in assigning ground classes, Mr. Mussger used Barton's and Bieniawski's rock classification procedures in practice. The two procedures work where the ground is clearly good or bad. However, they do not work for the more important intermediate cases; i.e., they cannot differentiate between different shades of grey. From his experience, Mr. Mussger concluded that the two systems are difficult to apply in practice, and they are complicated. The Austrian procedure is considered simpler to apply, since only one number, the ground class, has to be determined. In contrast, Barton's and Bieniawski's classification procedures require the determination of several parameters, from which a ground class* is determined. Thus, each of the parameters can be a cause of dispute, greatly increasing the likelihood of disagreement.

In the context of ground classification, the problem of extrapolation from a small pilot tunnel to the normal size tunnel is important. After a pilot tunnel is available, however, the extrapolation proves to be rather difficult and often fails. Particular examples where the extrapolation did not work properly are the Tarbela diversion tunnels and tunnels

*Ground class naturally includes implicitly many different factors.

in Austria (Mitterberg, Appendix A-14). No established rules exist or have been developed with regard to the extrapolation from pilot tunnel to regular size tunnel.

4) Roof Collapses

In recent times, several roof collapses occurred in Austrian tunnels. Their common feature is that they occur rapidly and often without warning. They are more likely in frictional ground, and rarely occur in cohesive ground. Ground with little tensile strength is most likely to experience roof collapses.

To prevent roof collapses a rapid ring closure is necessary; i.e., the ring with shotcrete has to be closed in the invert as fast as possible. Rock bolts do not, in general, provide sufficient support.

5) Information on Tarbela Tunnels

During the construction of the Tarbela Dam diversion tunnels, Mr. Golser was resident engineer once the NATM was used for driving these tunnels. A description of some details of these tunnels is given in Einstein et al. (1977). Mr. Golser delivered another paper on the Tarbela tunnels at the Salzburg Colloquium in 1977, and he provided us with a preprint which, however, did not include figures. During the interview, W. Steiner was able to gather some more data on ground properties at Tarbela, in particular, the strength and the block size of different rocks encountered. This information is presented below for 'basic rock', 'chlorite schists', and 'sugary limestone'.

5.1 Basic Rock (Gabbro)

The block size varies from inch size to cubic meters. In tunnel 2, the block size is approximately a cube, with the length varying from 5 to 20 cms. The joint surfaces are smooth and often coated with serpentine. The joints are slightly offset.

This rock was stable in the small exploratory drifts of 15 to 20 m^2 cross-section and in the large tunnels after breaks occurred (gebräches Verhalten).

5.2 Chlorite Schist

Chlorite schists were soft and plastic and developed only very small squeezing stresses and little loosening. Thus, it is considered by Mr. Golser to be of slightly better quality than the Tauern phyllite. A major difference compared to the Tauern is the small overburden of only 130 m (Tauern, 800 m).

5.3 Sugary Limestone

A mass friction angle of 40° has been estimated for this rock. In the area of the sugary limestone, dewatering was by wells driven from exploratory drifts, which reduced the danger of flowing ground. The seepage force towards the wells stabilized the ground. However, during excavation care had to be taken as to not disturb the ground.

APPENDIX: A-8

NAME: Laabmayr,
Consulting Engineers

ADDRESS: Schallmooser Hauptstrasse 22a
A-5020, Salzburg
Austria
or Rindermarkt 7, D-8000, Munchen 2,
Germany

DATE OF MEETING: 16th January, 1978

PERSONS MET: Mr. Laabmayr, owner

1) Introduction

Mr. Laabmayr was involved in the implementation of the NATM for the Munich Subway, but now is also in subway construction in other cities. He primarily acts as a consultant to contractors who want to submit an alternate proposal. His first involvement with subway construction came with the design of the Sendlingertorplatz Station of the Munich subway (section 9. Line U8/1); at that time Mr. Laabmayr was an employee of Dr. Pacher. Later, Mr. Laabmayr became an independent consultant with offices in Munich (Germany) and Salzburg (Austria).

In the following sections, the main points of the discussion with Mr. Laabmayr are reported. Problems in sign and construction, the effectiveness of support types, and a method to reduce spalling are discussed.

2) Subway Design Problems

In design practice for subway tunnels in Germany, plane strain finite element analyses are a standard procedure. The FE Analyses for section 9 (Sendlingertorplatz) in Munich were performed by Dr. Kovari of ETH, Zurich. Figure A-8.1 shows one of the results. Note, in particular, the large heave in the surface excavation (2 cm) relative to the other displacements. However, during the actual construction, no such heave was monitored. Subsequent FE Analyses with a method by Swobada of the Technical University of Innsbruck predicted no such heave. The problem of the larger surface heave in the first analysis seems to be primarily associated with the assumed boundary conditions.

3) Effectiveness of Different Support Types

According to Mr. Laabmayr, the most important factors to reduce surface settlements in shallow tunnel construction with the NATM are:

(i) properly placed and braced light steelsets,

(ii) shotcrete placed rapidly, and voids between ground and steelsets properly filled.

One notes that bolts are missing from this list. Bolts were used for the initial support of the Sendlingertorplatz station (the first NATM subway tunnel in Munich). However, bolts have not shown to be of significant importance in shallow tunnels; i.e., they seem not to lead to a reduction of settlements

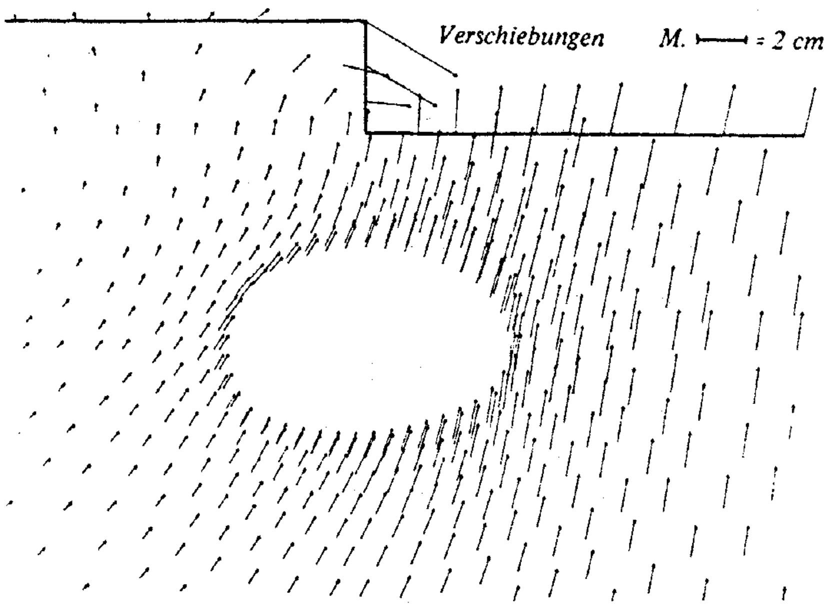


FIGURE A-8.1 PREDICTED DISPLACEMENTS FOR SECTION 9, SENDLINGERTORPLATZ (FROM KOVARI AND HAGEDORN, 1975)

Furthermore, they require much manual work and thus make construction more expensive. Thus, from an empirical as well as technical point of view, bolts for the support of shallow tunnels are not considered to be practical.

4) Spalling in Deep-Lying Tunnels

In the Ofenauer tunnel south of Salzburg (Figure A-8.2), spalling was observed. In the middle of the tunnel (length = 1.41 km, max. overburden = 500 m), a cavern allows emergency turns and will later be connected to a planned parallel second tunnel. The cavern was designed with vertical sidewalls, along which the spalling occurred. To prevent spalling, the sidewalls in another cavern were curved and no support was required for the walls of this cavern (Figure A-8.2).

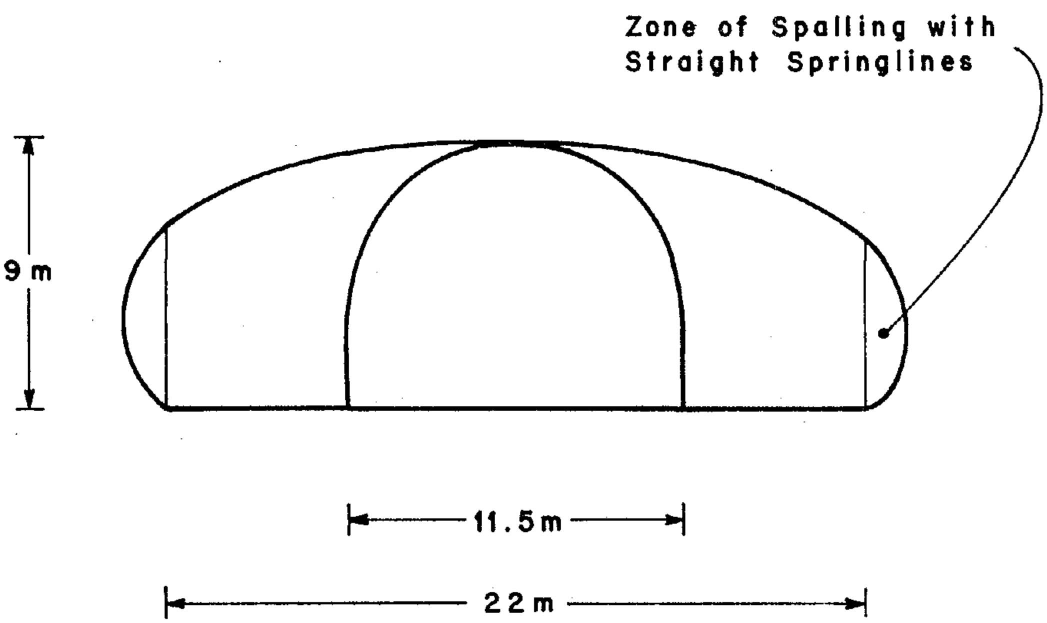


FIGURE A-8.2 PREVENTION OF SPALLING WITH CURVED SIDEWALLS, OFENAUER-TUNNEL (AFTER LAABMAYR, 1978)

APPENDIX: A-9

NAME: Dr. Pacher

Consulting Engineer

ADDRESS: Franz-Josefstrasse 3

A-5020 Salzburg, Austria

DATE OF MEETING: 17th and 18th January, 1978

PERSONS MET: Dr. Pacher, Owner

1) Introduction

Dr. Pacher is a geotechnical consultant and was a former collaborator of Professor Rabcewicz and Professor Müller. He has designed many tunnels in Austria and abroad, many of them in collaboration with these two professors. The most notable projects are the Tauern and Katschberg Tunnels in Austria. In addition, other smaller tunnels were designed by the firm of Dr. Pacher. Data on three shorter tunnels on the access highway to the Tauern were made available. These tunnels are the Brentenberg, Zetzenberg and Helbersberg Tunnels, which will be described in more detail in section 3.1. A general description of the Klamm Tunnels on the highway to Badgastein has been obtained (Section 3.2), also. The geologic data of the Klamm Tunnels have been obtained from the University of Graz (Brandecker and Vogeltanz, 1975).

However, before these tunnel projects will be described, a more general discussion with Dr. Pacher on tunneling problems will be presented.

2) Classification in Tunneling and Related Problems

This section summarizes some of Dr. Pacher's ideas on classification for tunnels. At the present time, a new Austrian standard for tunnel construction is being developed. This standard also includes recommendations on tunnel classification. Some significant changes compared to the present system will be included in this standard, in particular:

i) Seven major ground classes are considered.

ii) In ground classes where large deformations can be expected, the amount of necessary overexcavation has to be specified. This overexcavation is deliberately made to accomodate the displacements occuring after excavation.

iii) For each ground class, single standard quantities are no longer quoted, rather ranges of support. E.g., a range of bolt lengths and a range for the number of bolts to be specified.

Classification schemes may represent different aspects. Dr. Pacher differentiates between three types of classification for tunneling:

a) the rock mechanic, geologic classification

b) classification from the tunnel statics point of view

c) classification from the contractor's point of view

A rock mechanics geologic classification is a classification of the rock without consideration of the size of the tunnel and the overburden conditions.

A classification system from a tunnel statics point of view considers the support required as a function of ground conditions, overburden and support material (type). The classification also has to consider the expected displacements.

The classification from the contractor's point of view primarily addresses operational aspects, i.e., the method of excavation and also the support installation. The method of excavation to loosen the rock (blast, hydraulic excavator) or the specific amount of explosives are important. Another important aspect is the time, or distance to the face, where the support has to be placed.

A comprehensive classification has to include parts of all three systems.

2.1 Methods of Design

Depending on the ground classes, different types of design procedures for the support are necessary. Figure A-9.1 shows the qualitative relationship between rock strength and overburden. The third parameter, the size of the opening, is indicated; however, the relation is not known. For ground class I, generally, no support is necessary, thus no design is required. For ground class II, a load acts primarily in the crown; the support may be designed as an elastically embedded ring (Winkler foundation) with a crown load acting on it. For ground class III, the support may be designed with a finite element procedure. For ground class IV and V, a design with the

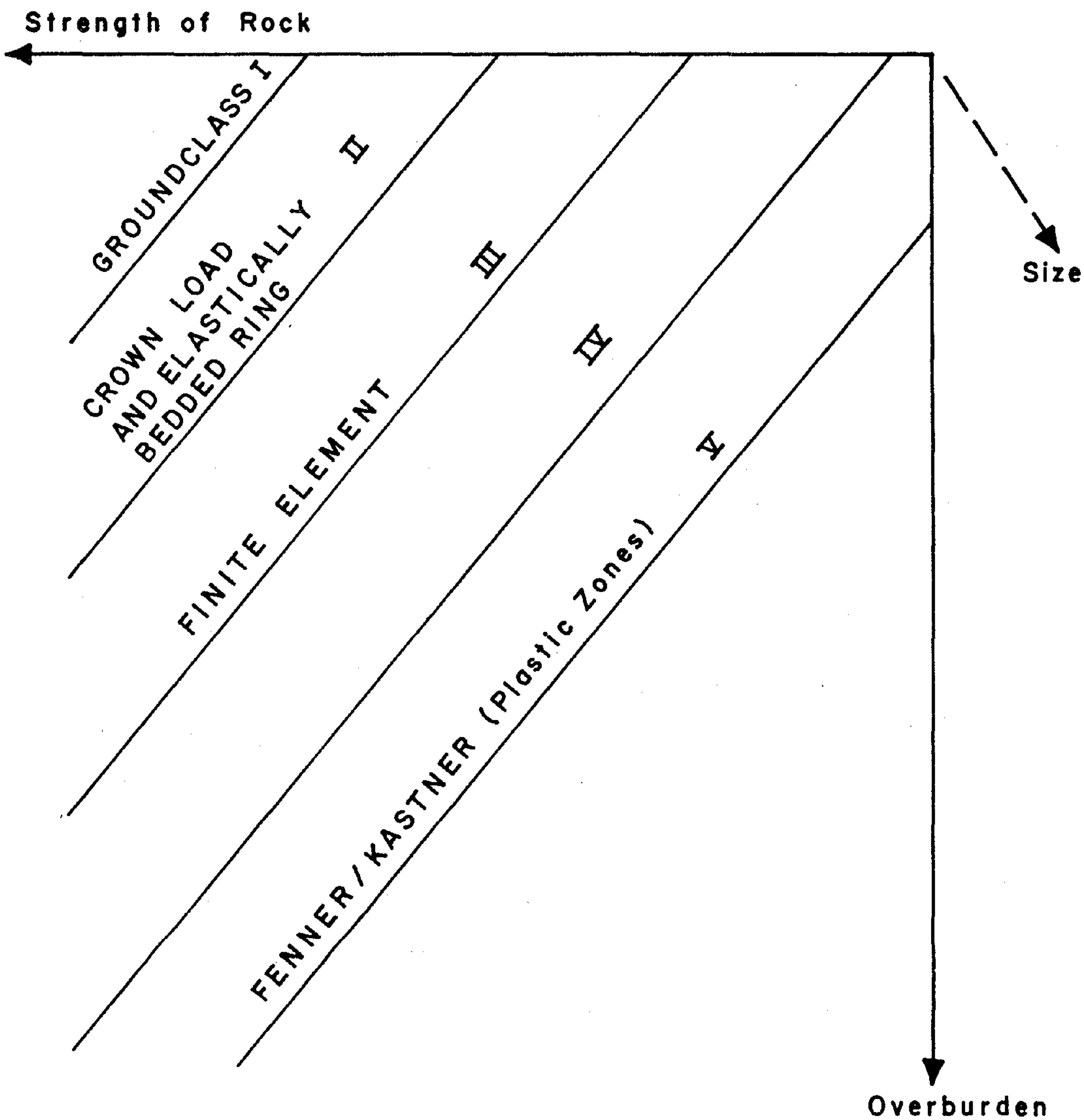


FIGURE A-9.1 APPLICATION OF DESIGN METHODS FOR DIFFERENT GROUND CLASSES AND QUALITATIVE INFLUENCE OF ROCK STRENGTH AND OVERBURDEN CONDITIONS (AFTER PACHER, PERSONAL COMMUNICATION)

Kastner-Fenner method may be used (sometimes for ground class I a finite element method may also be used).

2.2 Shallow vs. Deep Tunnels

For tunnels in ground class IV and V one has to consider the overburden. Shallow tunnels may primarily cause problems during excavation. For deep tunnels, the excavation may not cause problems; however, large convergences might be experience after excavation. In particular, the prediction of the over-excavation for deep tunnels has to be carefully studied to reduce contractual disputes.

2.3 Time-Dependent Behavior

In this context, Dr. Pacher mentions the notion of apparent viscosity to describe time-dependent behavior of excavations at great depths. A relation between ground conditions (rock mass properties) and rates of deformation should be established Then the tunnel designer and contractor would have an additiona criterion which would allow judging the performance of an opening: the rate of convergence. A satisfactory performance is achieved when the observed rate of deformation is below the limiting one. However, at present, no conclusive criteria are available.

From the experience at the Tauern and the Arlberg Tunnels, it is possible to conclude qualitatively that geologic structur has an influence on the time-dependent behavior. At the Tauern Tunnel, the discontinuities strike perpendicular to the tunnel and dip to the north. At the Arlberg Tunnel, the discontinuities strike subparallel to the tunnel. It could be observed that in both tunnels the convergence in the cross-cuts were substantially different from those in the actual tunnel. In the Tauern Tunnel, the cross-cuts experienced larger convergences than the tunnel; at the Arlberg Tunnel, the convergences in the cross-cut were significantly lower (one order of magnitude!). The most important difference, however, is the rate of residual deformation in the actual tunnel. In the Arlberg Tunnel, the rate of residual deformation was on the order of a few millimeters per month after invert closure.

2.4 Invert Heave

Invert heave has been observed at the Sendlingertorplatz Tunnel of the Munich subway. In this section (Figure A-9.2), the first two sidedrifts had been driven and supported by shotcrete and steelsets. During excavation of the central part and after placement of the crown support, fractures were observed where the support of the sidedrifts joined the crown (Fig. A-9.2). This is believed to have occurred due to invert heave near the face.

At the Tauern Tunnel, invert heave has not been completely verified. The picture shown in the book "Tauernautobahn" may be an optical illusion because some material was dumped in the area. The picture seems to indicate an upward bending of the ground. The impression of the picture has not been confirmed

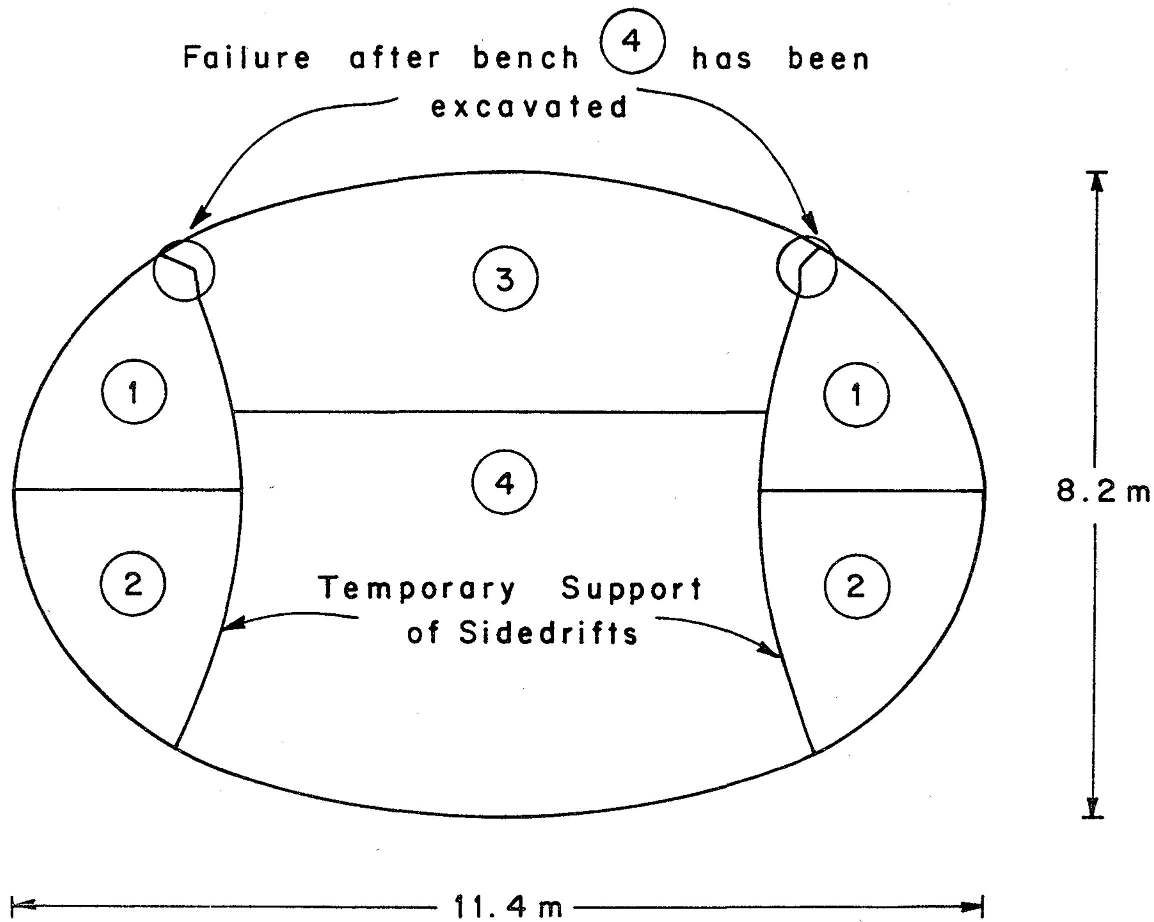


FIGURE A-9.2 INVERT HEAVE IN TUNNEL OF MUNICH SUBWAY OBSERVED BY FAILURE OF SUPPORT OF THE SIDE DRIFTS (PACHER 1978, PERSONAL COMMUNICATION) by measurements, simply because none were taken

3) Data

3.1 Werfen Tunnels

The Werfen Tunnels are three pairs of tunnels on the Tauern highway south of Salzburg (Figure A-9.3). The lengths of the tunnels are listed on Table A-9.1. The encountered geologic conditions are summarized in Table A-9.2. The support measures are shown in Table A.9.3.

The three tunnels encountered favorable ground conditions, as only ground classes I and an intermediate class between III and IV were found.

The observed convergences are small; the maximum measured is only on the order of 20 mm. Incidentally, this maximum convergence was observed in a zone classified as ground class I. In this area, widely spaced large discontinuities of 5 to 10 cm width, filled with sandy dolomite gouge (Dolomite grus), were encountered. These discontinuities were found to be associated with the Salzach Fault. The Salzach Fault is parallel to the Salzach valley through which this highway runs and is essentially parallel to the tunnel; however, the exact location is not known at the present time.

In the areas where no discontinuities were encountered, the convergence was within the measurement accuracy of the instruments (± 1 mm). Since no large convergences were observed, the

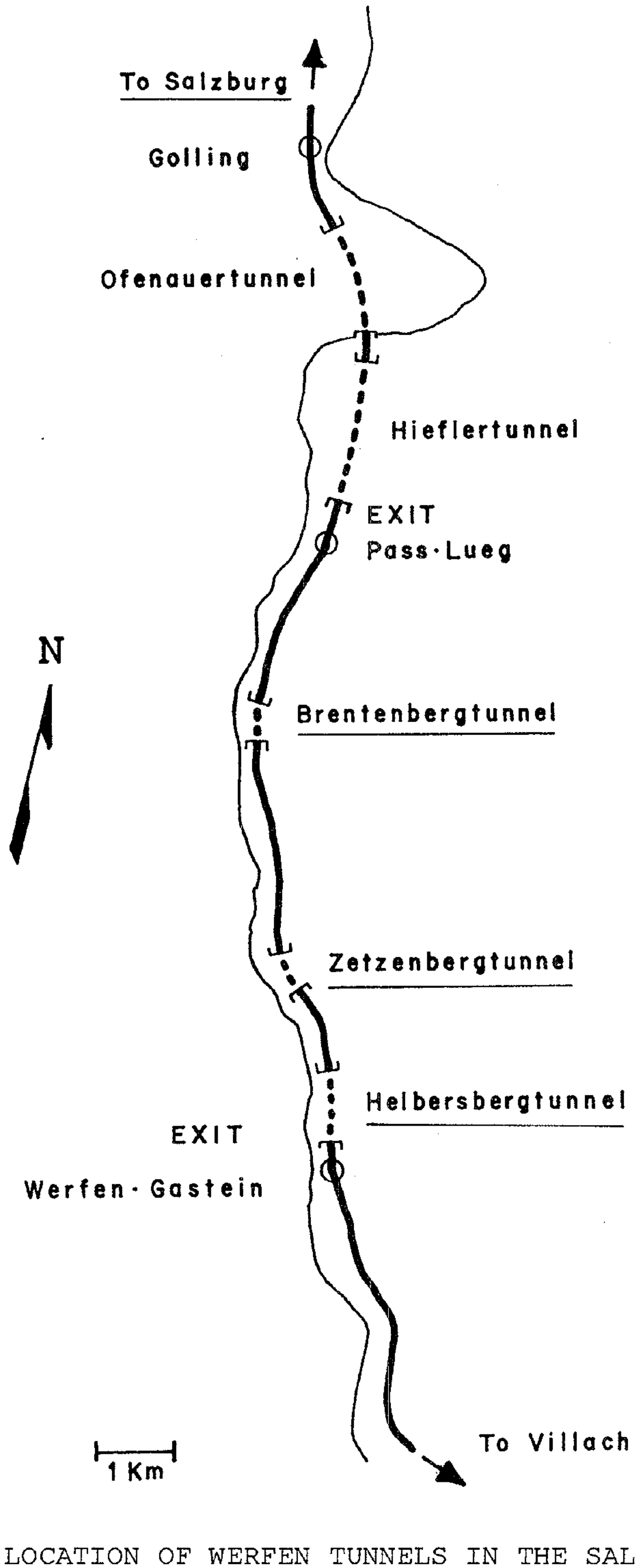


FIGURE A-9.3 LOCATION OF WERFEN TUNNELS IN THE SALZACH VALLEY (AFTER PACHER, 1975)

TABLE A-9.1 DATA OF WERFEN TUNNELS

Tunnel		Length (m)
Brentenberg	East	615
	West	510
Zetzenberg	East	540
	West	568
Helbersberg	East	836
	West	803

TABLE A-9.2 SUMMARIZED DATA FROM WERFEN TUNNELS

CASE	Stress Condition (Overburden)	Rock Description	Measured Convergence	Deduced Properties of Rock Mass
GROUND CLASS I				
Werfen Brentenberg	18 - 55 m	Dolomite, light gray. Singular large-scale discontinuities filled with dolomite grus. These discontinuities are parallel to the Salzach Fault (which runs parallel to the valley and highway, Fig.A-9.3, No other details known). These discontinuities are 5 to 10 cms wide and filled with dolomite grus. These discontinuities are spaced 5 to 10 m. Strike rel. tunnel 60-80° Dip = 70-80°	Without the widely-spaced discontinuities $\Delta H = \pm 2$ mm In case of the large discontinuities $\Delta H = + 20$ mm (convergence)!	One set of discontinuities spaced 5 to 10 m, diameter/block size = 1 to 2
GROUND CLASS II				
Werfen Brentenberg	100 m	Triassic Dolomite, fine grained, light gray (Guttensteiner Dolomite) Set 1: Str. rel. tunnel 80-120° Dip = 80° Spaced = 1 to 2 m Set 2: Strike = 45° Dip = 60° Set 3: Varies Overbreak likely where several joint sets intersect	3.5 mm to 15 mm	
Zetzenberg	20 - 100 m	Light gray, lightly stratified, Triassic Dolomite (Guttensteiner Dolomit) Discontinuities Set 1: Syr. rel. T = 45° Dip = 60-80° Set 2: Str. rel. T = 0° Dip = 40-60° orientation of bedding planes Set 3: Str. = 90° Dip = 60-90° Discontinuities spaced 0.5 m in some areas, not persistent. Orientation of discontinuities varies for Set 2. Afterbreaks where jointsets intersect.	spaced 1 to 2 m	Major blocks ~1 to 2 m D/B = 5 to 10 Persistence of rock not given, however, it exists or in echelon joints
GROUND CLASS III to an intermediate class III/IV				
Werfen Helberberg	0 to 40 m to 70 m	Colored quartzitic sandy to clayey schists (Werfener Schichten) Set 1: Str.rel. axis = 90° Dip = 50-80S = schistosity thickness of stratification = 3 to 10 cm Set 2: Strk.rel. axis = 30° Dip = 60-90° NE Spaced < 0.5 m One face collapse occurred (volume = 15 - 20 m^3) where Set 1 is dipping to the face. In one area a channel was crossed with only 9.5 m overburden, in this area more weathering and consequently more afterbreaks occurred.	0.3 to 5 mm Possible interpretation of small convergence monitored with orientation of discontinuities and thus convergence occurs immediately after each round	$(D/B)_1 = 100$ to 200 Persistence = unknown $(D/B)_2 = 20$

TABLE A-9.3 GROUND CLASSES FOR WERFEN TUNNELS

Support \ Ground Class	I	II	III	IV	V
Rock Bolts	in crown only	in crown only	crown and walls		
type	expansion	fully grouted	fully grouted	fully grouted	fully grouted
capacity (t)	15	18	18	18	18
prestress (t)	10	10	10	10	10
length (m)	3.0	3.0	4.0	4.0	3.0 | 5.0
quantity per meter of tunnel	3	2.4	4.8	.3	6.3 | 6.3
Total Length per meter of tunnel	9.0	7.2	19.2	25.2	19.5 | 32.5 52.0
Shotcrete		crown | walls			
thickness (cm)		7.5 | 5.0	10	15	20
$\beta_{28} = 280\ kg/cm^2$ area covered with shotcrete per lineal meter of tunnel		11.0 | 10.4	21.4	21.4 face support d = 3cm area = 20 m^2/m	21.9 face support d = 3cm area = 30 m^2/m
Wirefabric					
unit weight area in^2	3.12	3.12	3.12	3.12	3.12
total weight/ lin.meter tunnel	10.0	35.0	63.0	66.8	66.8
Steelsets					
type			TH 18	TH 18	TH 18
weight kg/cm	no	no	18	18	18
spacing (m)			only where necessary	1 5	1.0 m
AREA OF EXCAVATION	$67.5m^2$	$68.0m^2$	$68.8m^2$	$78.0m^2$	78.5m

measurements program was subsequently reduced.

At least twice, face collapses occurred; they are associated with the bedding plane (set 1) of the Werfen schists, which dip at angles of 60° toward the face. Also, overbreaks were observed in areas where two or more joint sets intersect. Overbreaks were more frequent in more weathered rock. Some of these overbreaks occurred after time had elapsed after each round and seem to have been associated with water inflow. In the report on the Helbersberg tunnel, it is stated that: "Small water inflows caused overbreaks which did not announce themselves. Little water inflow which rapidly dried up was sufficient to trigger afterbreaks of several m^3 in volume. This water caused a worsening of the ground classes by one or two classes. When dry, the rock was strong and brittle; however, in the presence of water, the ground behaved as strongly ravelling (stark gebräch) at the circumference and lightly squeezing (leicht druckhaft) at the face." This statement clearly demonstrates the importance of the cleft water pressure.

At the Zetzenberg Tunnel, the observation was made that: "...overbreaks occurred mainly in the crown and and at the face, sometimes also at the springlines." To prevent overbreaks at the springlines, the discontinuities had to be carefully observed, because once they intersect the sidewalls they lead to an undercutting of the sidewalls with subsequent sliding-out of rock wedges (Figure A-9.4).

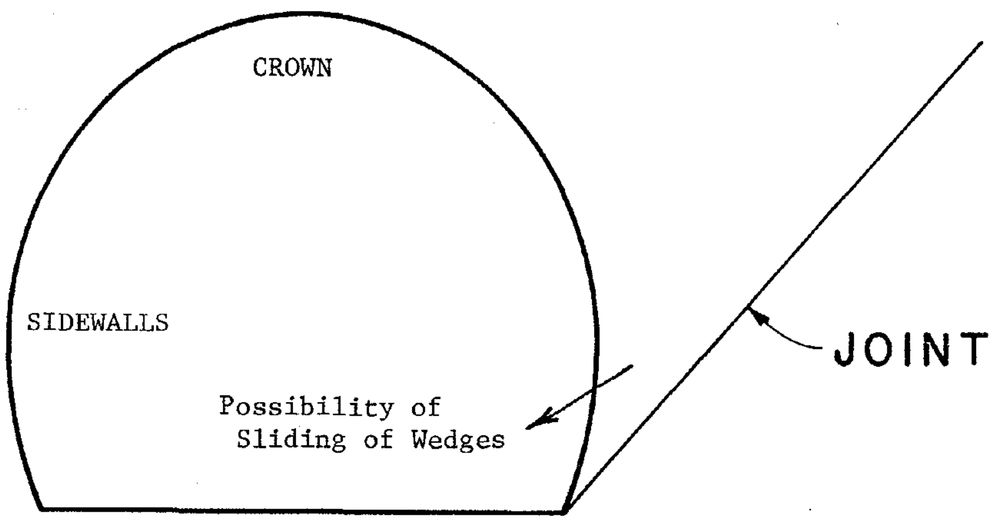


FIGURE A-9.4 UNDERCUTTING OF TUNNEL WALLS BY MAJOR DISCONTINUITIES

3.2 Data from the Klamm Tunnels

The reconstruction of the access road to the resort Bad astein involves four tunnels whose principal dimensions are listed in Table A-9.4. Three two-lane tunnels were built and one double-deck four-lane tunnel (Gigerach), Figure A-9.5. The Gigerach Tunnel is in the vicinity of a junction, thus necessating a two-level construction. The two-lane Klamm Tunnel is 1584 m long, and it first crosses steeply dipping limestone and phyllites in the north for 9/10 of its length. At the southern end, quarternary talus debris had to be crossed, extending over a distance of 140 m. The maximum overburden in the rock section is 510 m. The distribution of ground classes as they were predicted and encountered in the tunnel is given in Table A-9.5. Popping rock was encountered under the highest elevation, but could be controlled with shotcrete.

The support was a combination of shotcrete and rock bolts. In all ground classes, 7 cm shotcrete was placed in the crown after each round. In Ground Class I, this shotcrete was supplemented by occasional bolts that were placed a few meters behind the face. In Ground Class II, rock bolts were placed immediately after each round had been excavated. In Ground Class III, the support was shotcrete (10 cm) in the crown and at the sidewalls, with wire fabric steelsets and pattern bolting.

TABLE A-9.4 TUNNELS FROM LEND TO GASTEIN
(AFTER PACHER, 1977)

TUNNEL	BUILT	LENGTH	CROSS-SECTION excavated (m^2)	HEIGHT (m)	WIDTH (m)	COSTS Austrian Schillings
Mauth	1970/1972	208 m	80 m^2	6.58	9.78	32 million
Gigerach dual level (Fig. A-9.6)	1974/1976	180 m	type I 135 m^2 type II 175 m^2	15.07	12.5 m	105 million
Klamm	1971/1974	1600 m	80 m^2	7.00	10.65	290 million
Klammstein	1958/1959	103 m	75 m^2	6.65	10.0	N.A.

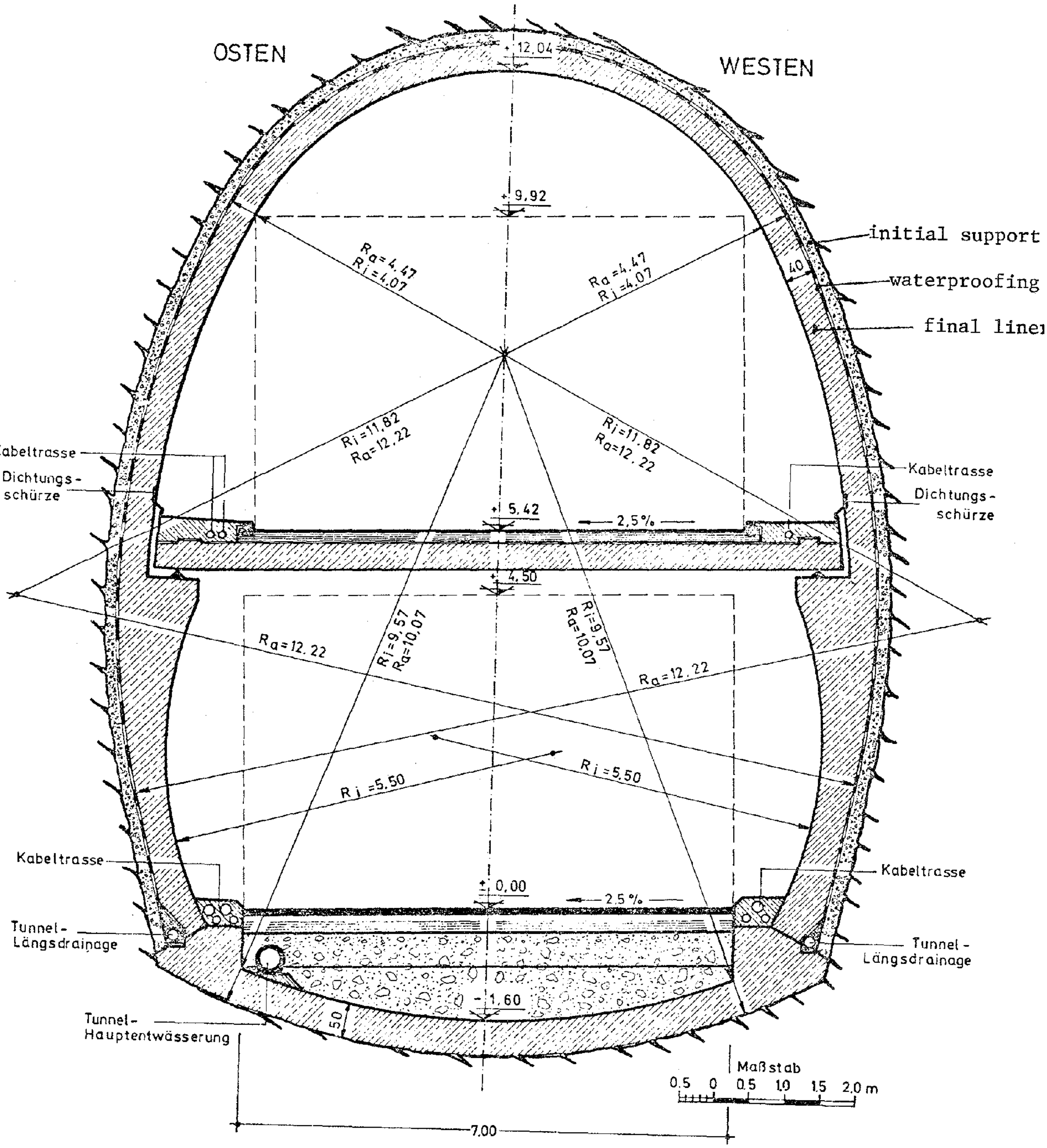


FIGURE A-9.5 CROSS-SECTION OF GIGERACH-TUNNEL (FROM PACHER, 1977)

TABLE A-9.5 ROCK CLASSES ENCOUNTERED IN THE KLAMM TUNNEL (FROM BRAUDECKER AND VOGELTANZ, 1975)

GROUND CLASS	PREDICTED (%)	ENCOUNTERED (%)	ROCK TYPES
I and II	74.5	79.1	Klamm-Limestone Klamm-Phyllites
III	11.8	11.0	Klamm-Phyllites Chloritic, Sericitic, Quartzitic Phyllites
IV V	3.2 10.5	2.4 7.5	Alluvium

In the Klamm-Limestone, three joint sets were observed (Brandecker and Vogeltanz, 1975). Joint set 1 strikes parallel to the axis of the tunnel and dips 75 to 90°. The trace length of these joints rarely exceeds 1m; perpendicular spacing is approximately 5 to 10 cm and persistence is approximately 50%. These joints were closed and did not influence the stability of the tunnel significantly. In some places, the springlines had to be supported with bolts.

Joint set 2 strikes perpendicular to the tunnel axis and dips 60 to 90° towards the face. Trace length is 0.5 to 3 m, perpendicular spacing is 10 to 50 cm, and persistence is 60 to 70%. Where this joint set was closely spaced (i.e., near the lower bound of 10 cm), the ground had to be rapidly supported with reinforced shotcrete.

Joint set 3 strikes diagonal to the axis and dips 50 to 90° to the face. The trace lengths of these joints are greater than 3 m; they are generally closed, but may be open for 5 to 10 cm, and in some instances they widen to small karstic tunnels. In some places, the joints are filled with clay or treated with calcite. Joint set 3 is parallel to a fault. Also, 3 was formed last and offset the first two sets by 10 cm.

From the construction point of view, set 3 did not cause major problems. For a detailed description of the geologic problems with the construction of the Klamm Tunnel, the reader is referred to Brandecker and Vogeltanz (1975).

APPENDIX:	A-10
NAME:	Tauern Autobahn AG,
	Tauern Highway Authority
ADDRESS:	Alpenstrasse 94
	A-5020, Salzburg,
	Austria
DATE OF MEETING:	18th January, 1978
PERSONS MET:	Mr. Köllensperger
	General Manager

1) Introduction

The Tauern Autobahn-AG (Tauern Highway Authority) owns and builds the central part of the highway from Salzburg to Villach (Figure A-10.1). The part crossing the Alps from Eben to Pongau to Rennweg has been opened to traffic on June 21st, 1975. The Tauern Tunnel (L = 6.4 km) and the Katschberg Tunnel (L = 5.4 km) form the key parts of this highway. Notably, the Tauern Tunnel posed severe problems during construction. The tunnels were excavated according to the NATM.

2) Information on Construction and Performance of the Tauern Tunnel

During the construction of the Tauern Tunnel, numerous instruments have been placed and many measurements have been made. A summary is given in Table A-10.1 and Figure A-10.2. Figure A-10.2 also summarizes information on geology and

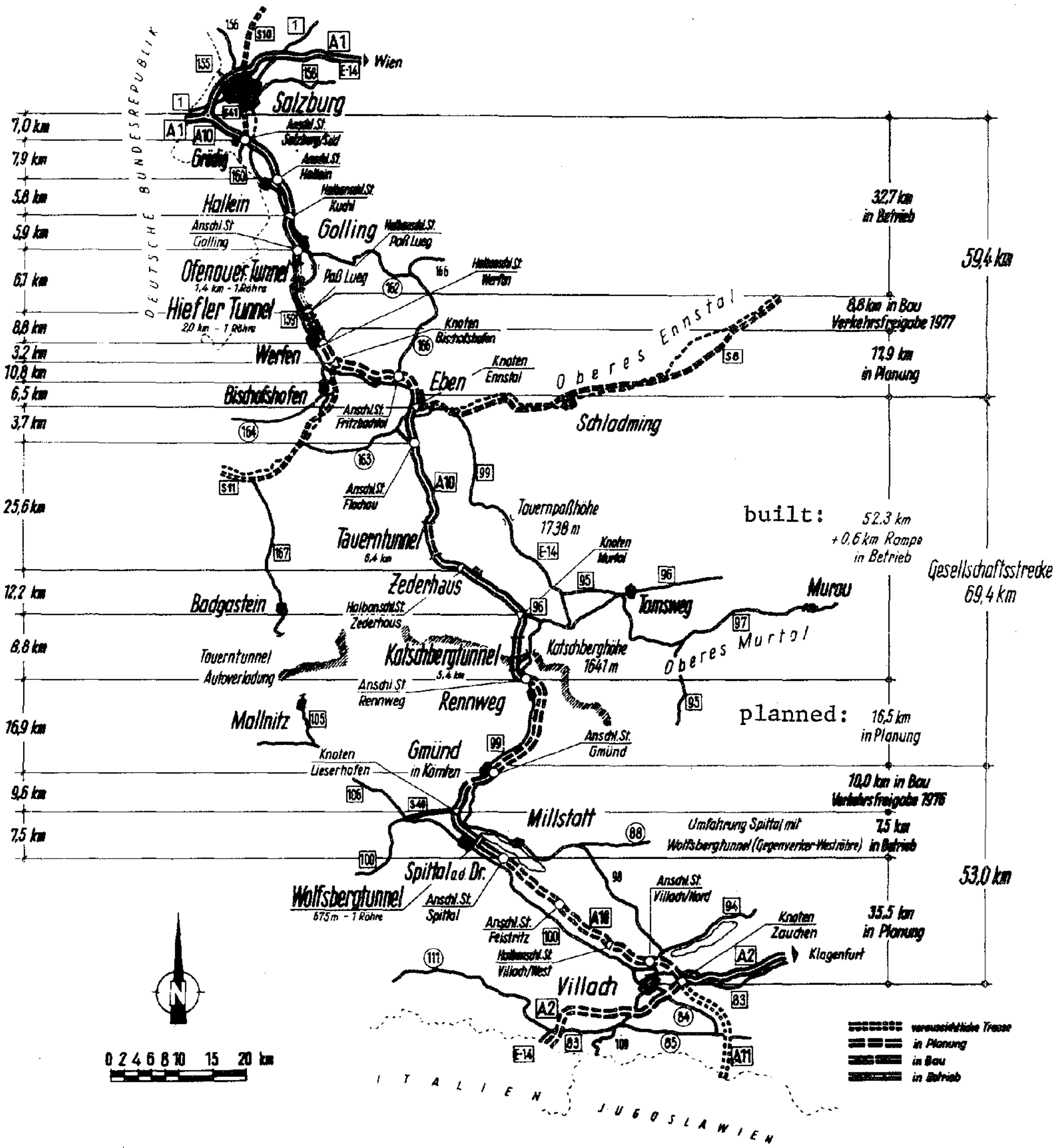


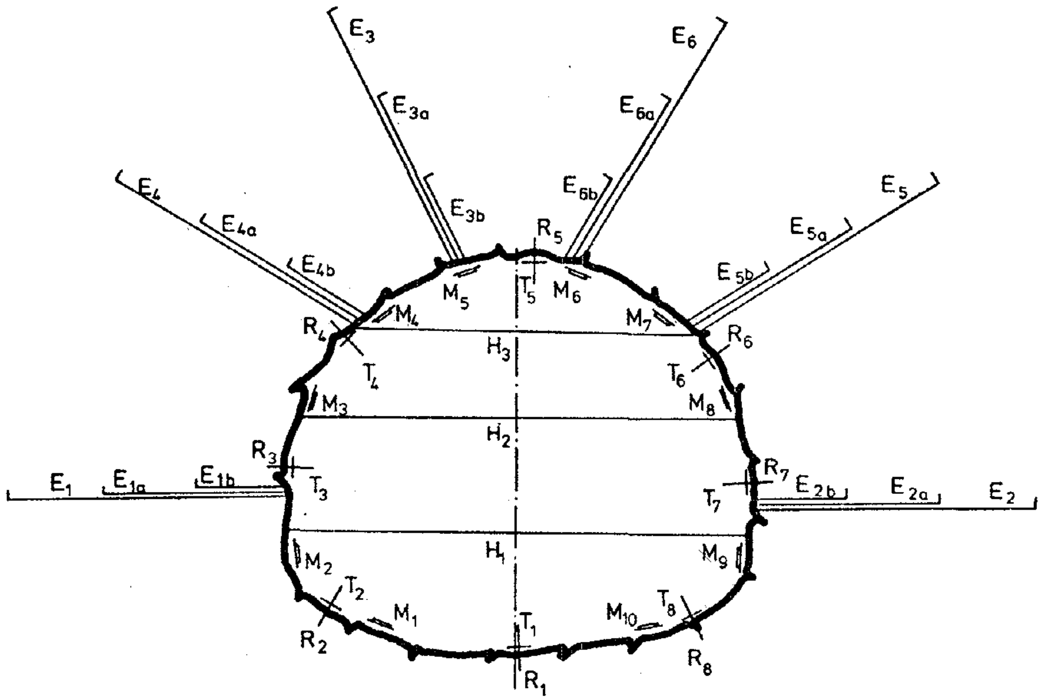
FIGURE A-10.1 MAP OF THE TAUERN HIGHWAY (FROM TAUERNAUTOBAHN, 1975)

TABLE A-10.1 SUMMARY OF MEASUREMENTS TAKEN FOR THE TUNNELS OF THE TAUERN-HIGHWAY

Instruments	Number of Instruments Placed	Number of Readings			
		TT	LS	KT	Total
Extensometers	350	4,240	850	822	5,912
Convergence measurements	608	8,950	85	640	9,675
Stress measurements	587	16,500	510	2,235	19,245
Anchor force measurement plates	30	526	16	-	542
Fissure meters	6	130	-	36	166
Thermometers	28	52	400	-	452
Total	1,609	30,398	1,861	3,733	35,992

(from Tauernautobahn, Vol. 1, p. 117)

TT = Tauern-tunnel
LS = ventilation shaft of Tauern-tunnel
KT = Katschberg-tunnel



H_1 = Horizontal Convergence Monitoring
E = Extensometers
R = Radial Stress Cell Between Shotcrete and Ground
T = Tangential Stress Cell in Shotcrete

FIGURE A-10.2a MONITORING CROSS-SECTION IN TAUERN TUNNEL (FROM TAUERNAUTOBAHN, 1975)

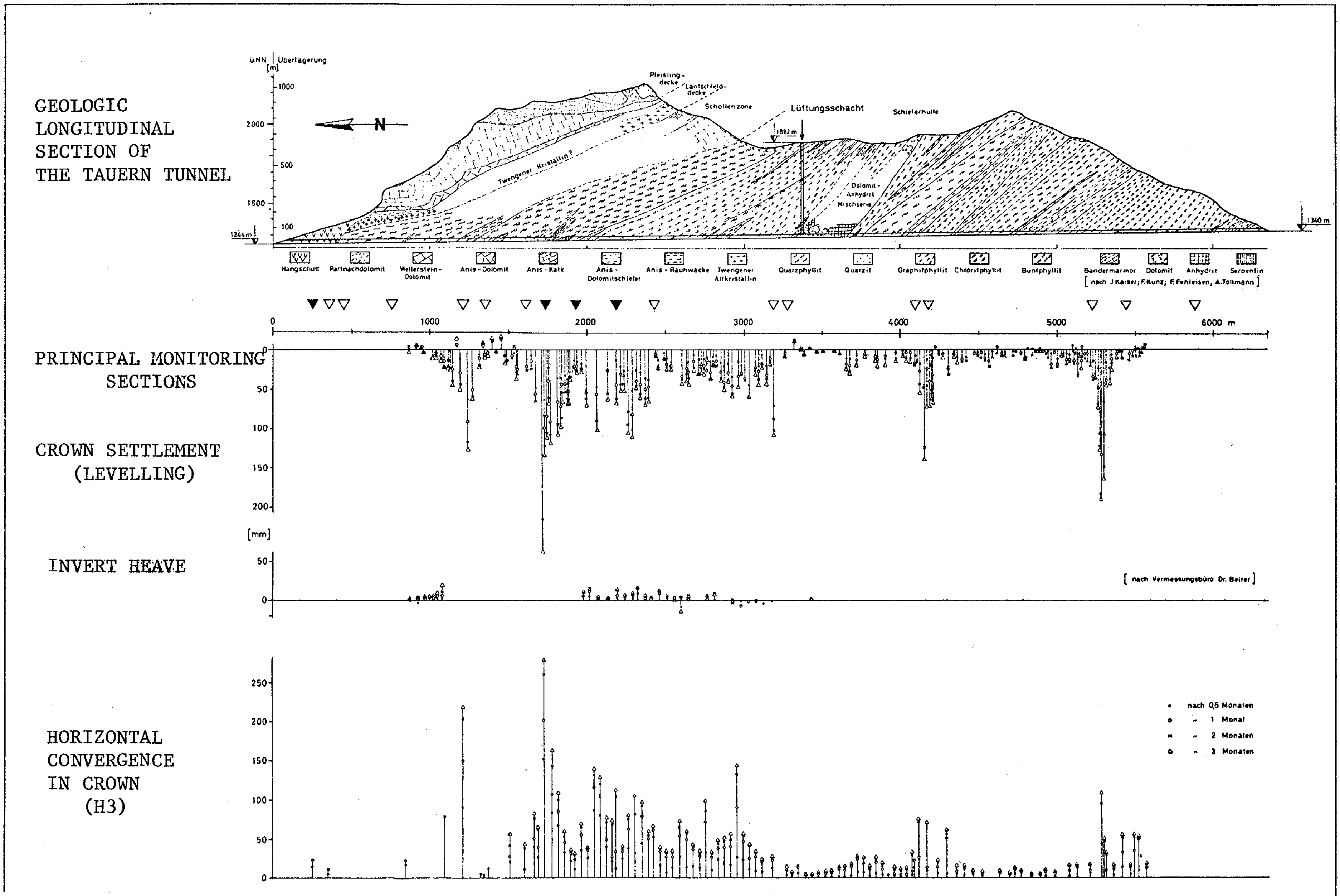


FIGURE A-10.2b TAUERN TUNNEL - SUMMARY OF CONSTRUCTION AND MONITORING PERFORMANCE

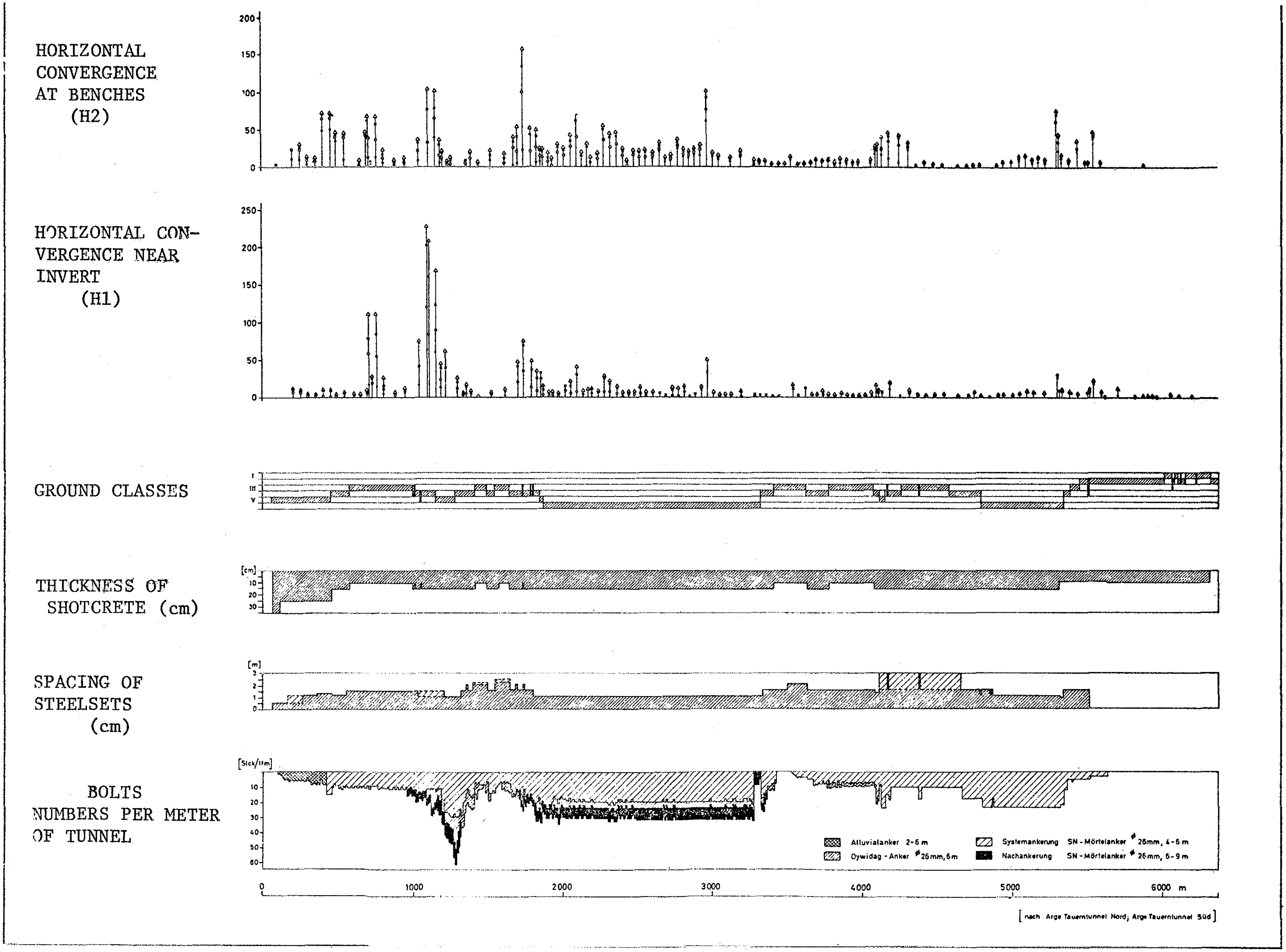


FIGURE A-10.2c TAUERN - SUMMARY OF CONSTRUCTION AND MONITORING PERFORMANCE construction details. In addition, detailed data sheets as shown in Figures A-10.3 and A-10.4 have been purchased from Tauern Autobahn-AG. Figure A-10.3 is an example of a data record sheet, combining a complete geologic map of the tunnel with details on water inflow and structural features, the ground classes, the support placed and grout injected, as well as monitoring measurement data. Figure A-10.4 shows the kind of information and detailed results collected at principal sections. The locations of these monitoring sections are shown in Figure A-10.2. It shall be recalled that in NATM tunneling, and thus in the Tauern, normal monitoring sections, where convergence and settlement measurements are made, are placed every 10 to 50 m depending on ground conditions (see Figure A-10.2). The principal monitoring sections provide more detailed information (stresses between support and rock, stresses in support, displacements in the ground mass, convergence and settlement) and are located to represent typical or particularly problematic ground conditions. This detailed information on ground conditions, support placed and performance was used by us to improve the knowledge on ground-structure interaction (Volume 2 of this report) and particularly for the work on empirical methods (Volume 5 of this report).

3) Other Aspects

During the discussion, Mr. Köllensperger mentioned the cross-cut at Station 1848 of the Tauern Tunnel. This cross-

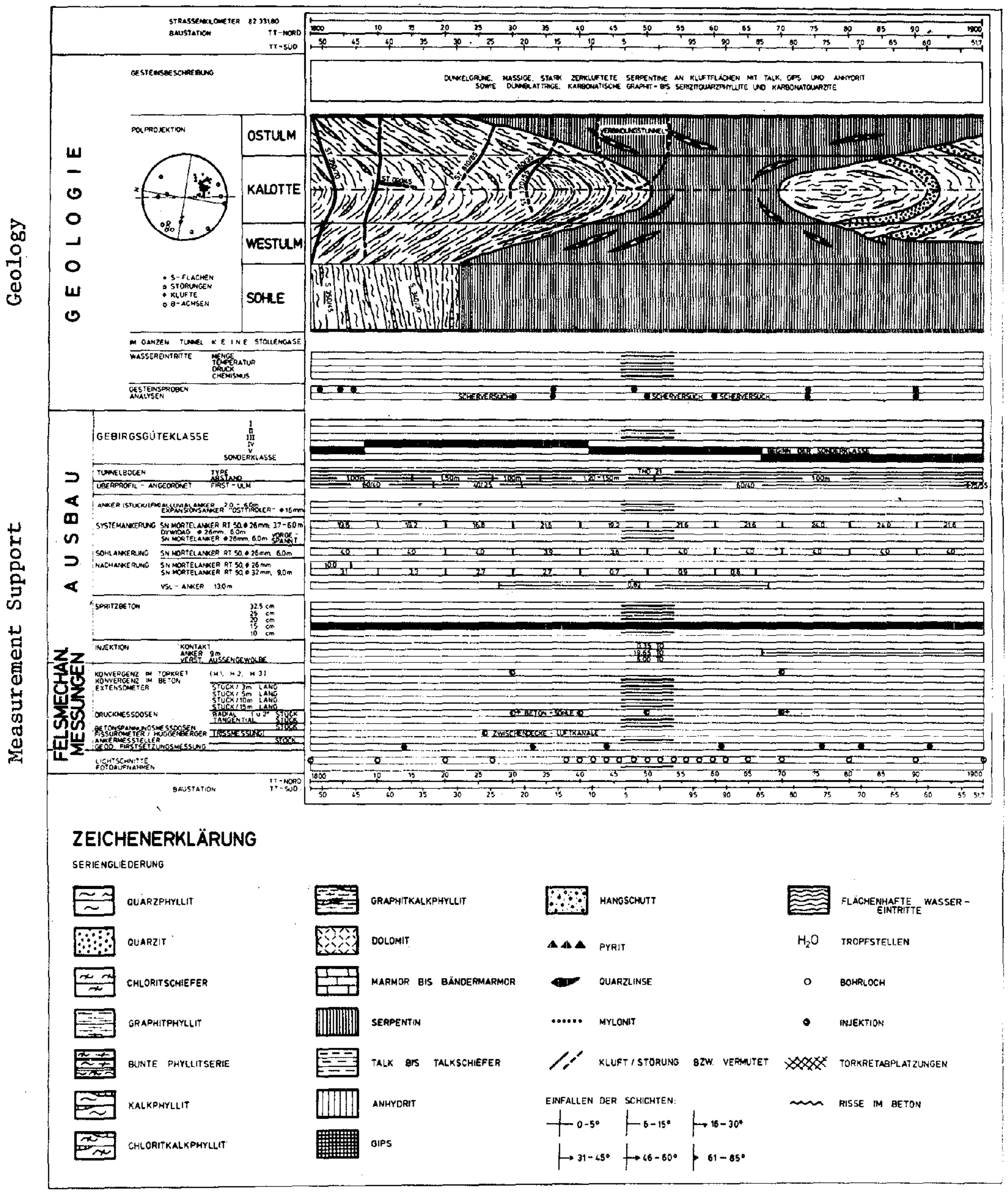


FIGURE A-10.3 DATA FORM THE TAUERN-TUNNEL
(STATION KM 1,800 TO 1,900)

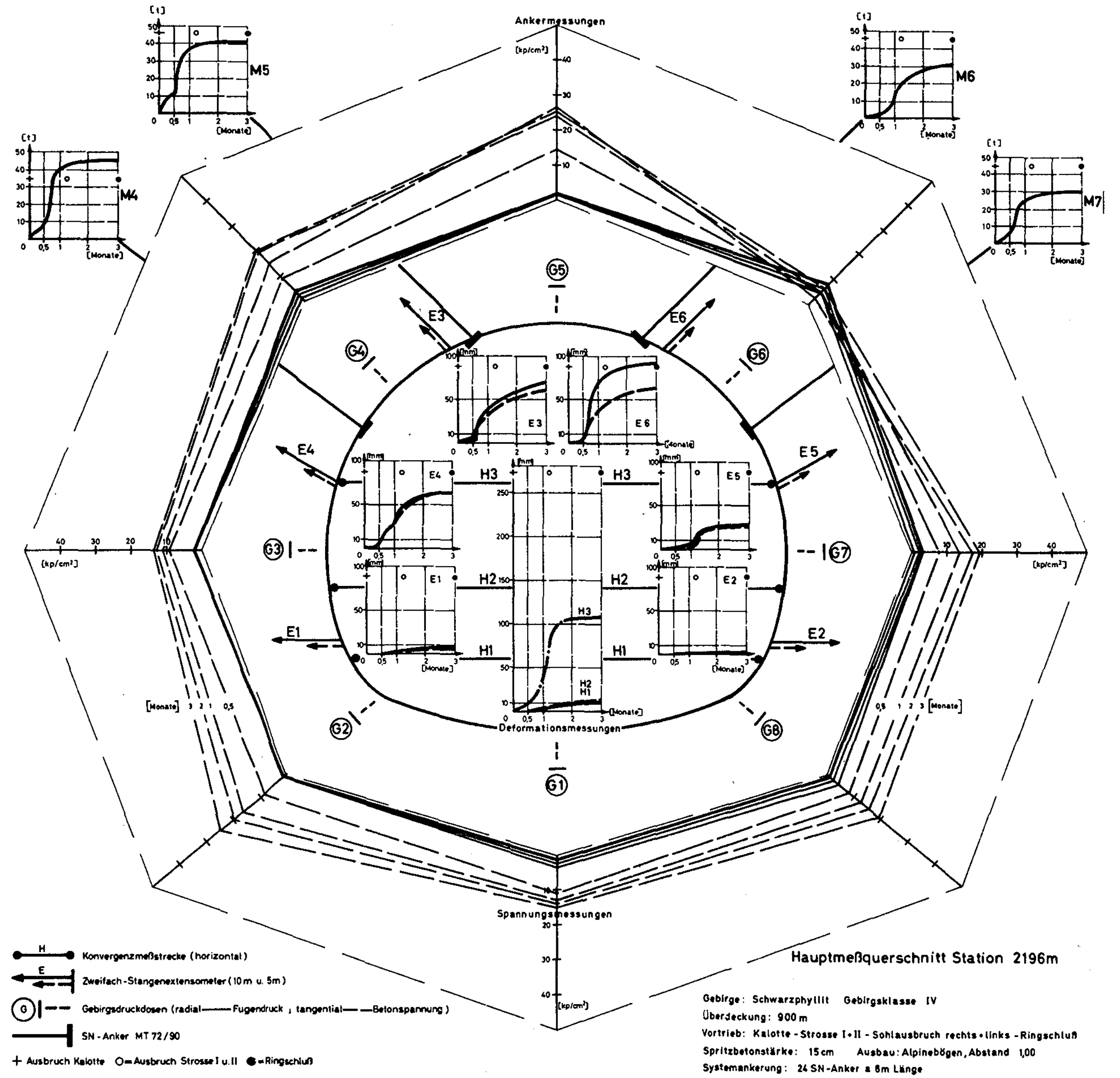


FIGURE A-10.4 SUMAMRY OF MEASUREMENT DATA FOR PRINCIPAL MONITORING CROSS-SECTION AT 2,196 IN. (FROM TAUERNAUTOBAHN, 1975) cut experienced very large deformations over a long period of time. Finally as the movements did not stabilize, it was decided to reduce the cross-section and to place a thick concrete lining (Figure A-10.5). The geologic section (Figure A-10.3) shows that a major shear zone filled with talc interested this cross-cut. (Strike parallel to cross-cut and dipping approximately 45°). The large deformations were mainly attributed to this discontinuity. According to Mr. Köllensperger, this could have been avoided by more carefully observing the geology in the main tunnel and by shifting the cross-cut by 50 to 100 m into a zone with more favorable conditions. This example clearly illustrates the importance of the conditions and orientation of shear zones relative to the opening.

Cross-over Tunnel at 1,848 m

Cross-section of Tunnel
with Breakdown Lane

Verbindungstunnel bei Station 1848,00

Regelprofil **Profil im Bereich der Trafonische** **Abstellnische bzw. Umkehrnische**

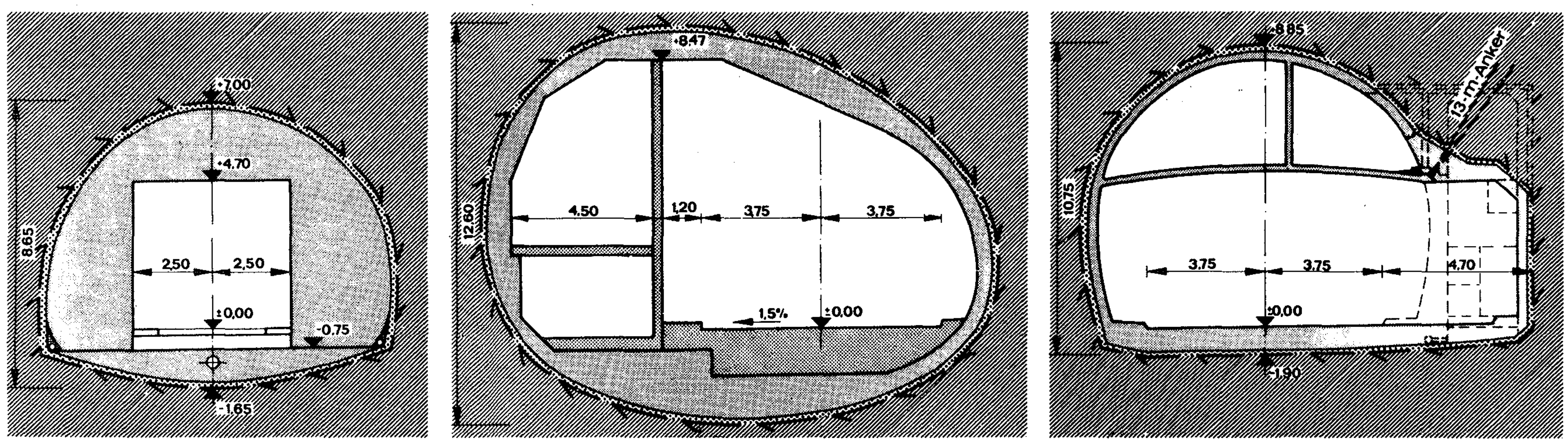


FIGURE A-10.5 CROSS-SECTION OF THE CROSS-OVER TUNNEL AT STATION 1848 OF THE TAUERN TUNNEL (FROM TAUERNAUTOBAHN, 1975)

APPENDIX: A-11

NAME: Studiengesellschaft fur unteridische Verkehrsanlagen (STUVA)
Research Institute for Underground Transportation Facilities

ADDRESS: Matthias-Bruggen-Strasse 41
D-5, Köln 30
Germany, (West)

DATE OF MEETING: 19th January, 1978

PERSONS MET: Dr. A. Haack
Dr. N. Klawa

1) Introduction

STUVA is a private research organization dealing with problems of underground construction. It was founded in the 1950's by the owner of a supermarket chain in the Ruhr district. During the fifties, the Ruhr district, an area of coal mining, went through a depression and this research institute's goals were to provide ways to construct underground traffic facilities in order to provide work for the unemployed miners.

The institute was previously located in Dusseldorf, some 50 km to the north, and also maintained a subsidiary in Hamburg. Last year, STUVA moved to its new quarters in the outskirts of Cologne. The facilities include a laboratory where large tests can be run. One of the research programs of STUVA deals with water proofing. The results have been published in several volumes of "Forschung und Praxis", the series published by ALBA-Verlag, Dusseldorf, on behalf of STUVA. At present, model tests on the ventilation requirements for partial face TBM are being conducted.

Besides these experimental studies, studies on the costs of tunnels have been performed, which will be described below.

2) Cost Studies Performed by STUVA

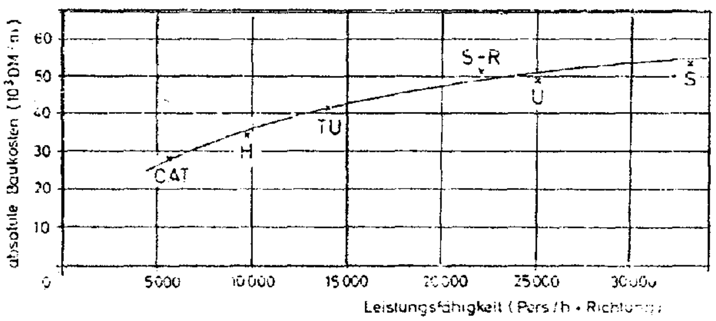
Two studies on the costs of tunnel construction have been performed by STUVA. One, based on an actual built subway in Hamburg, considers the cost and other characteristics of various types of rapid transit. A second study deals with the influence of the most important parameters on tunnel construction costs of subways in various subsoil conditions.

The first study has been published as Volume 16 "Baukosten von Verkehrstunneln" (Construction Costs of Traffic Tunnels) of "Forschung und Praxis" (Research and Practice) by ALBA-Verlag. It deals with construction costs and the capacity of various systems, notably PRT (Personal Rapid Transit), LRT (Light Rapit Transit), subway (U-Bahn in German terminology), and express subways (S-Bahn in German terminology). The data are actual subway construction costs incurred in Hamburg; a total of 25 route kilometers have been considered. Figure

A-11.1 shows average total construction costs per route meter as a function of the capacity of the systems. Figure A-11.2 shows the costs per route meter and person moved per hour in one direction for different subway transit systems. These are average costs and include tunnels and surface line sections and stations.

The second study compared construction costs for different design construction methods (Table A-11.1); different ground conditions and different section lengths per contract are considered. Two types of ground conditions are sandy-silty clay (similar to the ground conditions in Hamburg) and medium stiff clay (similar to the ground conditions in Frankfurt). We have obtained a preprint of this comprehensive STUVA report. Some of the results are preseneted in Section 3 of this report. The STUVA report on tunnel construction costs provides much valuable information, and a complete translation of this report, once finalized, is the best way of transmitting this information to U.S. practice.

Cost per Route Meter
(thousand DM)

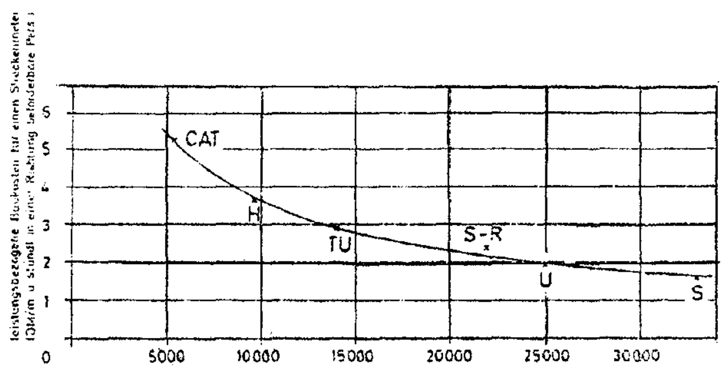


Transport Capacity (Persons per hour and direction)

FIGURE A-11.1 TOTAL CONSTRUCTION COSTS PER ROUTE METER AS A FUNCTION OF CAPACITY (FROM GIRNAU, 1975)

CAT= Personal Rapid Transit
H = H-Bahn= People-Mover
TU = Transurban = People-Mover
S-R= Stadtbahn Rhein-Ruhr = Light Rail Transit (LRT)
U = U-Bahn = Subway
S = S-Bahn = Express-Subway

Cost per Meter per Person
moved per Hour



Transport Capacity
(Persons per hour and direction)

FIGURE A-11.2 NORMALIZED CONSTRUCTION COST PER ROUTE METER AND PERSON MOVED PER HOUR IN ONE DIRECTION (FROM GIRNAU, 1975)

TABLE A-11.1 METHODS OF TUNNEL CONSTRUCTION STUDIED

OPEN CUT	CAST-IN-PLACE CONCRETE
	PRECAST CONCRETE
	TREMIE CONCRETE (SOHLE-WAND)
MINED TUNNELS	SHIELD
	NATM (SHOTCRETE, STEELSETS, BOLTS)
	PIPE-JACKING (ELEMENTS, RECTANGULAR IN CROSS-SECTION)

APPENDIX:	A-12
NAME:	U-Bahn Bauamt der Stadt Köln, (Subway Department of the City of Cologne)
ADDRESS:	Schildergasse 32 D-5, Köln 1 Germany (West)
DATE OF MEETING:	19th January, 1978
PERSONS MET:	Mr. Behrendt, Head of Design Department and Chairman of Subcommittee on Tunnel Construction Cost of the German Federation of Cities

1) Introduction

Mr. Behrendt is chairman of the subcommittee on subway construction costs of the German Federation of Cities (Deutscher Stådtetag). This subcommittee wanted to compare subway construction costs in German cities. Comments on the study will be made below. In addition, it was possible to discuss the design and construction of two subway sections built in Cologne.

2) Construction Cost of Subways (in general)

The German Federation of Cities considers it impossible to compare subway construction costs directly. There are many variables for each individual case; the costs vary considerably even within each city. The Federation decided neither to publish these costs nor to furnish it to outside groups. The reasons for this decision could not be determined. The goal of the subcommittee may have been set too high because it attempted to define construction costs as one single parameter, i.e., cost per cubicmeter of excavation or per cubicmeter of concrete placed. It was decided to use a representative "unit price" that includes the quantities shown on Table A-12.1. But, no cost data will be published.

Mr. Behrendt quoted costs for subway tunnels built by the NATM (Table A-12.2). The cost differences stem from different cross-sections (running tunnels, stations), as illustrated by the cost spread for Munich, but also from different ground conditions. The ground conditions for the Bochum and Dortmund Tunnels are more favorable than those in Munich and Frankfurt, and groundwater control is less of a problem.

In addition to these data, Mr. Behrendt provided us with detailed brochures on two subway sections in Cologne, which are described below.

3) Subway Sections Built in Cologne

3.1 Introduction

The City of Cologne is constructing a light rapid transit system. Conversion of street car lines to subways proceeds gradually as separate sections are built which are then linked to existing tunnels. The two subway sections where detailed

TABLE A-12.1 WEIGHTING PROCEDURE FOR AVERAGE QUANTITIES FOR SUBWAY CONSTRUCTION COSTS

MATERIAL	QUANTITY
Concrete	1 m^3
Formwork (horizontal)	0.2 m^2
Formwork (walls)	1.0 m^2
Reinforcing steel	50 kg
Soil excavation	4 m^3
Transportation of excavated soil	4 m^3

TABLE A-12.2 CONSTRUCTION COSTS FOR SUBWAYS IN GERMANY BUILT WITH NATM

CITY	COST PER ROUTE METER OF TUNNEL	
	in German marks	U.S. dollars
Munich	19,000-50,000	9,500-25,000
Frankfort	23,000	11,500
Dortmund	12,000	6,000
Bochum	18,000	9,000

data are available are interesting due to the following reasons:

(i) both are built in open-cut

(ii) the groundwater conditions are different

3.2 <u>Section Ost 1, Betzdorfer Strasse</u>

This section has been built by a contractors joint-venture with the following partners: Wayss & Freytag, Subsidiary Köln, and Peter Bauwens. The technical data is presented in Table A-12.3. Figures A-12.1 and A-12.2 show a plan view, longitudinal and trasverse cross-sections (of the western half of the section) as well as the construction procedure. The tunnel will be temorarily linked to the surface network; it thus contains a running tunnel extending over the entire length of the section, and a ramp extending over half of the section (once the subway is extended, only the ramp has to be demolished, since the tunnel under the ramp already exists).

The ground consists of permeable gravel with the water table lying 6 m above the invert; dewatering would thus affect existing wells in the vicinity. To reduce such effects, the wall-invert method (Figure A-12.2) is employed. The lateral support consists of an internally braced diaphragm wall. All the groundwater table excavation proceeds with a hydraulic shovel excavator; below the groundwater table, a clamshell excavating underwater is used. Tremie concrete is then placed, the water in the excavation pumped out, the tunnel structure placed, and the space above the tunnel backfilled.

TABLE A-12.3 TECHNICAL DATA FOR SECTION OF OST 1, BETZDORFERSTRASSE (FROM BROCHURE)

Technical data	
Construction period:	November 1974 - June 1976
Costs (gross):	16.8 million DM
Length of tunnel:	560 m
Length of ramp;	170 m
Depth of excavation:	14 m
Diaphragm walls:	16,000 m^2
Excavation:	80,000 m^3
Concrete poured:	28,000 m^3
Steel (reinforcement):	2,000 metric tons

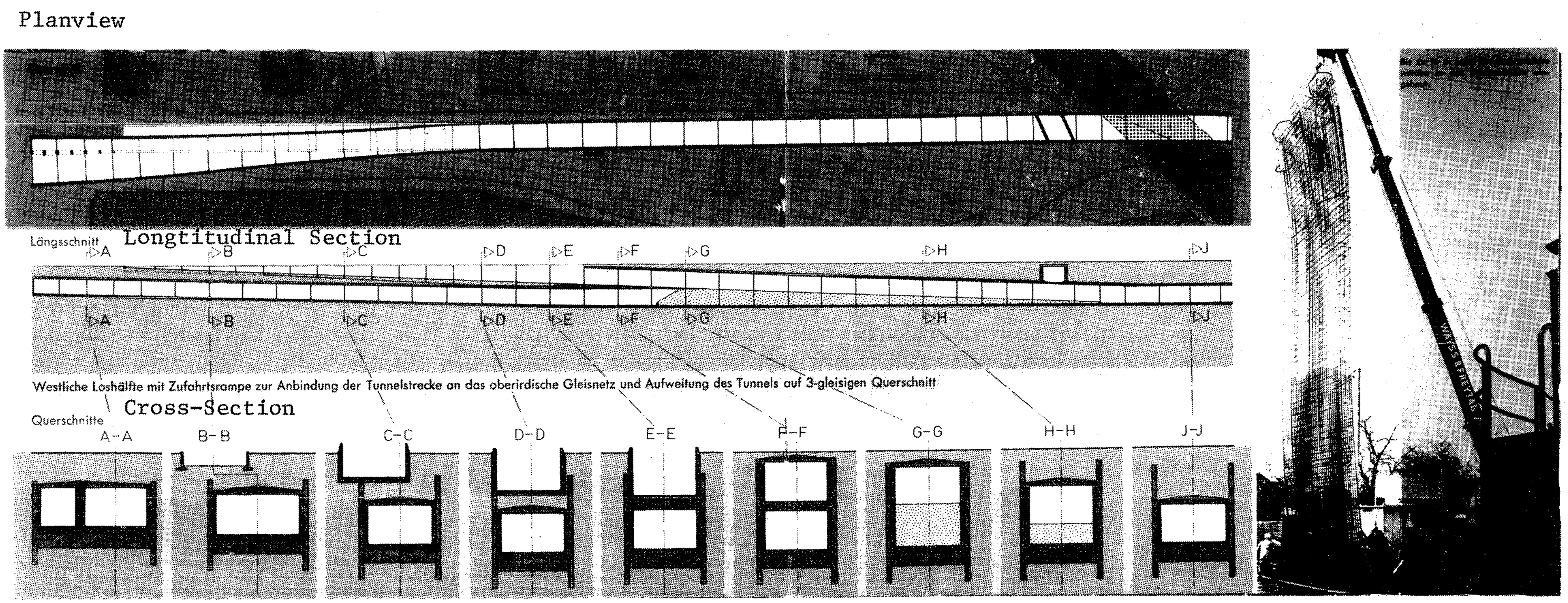
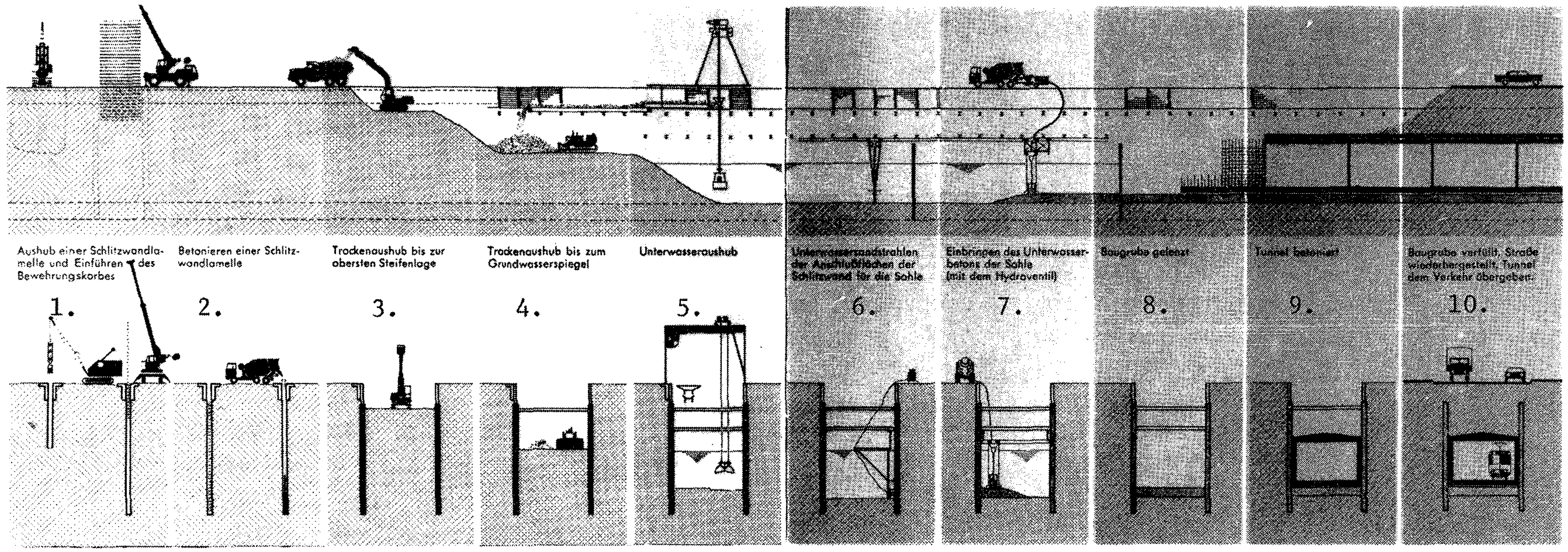


FIGURE A-12.1 SECTION OST 1, BETZDORFER STRASSE, KÖLN (FROM BROCHURE PUBLISHED BY JOINT-VENTURE WAYSS & FREYTAG, AND P. BAUWENS)



1. Excavation of slurry trench and placement of reinforcing cage.
2. Concreting of diaphragm wall.
3. Excavation in dry soil to first level of bracing.
4. Excavation in dry soil to groundwater level
5. Underwater excavation.
6. Underwater sandblasting of contact zones for invert slab.
7. Placement of invert slab with tremie method.
8. Dewatering of interior of excavation.
9. Tunnel constructed.
10. Site back-filled.

FIGURE A-12.2 CONSTRUCTION PROCEDURE WALL-INVERT METHOD BETZDORFER STRASSE, KOLN (FROM BROCHURE, CITY OF COLOGNE)

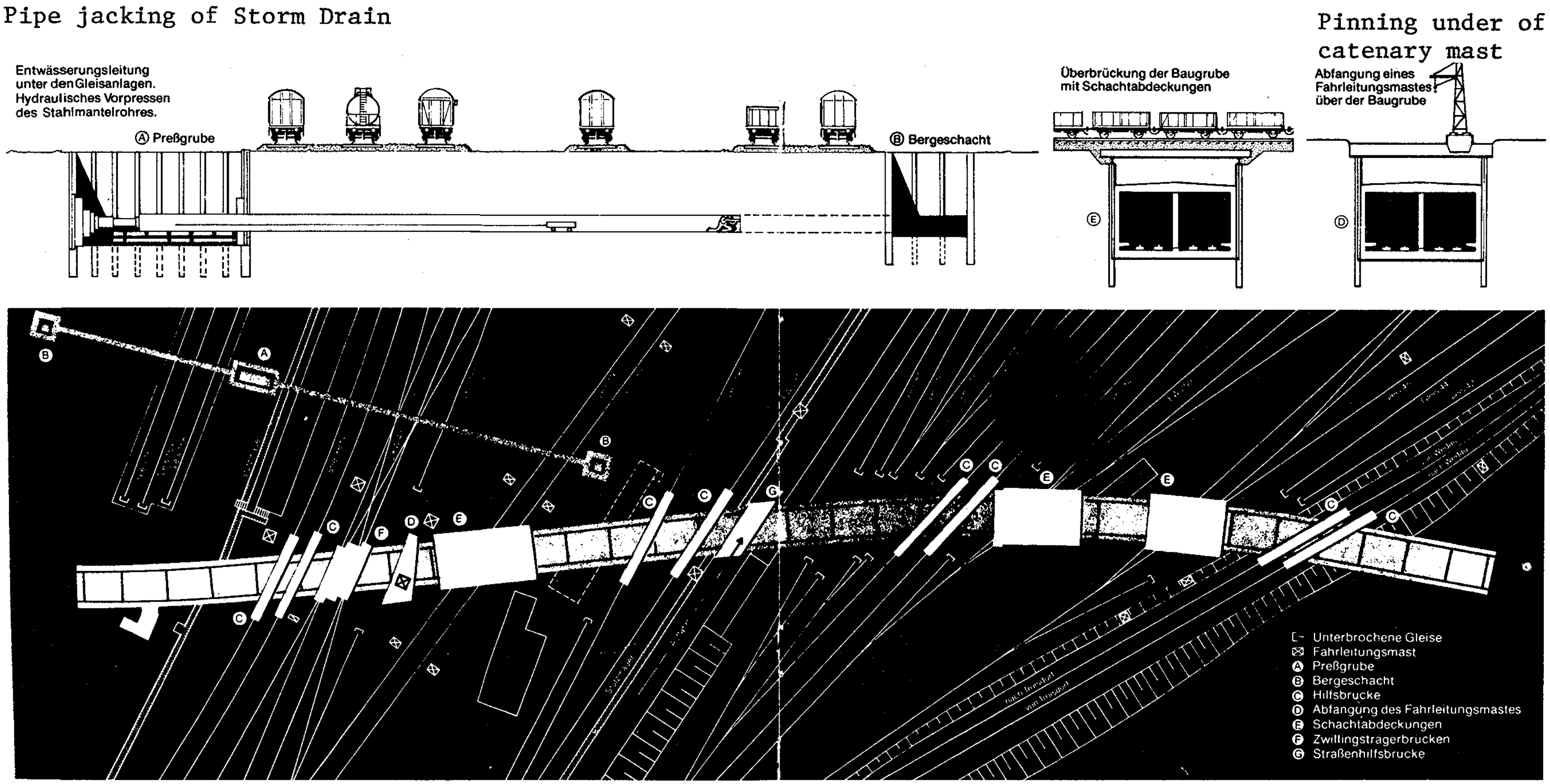
Construction cost per route meter (average for entire section) are 30,000 DM/m = 15,000 \$/m. The costs of the ramp which extends over half of the section are included, but no detailed data on the related cost increase are available. On the other hand, the ramp also reduced the backfill cost, and it may thus be expected that the ramp costs do not significantly influence the total average costs.

3.3 Section Ost 3, Kalk Nord

This section crosses a major railroad yard in Cologne, as illustrated in Figure A-12.3. Figure A-12.3 illustrates how the construction was influenced by the railroad tracks. Huta-Heberfeld, Cologne, was contractor for this section. Technical data and a cost breakdown are given in Table A-12.4.

The excavation is supported by an internally braced soldierpile wall with timber lagging. The tunnel is reinforced, impervious concrete. The groundwater table lies below the bottom of the excavation; thus, no dewatering was necessary. Crossing the railroad yard resulted in additional work in that a storm drain had to be placed as well as numerous temporary bridges.

The storm drain (length = 120 m, diameter = 1.2 m) was constructed by pipe-jacking (upper lefthand corner of Figure A-12.3) prior to construction of the tunnel. The temporary bridges and the interrupted tracks can also be seen in Figure A-12.3. Only nine of the twenty-seven tracks



A. Jacking shaft for storm drain
B. End shaft
C. Temporary bridges
D. Underpinning of catenary mast
E. Temporary slab and bridges
F. Twin-beam bridges
G. Temporary road bridge

FIGURE A-12.3 SECTION OST 3, KALK-NORD, OPEN-CUT TUNNEL UNDER RAILROAD YARD (FROM BROCHURE PUBLISHED BY HUTA-HEGERFELD)

TABLE A-12.4 TECHNICAL DATA FOR SECTION OST 3, KALK, NORD (FROM HUTA-HEGERFELD)

Technical data	
Construction costs:	10 million DM
Length of tunnel:	320 m
Excavation:	42,000 m^3
Area of support:	8,100 m^2
Concrete poured:	6,500 m^3
Steel (reinforcing)	400 metric tons
Number of train movements in railroad yard over site during construction:	350,000
over a period of 18 months:	
Cost breakdown:	
Site installation:	12.0 %
Demolishing and soil excavation:	6.6 %
Support:	22.4 %
Concrete:	20.0 %
Temporary railroad bridges:	8.4 %
Track reconstruction:	3.4 %
Storm drains:	6.2 %
Other work related to the railroad:	21.0 %

could be interrupted during construction. For the remaining eighteen tracks, temporary bridges had to be placed. Another obstacle was the catenary wires and posts, which interfered with construction equipment. In some places, special low-clearance equipment was necessary to place the soldier piles. Whenever possible, the piles were driven by a free-fall hammer (with a noise protection shield). The piles located under the catenary could not be driven and were placed in predrilled holes. Two sections of 5.8 m length were lowered and bolted together to form a rigid pile. The final tunnel is constructed in 32 blocks of 10 m length; no waterproofing layer is necessary since impervious concrete is poured.

This site had thus substantial constraints which will result in higher total costs. The total cost of this section, including the storm drain and the work related to the railroad, was 10.1 Million DM; the costs of the tunnel only are approximately 7 Million DM. The total construction costs per route meter are 31,500 DM/m (15,750 $/m, $1 = 2 DM), including storm drain and work related to the railroad; for the tunnel only the costs are 21,900 DM/m (10,950 $/m).

3.4 Comparison of Section Ost 1 and Ost 3

Site constraints are not considered in this comparison. However, the major difference between the two sections lies in the hydrologic conditions, which resulted in a 8000 DM/m (4000 $/m) higher cost for Section Ost 1.

APPENDIX:	A-13
NAME:	Allgemeine Baugesellschaft A. Porr (Porr, Contractor)
ADDRESS:	Rennweg 12 A-1031, Wien, Vienna Austria
DATE OF MEETING:	23rd, 24th January, 1978
PERSONS MET:	Mr. Köhler, Manager, Principal Mr. Pöchhacker, Executive Vice President Mr. Zotter, Chief Estimator

1) Introduction

Porr, an Austrian contractor, has extensive experience in tunnel construction, and they were involved in several of the major steps in the development of the "New Austrian Tunneling Method, NATM". Examples include the Waldeck Power Station Cavern in Germany, the Tauern and Katschberg Tunnels, and the Arlberg Tunnel.

The discussions with Messrs. Köhler, Pöchhacker and Zotter yielded a significant amount of information on the contractor's point of view in tunnel construction. The information obtained will be discussed as follows. Section 2 deals with bid estimates. Contractual aspects and aspects of ground classification are discussed in Sections 3 and 4, respectively.

2) Estimates

The principle of estimating labor costs are described first, followed by an example estimate for the Arlberg Tunnel.

2.1 Labor Costs

Labor costs are determined from prevailing wage rates, to which a surcharge is added. This surcharge varies depending on the type of construction contract and bid schedule.

The wage rate for heavy construction is determined by a single contract between the unions and the association of heavy construction contractors. The contract terms are renegotiated each year. The wage rate is set as well as other benefits (overtime pay, vacations, 13th and 14th monthly wages). (In Austria, twice a year, usually at the end of June and December, two monthly wages are paid to each worker, the 13th and 14th monthly wage). The cost of these 13th and 14th monthly wages has to be added to the basic hourly wage rate as well as the other benefits. Since the contract conditions change each year, a handbook is published (Bauhandbuch), which lists the factors that have to be considered in estimating labor costs and in determining the surcharge to the basic wage rate.

The surcharges reflect social cost, overhead, site operation costs, overtime payments, and fringe benefits which a contractor has to include in his estimate. The surcharges are expressed as a percentage of the basic wage rate, and for a particular project they are expressed as a percentage of the anticipated average wage rate for the specific site. The average wage rate for each site is based on the anticipated crew to be used on the project; i.e., the contractor has to know which trades and how many of each trade he wants to use on the site.

In Table A-13.1, examples of surcharges taken from the earlier mentioned manual are shown. The five cases shown differ in the contract conditions, i.e., whether site installation, site operation and equipment costs are separate items. In case 1, site installation, site operation and equipment costs are separate items and are thus not included in the the surcharges on labor costs. In case 2, equipment costs have to be included in the surcharge, and in case 3, only site installation costs are not included. In cases 4, all costs mentioned above are included. The wage surcharge for the cases shown in Table A-13.1 varies. Note in particular that social benefits amount to 105% of the basic wage. However, these five cases are only examples and illustrate the ranges of the surcharge on the basic wage rate. For the estimation of individual projects, the contractors have to determine the surcharge, based on: (i) the actual contract conditions, (ii) his experience gained from measuring work rates in previous projects, and (iii) his risk premium.

TABLE A-13.1 WAGE SURCHARGES IN AUSTRIA FOR DIFFERNT TYPES OF CONTRACTS (AS OF APRIL 1, 1978)

TYPE OF SURCHARGE TO WAGE RATES	WAGE SURCHARGE TYPE 1 BASIC TYPE	TYPE 2 OPERATING COSTS OF CONSTRUCTION SITE SURCHARGED TO WAGES	TYPE 3 OPERATING COSTS OF CONSTRUCTION SITE AND EQUIPMENT COSTS SURCHARGED TO WAGES	TYPE 4 SITE INSTALLATION COSTS, OPERATING COSTS OF SITE, EQUIPMEN COSTS SURCHARGED TO WAGES	TYPE 5 SAME AS TYPE 4 BUT HIGHER PROFIT MARGIN
PROFIT MARGIN	Profit = 4.3 %	Profit = 5.4 %	Profit = 6.5 %	Profit = 4.3 %	Profit = 5.4 %
Wages	100	100	100	100	
Social Payments	104.70	104.70	104.70	104.70	
Office Costs	1.84	1.84	1.84	1.84	
Small Equipments	4.14	4.14	4.14	4.14	
Wage Tax	2.00	2.00	2.00	2.00	
Liability Insurance	1.50	1.50	1.50	1.50	
General File Costs,			1.84	1.84	
Small Materials,				(6-15)* 7.87	
Misc. Freight	1.84	1.84			
Operating Costs of				(15-30) 15.75	
Site (time dependent costs)	-	17.91	} 22.71	(2-6) 3.80	
Equipment Costs	-	-		(8-30) 12.00	
Fabrication Cost	216.02%	233.93%	238.73%	255.44%	
Surcharge on total					
Overhead for home offc	8.50	8.50	8.50	8.50	
Interest payments	2.30	2.30	2.30	2.30	
Risk premium	2.20	2.20	2.20	2.20	
Profit	4.30	5.40	6.50	4.30	
	17.30%	18.40%	19.50	17.30	
	$\frac{17.3 \times 100}{100 - 17.3} \times 216.02$	$\frac{18.4 \times 100}{100 - 18.4} \times 233.93$	$\frac{19.5 \times 100}{100 - 19.5} \times 238.73$	20.92 x 255.44	
Surcharge	= 45.19%	= 52.75%	= 57.72	= 53.44	
Labor costs	261.21%	286.68%	296.55%	308.88%	
Wage surcharge	161%	187%	197%	209%	213%

*values in parentheses give the range.

The actual determination of the wage surcharge has to be performed on a form and is included in the construction contract (The Austrian Standards B2111 require that the contractor shows to the owner how he obtained the wage surcharge). Also, the average wage rate for each site has to be determined on a form and included in the contract documents.

The present hourly basic wage rate is approximately 45 AS = \$3. For a tunnel construction site, the actual labor costs per hour vary in the range from 170 to 220 AS (\$11.3 to 14.7). These labor costs reflect higher surcharges than those presented in Table A-13.1, mainly due to the "remoteness" payment and higher fringe benefits and premiums for these tunnel sites.

Actually paid wages are also considerably higher, in particular for the miners. Bonus payments for miners may reach 200%. The average bonus payments for a single tunnel site are 110 to 120%. Thus, with a basic hourly wage of 45 AS = \$4, a miner gets paid up to 135 AS = \$9, or for a 40-hour work week, \$360/week. The average pay is 95 to 100 AS/hour or \$6.3 to 6.6/hour, equalling 250 to 270 \$/week.

2.2 Estimate of Costs for Tunnel Excavation and Support

Table A-13.2 shows bid estimate data for the western section of the Arlberg Tunnel. Based on the support quantities specified in the bid documents and the contractor's experience, the work rates for each item have been estimated. In this particular case, the experience from the Tauern Tunnel was

TABLE A-13.2 ESTIMATED WORKHOURS FOR CONSTRUCTION OF ARLBERG WESTERN SECTION PER GROUND CLASS

		UNIT	CLASS Ia			Ib			II			IIIa			IIIb			IV			V		
EXCAVATION	Excavation-total length	m	70			110			1145			1275			1160			945			395		
	-invert	m^2													7.9			8.4			8.9		
	-area	m^2	90.9			90.9			90.9			90.9			90.9			92.2			93.5		
	-overbreak	cm	30			30			25			20			20			15			10		
	Round length	m	3.0			3.0			3.0m			1.5m			1.5m			1.0m			< 1.0m		
	Blasthole/m^3	m/m^3	1.85			1.85			1.85			1.85			-			1.55			1.50		
	Workhours/meter for excavation	hr/m	105.72			105.72			99.32			118.44			117.62			161.46			180.38		
SUPPORT			units	time per unit	(hrs) to-tal	units	time per unit	(hrs) to-tal	units	time per unit	(hrs) to-tal	units	time per unit	(hrs) to-tal	units	time per unit	(hrs) to-tal	units	time per unit	(hrs) to-tal	units	time per unit	(hrs) to-tal
	thickness																						
	Shotcrete:																						
	crown cm	m^2				12.3	0.63	7.75	12.2	0.88	10.74	12.2	0.88	10.74									
	springline cm	m^3							13.8	0.63	8.69	13.7	0.88	12.06	25.8	1.13	29.75	25.9	1.38	35.74	26.0	1.66	43.16
	face cm	m^3																6.7	0.63	4.22	9.3	0.88	8.18
	Wire fabric:																						
	crown	kg	13.7	0.25	3.43	-			37.7	0.14	5.28												
	springline	kg	-	-	-				-	-	-	80	0.14	11.20	80.	0.14	11.2	80.6	0.14	11.28	78.3	0.14	10.96
	Bolts:																						
	crown	m	4.25	0.43	1.83	-			8.75	0.43	3.76							21.7	0.38	8.25			
	springline	m							-	-	-	17.25	0.43	7.53	33.3	0.38	12.65	30.0	0.31	9.30	100.	0.31	31.00
	invert	m													9.0*								
	Forepoling sheets:		-			-			-			-			-			-					
	crown	m^2																					
	springline																	-	-	-	26.5	2.80	74.20
	Steelsets:																						
	crown & springline }	m	-			-			-			-			-			15.6	1.02	15.90	34.6	10.0	34.6
	Concrete:																						
	in invert	m^3	-			-			-			-			4.8*			5.4*			6*		
	Workhours for support/m	hrs/m	5.25			7.75			28.47			41.53			53.0			84 69			202.70		
	Total work hours/m	hrs/m	110:97			113.47			127.79			159.97			170.62			246.15			383.08		
	Hours per m^3 (ex-cavation & support)	hrs/m^3	1.22			1.25			1.41			1.76			1.73			2.45			3.74		

*invert support only if necessary, (not included in work hours for excavation - support) incorporated. In the Arlberg Tunnel, work for placement of the support but not the support material had to be included in the excavation unit price. Thus the work for excavation and the placement of standard support quantities is obtained per lineal meter of tunnel. Based on the average labor costs per hour, the total labor costs (including this excavation and support placement) per meter of tunnel can be estimated. The total excavation unit price also includes some materials (mainly explosives).

Note that the work hours per cubic meter of excavation increase by a factor of 3 from the best to the worst ground class. A particular problem which surfaced at the Arlberg Tunnel relates to the contract requirement that support placement work should be included in the excavation unit price. For class V, a total of 100 bolts were designed; however, in some sections the actual length of bolts placed was more than 500 m per m. By assuming that the same work rate applied, one has to conclude that the actual work increased by 124 hours per meter of tunnel. With an average labor cost of 210 AS/hr (=$14) the contractor would thus have to carry 26,040 AS (=$1,736) per meter on his own with these contract provisions. The profit of the contractor may be further reduced by bonus-malus payment provisions for support quantities (Section 4). Obviously, under such payment provisions divergence of anticipated ground conditions from actually encountered ones must lead to disputes. The particular dispute at the Arlberg is not yet entirely solved.

Another problem in estimating is related to overexcavation to accomodate large convergence. Excavation is paid to a theoretical line of excavation which includes overexcavation. In general, the contractor tends to generously overexcavate, i.e., more than theoretically necessary, and use more concrete (i.e., effectively construct a thicker final liner) rather than to risk re-excavation. Re-excavation is more expensive than additional concrete, and it is practically impossible if high density bolting exists. It should be noted that the contractor has to include in excavation and concrete prices the necessary overexcavation and possible concrete refill. Thus, if the overexcavation is much greater than anticipated, disputes may occur, as was again the case at the Arlberg Tunnel.

3) Contractual Relations

3.1 General

Porr has not only great experience in tunnel construction in Austria but also in other countries, and thus under different contractual set-ups.

3.2 Award of Contracts

In Austria, usually, the low bidder is considered to be the best bidder and is awarded the contract. Only technical reasons may lead to exclusion of the low bidder. (It is also often the case that a low bidder has to beef up the price by means of "justified" claims, like the ones discussed above. These justified claims rarely lead to a court suit.) Due to the present depressed economic situation, most large projects are bid by joint-ventures; in this case, the low bidder is almost certainly awarded the contract since it is nearly impossible to exclude a joint-venture for technical reasons. However, the formation of these joint-ventures is not a technical necessity, but rather one of economics such that everybody gets a share of the pie.

Contrary to Austria, in Iran, where Porr has been working for more than 30 years, the low bidder is not considered the best bidder. In brief, the procedure of awarding a contract is as follows:

(i) the highest and lowest bidder are excluded

(ii) the average price is determined from the remaining bids

(iii) the contract is awarded to the bidder nearest to this average

This bidding practice leads to a strict contract interpretation and claims are rare.

4) Ground Classification

This section primarily reflects the ideas of Mr. Pöchhacker. He was chief site engineer at the Tauern Tunnel and has now moved to a management position in Porr's main office. His ideas were developed from the experience gained during the construction of the Tauern and Arlberg Tunnels. The ideas have been submitted to the Austrian Committee for Tunnel Contracts, to be included in the new standards.

In a comprehensive classification procedure one should consider the behavior of the ground at the face and the circumference separately. One would thus have to consider a two-dimensional field of possible combinations (Figure A-13.1); by only considering "reasonable" possibilities, one would arrive at the central band. Classes I and VII would behave similarly at the face and circumference, i.e., class I would be good everywhere whereas class VII causes problems at the face and circumference. However, for intermediate classes it is possible to have favorable behavior at the face and unfavorable behavior at the circumference and vice versa. This system may also be used on a single linear scale; this requires, however, subclasses for each principal class. The entire system is still highly qualitative.

Objective criteria are necessary for ground classification; in particular, the required support and the convergence of the tunnel should be predicted. This goal may be achieved by statistical analysis of support placed in built tunnels. Mr. Pöchhacker cautions that even such an approach does not consider all possible combinations.

5) <u>Crew Quality, Site Organization and Equipment</u>

Crew quality, site organization, and type of equipment are very important for successful tunnel construction in general and the New Austrian Tunneling Method in particular.

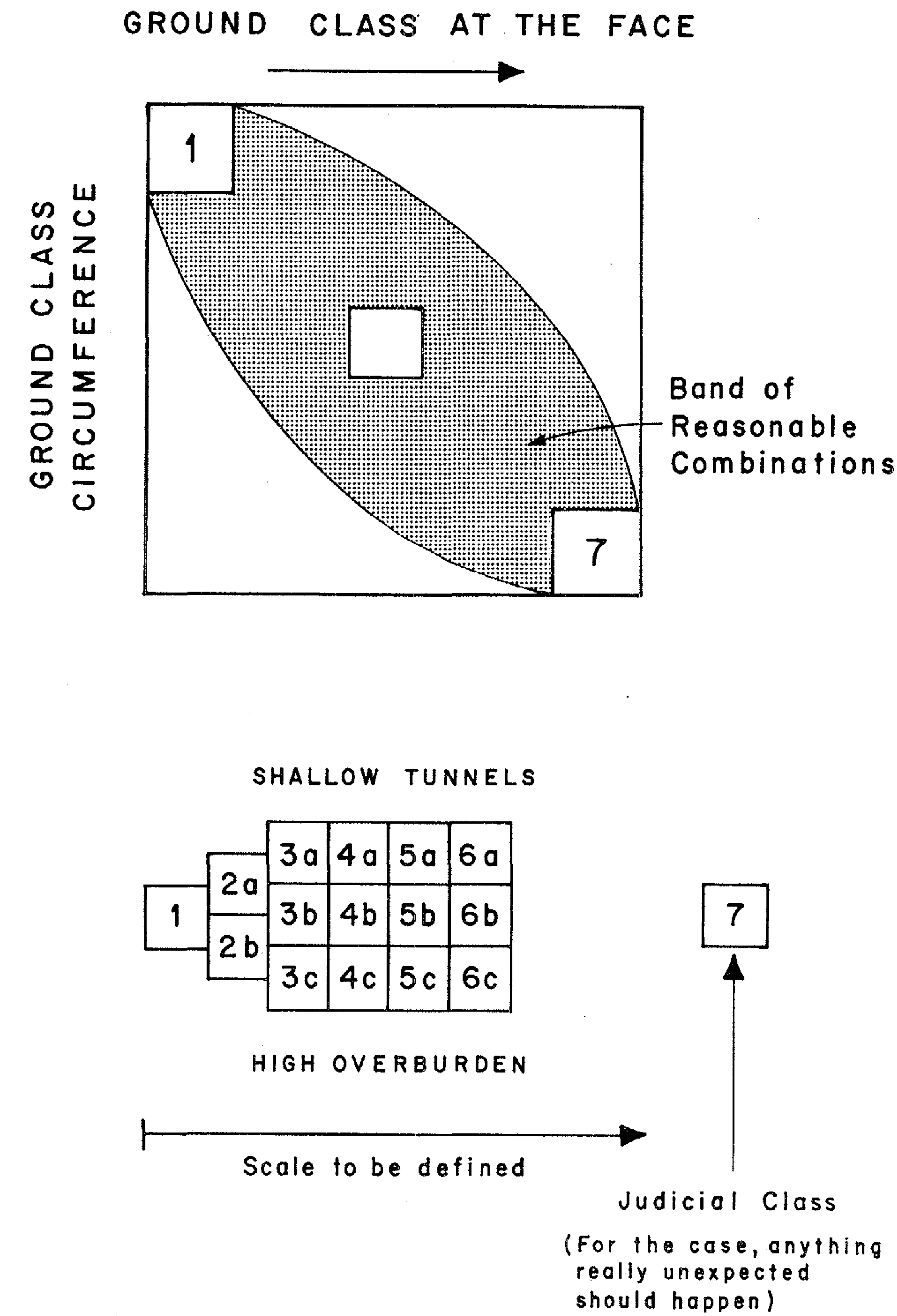


FIGURE A-13.1 CLASSIFICATION SCHEME AFTER POCHHACKER

5.1 Crew Quality

The New Austrian Tunneling Method has been developed by training young miners on site and in small steps. Miners must be able to perform various types of work; e.g., they have to know exactly how to place a rock bolt, how to hold the shotcrete nozzle, how to place the wiremesh and the steelsets. According to Mr. Köhler, it is important that rockbolts are placed a short time after the excavation; often a delay of a few minutes may be crucial, and a delay may require more support. However, at the present time these details have not yet been quantified.

The miners require continuous on-the-job training, and it may be often difficult to retrain older miners to new types of equipment. In particular, miners used to pneumatic hand drills may not get used to the new electrohydraulic boom jumbos. Thus, management has to consider these points when assigning personnel to specific tasks. (Fortunately, no union rules limit this task of the manager).

5.2 Site Organization

Among major considerations for site organization are: (i) the length of the tunnel section, (ii) the schedule, (iii) the remoteness of the site. The length of the tunnel has an influence on the type of haulage that is chosen. In longer tunnels, track haulage may be more favorable because less exhaust is produced, thus reducing ventilation requirements.

In tunnel sections shorter than 3 km,trackless operation is more economical and ventilation causes no major problems.

The selection of the shift arrangement based on the schedule and remoteness of the site has been discussed in Section 5.

5.3 Equipment

Equipment used in tunnel construction has undergone significant development. The most important development during the last few years is the electrohydraulic drill. At the Tauern Tunnel penumatic drill hammers were used, and only after the construction of the Arlberg Tunnel had started did reliable electrohydraulic drills become available. Table A-13.3 shows a comparison of drill rates. In addition to having roughly twice the drill rate of penumatic drill hammers, electrohydraulic drills use less energy and are quieter.

6) The Vienna Subway

Currently in Vienna a subway is under construction. The first line began operation in the spring of 1978. The subway is primarily constructed by shield tunneling with segmented steel liners. The subsoil underneath the inner city in Vienna is Vienna Tegel (Wiener Tegel), an inhomogeneous ground, varying from sand to silt to clay. The ground conditions required excavation under compressed air to avoid dewatering. In areas where the surface effects had to be minimized (inner city, St. Stephen's Cathedral), the zone around the shield was grouted to further reduce the risk of running and ravelling. Shotcrete

TABLE A-13.3 PENETRATION RATES OF DRILL HAMMERS

RATE	PNEUMATIC	ELECTROHYDRAULIC
Gross drill rate, cm per minute	90 - 100	200
Net drill rate, cm per minute (includes time to change drillsteel and switch to new boring)	60 - 70	140 - 160

support according to the NATM has not been used on the section built by Porr, although in some other sections it has been used for station cross-cuts. This example illustrates that there are some limitations to the NATM. In addition, there seem to have been political reasons - the supplier of the segmented steel liners (which are innovative also) is the government owned firm VOEST.

For the time being, the Vienna subway will not be further described, but we might consider it in a later stage of the research.

APPENDIX:	A-14
NAME:	Amt der Steiermarkischen Landesregierung (Department of Public Works, State of Styria
ADDRESS:	Landhausgasse 7 A-4010 GRAZ Austria
DATE OF MEETING:	25th and 26th January, 1978
PERSONS MET:	Dr. W. Gobiet Head of the Tunnel Design Department

1) Introduction

The Department of Public Works of the State of Styria (Steiermark) supervises the construction of tunnels on public highways that are not built by a separate authority. The government of Styria has long-standing experience with the New Austrian Tunneling Method. The first well known case where the NATM was essential is the Massenberg Tunnel on the bypass road of the City of Leoben (Rabcewicz, 1965; Einstein et al., 1977, Figure A-14.1). The present boom in highway tunnel construction started only a few years ago, and most of the tunnels are thus under construction or planned.

2) Highway Tunnels in Styria (Steiermark)

Tunnels are mainly necessary along the two planned major

Das Netz der
Österreichischen Autobahnen
laut Bundesstraßengesetz 1971

A 1 West Autobahn
A 2 Süd Autobahn
A 3 Südost Autobahn
A 4 Ost Autobahn
A 5 Nord Autobahn
A 6 Preßburger Autobahn
A 7 Mühlkreis Autobahn
A 8 Innkreis Autobahn
A 9 Pyhrn Autobahn
A 10 Tauern Autobahn
A 11 Karawanken Autobahn
A 12 Inntal Autobahn
A 13 Brenner Autobahn
A 14 Rheintal Autobahn
A 15 Bodensee Autobahn
A 20 Wiener Gürtel Autobahn
A 21 Wiener Außenring Autobahn
A 22 Donauufer Autobahn
A 23 Autobahnverbindung Wien/Süd
A 24 Autobahnverbindung Wien/Ost
A 25 Linzer Autobahn

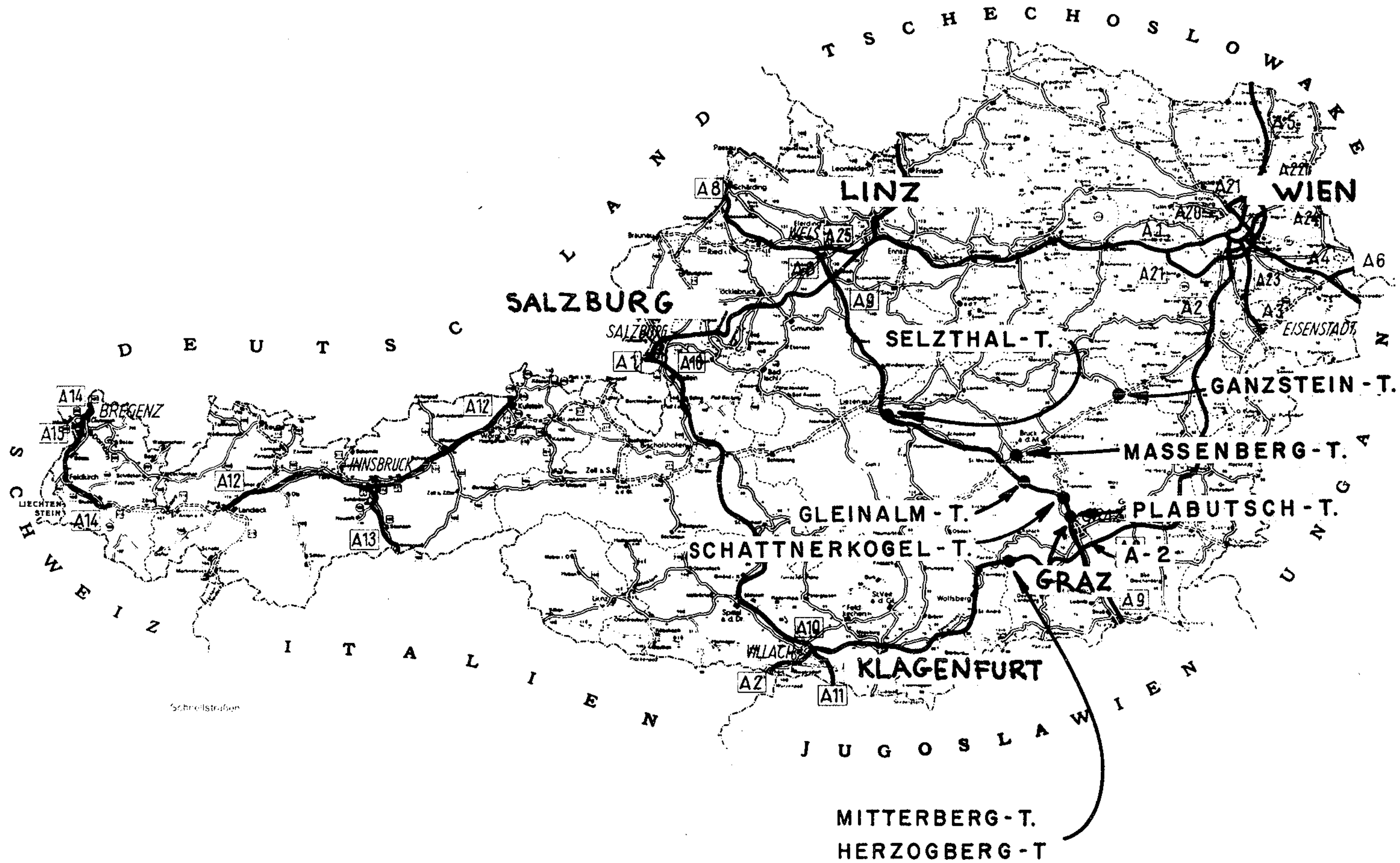


FIGURE A-14.1 MAJOR HIGHWAY TUNNELS IN THE STEIERMARK, AUSTRIA, BUILT, UNDER CONSTRUCTION OR PLANNED highways, the Highway A2, "Sudautobahn", linking Vienna to Graz and Klagenfurt, and the other major route, the Phyrnautobahn A9, linking Yugoslavia and West Germany from Maribor to Passau, via Graz, Selzthal and Linz. A secondary major highway is the Semmering Route S6, which needs improvement (Figure A-14.1), since it carries the traffic from Vienna to Graz until the Sudautobahn will be opened.

At the present time, the Sudautobahn, A2, is under construction west of Graz to form a link to Klagenfurt in Karnten. To our knowledge, two major tunnels are under construction, namely;

the Mitterberg Tunnel (L ≃ 1.1 km) and

the Herzogberg Tunnel (L ≃ 2.0 km)

Work at the Mitterberg Tunnel is suspended at the present time because one of the contractors of the joint venture has just initiated bankruptcy proceedings. The geologic data of the Mitterberg Tunnel were available in the office in Graz; they will be summarized in Section 3 of this appendix. The second tunnel, the Herzogberg Tunnel, is located some 50 miles to the west of Graz and can only be reached by car. Due to time limitations, W. Steiner decided not to visit this site. According to Dr. Gobiet, the geologic conditions at the Herzogberg and Mitterberg Tunnels are very similar.

In addition, several tunnels have been built, are under construction, or are planned on Phyrnautobahn, A9; they are listed in Table A-14.1.

TABLE A-14.1 TUNNELS OF THE PHYRNAUTOBAHN, A9

TUNNEL	LENGTH (km)	STAGE	REMARKS
Plabutsch	9.5	planning (test adit)	under direction of state by-pass of Graz
Schattnerkogel	1.5	completed (tunnel)	Phyrnautobahn-Gesellschaft
Gleinalm	8.3	completed (1 tunnel)	Phyrnautobahn-Gesellschaft
Selzthal	1.01	under construction	State of Styria (Steiermark)
Bosruck	5 (?)	planned	Phyrnautobahn-Gesellschaft

The Plabutsch Tunnel is a part of the Graz bypass highway. Initially, it was planned to build the highway through the outskirts of Graz. In 1973, the elections to the city government were won by a party running on a ticket against the proposed location of the highway and requesting its relocation in a tunnel. (The weekend following Steiner's visit, new elections were held in Graz, and prior to these the test adit had to be presented to the press and public as proof of progress and feasibility of the tunnel.)

The other two tunnels north of Graz, the Schattnerkogel and the Gleinalm Tunnels, have been built by the Phyrnautobahn Gesellschaft (tentative opening date in mid-1978). No data on these tunnels has been collected. The Selzthal Tunnel has been visited and is described in Appendix B-6. The Bosruck Tunnel is planned and will probably be built by the Phyrnautobahn-Gesellschaft.

3) The Mitterberg Tunnel

Geologic data on the Mitterberg Tunnel were available in Graz. Two parallel two-lane tunnels approximately 1.1 km in length are built. General data were available for both tunnels; however, detailed data were only available for the southern tunnel. It extends from Stations 233.853 to 234.938 km of the Sudautobahn, A2. At the time of W. Steiner's visit to Graz, construction had been halted because of the aforementioned bankruptcy of the principal contractors.

A pilot tunnel had been driven prior to bidding. Fig. A-14.2 shows a cross-section (sketch) of the tunnel and the approximate location of the pilot tunnel. In Table A-14.2, predicted and encountered lengths of the five ground classes in both tunnels are shown. The deviations from the predicted geology seem to be a source of dispute and a point of changed condition claims (the contractor went into bankruptcy).

Standard support quantities for each ground class are listed in Table A-14.3. Detailed data on ground conditions for the southern tunnel are presented in Table A-14.4. The geologic data available were collected continuously in the heading of the tunnel by a consulting geologist hired by the contractor (see Figure A-14.2).

The data collected indicate that (i) there are three major joint sets, (ii) most joints are persistent, and (iii) the joints are often coated by talc and montmorillonite. The major parameters distinguishing ground classes seems to be the block size relative to the size of the opening. In ground class III there are approximately 5 to 10 blocks per tunnel diameter; in ground class IV the number of blocks is 10 to 15 or more.

Since the discontinuities were persistent problems when geologic overbreaks were encountered, the tunnel circumference primarily follows the discontinuities rather than the

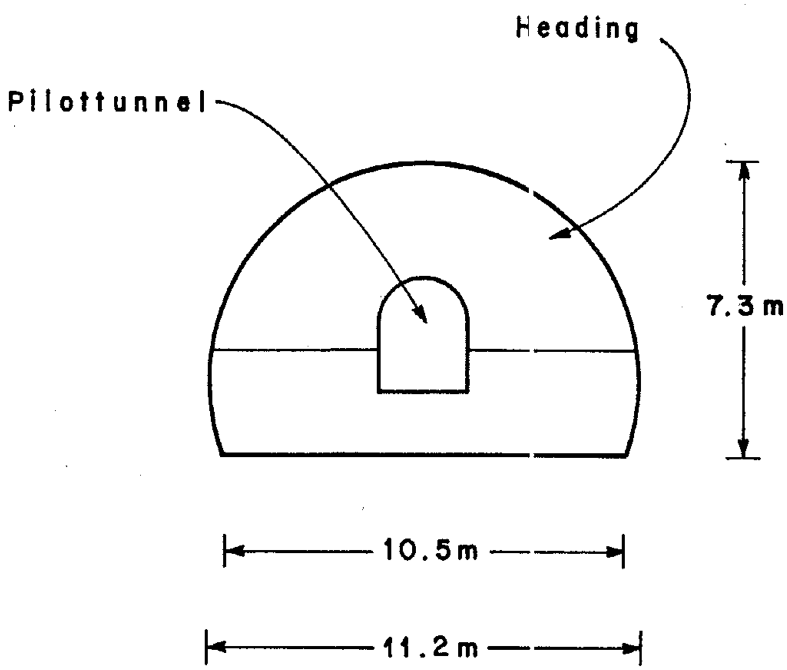


FIGURE A-14.2 CROSS-SECTION OF MITTERBERG TUNNEL (PRINCIPAL DIMENSIONS)

TABLE A-14.2 COMPARISON OF PREDICTED AND ENCOUNTERED GEOLOGY OF MITTERBERG TUNNEL

GROUND CLASS	LENGTH DIMENSIONS PREDICTED (meters)	SOUTHERN TUNNEL Encountered	% Deviation	NORTHERN TUNNEL Encountered	% Deviation
I	60	59.5	-0.8	50	-16.7
II	150	165	+10.0	197	+31.3
III	234	207.5	-11.3	176	-24.8
IV	120	100	-16.7	121.3	+1.1
V	36	68	+88.9	55.7	+54.7

TABLE A-14.3 STANDARD SUPPORT QUNATITIES FOR GROUNDCLASSES OF MITTERBERG TUNNEL

SUPPORT \ GROUNDCLASS	I	II		III	IV	V	
ROCK BOLTS;							
Type	expansion	expansion	grouted	grouted	grouted	grouted	
Size	ϕ 20 mm	ϕ 20	ϕ 24	ϕ 24	ϕ 24	ϕ 24	
Length of Bolt	2.5 m	2.5	4.0	4.0	4.0	4.0	
Quantity per lineal meter of tunnel	3.0 p	3	3	6	8	4	7
Quantity/Meter	7.5 m/m	7.5 m/m	12.0 m/m	24 m/m	32 m/m	44 m/m	
PRE-STRESS			20 t	20 t		No	20 t
SHOTCRETE;							
Thickness β_{28} = 220 kg/cm^2	5 cm	7.5 cm		10 cm	15 cm	20 cm	
Area shotcreted per lineal meter of tunnel	20.3 m^2	20.4 m^2		20.6 m^2	20.7 m^2	23.9 m^2	
WIREFABRIC;							
Unit weight kg/m	1.9	3.12		3.12	3.12	3.12	
Area m^2	12	(13 + 20.5) 1/2		20.6	20.7	23.9	
Total weight	22.8 kg	52.3		64.2	64.5	74.7	
STEELSETS;							
Type	NO	NO		NO	WELDED TRUSS	WELDED TRUSS	
Weight					equiv. THŌ 21 (21 kg/m)	equiv. THŌ 21 (21 kg/m)	
Spacing					1.40 m	1.40 m	

TABLE A-14.4 DESCRIPTION OF ENCOUNTERED GROUND CONDITIONS AND ROCK MASS PROPERTIES FOR MITTERBERG TUNNEL

CASE	STRESS CONDITIONS (overburden)	ROCK DESCRIPTION	MEASURED CONVERGENCE	ROCK MASS PROPERTIES
GROUND CLASS III (GGKL III)				
Mitterberg south tube	130 m	Gneiss, gray to light gray. After mucking and during placement of support rooffalls occurred. Joint spacing approximately 1m. Joints and planes of schistosity filled with mylonite to some extent but not completely Set 1: Strike rel. tunnel = 30° attitude in CS = 30° to the south Joints persistent, spaced 3 to 4 m Set 2: Strike rel. tunnel ~ 30° attitude in CS approx. vertical Set 3: dips to the north at 40 - 50°, spaced 1 - 2 meters, length of joints and intact rock bridges approx. 2 m	after heading excavation: 7mm after bench excavation 17 mm Final: 20mm	Rock blocks per diameter ~ 5 - 12 Block shape rectangular to cube Block size 0.5 to 2.0 m
GROUND CLASS IV (GGKL IV)				
Mitterberg south tube	130 m	Gneiss, light gray, dark band of schistosity (5-7 cm thickness) wedging out to the north, gneiss has been altered Large size discontinuities parallel to face. Dark garnet mica-schist, (Granatglimmerschiefer) schistosity dipping SSW, i.e., towards the face. Strong disturbance above pilot tunnel (cave-ins when drilling for main tunnel. Jointing narrowly spaced (1-10cm)	Range of convergences = 6 to 53mm	Major joint spacing 0.5 to 1 m. Joint filler with kaolinite and talc
GROUND CLASS V (GGKL V)				
Mitterberg south tube	50 m	Mica schist with garnet, banked to platy, partly thin-layered. Schistosity dipping flatly to the south. In the crown large grey mylonite with embedded tectonically disturbed quartz and feldspar layers. Surface of the mylonite zone shows striated slickensides (which act as preferred plane of separation).	Station ΔH (mm) Overburden 1037 8.5 50 810 12.3 105 mean ≈ 10 mm (only!)*	Similar to Class IV Dispute on GGKL IV or V? (oversupported?)

CS = cross section

*The small convergences of approximately 10mm for Ground Class V, which is less than the observed convergences in Classes III and IV, may indicate that the ground is "oversupported" in Class V.

theoretical line of excavation. Even in the pilot tunnel, overbreak in the crown occurred; this was aggravated in the main tunnel, where the larger cross-section facilitates movements and fall-outs of rock blocks.

Dip of the discontinuities relative to the face can have a significant effect, possibly leading to a change in ground class. For example, near station 743 m, a fold, whose axis strikes perpendicular to the tunnel axis and dips to the south, was encountered. Before Sta. 743, the discontinuities are dipping away from the face. At Sta. 743, the ground was classified as class III, at 745.5 as class IV. The change in discontinuity dip was one cause; another factor contributing to the change in ground class was the more intense weathering at station 745.5 m (the axis of the fold).

At the Mitterberg Tunnel, considerable disputes seem to have arisen about the ground classification. These disputes may have two reasons: (1) an unreasonably low bid, (2) the classification criteria was incomplete. In the following, the second argument will be further studied.

The original (prior to construction) classification criteria, which are primarily of mineralogic nature, are presented. During tunnel construction, different rock types were encountered in the same cross-section, and it was thus not possible to assign a ground class unequivocally. The dispute centers now around this gap in classification criteria.

Another issue was raised by the contractor, respectively its engineering geologist; they claim that the conditions in the pilot tunnel have not been carefully mapped and that several major discontinuities filled with talc and montmorillionite were omitted. We cannot judge whether this claim is correct.

However, we concluded that this case once more confirms the problem of extrapolation from a small pilot tunnel of approximately 2 m diameter to a tunnel of 10 m in diameter. Also, classification criteria based on mineralogic considerations alone are not appropriate,

4) Contractual Aspects

4.1 Introduction

This section reflects the information gathered during the interview with Dr. Gobiet, covering both experience gained in the tunnels in the Steiermark as well as Dr. Gobiet's personal opinions. (Information of a general nature on Austrian Practice has also been obtained; this information has been included in the main body of this report). The following topics of interest were discussed: (i) cost of tunnels, (ii) award of contracts, (iii) prevention and resolution of disputes, which includes ground classification, and (iv) problems related to overbreak.

4.2 Costs of Tunnels

In estimating tunnel costs Dr. Gobiet uses the following approximations. A single two-lane tunnel in rock with not too difficult ground conditions costs approximately 100 million

Austrian Schillings/km = 10.7 million U.S. dollars/mile. If two parallel two-lane tunnels are constructed simultaneously, these costs have to be multiplied by 1.8. However, for different ground conditions the above-mentioned values may double or triple. Irrespective of the length of the tunnel, the amount of 20 million AS = 1.33 million U.S. $ has to be added for site installation (camp, repair shop, supporting services) at each heading.

4.3 Award of Contracts

As a general rule a contract is awarded to the low bidder. It is theoretically possible to exclude an unreasonably low bidder; however, in practice this proves to be difficult since political pressure may be exerted, e.g., the DPW might be accused of wasting tax money. (However, often a contract awarded to an unreasonably low bidder may finally be more expensive than a reasonable one due to delays and approved changes).

It is possible to exclude contractors from bidding on public works projects if they have been proven to be previously unreliable. The federal ministry (Austrian Department of Public Works) publishes a list of unreliable contractors, which are then excluded for some time. This list includes mostly smaller building contractors and frequently concerns ready-mix concrete plants. Most tunnel contractors (heavy contractors) are reliable.

Preconstruction bonding is generally not required and the technical prequalification can be easily fulfilled. However, sometimes bonding and more stringent prequalifications may be desirable. This may exclude young promising firms from a successful start in tunnel construction.

4.4 Prevention and Resolution of Disputes

Disputes can best be prevented with complete contract documents and a complete continuous project supervision. Also, Dr. Gobiet thinks a mediator-arbitrator nominated and accepted by both parties before (or when) signing the contract would be helpful. The costs for mediator-arbitrator should be borne by both parties, the contractor and the owner.

5) Ground Classification

At present, there is no unique ground classification system available in Austria. A ground classification is developed for each tunnel. Notably, the displacements cannot be used as classification criteria as there are considerable difference in observed displacements in the same class but different geolo conditions. For example, at the Mitterberg Tunnel, horizontal convergences monitored are in the order of 5 cm for ground clas IV, whereas at the Selzthal Tunnel crown settlements in the order of 50 cm were observed (compare Appendix B-6). Also, observations do not necessarily prevent a collapse, as evidence in the Ganzstein Tunnel (Appendix B-5), where, despite convergence measurements, a collapse occurred.

6) Overbreak

Particular attention should be paid to overbreak caused by geological structure (e.g., Mitterberg Tunnel). At the present time the contractor carries the risk of overbreak; overbreak has to be included in both the unit price for excavation and the inner liner (concrete), which are both paid by theoretical quantity. The inner liner is paid by volume corresponding to the theoretical thickness, which is, in most cases, 30 cm. The actual average liner thickness, however, is much more and often reaches 60 cm.

It might be preferable to pay overbreak exceeding some limit as a separate item. The quantity exceeding the limit might be paid at a reduced rate. With this procedure, disputes with respect to overbreak should be reduced.

APPENDIX B

SITE VISITS

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APPENDIX:	B-1
SITE:	Section 16 Line U8/1
OWNER:	Subway Authority of Munich
DESIGNER:	Not known (Alternate proposal)
CONTRACTOR:	Kunz AG Contractors Bilfinger & Berger and others in joint venture
DATE: OF VISIT:	4th January, 1978
PERSONS MET:	Mr. Weber, Baudirektor, Head of the Design Department
GUIDE:	Mr. Nixdorf, Head of the Construction Supervision

1) Site Description

Figure B-1.1 shows a general map of the rapid transit network of Munich. Section 16 of Line 8/1 is marked. This section extends from the northern end of station "Hauptbahnhof" (Central Railroad Station) northbound, to station "Theresienstrasse" of Line 8 and includes station Königsplatz (Fig. B-1.2). Line 1 branches off to the west immediately north of the four track "Hauptbahnhof" station. In the alignment of Line 8, also just north of "Hauptbahnhof", a wye is built, thus requiring a three-track tunnel of 150 m^2 cross-section. This section includes a series of unique features which will be described in greater detail. At present, no detailed map is available; the

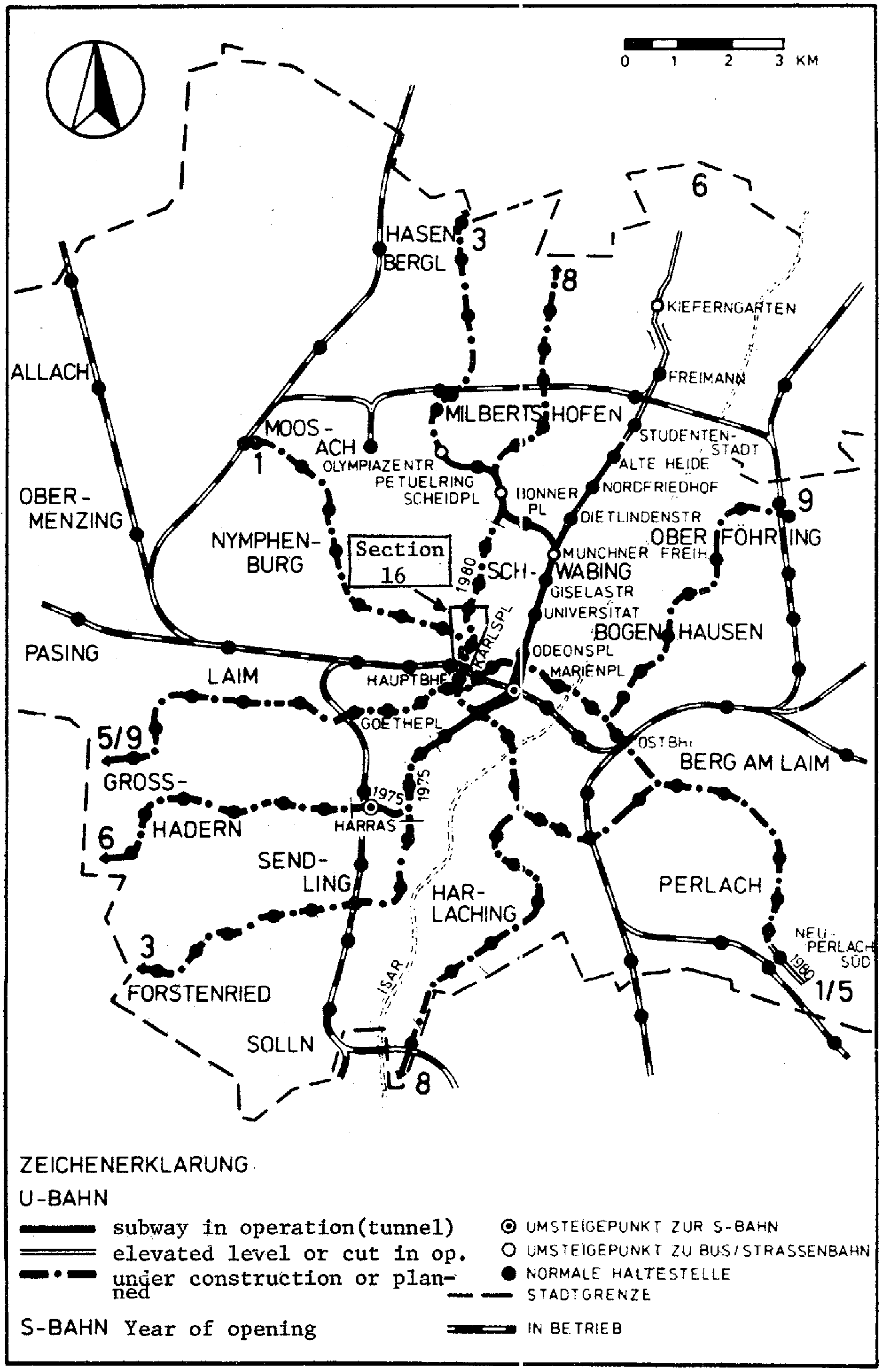


FIGURE B-1.1 MAP OF THE MUNICH SUBWAY, WITH LOCATION OF SECTION 16 (FROM BLENNEMANN, 1975) reader is referred to the sketch (Figure B-1.2 and Figure B-1.3) which shows a view of the intermediate access shaft "Sophienstrasse" between Hauptbahnhof and Königsplatz.

The Königsplatz Station is built by the under-the-roof construction method with diaphram walls (thickness = 80 cm, trench length = 4.5 m in non-built over areas and 1.5 m in builtover areas). The quality (watertightness, smoothness) of the diaphragm walls is excellent. However, an additional interior wall of imprevious concrete will be poured (without waterproofing layers). The base slab is keyed into the diaphragm walls to prevent uplift.

From the shaft "Sophienstrasse" and the north end of the "Königsplatz" station, a total of 10 headings are advancing simultaneously. The type of headings vary and are adapted to the final cross-section of the tunnel. The different types of heading are described below. A total of 220 men work in this section.

Two running single-track tunnels are driven from the north end of the Königsplatz station towards the Theresienstrasse Station. The alignment is S-shaped, as one block is crossed to a parallel street. The excavation and initial support follows the principles of the NATM but is somewhat modified and called the ramp construction method; it has been developed by Kunz's contractors. Figure B-1.4 explains the method with a longtitudinal section and several cross-sections; Figure B-1.5 shows a photograph of a heading. The heading

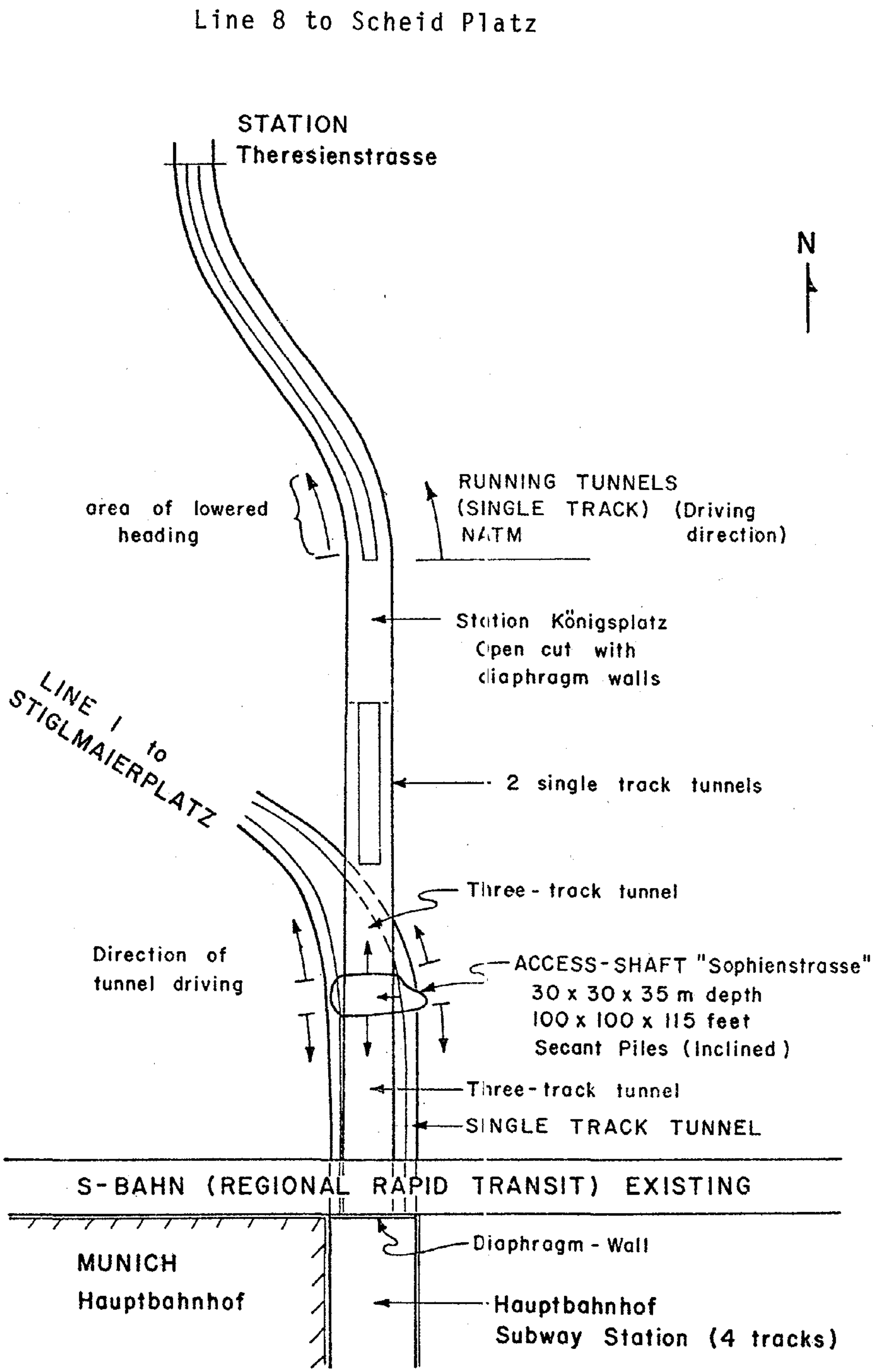


FIGURE B-1.2 SCHEMATIC PLAN VIEW OF SECTION 16 (LINE U 8/1) OF THE MUNICH SUBWAY

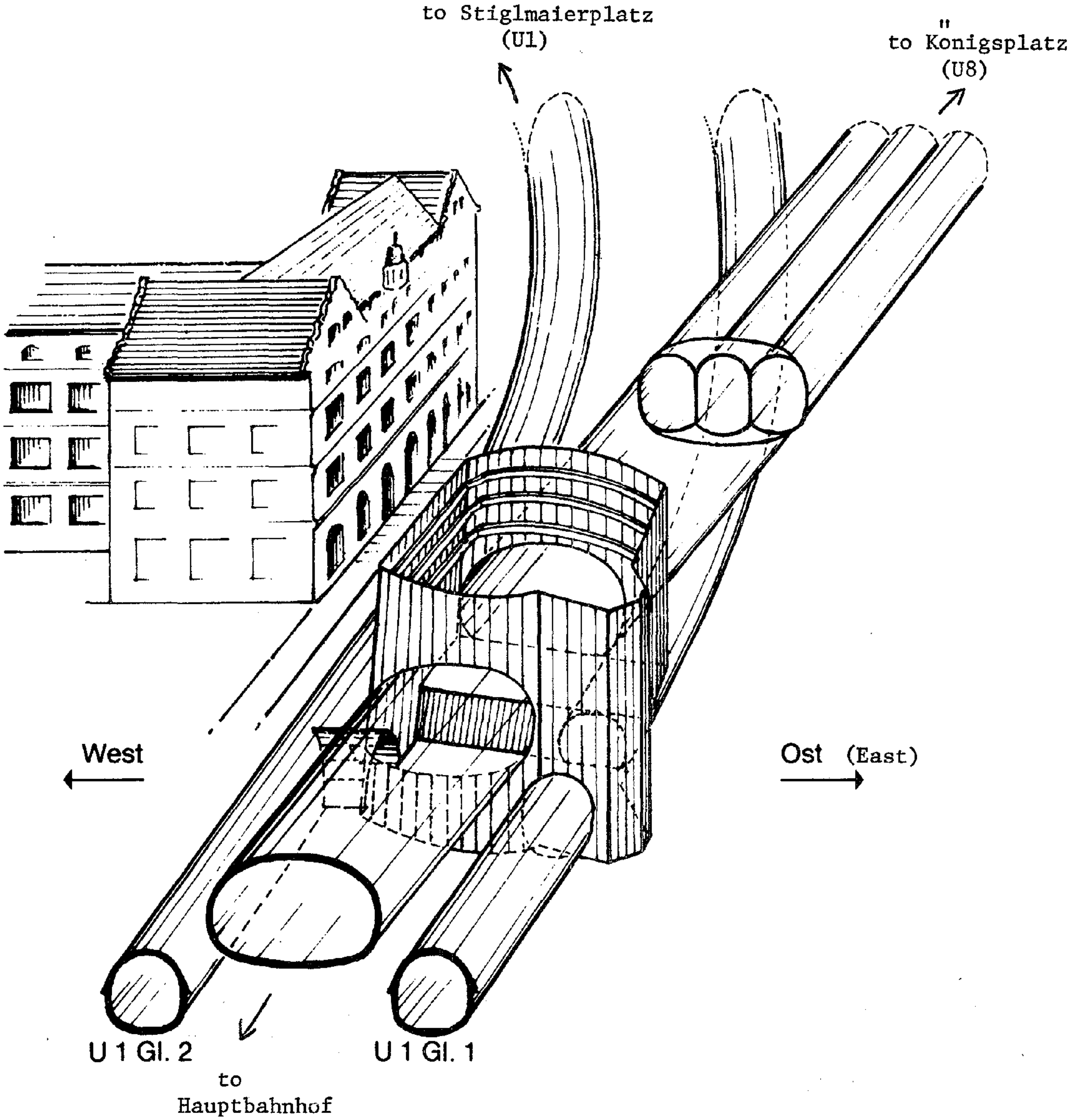


FIGURE B-1.3 PERSPECTIVE VIEW OF THE ACCESS SHAFT "SOPHIENSTRASSE" WITH RUNNING TUNNELS (FROM BROCHURE BY KUNZ CONTRACTORS, 1978)

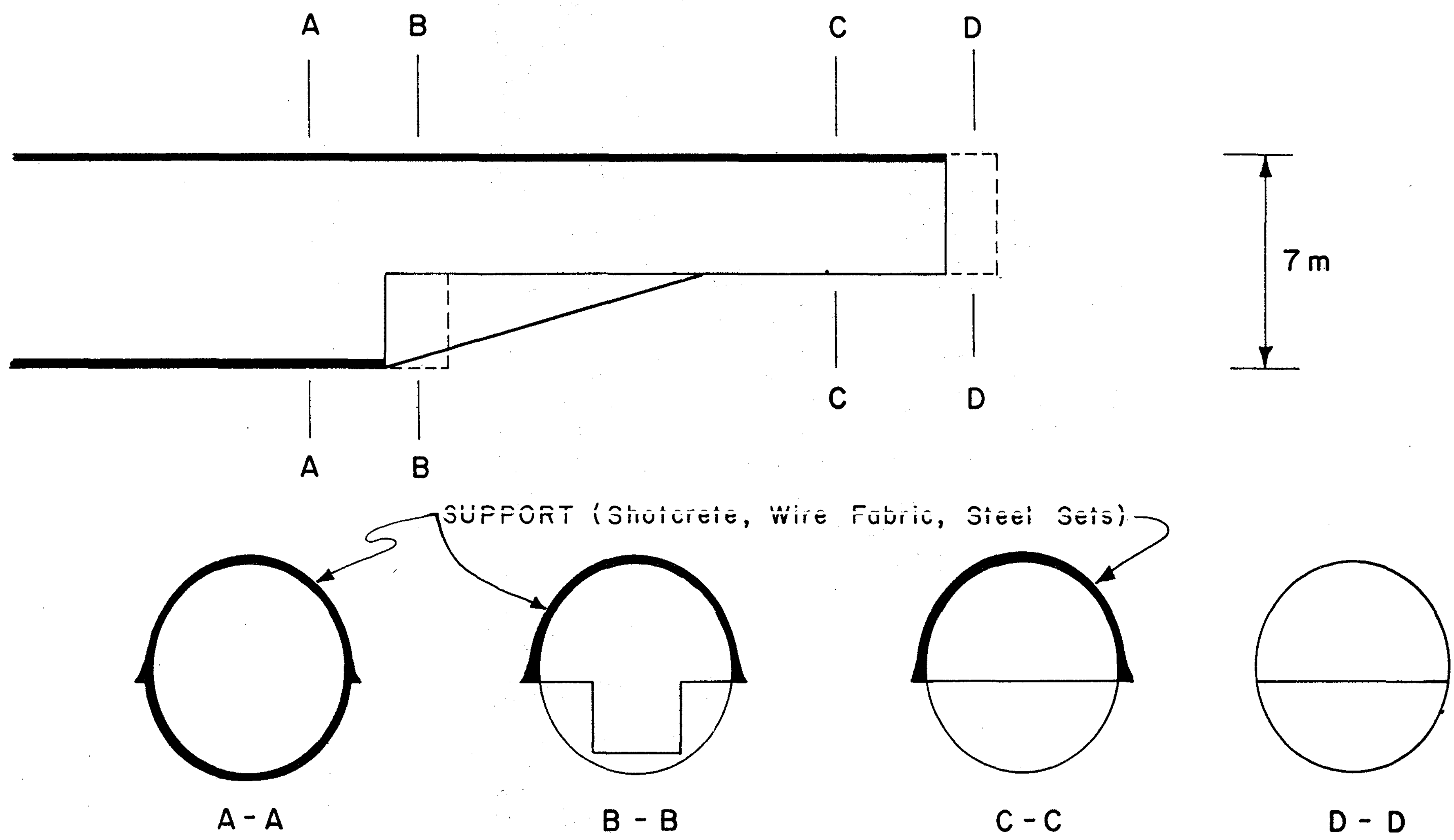


FIGURE B-1.4 RAMP TUNNELING METHOD

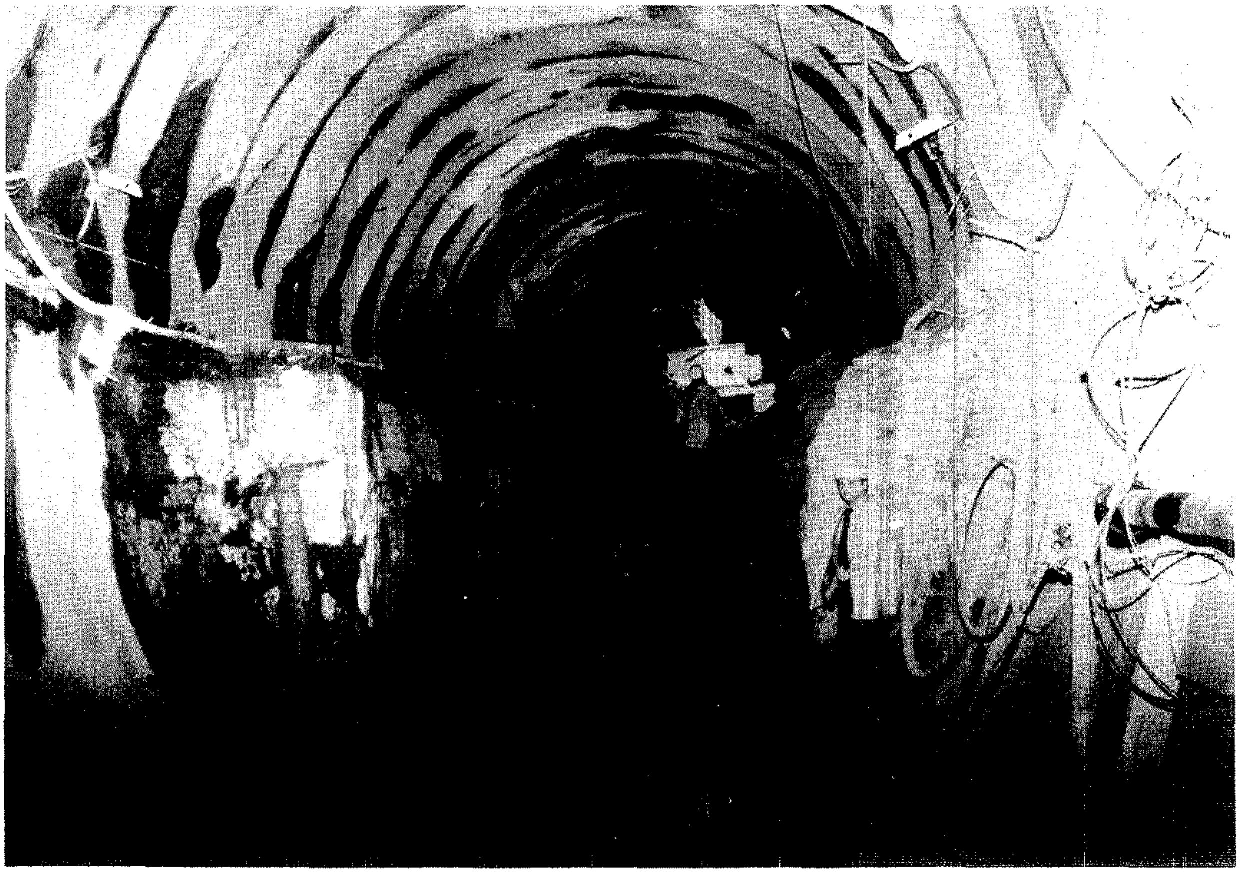


FIGURE B-1.5 RAMP CONSTRUCTION METHOD (PHOTO, W. STEINER, 197[illegible] precedes the invert closure by approximately 20 m; also, the heading is linked to the invert by a central ramp, leaving the sides of the bench. The bench and invert are excavated simultaneously behind the ramp. This method allows for a complete separation of heading and benching operation. Excavation and placement of support alternates between heading and bench (and invert).

The construction sequence details are as follows:

(i) The heading is excavated by means of a partial face tunnel boring machine (TBM) of the type Westfalia-Dachs. The muck is directly loaded onto a dump truck waiting on the ramp.

(ii) The heading is supported by steelsets (spaced 0.9 to 1.5 m , GI 100 $\cong$ W4 x 13), one layer of welded wire fabric (3 kg/m^2) and shotcrete (15 cm). The support of the heading is widened at the springlines to form a footing and a longitudinal beam.

(iii) Simultaneously with (ii) the bench and invert are excavated with the TBM.

(iv) The invert is closed with shotcrete and wire mesh, and the cycle starts again.

When water percolates into the heading, forepoling plates are placed and covered with shotcrete (Figure B-1.6) as a special precaution against ravelling; when no forepoling plates are needed, a fine wire mesh of stretchable metal is

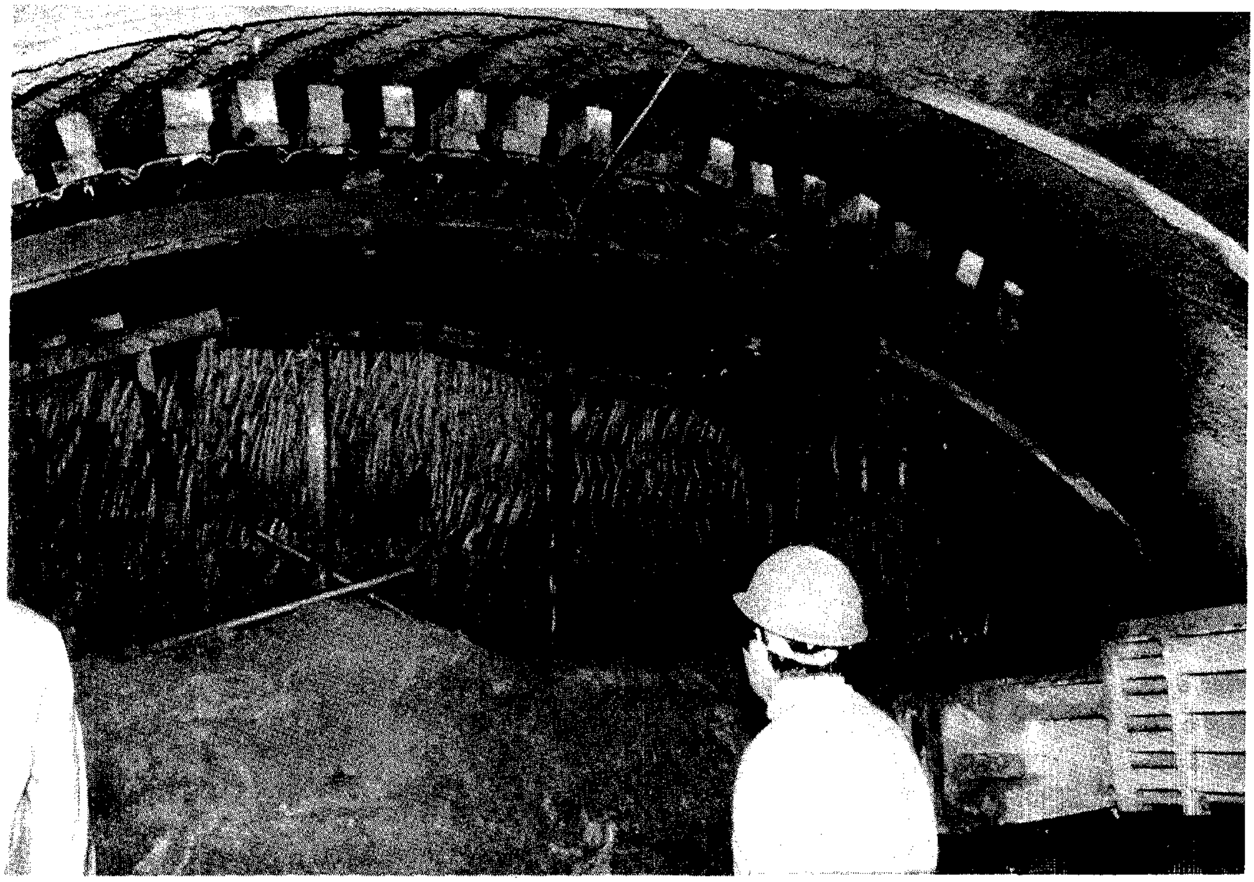


FIGURE B-1.6 FACE IN TOP HEADING OF RAMP-CONSTRUCTION METHOD (SUPPORTED WITH FOREPOLING PLATES) (PHOTO W. STEINER, 1978) placed on the ground in the crown immediately after excavation and nailed to it, whereupon shotcrete is placed. To prevent the invert from uplifing, the water table is lowered by means of wellpoints from the tunnel. Immediately north of the Königsplatz Station, the tunnel does not lie completely in the tertiary marl and the construction procedure shown in Appendix A-1 was applied: the heading was temporarily lowered and the upper part of the tunnel was excavated once groundwater control measures (controlled dewatering or grouting) had taken place. Figure B-1.7 is a photograph of the north end of the Königsplatz Station with the running tunnels. For both tunnels the heading had been lowered according to the above-mentioned procedure; the right tunnel (in Figure B-1.7) has already been brought to its final size.

The face is shotcreted during work stoppages like weekends or holiday periods, or in case the stand-up time is too short. (As W. Steiner's visit fell during the Christmas holiday period, the faces of these tunnels were covered with shotcrete and excavation work had not started again.

The southern part of section 16, illustrated on Figure B-1.3, starts from the access shaft "Sophienstrasse", which is 35 m deep (120 feet) and has horizontal dimensions of 30 by 30 m (100 feet by 100 feet).

It is supported by outward sloping tangent piles. This is required due to surface constraints; the road is not as wide as the maximum width of the tunnels (Figure B-1.3).

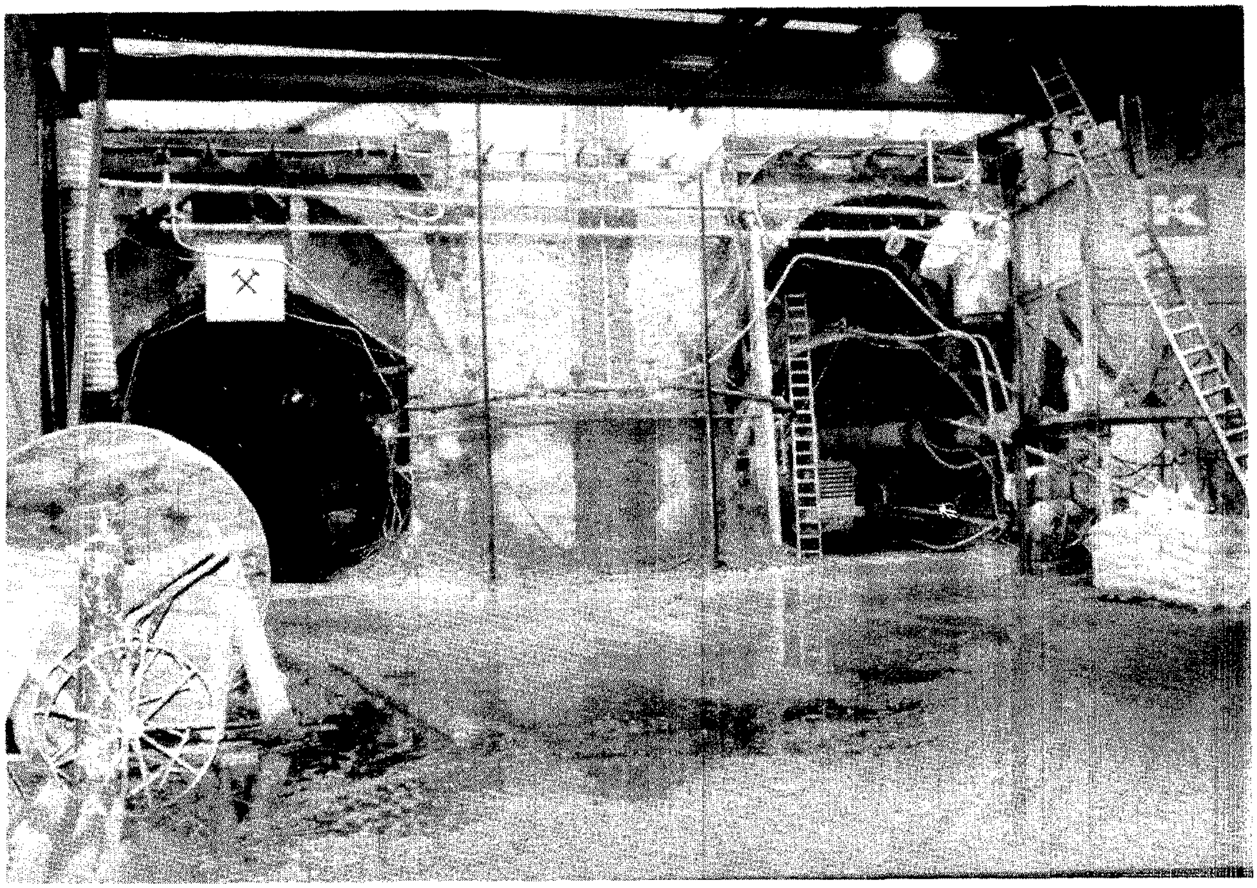


FIGURE B-1.7 NORTHERN END OF KÖNIGSPLATZ STATION WITH RUNNING TUNNELS (BOTTOM HEADING TO PASS UNDER LOCAL DEPRESSION IN MARL SURFACE) (PHOTO, W. STEINER, 1978)

Construction simultaneously progresses to the south (Lines 8 and 1, as indicated by the arrows on Figure B-1.2), to the north (Line 8) and to the north-west (Line 1). The single track tunnels (Line U1) are excavated by the ramp-NATM. However, for line U8 and wye-track, a central three track tunnel (Figures B-1.1, B-1.3 and B-1.8), whose cross-section is 150m^2 (width 16 m, height 11 m), is constructed with the sidedrift method. Figure B-1.8 shows a cross-section (sketch) and Fig. B-1.9 a photograph of this three track tunnel. At the time of Steiner's visit (January 5, 1978), the two side drifts (1 in Figure B-1.8) of the southbound tunnel had reached the north diaphragm wall of the excavation of a the Hauptbahnhof station (Figure B-1.2), passing under the existing tunnel of the S-Bahn (Regional Rapid Transit) without special measures and disruption. The side drift support consists of shotcrete, steelsets, wire fabric and bolts. The bolts were not placed with ongoing excavation of the sidedrifts; rather, they were placed once this excavation was completed. The bolts at the springline (Figure B-1.8, length = 4 m, diameter = 20 mm, spacing 1.5 by 1.5 m, capacity = 20 metric tons, prestress = 14 metric tons) had been placed at the time of this visit. Bolts were only placed at the springlines to avoid any protrusion of their ends into the ends into the quarternary soil (Figure B-1.8) which could lead to groundwater penetration into the bolt holes and cause a loss of ground; a minimum "design overburden" for the bolt ends of 0.5 m has been specified.

Light steelsets were placed on the outer sides of these

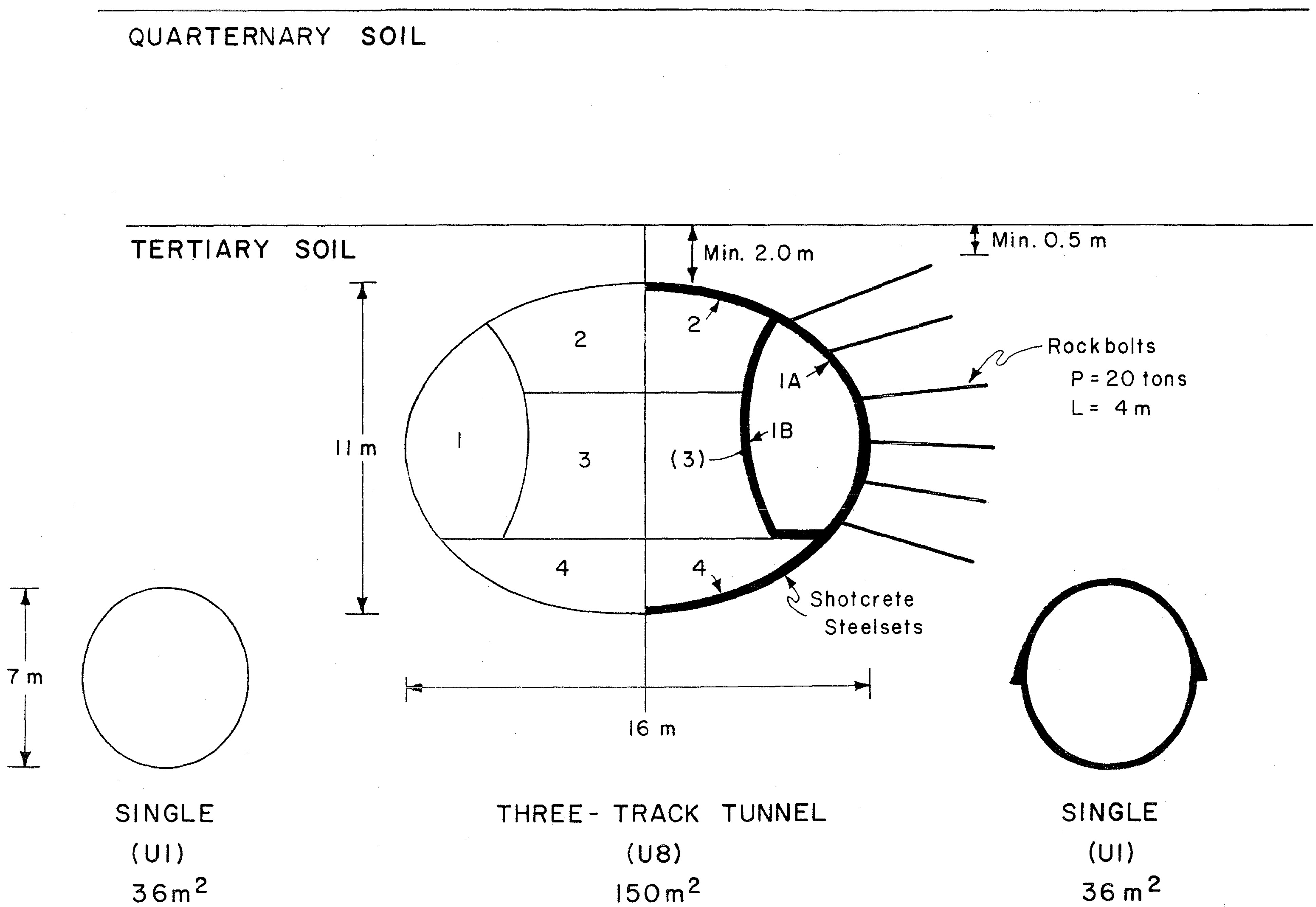


FIGURE B-1.8 SCHEMATIC CROSS-SECTION THROUGH SECTION 16, sidedrifts and covered by shotcrete (30 cm). The steelsets (GI 120, weight = 26.4 kg/m, approximately equivalent to W5 x 16) are spaced at 90 cm. The upper ends of these steelsets in the sidedrifts have been prepared to form a base for the steelsets that will be placed in the crown. These ends are protected by styrofoam in order that they are not covered by shotcrete.

The inner sidewalls of the sidedrifts consist of unreinforced shotcrete (15 to 20 cm) without steelsets, since they will be removed during excavation of the central core. The construction of the southbound top-heading ("2" in Figure B-1.8) has just started, and the excavation and support has advanced a few meters, as illustrated in the photograph (Figure B-1.9).

The construction of the northbound three track tunnel was less advanced; excavation of the side drifts was not yet complete. However, the dimensions of this northbound three-track tunnel will actually be larger because the central wye-track will be level and the two outer running tracks are climbing (Figure B-1.3). In this northbound three-track tunnel, the construction procedure will be somewhat different. The two sidedrifts are larger and have the size of a single track tunnel. Cast-in-place concrete walls will separate the tracks. In the sidedrifts, a slot in the shotcrete is left where the concrete walls will be placed and keyed into the shotcrete; the wirefabric, however, is continuous through this slot.

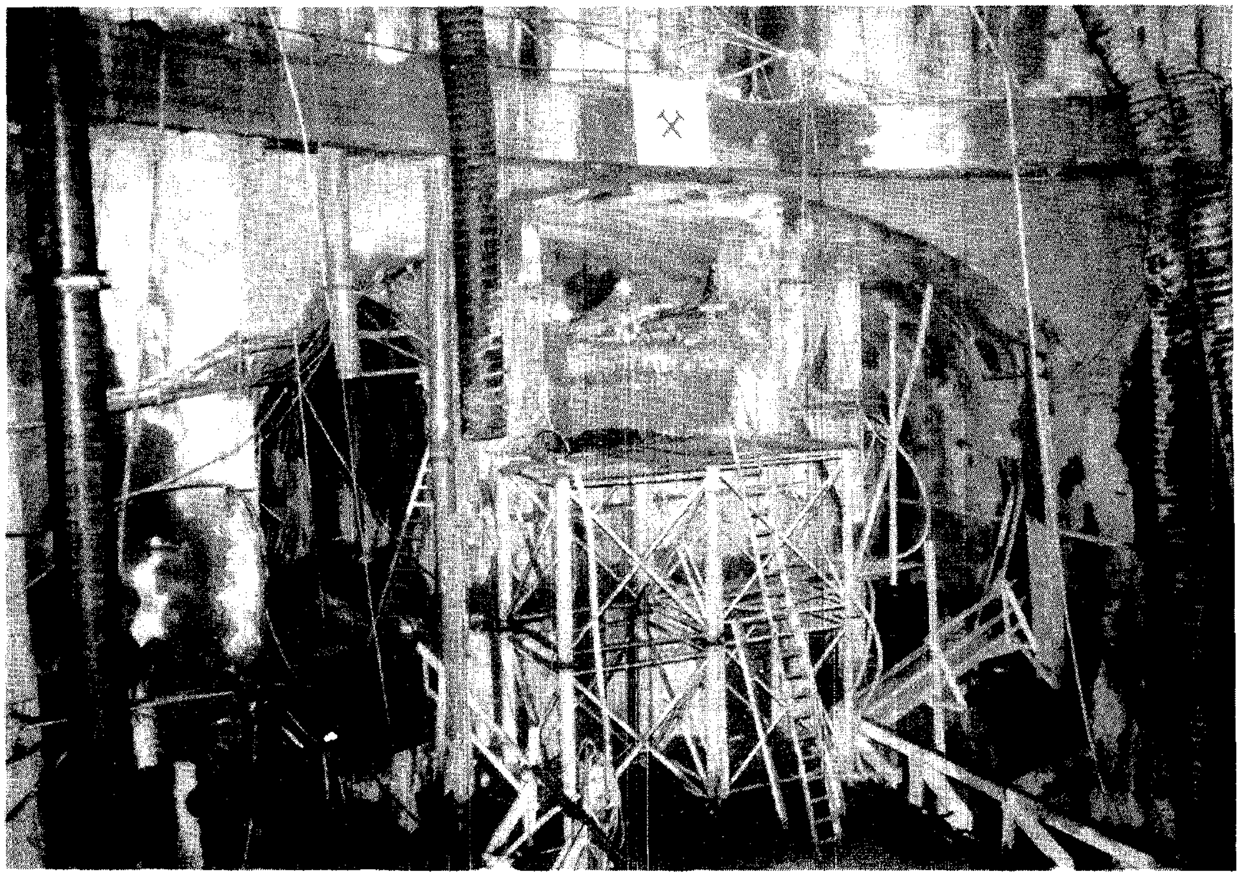


FIGURE B-1.9 ACCESS SHAFT "SOPHIENSTRASSE" WITH THREE-TRACK TUNNEL TO THE SOUTH (PHOTO, W. STEINER, 1978)

Following the three track tunnel, Line U8 will continue in two single track tunnels to the Königsplatz Station with the Ramp NATM. The two single track tunnels of Line U1 (Figure B-1.3) are driven in northwesterly direction one level below. The northwestbound eastern tunnel (U1, Track 1) crosses under the three track northbound tunnel. The tracks of lines U8 and U1 thus form a mined subterranean fly-over. The construction procedure for these lower level single track tunnels is the Ramp-NATM described earlier.

2) Monitoring Measurements

At the stage of excavation during the visit, the surface settlements south of the shaft in the Sophienstrasse were on the order of 3 cm (Figure B-1.3). Convergences measured in single track tunnels extending south from the shaft are approximately one centimeter; no further details are presently available.

In the large three track tunnel (Figure B-1.10), the convergence of the final cross-section during excavation of the heading and bench will be monitored as shown in Figure B-1.10. For this purpose, a connecting borehole will be drilled between the two side drifts. Additional convergence measurements will be taken in the three-track cross section, as shown in Figure B-1.10.

At present, no inclinometers have been used to monitor lateral displacement; they are considered to be too expensive. Settlements at depth are monitored with extensometers from the surface as illustrated on Figure B-1.10.

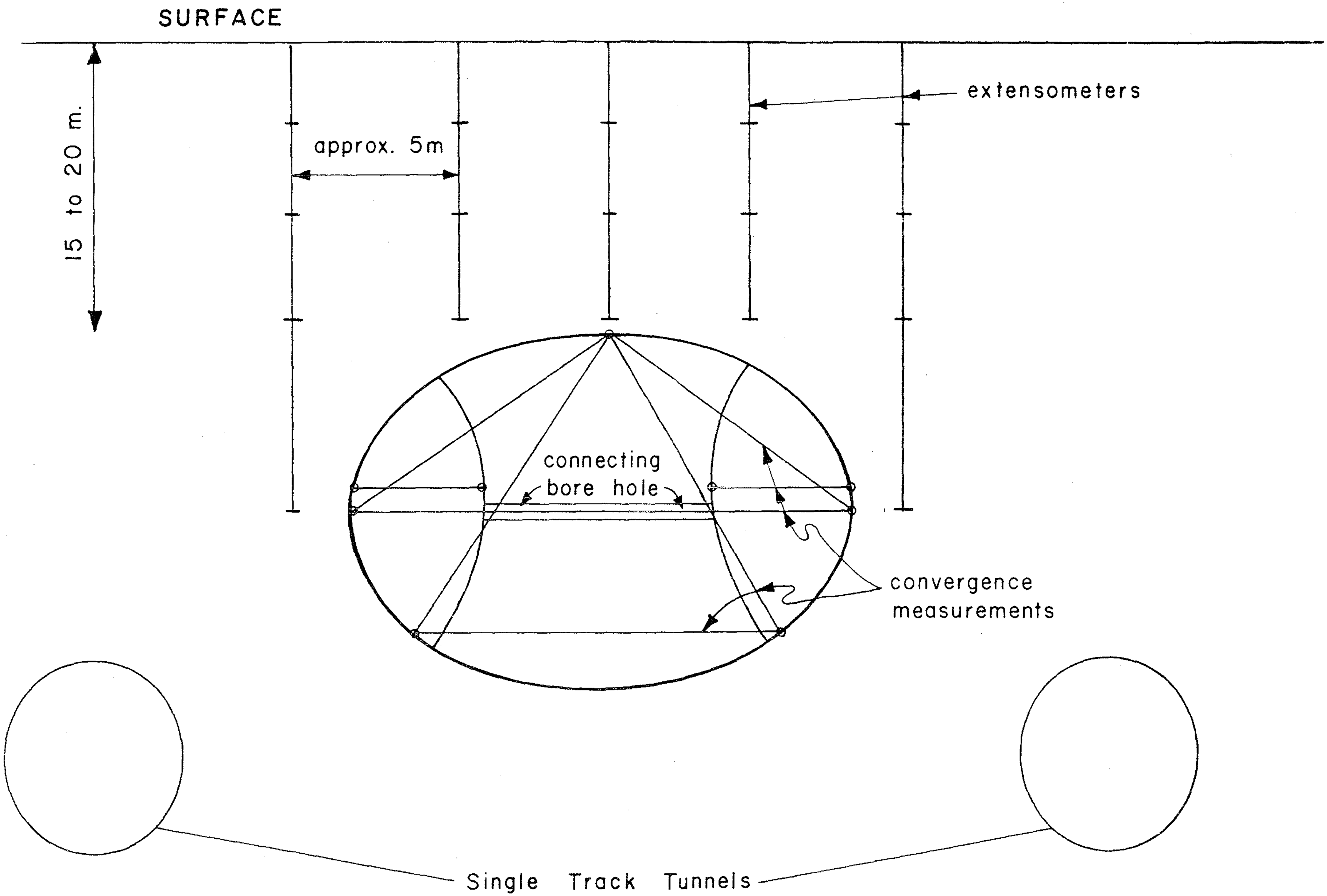


FIGURE B-1.10 SCHEMATIC MONITORING CROSS-SECTION IN SECTION 16 OF LINE U8/1

APPENDIX:	B-2
SITE:	Section 24 (Mined tunnel)
OWNER:	City of Essen
DESIGNER:	Alternate Proposal
CONTRACTOR:	Beton-und Monierbau in joint venture
DATE OF VISIT:	19th January, 1978
PERSONS MET:	Dr. H. Wagner, Head Design Department, Beton-und Monierbau, Innsbruck
GUIDE:	Mr. Wachtlechner, Site Manager, Beton-und Monierbau

1) Introduction

Section 24a of the subway (Stadtbahn) of the City of Essen comprises the University Station, built in an open-cut, and the two adjacent south-bound single track tunnels of 36 m^2 cross-sectional area and a length of 305 m each (Figure B-2.1).

The section is built by a joint venture, and the responsibilities for parts of this section are assigned to individual partners of the joint venture. Beton-und Monierbau builds the two mined tunnels.

In the following, the ground and surface conditions are described first, followed by discussions of technical problems, site organization and monitored performance.

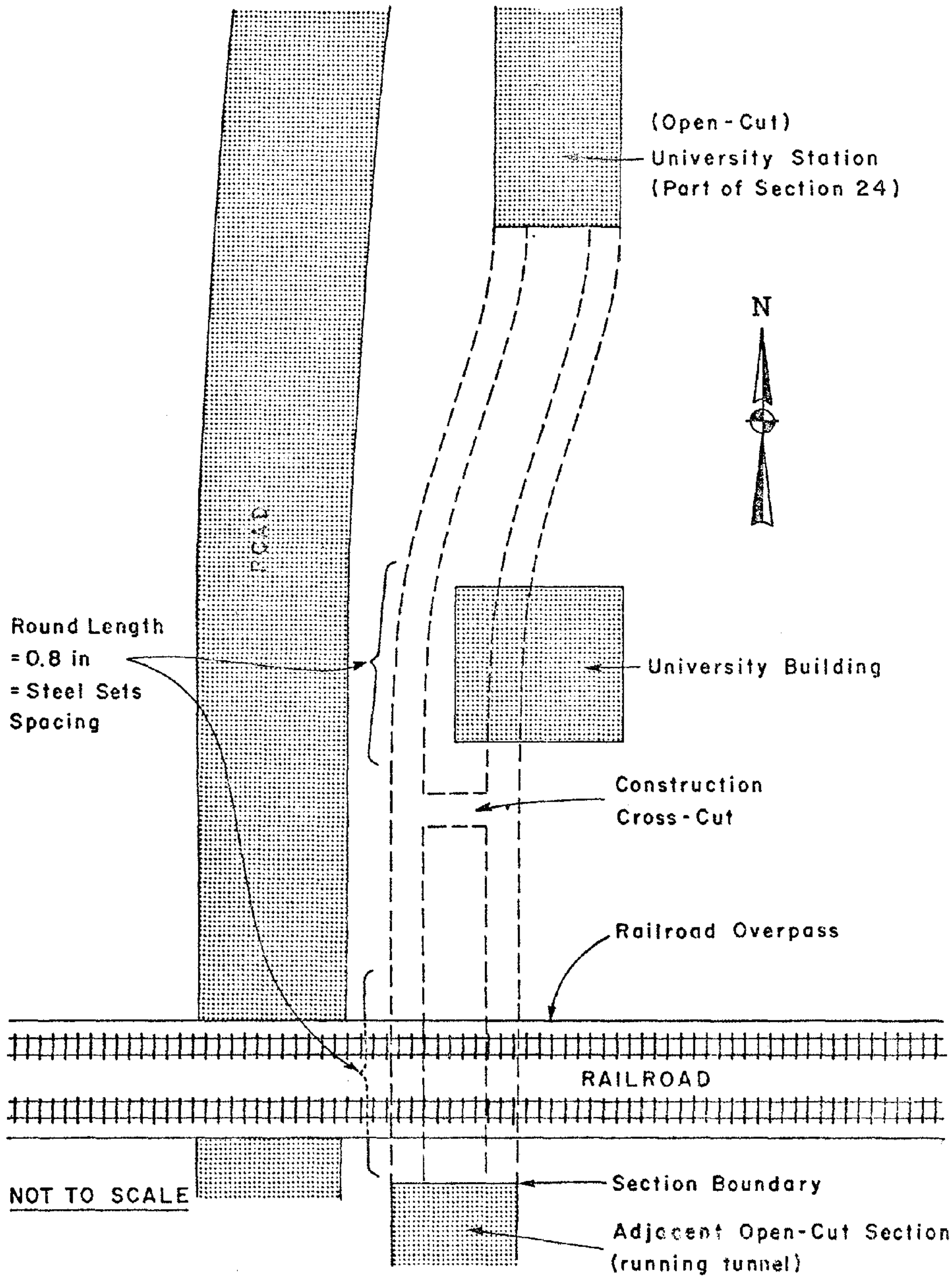


FIGURE B-2.1 SCHEMATIC PLAN VIEW OF SECTION 24 IN ESSEN

2) Ground and Surface Conditions

The two, mined single-track tunnels, start in the open cut of University Station and end in an open-cut running tunnel (Figure B-2.1). The mined tunnels pass under a staircase tower of a university building and also under a major railroad line on an embankment. The subway runs parallel to a road, which passes under the railroad close to the location where the subway will pass under the embankment. This road underpass is very sensitive to settlements (the type of the bridge structure could not be determined), particularly since one abutment will be more strongly influenced. Ground conditions are a sandy clay (mica, green sand marl) with a modulus of elasticity $E = 300 - 500\ kg/cm^2$. The permeability is low; the ground water table has been lowered with gravity wells below the invert.

3) Work Progress, Method of Excavation

The tunnels are built according to the NATM with steel-set and shotcrete support. Work for the tunnels started October 3, 1977, and on January 20, 1978, 200 m of each tube had been excavated. The average daily rate of advance is 3.5 m; the maximum rate of advance was 7.5 m at the beginning of the tunnels. The rate of advance dropped off as the tunnel advanced because the hydraulic crawler shovel excavates alternatively in one of the tubes (as shown in Figure B-2.2), and the time to move it from one tunnel to the other

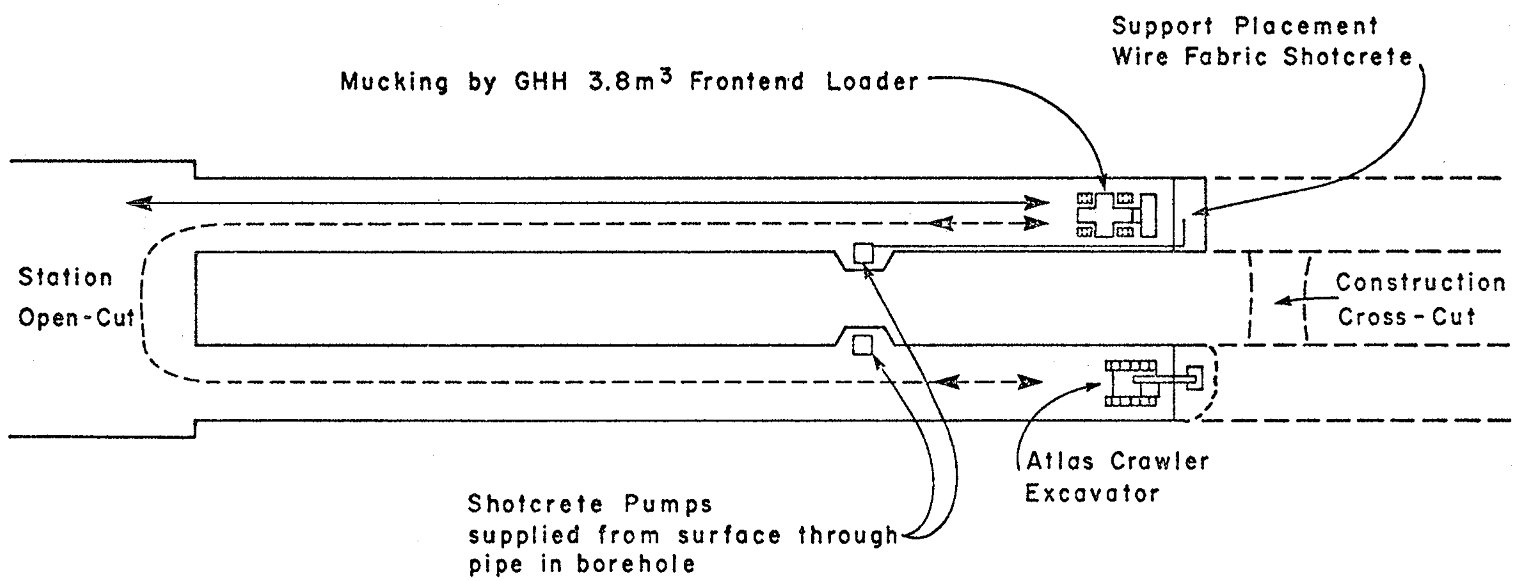


FIGURE B-2.2 CONSTRUCTION PROCEDURE IN SECTION 24, ESSEN increased with tunnel length (with 200 m long tunnels, 20 min. are required to move it from one tunnel to the other). The excavated ground is temporarily dumped in the invert. Mucking also alternates between tubes and is performed by 3.8 m^3 frontend loaders; once the excavator has moved to the other tunnel, the muck is then brought to the open-cut station. Immediately after the crawler excavator has completed the excavation of a section of heading, bench, or invert (see Figure B-2.3), support is placed. The support is a combination of shotcrete, steelsets, wire fabric and bolts. The support quantities vary depending on the surface conditions, as listed in Table B-2.1 (in areas where there are no buildings in the vicinity of the tunnel the round length was greater and the support lighter.) A mini-wall beam (Vorpfändschienen) ties the steelsets together. The mini-wall beam is bolted to the vertical steel sets. Thus, the support in the heading forms a cantilever. To provide further protection for the support in the heading, which is particularly vulnerable during excavation, rock bolts of 3 m length tie the steelsets to the ground. The danger of accidentally tearing off the support with the excavator is thus greatly reduced.

As mentioned earlier, it takes 20 minutes for the crawler excavator and crews to move from one tube to the other (with a tunnel length of 200 m), and this results in a loss of production time and a drop in advance. The contractor is thus

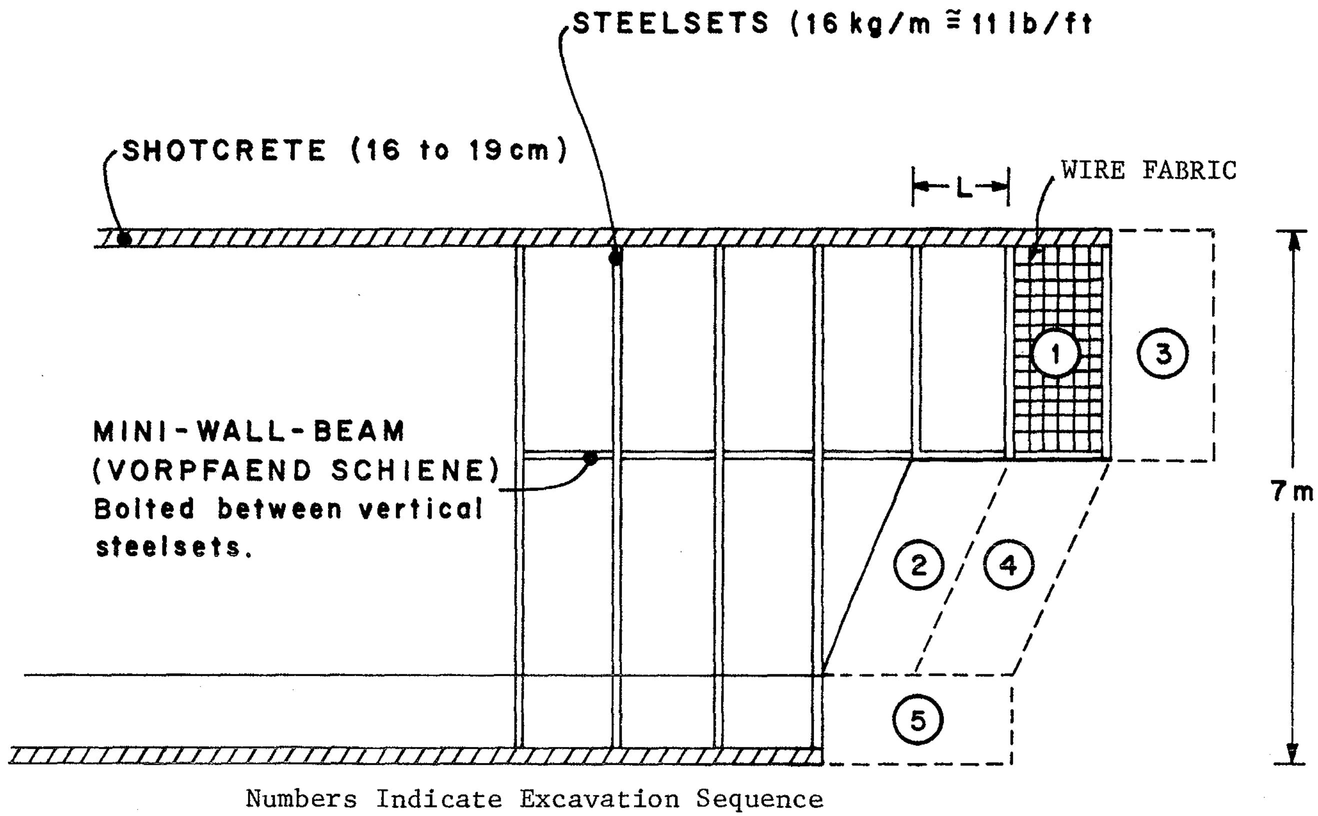


FIGURE B-2.3 EXCAVATION AND SUPPORT PROCEDURE AT THE FACE

TABLE B-2.1 INFLUENCE OF SUPPORT AND EXCAVATION PROCEDURE ON DEFORMATIONS

TYPE	ROUND LENGTH = set spacing	STEEL SETS	WIRE FABRIC	SHOTCRETE	RING CLOSURE DISTANCE FROM FACE	SURFACE SETTLEMENTS	CONVERGENCE
No Buildings	1.25 m	16 kg/m in heading & at springline	1 layer	nominal > 16 cm (effective ≃ 20 cm)	6 m	4 cm	1.0-1.5 cm
Under Buildings and Railroad	0.85 m	16 kg/m circumferential	2 layers	nominal > 19 cm (effective ≃ 22 cm)	3.2 m	2 cm	1.0-1.5 cm

excavating a temporary cross-cut, costing on the order of 40,000 to 50,000 DM (20,000 to 25,000 U.S. $) compared to an estimated savings of 145,000 DM ($72,000 U.S.) due to higher productivity. Furthermore, the wear and tear of the crawler is reduced. Mucking with the 3.8 m^3 frontend loaders should not affect the advance rate, according to the site engineer.

There is one shotcrete pump with supply silo placed in a niche in each tunnel. To limit the requirements for compressed air, the shotcrete pumps are never more than 100 m behind the face. The shotcrete silos are supplied through a cased drill-hole from the surface with dry-mix shotcrete.

Table B-2.2 is a listing of the major equipment and personnel for this tunnel. The crew consists of 10 men per shift, with two shifts of 10 hours per day. This crew performs excavation, support, mucking and surface supply operations. The crew members performing a specific task move from one tube to the other. Many of the crew members are Austrians who were brought to Essen by Beton-und Monierbau.

4) Cost Data

At Beton-und Monierbau's tunneling headquarters in Innsbruck, W. Steiner received cost and cost component data for this tunnel. Man hours per lineal meter of single tunnel are 35 h/m for excavation and initial support. This compares favorably with 150 manhours/meter in section 25 of the Frankfort subway, where the NATM was used the first time in subway construction. However, the ground conditions in Frankfurt are

TABLE B-2.2 EQUIPMENT & PERSONNEL, ESSEN SECTION 24

EQUIPMENT

1	Hydraulic Crawler Excavator Atlas 1602 (150 hp)
1	Front End Loader Gute Hoffnungshutte with 3.8 m^3 bucket
2	Shotcrete Pumps GM57 with intermediate silo and accelerator mixer (one for each tunnel)
1	Truck on surface for shotcrete supply + compressor

PERSONNEL

1	Excavator Operator
1	Loader Operator
1	Shotcrete Pump Operator
6	Workmen in tunnels (3 in each)
9	
1	Foreman
10	per shift

2 shifts at 10 hours per day

somewhat less favorable (medium clay with hard limestone beds which require blasting).

The average hourly wage rate was 31.49 DM, which includes premiums and fringe benefits and apportioned foremen's wages. The individual components are presented in Table B-2.3.

The cost components of one lineal meter of tunnel excavattion and initial support without final liner are shown on Table B-2.4. The itemized costs do not include rentals, overhead and the cost of energy. The total costs for initial support and excavation are estimated based on the usual percentage of labor costs. These data are for a very efficient and well-organized construction site and ought to be generalized with caution.

5) Performance Data

The data presented on Table B-2.1 illustrate the influence of support delay and support stiffness on the surface settlements. The larger round lengths (1.25 m) and less stiff support (see Table B-2.1) were used in non-built-over areas (see Figure B-2.1) and resulted in settlements on the order of 4 cm. The shorter round length and stiffer support, applied where the tunnel is located under a staircase tower of a university building, resulted in maximum settlements of 2 cm.

For the crossing of the railroad embankment (Figure B-2.1), the support was increased and the round length decreased,

TABLE B-2.3 WAGE RATES FOR TUNNEL CONSTRUCTION WORK IN ESSEN, SECTION 24 (1 U.S. $ = 2.00 DM)

Item	DM	U.S. $
Base Pay	10.84	5.42
Overtime surcharge	0.75	0.38
Surcharge for difficulties	0.94	0.47
Apportioned Foremen wages	3.08	1.54
Fringe Benefits	3.78	1.89
Overhead	1.78	0.89
Social Payments	10.32	5.16
Total	31.49 DM	15.75

TABLE B-2.4 COST PER METER IN INITIAL SUPPORT OF SINGLE-TRACK TUNNELS (1 U.S. $ ≃ 2.00 DM)

Item	DM	U.S. $
Labor	1300.	650
Steel (sets, wire fabric)	461.	230.5
Shotcrete	693.	346.5
Accelerator for Shotcrete	100.	50
Estimated Cost for excavation and initial support (based on Labor = 30% of total cost)	4333.00 DM/m	2166.5 $/m

as under the university building (Table B-2.1). Monitored convergences were in the order of one to one and a half centimeters.

APPENDIX:	B-3
SITE:	Section 17a Rüttenscheider Strasse
OWNER:	City of Essen
DESIGNER:	Alternate Proposal
CONTRACTOR:	Joint Venture Hochtief AG Dyckerhoff & Widmann AG Bilfinger & Berger Bau AG Wayss & Fretag AG
DATE OF VISIT:	21 January, 1978
PERSONS MET:	Mr. Kondmann, Deputy Site Engineer for Joint Venture

1) Introduction

The visit to section 17 of the Essen Subway was organized by Dr. Wagner of Beton-und Monierbau through Hochtief AG, who is a partner in the joint ventures of section 17 and 24. Hochtief has the technical direction of section 17. In this section, innovative tunneling has been used, a blade shield with immediately following cast-in-place liner of impervious concrete. This appendix is based on the interview on the site as well as a recently published article in Tunnels and Tunneling (Gruner, 1978). In the following lines the ground conditions will be described, then the method of construction, site organization, and finally, performance monitoring.

2) Geologic and Surface Conditions

Section 17 is 460 in long; it forms an extension of an existing LRT line. The tunnel is shallow and the minimum cover over the crown in only 5 m (Figure B-3.1) below the pavement in a street with buildings on both sides.

A geologic section is shown in Figure B-3.3. The top layer is a clay sand (marley sand) underlain by green sandy clay (green sand, marl underlain by shale and sandstone). The water table is 5 m above the invert. As can be seen from F. B-3.3 (bottom), the interface green sandy marl-sandstone varies, and thus mixed face conditions were encountered in the tunnel. At the end of the section, the sandstone reaches the crown of the tunnel.

3) Technical Problems

The blade shield is part of an alternate proposal submitted by the joint venture under the direction of Hochtief AG. The official proposal called for a circular shield and a liner of cast-iron segments.

A longtitudinal section of the blade shield (Manufacturer: Westfalia Lüen) is presented in Figure B-3.2. A frame of horseshoe shape (Figure B-3.1) carries 40 forepoling plates (32 in the crown and springlines, 8 in the invert). The forepoling plates are called cutters in Figure B-3.2. Each plate can be individually advanced with a 60 metric ton

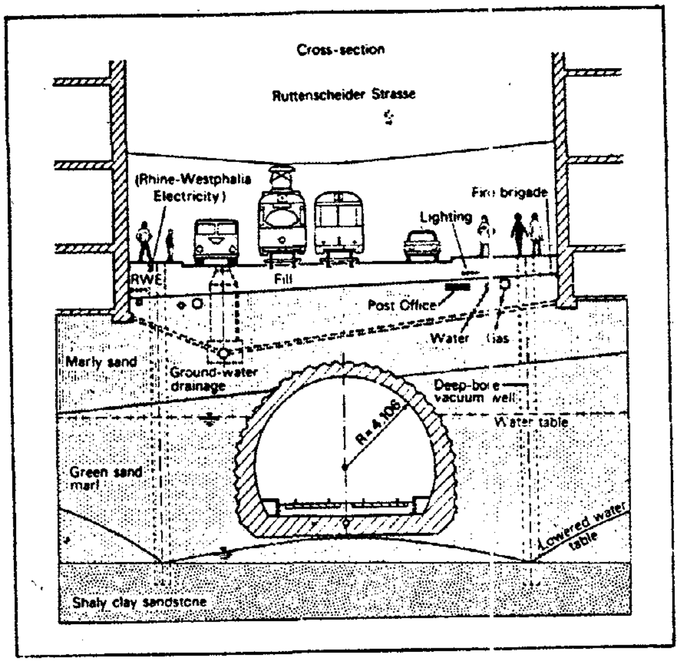


FIGURE B-3.1 CROSS-SECTION, ESSEN SECTION 17, RUTENSCHEIDER STRASSE (FROM GRUNER, 1978)

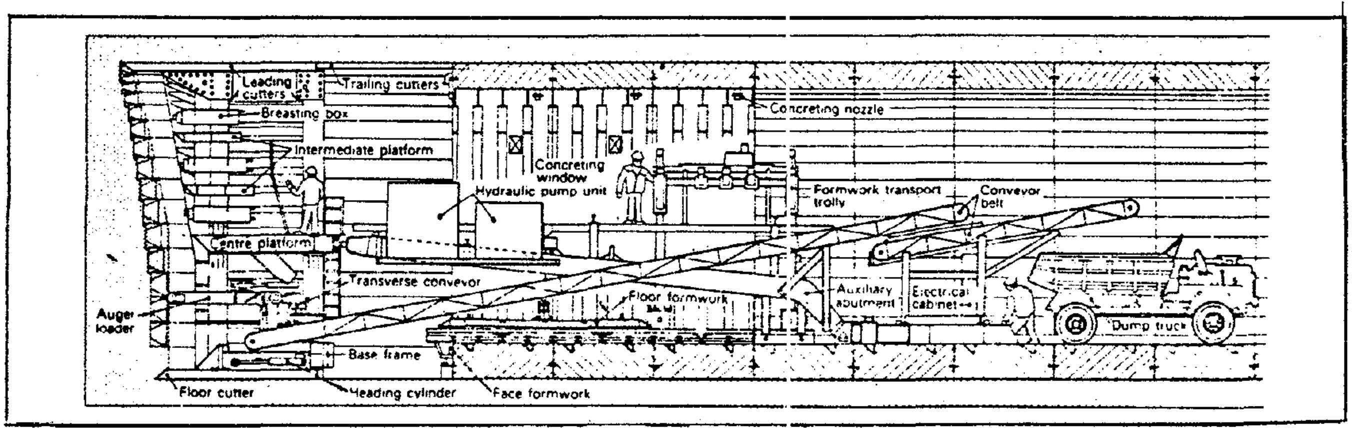


FIGURE B-3.2 LONGITUDINAL SECTION OF BLADE SHIELD, ESSEN SECTION 17 jack (616 kN). Since each plate is advanced individually at a time, the jacking forces are taken by the shield only and do not exert forces on the freshly-poured liner. The face of the tunnel may be supported by intermediate platforms in the shield, and if necessary, the face may be entirely breasted. An "auxiliary abutment" has been designed to support the face in case of instability; it transfers the forces from the face to the already concreted floor.

The liner is poured in sections of 2.5 m length under the protection of the trailing plates. The weight of the reinforcement is 3 metric tons per section of 2.5 m length (= 1.2 t/m). The placement of the reinforcement is quite tedious, since it has to be brought in from the back between formwork for the liner and the shield machinery (Figure B-3.3). Once the reinforcement has been placed, the last ring of the formwork is moved forward to the front position and the concrete is pumped in through "concreting windows" at the springline and a valve in the crown. To prevent the trailing plates from sticking to the concrete during hardening, they are advanced so that only one meter of their tail end is in contact with the freshly poured concrete. The tail void is grouted with a lime mix.

The water table is approximately 5 m (Figure B-3.1) above the invert. Dewatering by vacuum wells was necessary. The wells are spaced at 10 m along the alignment and placed at a minimum distance of 3 m from the tunnel; a total of 92 wells were sunk. Dewatering started 30 m ahead of the face

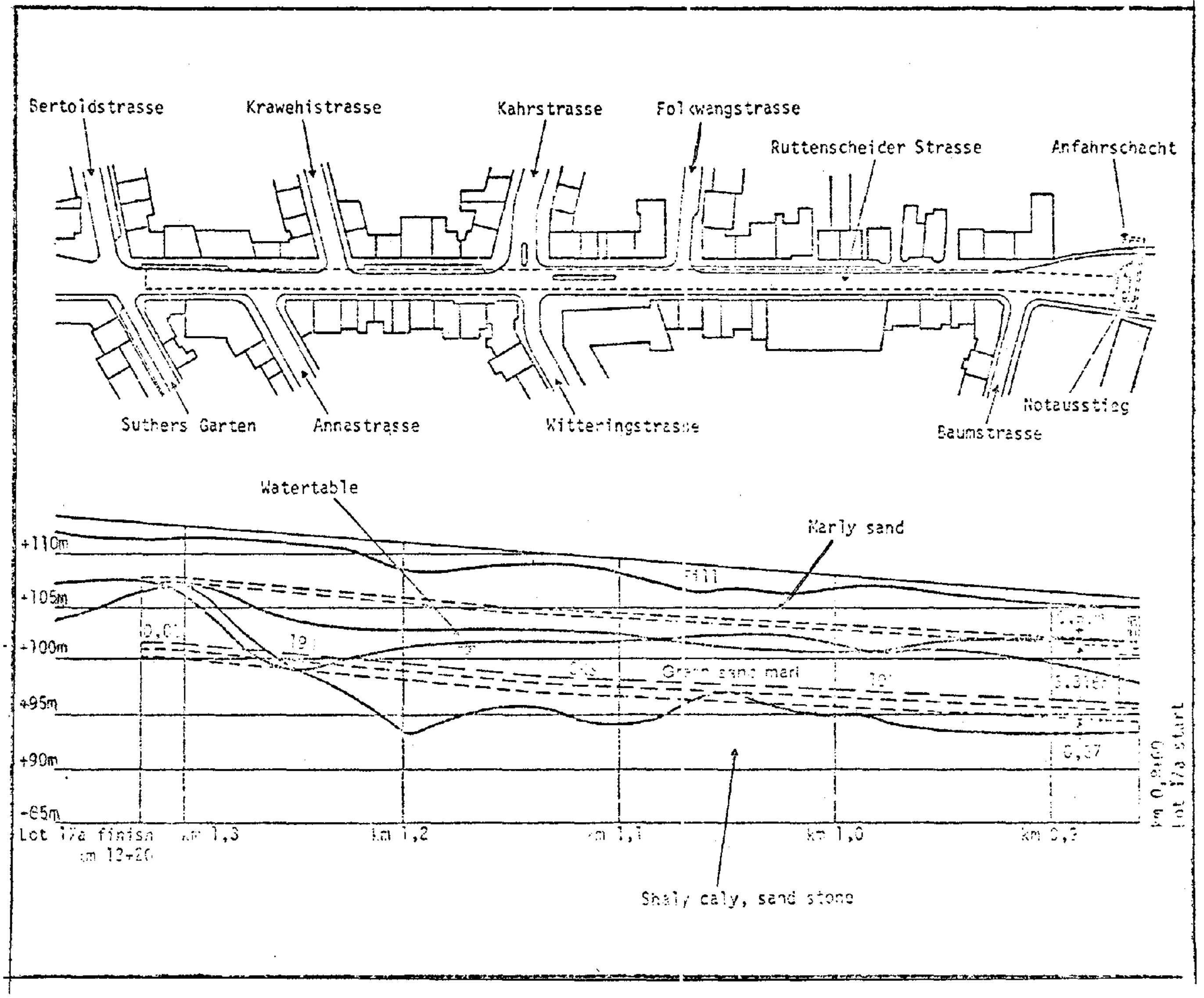


FIGURE B-3.3 PLAN VIEW AND GEOLOGIC SECTION OF SECTION 17, ESSEN (FROM GRUNER, 1978) and continued for 30 m behind the face.

4) Site Organization

Work progresses 5 days a week with two shifts of 10 hours each. The crew size is approximately 17 men per shift. During the initial stage, the advance rate was only one ring (2.5 m) per week, however, this increased to 5 rings a week (12.5 m week). Access to the site is from an access shaft at the northern end of the tunnel.

5) Performance Monitoring

Surface settlements were measured, supplemented by three principal monitoring cross-sections (with quadruple extensometers). The settlements were concentrated in the vicinity of the tunnel, and no damage to buildings was recorded. The settlements reach a maximum of 8 cm and on the average 5 to 6 cm above the crown; however, they vary quite erratically. Gruner (1978) attributes the concentrated erratic settlement (in monitoring section 1) to the mixed face conditions. However, we believe that the construction procedure has had a significant influence. In particular, incomplete grouting of the tail void (of the blade shield) may have led to settlements. Also, the settlements have been concentrated immediately above the shield within the area of the street.

APPENDIX: B-4

SITE: Sections A2, A3/5
Bochum Subway

OWNER: City of Bochum

DESIGNER: Subway Department, City of Bochum, with the collaboration of H. Waring, Consulting Engineer (alternate proposal by Beton-und Monierbau)

CONTRACTOR: Joint venture with:
Beton-und Monierbau,
Thyssen Schachtbau,
A. Pape, KG.

DATE OF VISIT: 20th January, 1978

PERSON MET: Dr. Wagner, Head of Design Department, Beton-und Monierbau

1) Introduction

Bochum, a city in the Ruhr district, is building a light rapid transit network which will be part of the overall system planned for the Ruhr district. The first sections have been completed in the inner city and will first be used by streetcars. In the two sections visited, track installation is under way at the present time. Section A-2 is described in detail in Einstein et al. (1977) and only the location is shown here. Section A3/5 will be described in more detail in this appendix. For section A3/5, Dr. Wagner was project engineer for the contractor, Beton and Monierbau, who built this section based on an alternate proposal. This appendix is also based on a brochure by the City of Bochum and an article by Müller and Spaun (1977).

2) Description of the Sections

Figure B-4.1 shows the general situation of sections A2 and A3/5. The alignment is S-shaped with curves of varying radii. In addition, the elevation varies considerably; thus this tunnel is continuously changing in direction and in cross-section (shape and area). Also, twin track tunnels are followed by single track tunnels, bifurcation and a station. Figure B-4.2 gives a more detailed plan view of section A3/5 and of the existing buildings. Figure B-4.3, B-4.4, and B-4.5 show some typical cross-sections of the tunnel in section A3/5. The minimum distance from the tunnel crown to a footing was 3.5 m. None of the buildings was underpinned, however. Initially, the station "Berlinerplatz" was planned to be constructed in an open-cut (official design). The contract was awarded for a mined excavation based on an alternate proposal (NATM) submitted by the contractor. (Figure B-4.2 shows how an open-cut construction would have virtually shut-down the shopping district in the Brüderstrasse and Kortumstrasse). Access to the tunnel was by a single shaft at the northern end of the station "Berlinerplatz". Driving of the running double-track tunnel proceeded to the

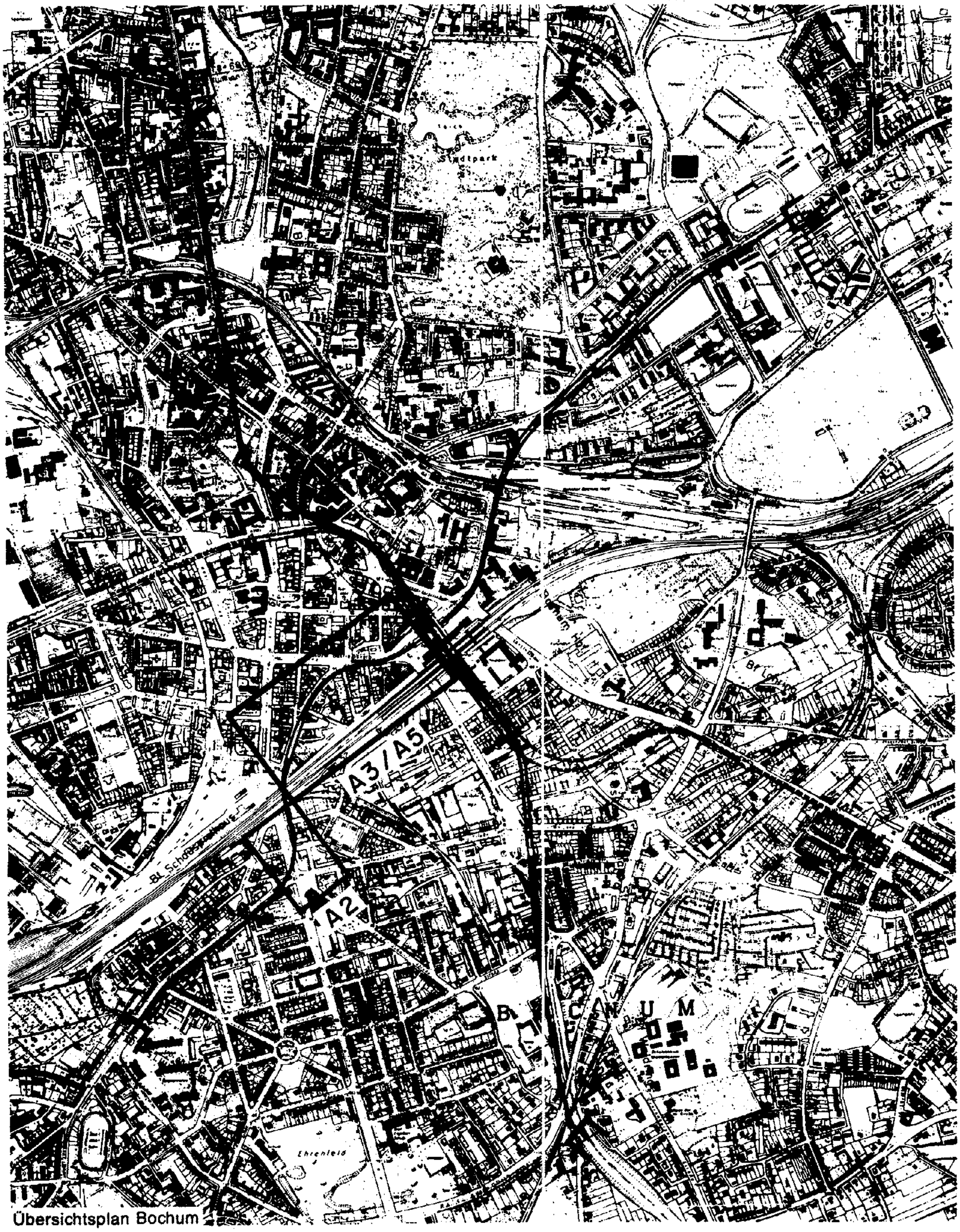


FIGURE B-4.1 MAP OF BOCHUM SHOWING LOCATION OF SECTIONS A2 AND A 3/5 (FROM BROCHURE PUBLISHED BY CITY OF BOCHUM)

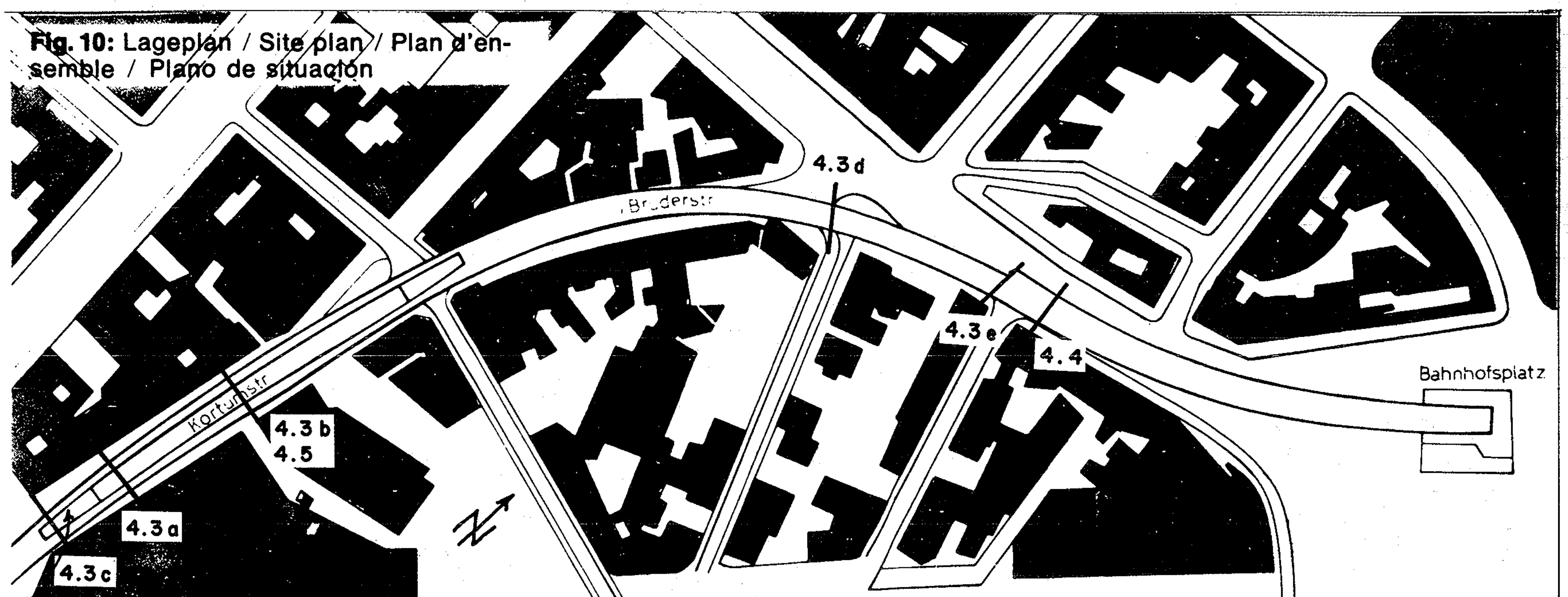


FIGURE B-4.2 PLAN VIEW OF SECTION A 3/5 IN BOCHUM. THE NUMBERS ON THIS FIGURE REFER TO FIGURE NUMBERS FOR THE CROSS-SECTION (FROM BETON-UND MONIERBAU, 1976)

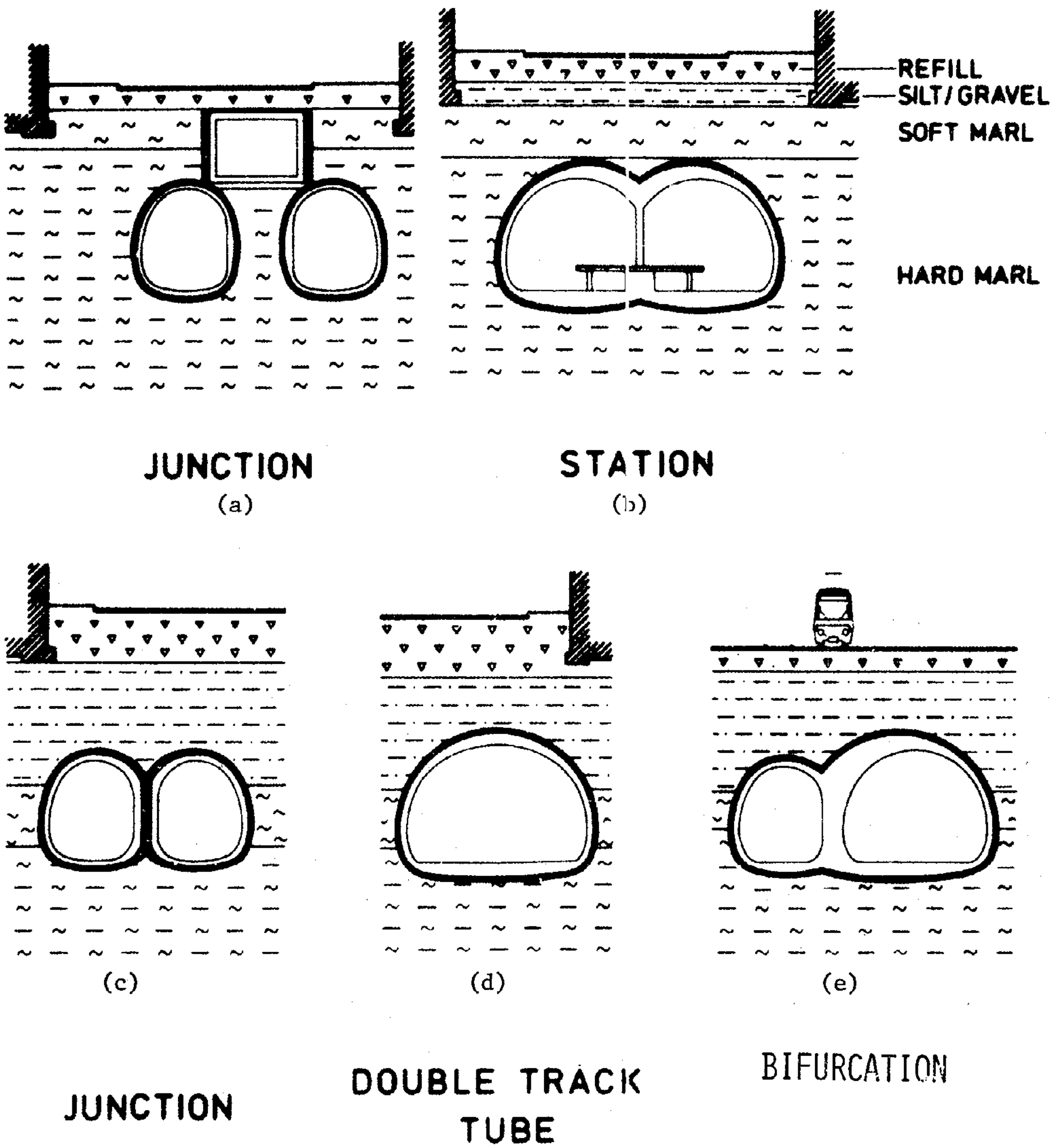


FIGURE B-4.3 CROSS-SECTIONS OF THE TUNNEL, SECTION A 3/5, BOCHUM (FROM MULLER AND SPAUN, 1977)

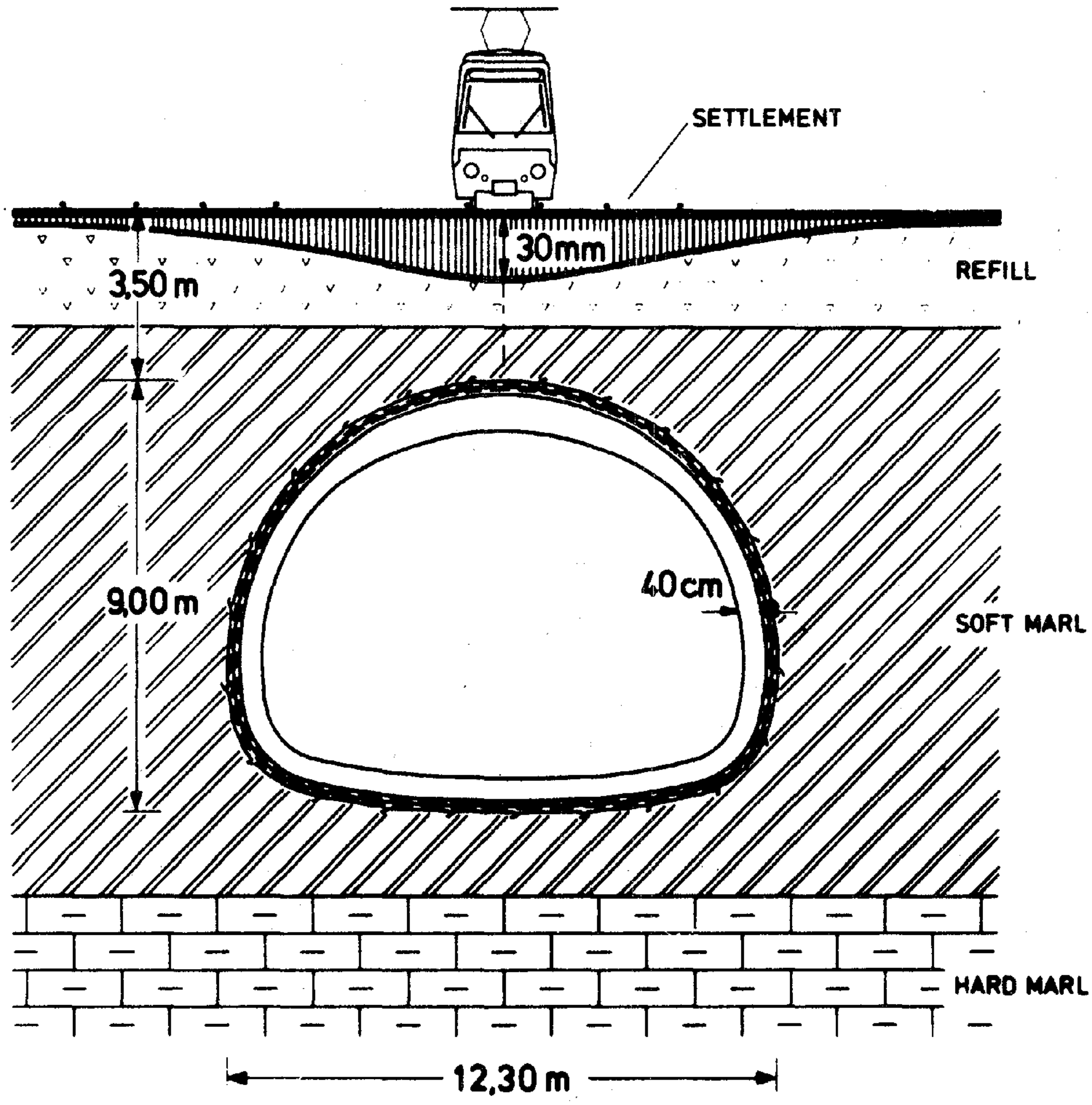


FIGURE B-4.4 SETTLEMENT ABOVE BIFURCATION-TUNNEL BOCHUM (FROM MÜLLER AND SPAUN, 1977)

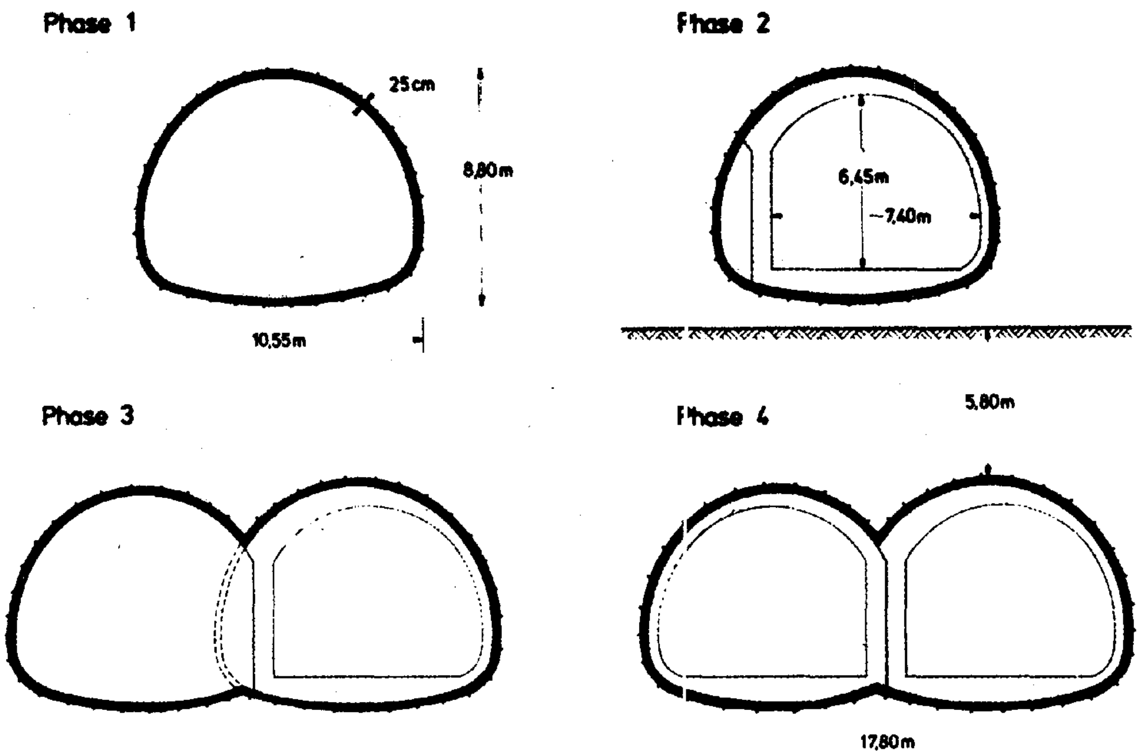


FIGURE B-4.5 CONSTRUÇTION PHASES OF "BERLINERPLATZ" STATION (FROM MULLER ANS SPAUN, 1977) north, and the station "Berlinerplatz" was driven by means of the Bochum Station Construction Method (Bochumer Bahnhofsbauweise). In a first phase (Fig. B-4.5), a tunnel with a width of 10.55 m and height of 8.8 m was driven and supported with shotcrete (25 cm), steelsets (spacing not given, estimated at 0.8 m). In a second phase (Fig. B-4.5), the inner liner, 40 cm of impervious concrete, was placed, as well as the central columns of the station. Then, in Phase 3 a parallel tunnel was excavated and supported with shotcrete and steelsets while the overlapping shotcrete of the first tunnel was excavated. Finally, the interior liner in the second tunnel was placed. The maximum width of the completed tunnel is 17.8 m, with only 5.8 m of overburden over the crown. The southern entrance to the station "Berlinerplatz" was excavated from the surface and supported by soldierpiles once the tunnel had been excavated and supported.

In the running tunnel to the north, a service track will later join the tunnel. In the area of this bifurcation, the tunnel was 12.30 m wide, 9.00 m high, with only 3.5 m of overburden (Figures B-4.4 and B-4.3e).

3) Monitored Performance

Figure B-4.4 shows the measured surface settlements for the 12.3 m wide tunnel in the area of the bifurcation. The maximum settlement above the crown of the tunnel is only 30 mm. The performance in the other sections (smaller span) was even better, with average settlements in the order of 20 mm. No damage to existing buildings was recorded.

4) Support Quantities and Costs

A summary of quantities (excavation, support placed) and cost data is given in Table B-4.1.

5) Concluding Remarks

Sections A2 and A3/5 of the Bochum Subway consist of tunnels of variable cross-sections and constantly changing alignment. The adaptability of the shotcrete support (includes light steelsets, wirefabric and shotcrete) made it possible to follow these changes and to do this without causing damage to existing buildings.

In cases where formwork would have required special shapes and large quantities of concrete, the placement of the interior liner by the wet shotcrete process proved to be more economical. This procedure was employed at the ends of the station, where the double track tunnel separated into two short single track tunnels before reaching the platform tunnel. The final support for this "nose" was by "wet" shotcrete.

No waterproofing was placed; the inner liner was "impervious" concrete. During this visit, the tunnel was completely dry, with the exception of one leaking construction joint.

TABLE B-4.1 TECHNICAL DATA OF SECTION A 3/5 FROM BROCHURE CITY OF BOCHUM AND BETON-UND MONIERBAU

Lengths	
Length of Section	614 m
Length of mined tunnel	552 m
Cover	3.5 to 8.2 m
Smallest distance from crown to a footing	3.5 m
Cross-section (excavated)	$35 m^2$ to $95 m^2$
Shotcrete thickness	25 cm to 40 cm
Inner liner(impervious concrete)	40 cm
Quantities	
Excavation of mined tunnels	50,000 m^3
Open cut "Hauptbahnhof"	17,000 m^3
Station Berlinerplatz	9,000 m^3
Total Excavation	76,000 m^3
Shotcrete placed	11,000 m^3
Concrete	19,000 m^3
Steel	2,000 metric tons
Costs	
Total (approximately)	26 million DM or $13 million U.S. ($1 = 2.00 DM)

APPENDIX: B-5

SITE: Ganzstein Tunnel

OWNER: Department of Public Works
State of Steiermark

DESIGNER: Dr. Pacher, Salzburg

CONTRACTOR: Beton-und Monierbau, in joint venture

DATE OF VISIT: January 25, 1978

PERSONS MET: Mr. Müller, Site Manager

1) Introduction

Mürzzuschlag lies on the road and railroad linking Vienna and Styria (Steiermark) by the Semmering Pass (Figure B-5.1). W. Steiner visited the Ganzstein Tunnel during a stopover on the trip from Vienna to Graz. Discussions were held only with Mr. Müller, the contractor's representative, since the representative of the owner, Mr. Rausch, was at a meeting in Graz at that time.

2) The Importance of the Ganzstein Tunnel

At present, the major highway linking Vienna to Graz and Italy is the Semmering Highway, S6. The planned motorway, A2, the future main route, when completed will take a more southerly route (Figure B-5.1) directly into Graz and on to Klagenfurt and Italy. S6, which is very heavily used at the present time, passes through the town of Mürzzuschlag (15,000 inhabitants); this made it necessary to build a bypass, which

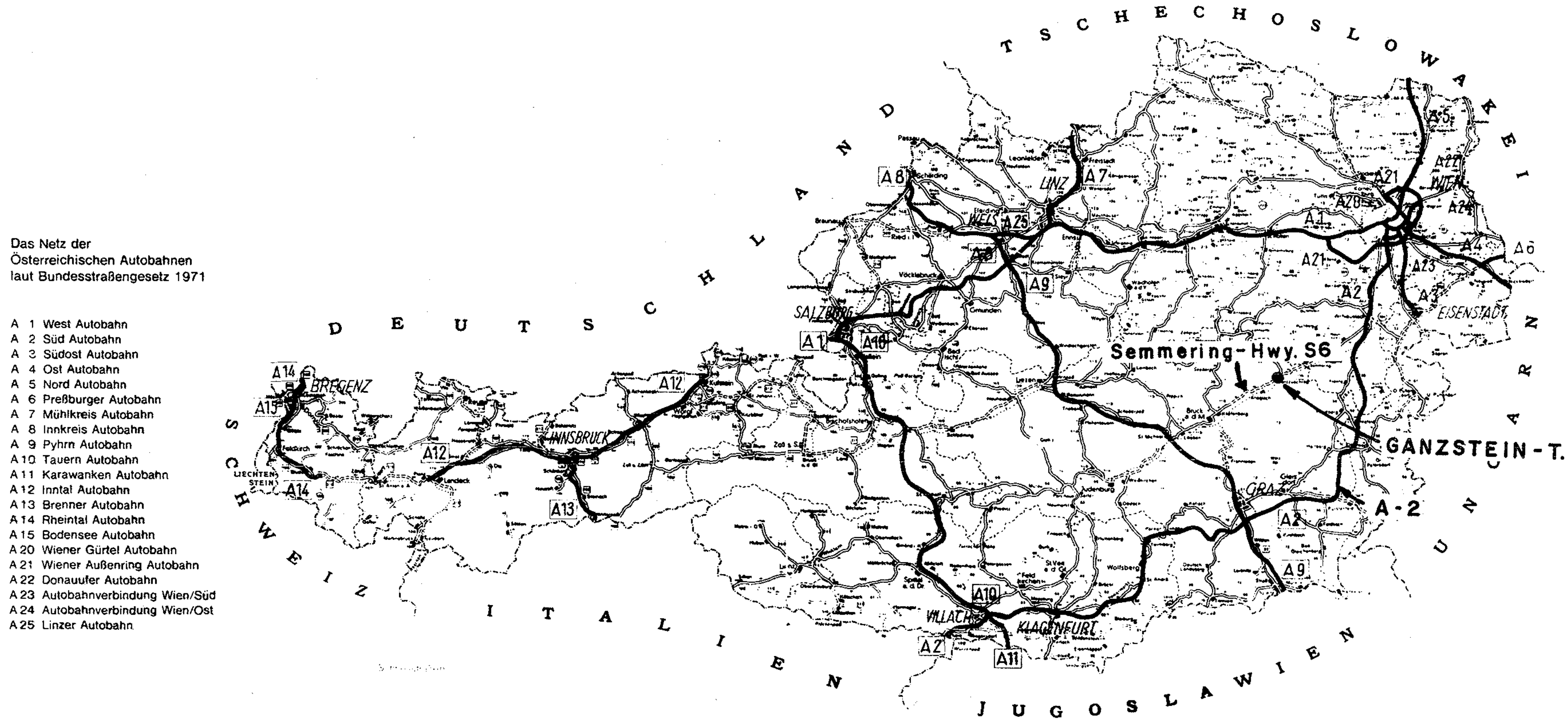


FIGURE B-5.1 LOCATION OF THE GANZSTEIN TUNNEL included the Ganzstein Tunnel.

At present, a single dual-lane tunnel of 2.2 km length is constructed.

3) Ground Conditions and Problems Encountered

A general geologic sketch is given in Figure B-5.2. The tunnel passes underneath the Michelbauerhöhe and the Ganzstein. The 200 m at the eastern end lie in phyllites that caused major problems; the remainder of the tunnel lies in limestone, where some caverns have been encountered.

In the limestone zone, excavation proceeded with partial face DEMAG machines, although blasting was also necessary. The ground is primarily classified as Ground Classes II to IV; the advance rate was on the order of 3 to 4 m per day, and the measured convergences were approximately 20 mm (2 cm).

The tunnel will be ventilated from a central cavern, which has been built in an open-cut (Figure B-5.2) supported by tangent piles. The heading of the tunnel in the immediate vicinity of the cavern has already been excavated. Due to the difficulties at the eastern end, the excavation from the western end will proceed through the ventilation cavern.

The difficulty at the eastern heading has the following cause: geological exploration was performed along a different alignment, some 70 m to the north. From borings placed along this alignment, the geologist concluded and predicted that non-water bearing phyllites would be encountered. The

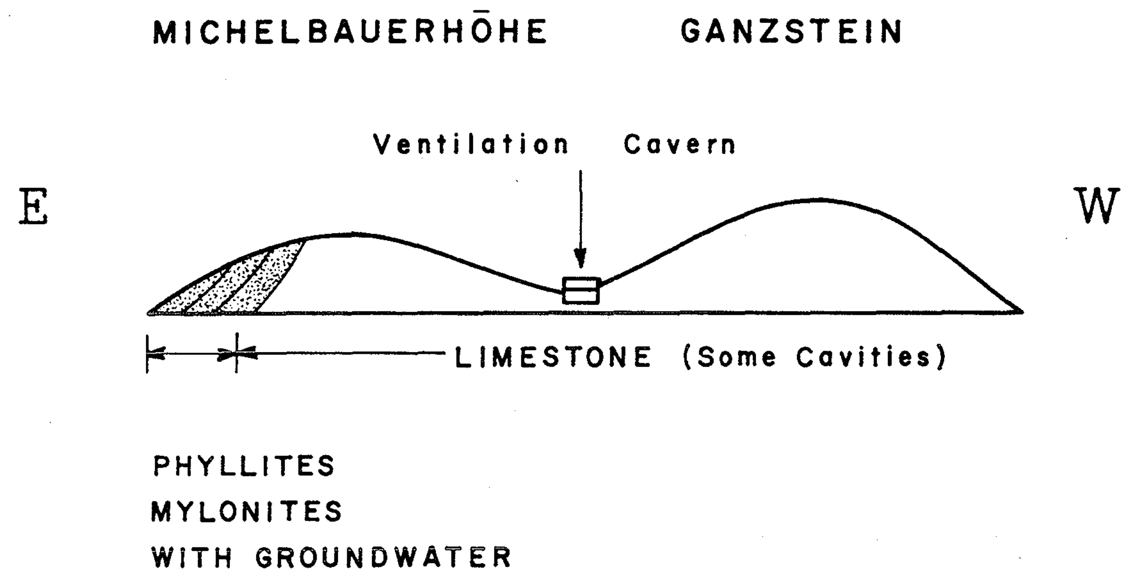


FIGURE B-5.2 SCHEMATIC GEOLOGIC SECTION THROUGH GANZSTEIN TUNNEL

tunnel excavation started from a start-up trench that had been excavated under the protection of tangent piles. The first part of the tunnel was constructed according to Class V (see Section 5, Table 5.1), but the face collapsed as water-bearing phyllites and mylonites were encountered. The failure reached to the ground surface, since the overburden above the crown is only 15 m. The ground flowed also around the already completed shotcrete shell, which experienced only minor damage (Figure B-5.3). The rock bolts were punched into the tunnel by the flowing ground. The ground seems to have behaved thixotropically and flowed when subjected to vibrations of the equipment in the tunnel. The contractor shut down this heading for approximately 10 months and called in his own experts; the contract was renegotiated, reflecting the changed conditions (water-bearing mylonites and phyllites instead of dry phyllites) in this eastern part. The contractor proposed to construct this section employing ground freezing procedures which were not accepted by the owner. The finally adopted solution can be described in general terms only, as the length and cross-sections are not exactly known: the zone where the collapse occurred is supported by caissons (Fig. B-5.4) placed from the surface (tangent piles). They are only concreted to the elevation of the crown. The shotcrete shell is keyed into these piles (Figure B-5.4). Excavation through the remainder continued with two side drifts (Figure B-5.5) in the mylonite zone. These side drifts were

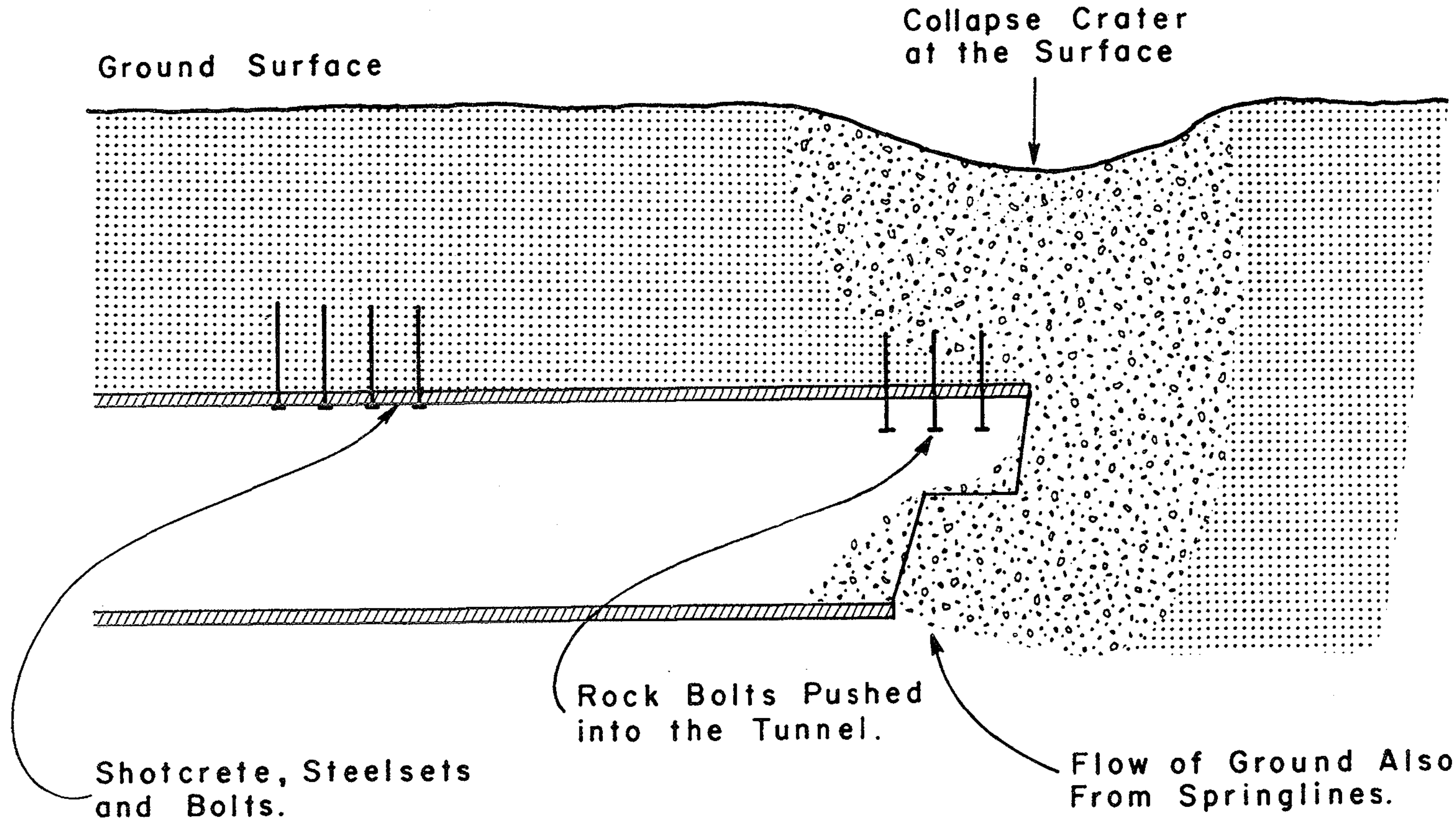


FIGURE B-5.3 COLLAPSE OF THE FACE IN THE GANZSTEIN TUNNEL

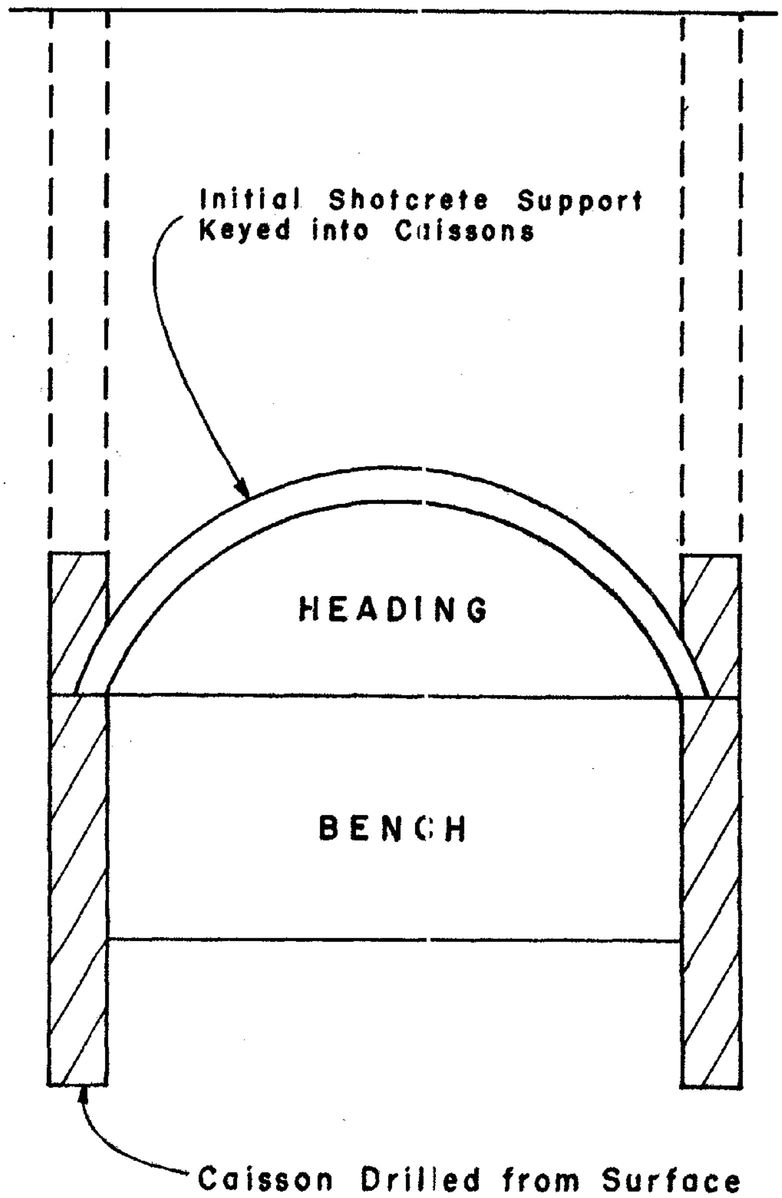


FIGURE B-5.4 SCHEMATIC CROSS-SECTION OF TUNNEL THROUGH COLLAPSED ZONE

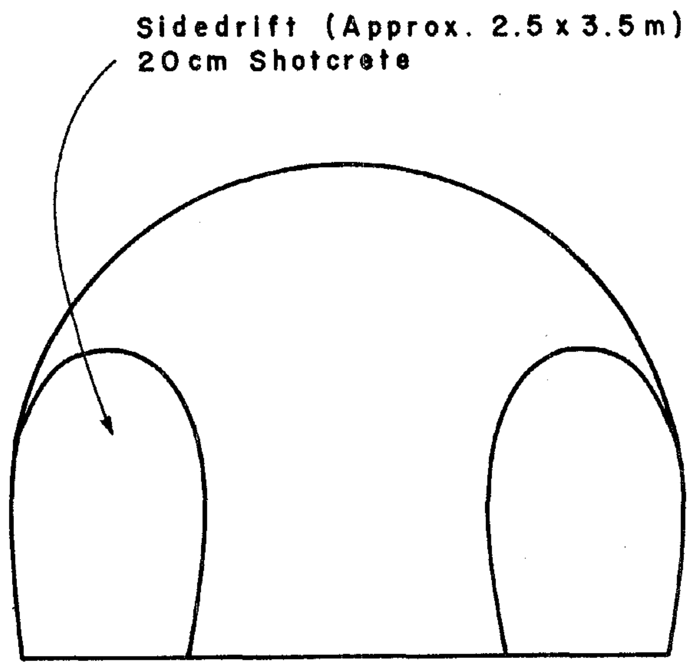


FIGURE B-5.5 TUNNELING DRIVING THROUGH MYLONITE ZONE WITH SIDEDRIFTS supported by shotcrete and steelsets (steelsets on the outside of the main tunnel only) and served as footings for the top heading, which followed later. After the excavation reached phyllite, which is more stable and not water-bearing, the method of excavation was changed back to the heading and benching procedure of class V. Still, some special measures had to be taken; e.g., the steelsets had footing plates of 30 x 30 cm to prevent bearing capacity failure. In addition to the above described measures, the ground water table was lowered by wells in the entire mylonite/phyllite section. Wells were placed at the springline (outside the tunnel cross-section); others were placed in the center line of the tunnel, extending to the crown of the tunnel.

According to Mr. Müller, the wells outside the tunnels worked well and brought an improvement of the stability of the ground. The wells in the crown (centerline of tunnel) proved to be rather unfavorable, since in the vicinity of the bottom of the well water actually concentrates and the zone was thus not dewatered.

The rate of advance per working day through the phyllite/mylonite zone was on the order of 1 m per day. Convergence measurements are taken by the contractor, but their value is limited as they are rather crude. The observed convergence is in the order of a few centimeters.

APPENDIX:	B-6
SITE:	Selzthal Tunnel
OWNER:	Department of Public Works, State of Steiermark
DESIGNER:	Toebich, Consulting Engineer, Wien
CONTRACTOR:	C. Baresel, Stuttgart, Germany G. Hinteregger, Salzburg, Austria
DATE OF VISIT:	27th and 28th January, 1978
PERSONS MET:	Mr. G. Sieberer, Site Engineer Department of Public Works

1) Introduction

The role of the Selzthal Tunnel in the Austrian and European road network will be described, followed by a discussion of problems encountered during construction and a summary of collected data.

2) The Role of the Selzthal Tunnel in the Austrian and European Highway Network

At the present time, the roads from Salzburg and Linz to Graz are only narrow two-lane highways with many town and railroad crossings. This route is heavily traveled by trucks exporting goods from West Germany to the Near and Middle East. In addition, during vacation periods and before extended holidays (Christmas and Easter), it is heavily used by foreign workers returning from West Germany to their native countries (Yugoslavia and Turkey). Many fatal accidents occur, and the road has one of the highest fatality rates in Western Europe.

The long-term plan envisions a four-lane divided highway, the Phyrnautobahn, A9, starting at the West German boarder in the vicinity os Schärdina as A8 (Figure B-6.1) and changing its numbers to A9 in the vicinity of Wels Highway A9 passes then through the Bosruck Tunnel under the Phyrn-Pass and by way of Graz to the Yugolavian border. Part of this highway is under the jurisdiction of a highway authority, the Phyrn Autobahngesellschaft, and will be a toll road. Other sections are built by the government. At present, the Phyrn Highway Authority builds the section from Graz northbound to St. Michael. This section, built for the time being as a two-lane road, includes two major tunnels: the Gleinalm Tunnel, of 8.3 km length and the Schattnerkogel Tunnel of approximately 1.5 km length, which lie in relatively good rock and did not cause major problems. The Schattnerkogel Tunnel caused problems, but has now been holed through. This section reduced the distance from Graz to St. Michael by 30 km (Figure B-6.1). Its opening to traffic was scheduled for the summer of 1978.

North of St. Michael, no major obstacles are encountered up to a point south of Selzthal where the valley narrows and the gorge is already congested with the present road and a single track railroad (Figure B-6.2). The Selzthal Tunnel,

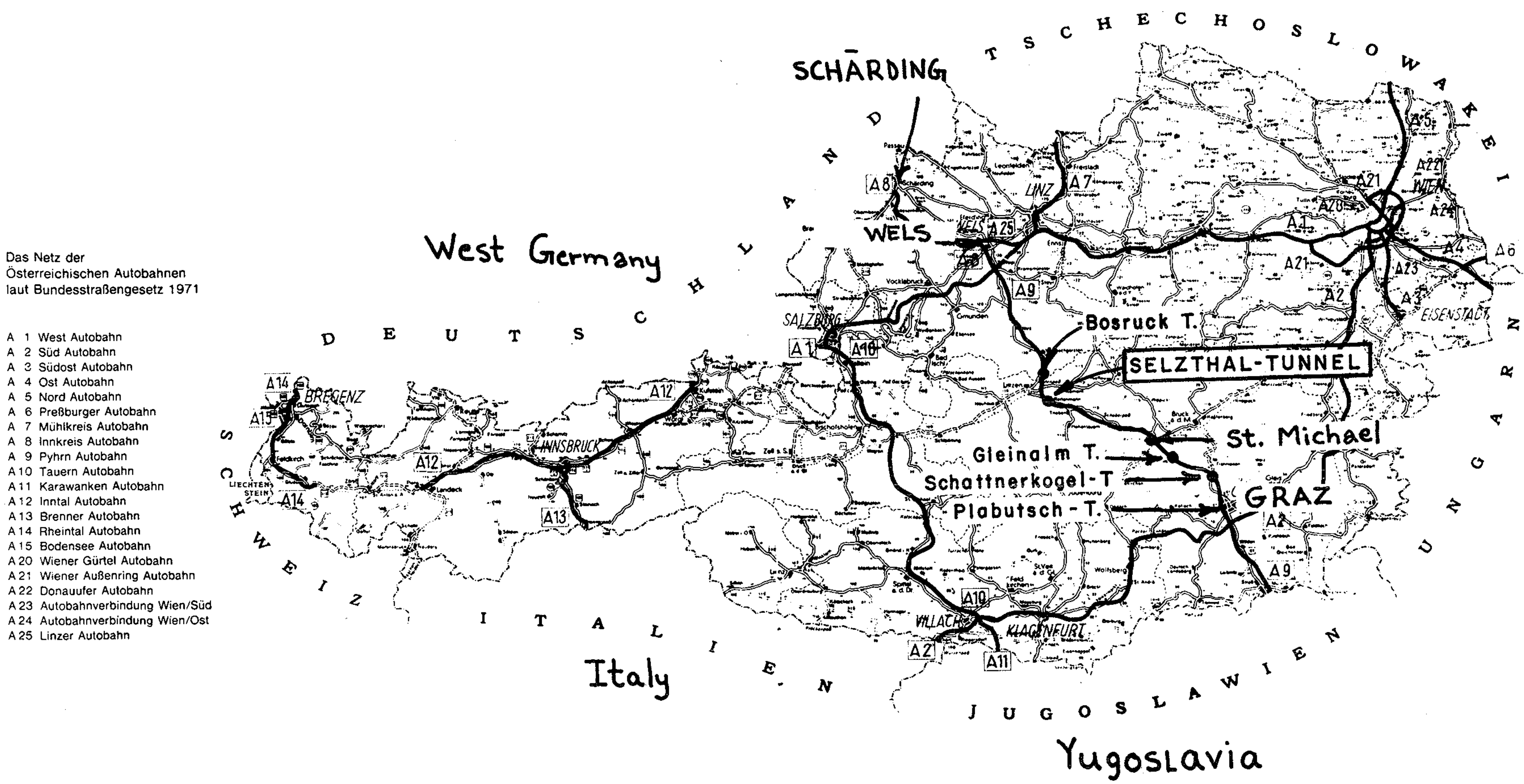


FIGURE B-6.1 TUNNELS ON THE PHYRN-AUTOBAHN, A9

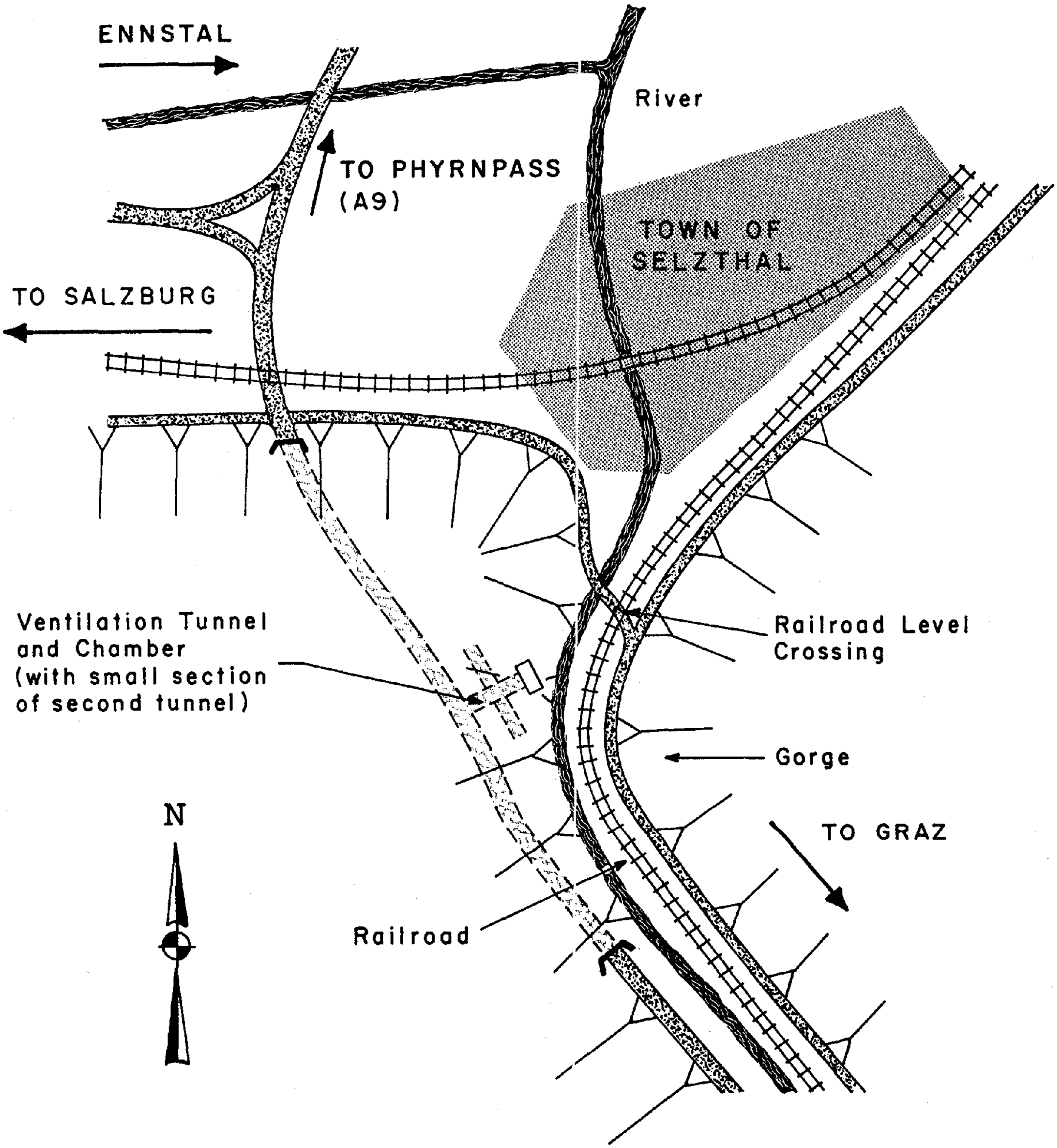


FIGURE B-6.2 SCHEMATIC MAP OF SELZTHAL AND THE TUNNEL of 1 km length, which was the subject of this visit, leads the new highway past the congested gorge into the Ennstal.

The highway will then cross the wide Ennstal and pass under the Phyrn Pass through the Bosruck Tunnel, which is parallel to a railroad tunnel built at the beginning of this century. The construction of the Bosruck Railroad Tunnel was hampered by heavy water inflows. It is expected that this railroad tunnel provides beneficial drainage for the highway tunnel. However, a pilot tunnel with full-size test sections will be built first. The new Bosruck tunnel is under the jurisdiction of the Phyrn Highway Authority. The Selzthal tunnel, however, is built under the direction of the local government of the province of Styria (Steiermärkische Landesregierung).

3) The Selzthal Tunnel

3.1 General Description

As mentioned before, the Selzthal Tunnel avoids the narrow gorge and the town of Selzthal (Figure B-6.2). Of the two planned parallel tunnels, only the western tunnel (length = 1010 m), the central ventilation cavern and 80 meters of the second (eastern) tunnel near the cavern are constructed (Figure B-6.2). Fresh air is supplied to the tunnel with a ventilation tunnel of approximately 100 m length extending to the gorge. At the time of the visit, the western tunnel had been holed through; waterproofing and final liner were being placed (western tunnel) while the excavation of ventilation tunnel and cavern continued.

Construction work started in March 1976 and was initially scheduled to last for 22 months. An extension of 6 months had been granted due to difficult ground conditions. During construction of Selzthal Tunnel, various problems related to geology, construction and contractual procedures were encountered.

3.2 Geology of the Selzthal Tunnel

No detailed geologic map or section of the areas was available. However, the ground conditions can be characterized as follows. The entire tunnel lies in metamorphic rocks, phyllites to green schists. Heavily weathered phyllites were encountered near the portals; the deeper the tunnel reached, the better became the rock. In the center of the tunnel, green schists were encountered.

Figure B-6.3 is a photograph of the northern portal showing that the entire slope above the tunnel is an active slide area. The overburden of the tunnel reaches a maximum of 150 m; however, high horizontal stresses seem to exist in the vicinity of the tunnel due to that active slide (this will be further discussed in section 4, where the data are summarized).

During construction, other geologic features were discovered which were not anticipated. In particular, a zone of weakness was discovered at station 220 m from the south portal which was correlated with a small valley hidden in a forest

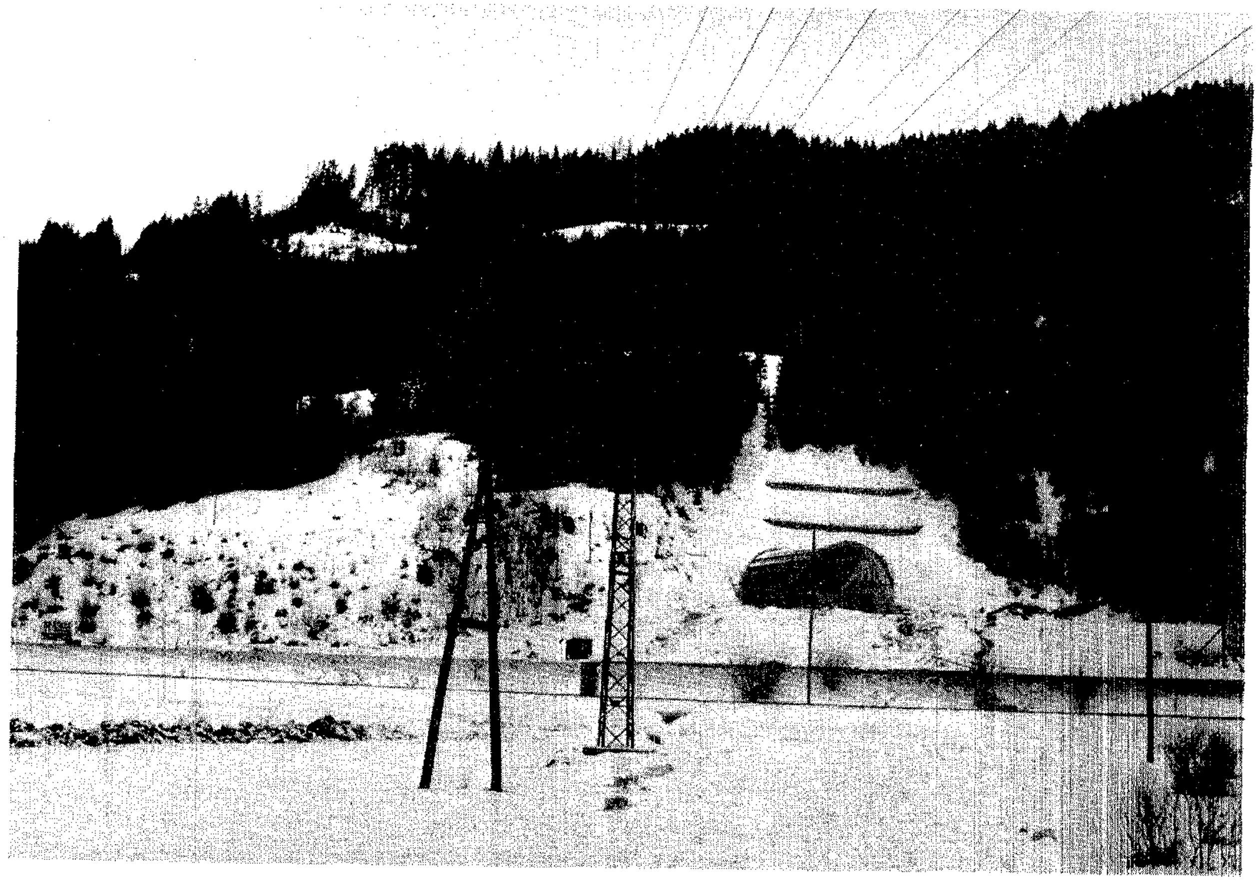


FIGURE B-6.3 UNSTABLE SLOPE NEAR THE NORTHERN PORTAL OF THE SELZTHAL TUNNEL (PHOTO, W. STEINER, 1978) on the surface.

For exploration purposes, 9 borings with a total length of 440 m were sunk; the borings all reached below the invert of the tunnel (for approximate location of borings, see Figure B-6.6).

3.3 Construction and Contractual Aspects

The bid documents for the Selzthal Tunnel were based on the NATM and included a ground classification relating ground conditions and support requirements (details on ground classification can be found in Section 5).

However, the low bidder submitted an alternate proposal based on the Bernold System combined with a forepoling shield. (Figure B-6.4), followed by a cast-in-place concrete liner poured between Bernold sheets and forepoling plates. The latter are then advanced and the void between rock and concrete is grouted. With the forepoling shield the work area is protected and an immediate roof support is possible. A more detailed description of the Bernold System can be found in the Bernold News of February, 1974; there the construction of the Sonnenburgerhof Tunnel in Innsbruck is described, which was built by the same contractor and with the same shield as the Selzthal Tunnel.

Construction started from the southern end with the Bernold Method and proceeded for 28 m, at which point the shield got stuck as the crown settled excessively (8 cm) and

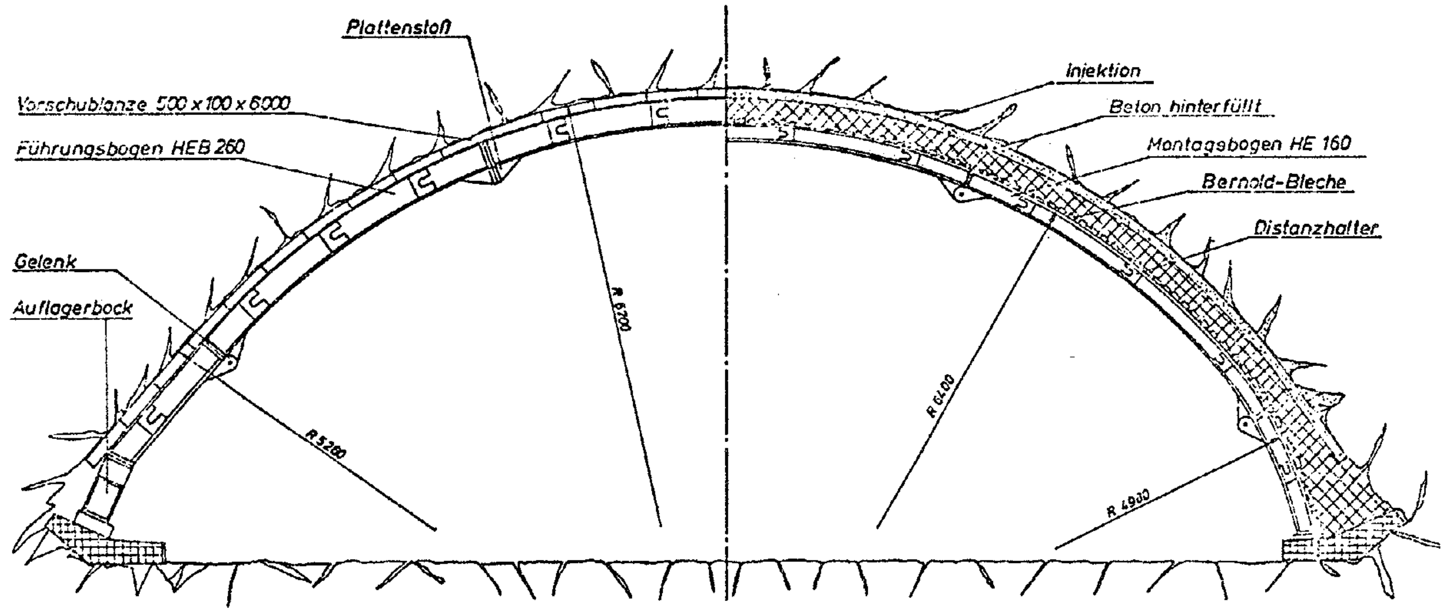


FIGURE B-6.4 CROSS-SECTION BERNOLD METHOD (FROM BERNOLD NEWS, FEB. 1974)

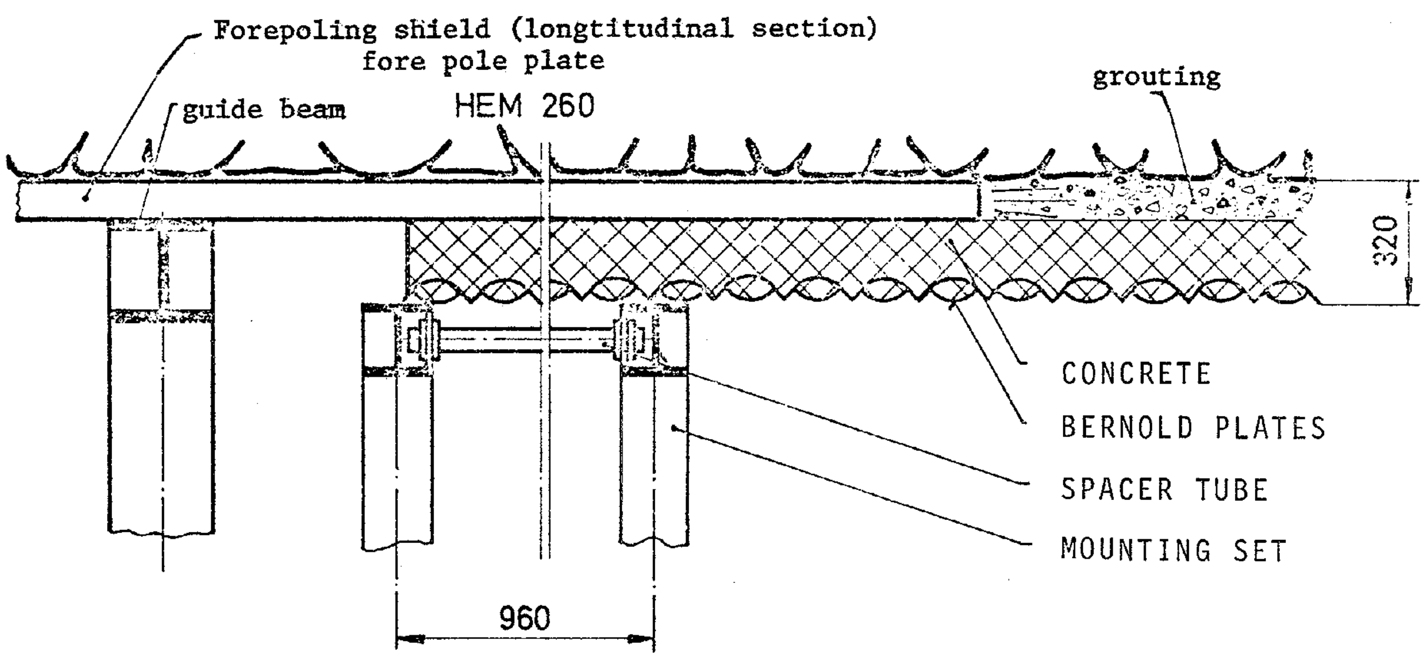


FIGURE B-6.5 LONGITUDINAL SECTION, BERNOLD SYSTEM (FROM BERNOLD NEWS, FEB. 1974) the poling plates had to be abandoned. Construction had then to continue according to the specifications of the design engineer, i.e., the NATM, but no price increase was granted.

The problems with the Bernold construction procedure can be related to several causes. (i) Rock bolts could only be placed after the forepoling plates of 6 m length had advanced, i.e., at a distance of 8 to 10 meters from the face. (ii) the steelsets on which the forepoling plates rest settled under the crown load, causing the forepoling plates to tilt.

Besides these problems in the heading, other difficulties occurred. The concrete liner in the crown was rather thick (> 30 cm); however, the walls below the springlines (bench excavation) were supported by shotcrete of 15 to 20 cms thickness. There was thus a sudden change in liner thickness (and stiffness); also, the reinforcement was not continuous, since the reinforcement in the concrete crown was by the Bernold sheets, but by wirefabric in the shotcrete. Thus, cracks developed where crown and springline support joined.

In addition, invert closure did not follow the excavation in the heading by 30 days as specified; it followed much later, often by 150 days.

In the already supported area (Bernold System), approximately 10 meters from the south portal a roof collapse occurred during a thunderstorm that followed a period of drought. A usually dry surface run-off channel filled with water that percolated into the ground leading to the collapse. The contractor had not taken the necessary steps as requested by the owner to divert such flows and thus had to carry the costs of this failure.

However, even after the excavation-support procedures had been changed to the NATM a roof collapse occurred at stations 214 to 225 m. This zone was classified as ground class III. The support in ground class III usually consists of shotcrete (10 cm), wirefabric (one layer = 3.11 kg/cm^2), a welded truss ($\simeq$ 7 kg/m) and pre-stressed concrete bolts of 4 m length (6-1/3 bolts per meter of tunnel). However, in this section, instead of a single wire fabric, welded truss and shotcrete, two layers of wirefabric and shotcrete were used as support.

The roof collapse which started near station 220 propagated through this zone of ground class III (the invert had not been closed) forward to station 280 m where the support changed to ground class IV (with steelsets). The area where the collapse started was later associated with a shear zone which could be correlated to a small valley in the forest that was not discovered before (see Section 3.2). The collapsed area was holed through with an excavation and support procedure according to ground class V. (Table B-6.1).

The support procedure for other parts of the tunnel was also changed and adapted to available equipment. For instance, the length of the roof bolts was reduced to 4 m (for ground class IV, Table B-6.1) in order to place them more rapidly

TABLE B-6.1 GROUND CLASSES FOR SELZTHAL TUNNEL

GROUND CLASS	CLASSIFICATION CRITERIA Method of loosening	ROUND LENGTH	SUPPORT	QUANTITY PER METER OF TUNNEL	TOTAL QUANTITY PER METER OF TUNNEL
III	Blasting required	1.5 to 2.0 m	Shotcrete: 10 cm	20.85 m^2	20 85m^2
			Wire fabric: 3.11 kg/m^2	20.85 m^2	64.84 kg
			Welded Truss (spaced: 1.5 m) ~ 10kg/m	--	116.26 kg
			Prestressed grouted bolts: L = 4 m	6 1/3	25.33 m/m
IV	Blasting may be necessary, may possibly be excavated by hydraulic shovel	1.5 m	Shotcrete: 15 cm	20.94 m^2	20.94 m^2
			Wire fabric: 3.11 kg/m	20.94 m^2	65.12 kg/m
			Steelsets: 27 kg/m (spaced 1.5 m)	--	370 kg/m
			Grouted bolts: L = 6 m	9	54 m/m
V	With hydraulic shovel	1.0 m	Shotcrete: 20 cm	21.03 m^2	21.03 m^2
			Wire fabric: 3.11 kg/m	21.03 m^2	65.4 kg/m
			Liner plates d = 3 mm (40% of circumference)	--	400 kg
			Steelsets: 27 kg/m (spaced 1.0 m)		557.3 kg/m
			Grouted bolts: L = 6.0 m	13.5	81 m/m

(drilling of 4 m bolt holes does not require a change in drill steel as the originally planned 6 m bolts did).

All these incidents led to substantial delays. To avoid further delays once the tunnel was to reach the weathered zone near the northern portal, a second heading was required starting from the northern portal, making it possible to catch up with the schedule.

The contract for tunnel construction, as mentioned before, was awarded on March 19, 1976, with a planned construction time of 22 months initially. Due to the unanticipated difficulties, the contractor was granted an extension of 6 months for completion of the tunnel (Delays caused by the contractor are subject to a penalty of 5,000 AS/day≃ $340/day (U.S.)) At the time of the visit, it appeared that the tunnel should be completed by the extended deadline.

When comparing the bid for the Selzthal Tunnel with other tunnels in similar ground conditions (see Section 3), one has to conclude that the bid was unreasonably low. In addition, other factors may also have led to problems. Both the owner and the contractor hired additional geotechnical consultants after tunnel construction had started. One can conclude that the problems surfaced which were not anticipated by the owner, the design engineer or the contractor.

4) <u>Data Collected</u>

The data from the Selzthal Tunnel are interesting, since they document behavior that cannot be found in other tunnels. The data on ground conditions are not as detailed as they ought to be for the development of good correlations between ground conditions and support requirements. For the description of the ground conditions, we have to rely primarily on the classification criteria in the bid documents and brief descriptions on the measurement data sheets. Figure B-6.6 shows the predicted and the actually encountered ground classes in the Selzthal Tunnel. 25% of the length was predicted to be in ground class V, while actually 51% fell into this class, with corresponding reductions of class III and IV.

Of particular interest is the monitored performance of the tunnel. Large crown settlements were observed, with values of up to 50 cm. However, the owner's site engineer questions these values and suspects a measuring error. This led the owner to perform a precision survey in some cross-sections. However, when these measurements were made, the excavation was complete and the support in the crown and at the springline was placed. Nevertheless, incremental displacement observed with the precision survey (Figure B-6.7) showed displacement vectors which were essentially vertical, with crown settlements of 6 to 10 cm per month before the invert was closed. Once the invert was installed the movement stopped abruptly and the deformations were only on the order of millimeters. This behavior may be explained by high horizontal stresses and

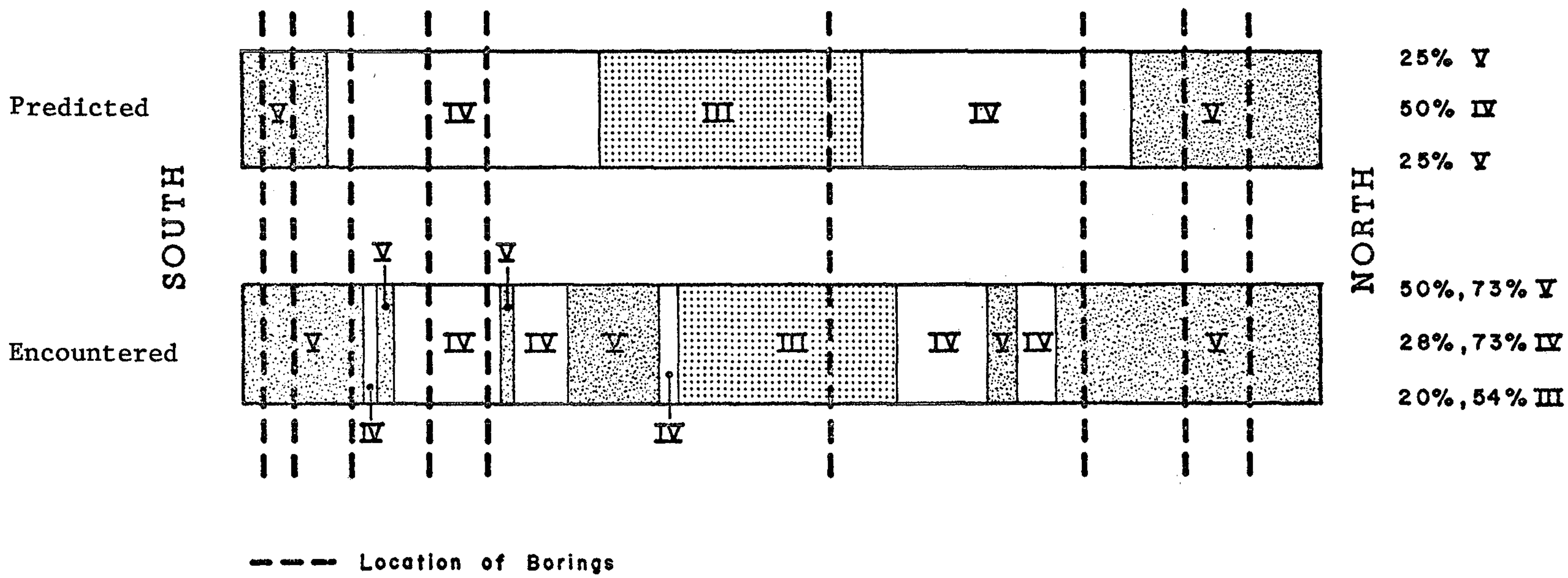


FIGURE B-6.6 COMPARISON OF PREDICTED AND ENCOUNTERED GROUND CLASSES WITH LOCATION OF BORINGS

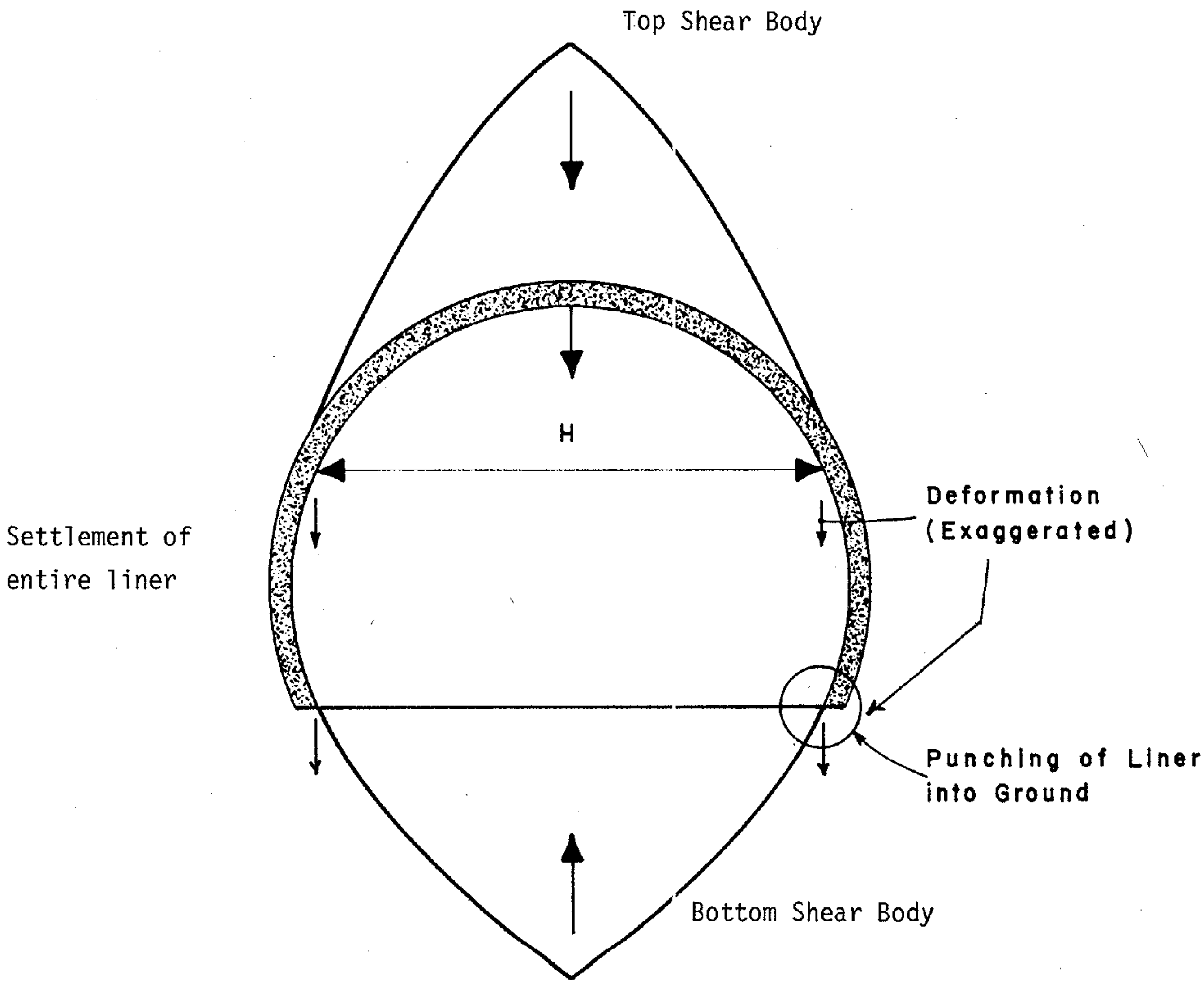


FIGURE B-6.7 SHEAR BODIES IN CROWN AND INVERT LEADING TO LARGE VERTICAL DISPLACEMENTS

Rabcewicz's shear body theory. The shear body develops perpendicular to the direction of the major principal stresses; in case of high horizontal stresses, the shear body forms in crown and invert. As long as the invert is not placed, the invert shear body is pushed into the opening, while the crown shear body and the support settle and the footings of the springlines are punched into the ground (Figure B-6.7).

In addition to these precision surveys, regular horizontal convergence measurements were taken (Figure B-6.7); the placement of the measuring bolts and the first reading were, however, often made at a considerable distance from the face and at a considerable time after excavation (sometimes the delay was more than a month). Thus, the convergence measurements are not very useful. A few measurements were made at and are reported here: the convergences monitored in ground class IV were on the order of 5 cm; in ground class V, these values are greater and reached 21 cm in one section.
the proper distance and time and are repoted here: the convergences monitored in ground class IV were on the order of 5 cm; in ground class V, these values are greater and reached 21 cm in one section.

It may be worthwhile to mention that the unstable state of the slopes in the vicinity of the Selzthal Tunnel is reflected in the behavior of a transmission line tower. During the driving of the tunnel the tower tilted with a horizontal displacement of 23 cm.

5) <u>Costs</u>

The bid for the 1 km western tunnel, the central cavern and the construction of a road embankment of 400 m length to the south was let for 112 million Austrian schilling (= \$7.47 million U.S., \$1 U.S. = 15 Austrian schillings). However, this does not include escalation and changes due to different ground classes and possible approved changed conditions.

APPENDIX:	B-7
SITE:	Arlberg Highway Tunnel (Eastern and Western Section), Flirsch Tunnels
OWNER:	Arlberg Highway Authority
DESIGNER:	Arlberg Tunnel: Ingenieurgemeinschaft Lässer-Feizlmayr Flirsch Tunnel: Dr. Pacher
CONTRACTOR: (EAST)	Arlberg Tunnel East Contractors with: Oberranzmeyer, Innsbruck; Innarebner and Mayer, Innsbruck, Soravia-IL Bau, Spittal.
DATE OF VISIT:	31st January, 1978
PERSONS MET:	Mr. Treichl, Site Manager; Mr. Schefzik, Deputy Site Manager; Dr. F. Kunz, Field Geologist, ASTAG.
CONTRACTOR: (WEST)	Arlberg West Contractors with: Jaeger, Schruns; Mayreder, Linz; Porr, Wien; Universale, Wien; Hinteregger, Salzburg; Rella, Wien.
DATE OF VISIT:	1st and 2nd February, 1978
PERSONS MET:	Mr. Mayrhauser, Site Manager Mr. Obst, Site Engineer Dr. J. Kaiser, Field Geologist, ASTAG

1. Introduction

The Arlberg Highway extends from Flirsch (Tyrol) to Dalaas (Vorarlberg). At the present time, only two lanes of a four-lane divided highway are built.

Initially, only the Arlberg Highway Tunnel from St. Anton to Langen (Figure B-7.1) was to be built by the Arlberg Highway Authority (ASTAG). In 1976, the law authorizing tunnel construction had been amended to include also the eastern access road from Flirsch to St. Anton and the western ramp from Dalaas to Langen, a total length of 36.2 km, all of which will be a toll road.

The main tunnel, with a total length of 13.98 km, has been holed through on October 9, 1977, five months ahead of the originally planned date. The date of opening, originally scheduled for mid-1979, was December 1, 1978. (Figure B-7.2)

On the eastern ramp, the two tunnels near Flirsch are presently under construction and have been visited. For the Dalaas Tunnel, of 1.6 km length, construction work will start soon. The new improved ground classification for the Dalaas Tunnel has been obtained from the design engineer (ILF).

First, some cost issues and problems regarding the entire project will be discussed; then each of the sites visited will be described. At each site it was possible to talk to the contractors as well as to the field geologists of the owner.

2. General Problems and Costs

The Arlberg Tunnel Authority is a public company, with the stock being held by the Austrian government and the state governments of Vorarlberg and Tyrol. Financing is mostly through the issue of bonds.

According to Rainer (1978), the construction costs for the main tunnel will be 3,881 million Austrian schillings (= 259

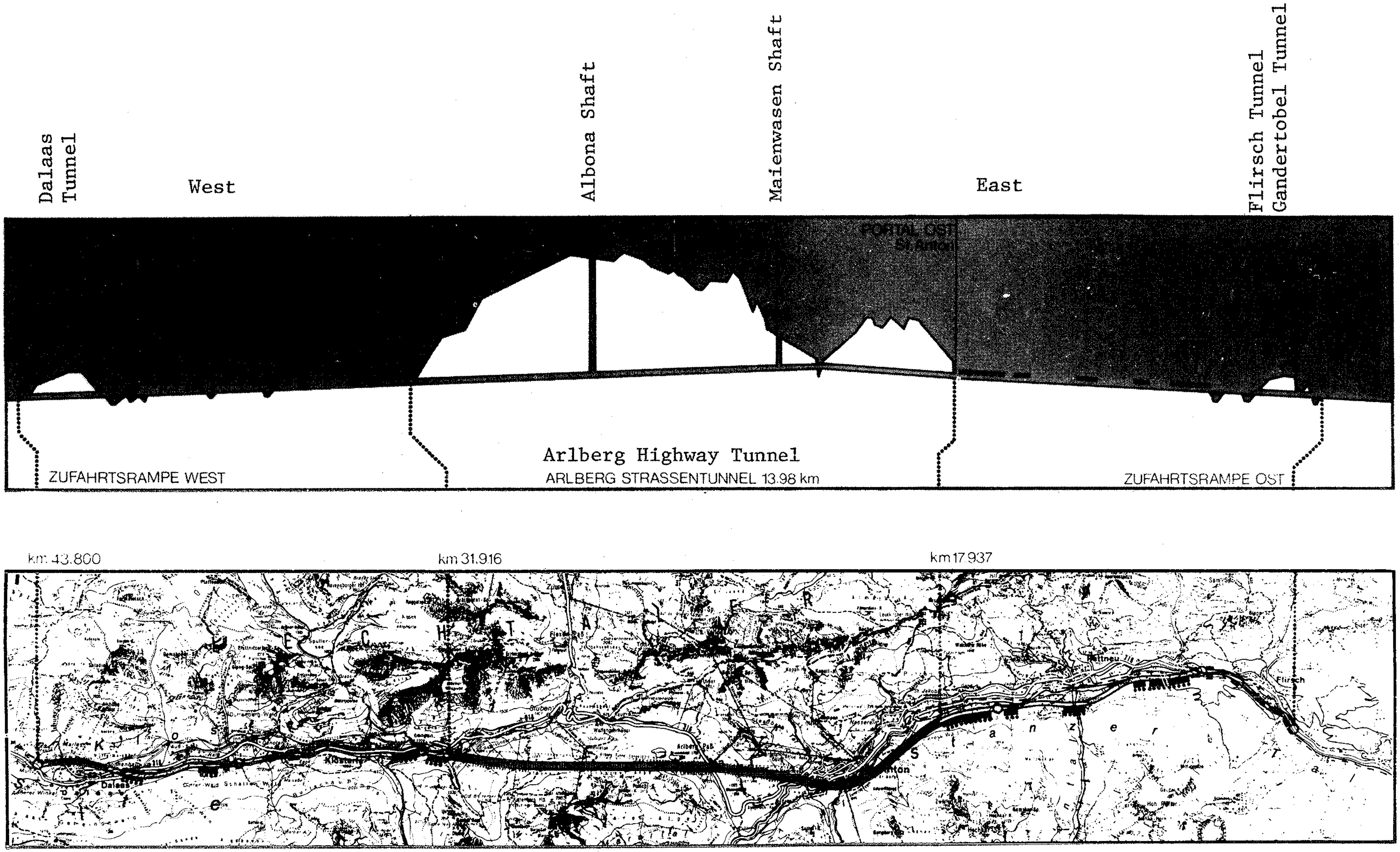
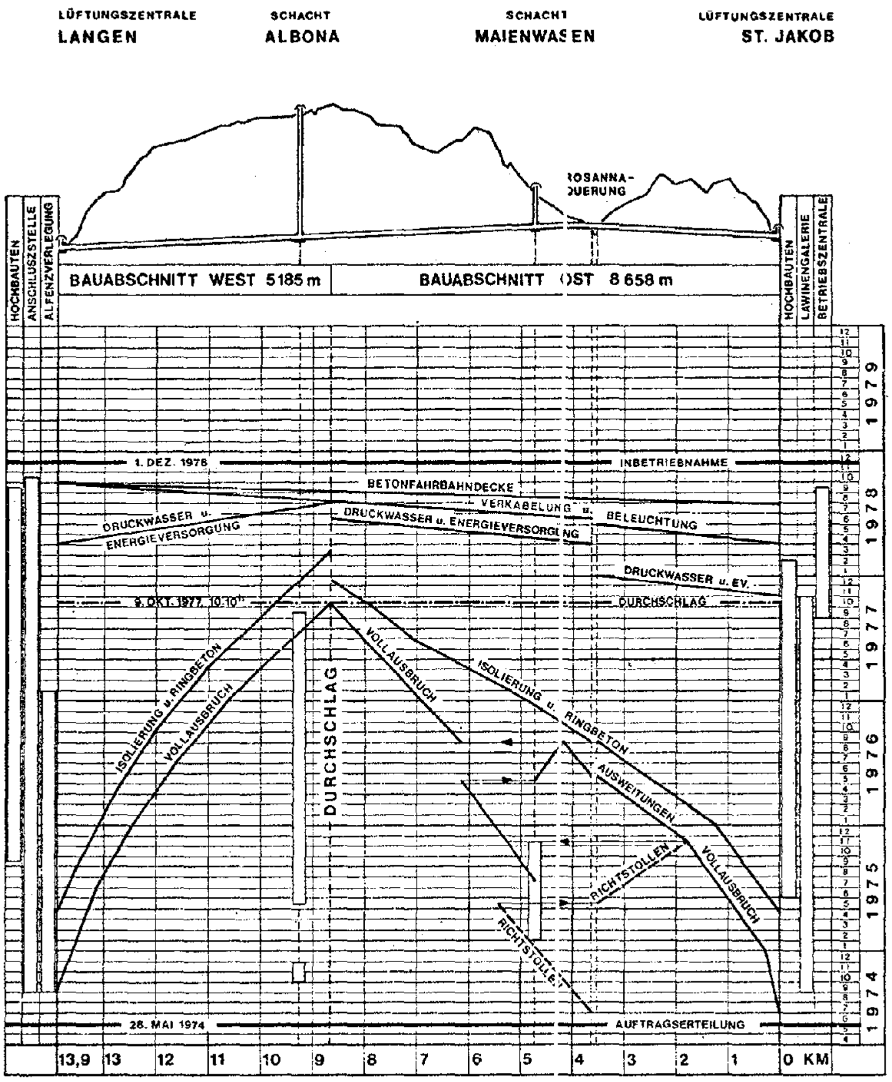


FIGURE B-7.1 MAP AND SECTION THROUGH ARLBERG (FROM ARLBERG HIGHWAY AUTHORITY, 1977)



Auftragserteilung:	Award of Contracts
Richtstollen:	Pilot Tunnel
Ausweitungen:	Widening
Vollausbruch:	Excavation of entire cross-section
Isolierung:	Waterproofing
Ringbeton:	Final Concrete Liner
Druckwasser:	Pressurized Water Supply
Energieversorgung:	Energy Supply
Verkabelung:	Placement of Supply Cable
Beleuchtung:	Illumination
Beton Fahrbahndecke:	Concrete Pavement
Inbetriebnahme:	Opening
Lüftungszentrale:	Ventilation Plant
Schacht:	Shaft
Rosanna Querung:	Crossing of the Rosanna River

FIGURE B-7.2 SCHEDULE FOR ARLBERG TUNNEL million U.S.) ($1 U.S. = 15 AS) on the price basis of 1972.

The tunnel has a cross section of 90 m^2 (Fig. B-7.3). In addition, there are two ventilation shafts which are also part of the tunnel. The shafts are 735 m and 230 m deep, with ventilation chambers.

3. Site Visit at Arlberg Ost and Flirsch

3.1 Introduction

At Arlberg Ost (East), W. Steiner visited the contractor ARGE ATO (Joint venture Arlberg East). Mr. Treichl, Site Manager, and Mr. Schefzik, Deputy Site Manager, are responsible for the site and have been involved in it from the beginning. The construction was nearly complete and the interior liner over the entire length had been placed. The ARGE ATO had recently been awarded the contract for the two tunnels at Flirsch, where work was in progress.

Besides talking to representatives of the contractor, W. Steiner talked to Dr. Kunz, the field geologist of the owner, ASTAG.

3.2 The Gandertobel and Flirsch Tunnels

These two tunnels on the eastern access route were under construction. Since the Arlberg East contractors were constructing the tunnels, most of the equipment was already on site. Notably no or only minor site installation costs occurred. This joint venture thus had a definite advantage in bidding. The two tunnels, although only separated by a few hundred meters of open road, lie in completely different ground conditions and have different cross sections. The eastern tunnel, the

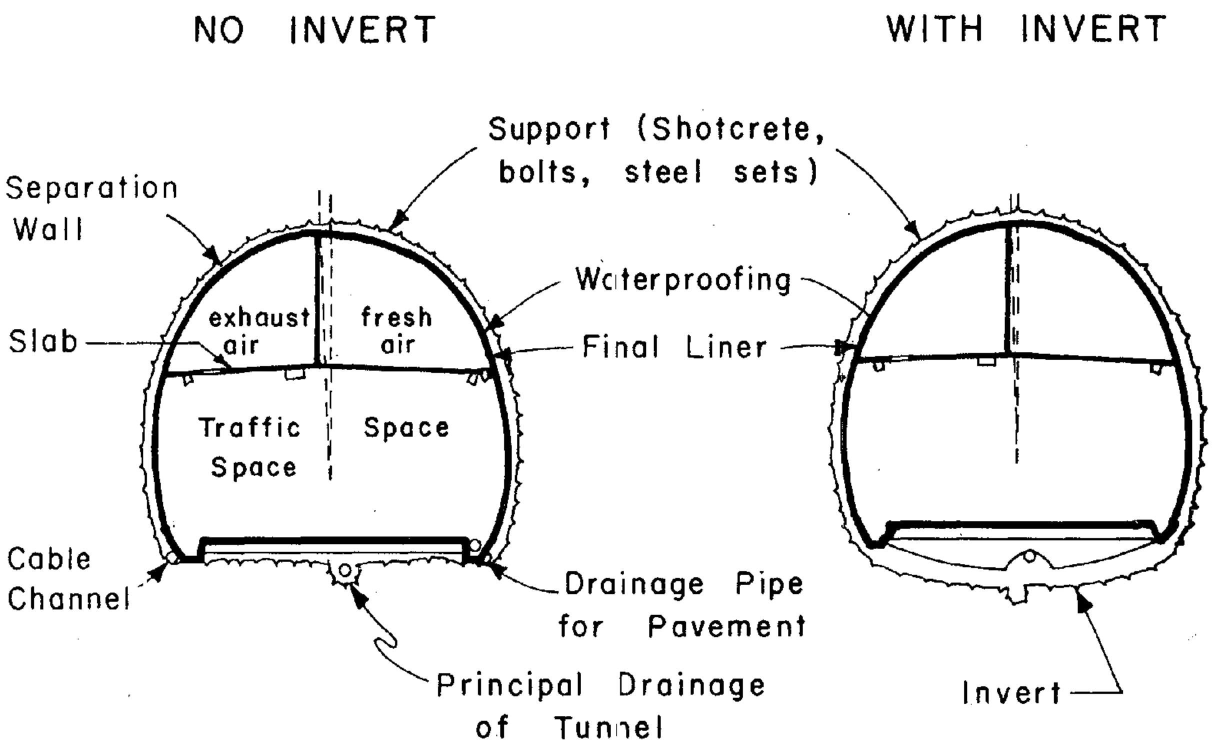


FIGURE B-7.3 CROSS-SECTIONS FO ARLBERG HIGHWAY TUNNELS

Gandertobel Tunnel, is 320 m long and has a cross section of 150 m^2. It crosses an old landslide consisting of slide debris and talus material and passes under a small stream. The large cross section of the Gandertobel Tunnel is necessary because an entrance ramp is located immediately to the east of the tunnel, requiring a three lane roadway. The western tunnel is the Flirsch Tunnel, which will be 820 m long and has a cross-section of 90 m^2. It crosses metamorphic rock of gneissose-schistose nature; the discontinuities strike primarily parallel to the tunnel axis. Two sets of discontinuities form wedges which tend to drop from the crown into the excavation. Each tunnel will now be described in some detail.

Gandertobel Tunnel

As discussed above, the Gandertobel Tunnel passes through slope debris, whose matrix is a silty sand but may also contain room-size boulders. The maximum overburden is 30 m but is as low as about 3 m where a stream crosses the alignment. To reduce problems caused by the small stream (ground water!) above the tunnel, the tunnel was driven in winter, when no surface run-off was expected.* The original design called for two side drifts supported by shotcrete similar to the subway tunnels in Munich. (Compare Appendix B-1, Figure B-1.4). Each side drift would have been supported by shotcrete and steelsets. Then the top heading and core excavation would have followed, also with shotcrete and steelsets. The contractor constructed the northern side drift for approximately 150 meters. The method was subsequently changed to a heading and bench operation with

*Nevertheless a rooffall and daylighting occurred as the tunnel pased under this stream. Since the volume of the rooffall was small, no damage was caused and construction proceeded normally.

invert closure approximately 50 m behind the face (Figure B-7.4 and photograph Figure B-7.5a); the top heading with a central supporting core is excavated and supported by shotcrete, steelsets, wire fabric and 6 m long driven grouted bolts. The final shotcrete thickness is approximately 25 cm. The spacing of the steelsets varies; it is on the order of 1 m.

No construction ventilation was required in this tunnel. Since the tunnel was driven downward, heat and exhaust produced by the equipment was sufficient to produce a natural draft.

The picture in Figure B-7.5a gives an overview of the excavation procedure and the equipment used; the type of equipment is marked on the picture. Figure B-7.5b is a photo taken close to the face, illustrating the activity in the heading. Note the heavy equipment used for the excavation and in particular that work continues while a convergence reading is taken. Convergence measurements require little time to take (a few minutes), and work is not impeded by the measurement.

Only 14 men work in the heading (without measuring crew); actually, more personnel would hinder the advance of the tunnel. Messrs. Treichl and Schefzik provided a list of the men and equipment and also the type of work they perform. This information is presented in Tables B-7.1 and B-7.2. Mr. Treichl reported a discussion with a Canadian visitor, during which they concluded that to perform the 44 tasks listed in Table B-7.2 and adhering to union rules 70 crew members would be required in North America instead of the 14 in Austria. As mentioned earlier, a larger crew would actually reduce advance rates due to hindrance and additional exchange time. Lower

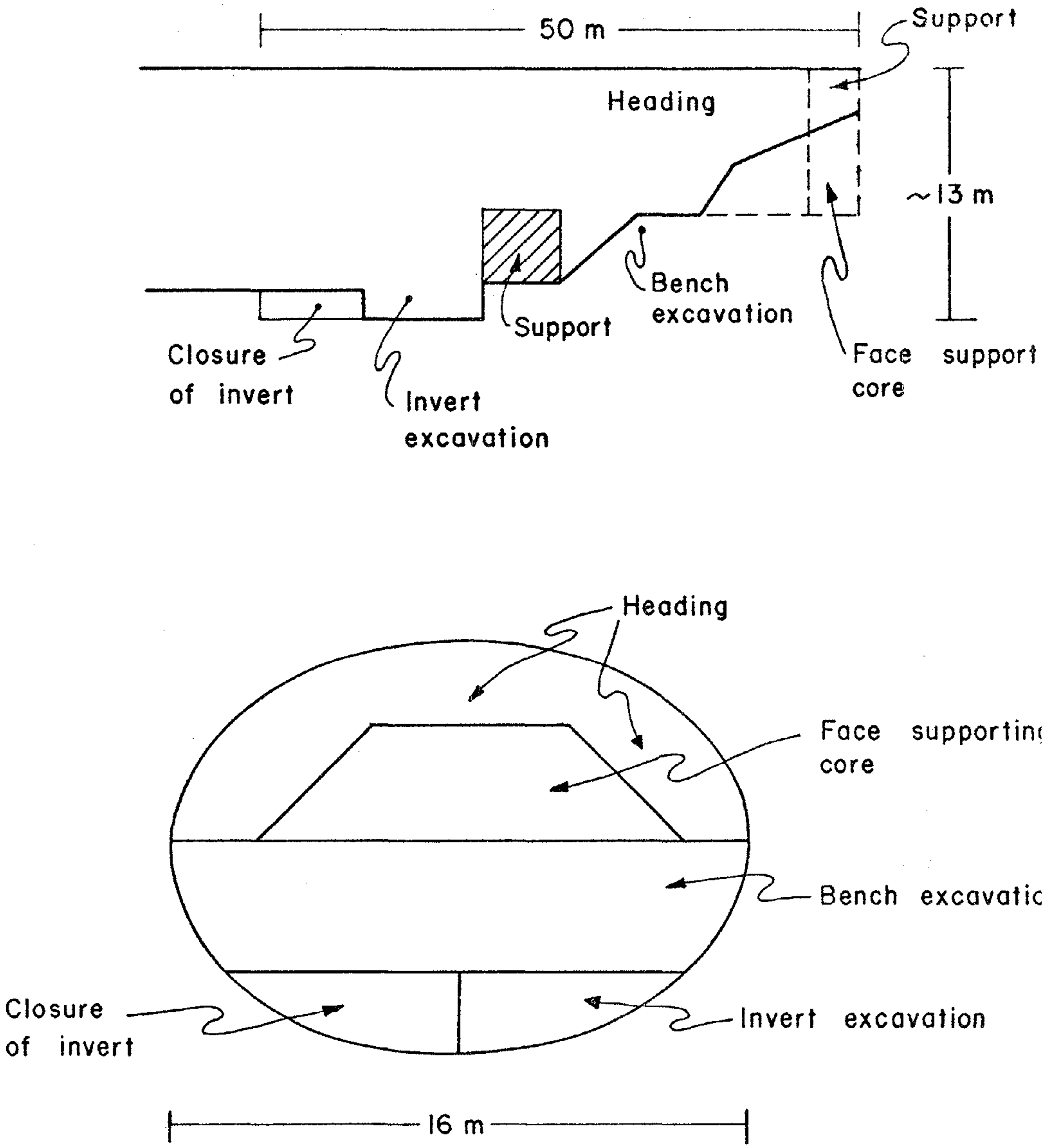
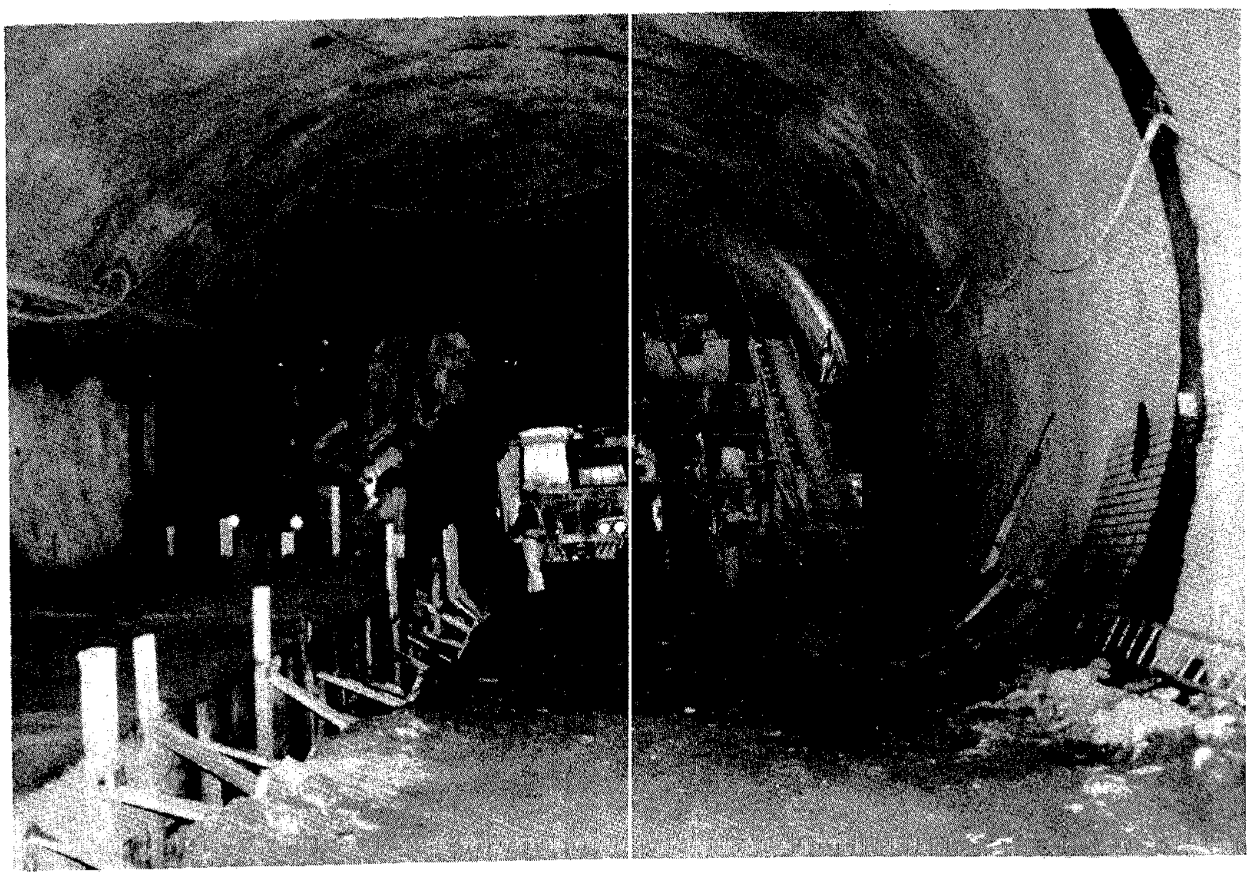


FIGURE B-7.4 SCHEMATIC VIEW OF EXCAVATION OF GANDERTOBEL TUNNEL (LONGITUDINAL AND CROSS-SECTION)

Cat 977 Broyt X-4EL



Invert Poured Kiruna K-250 Shotcrete Plant

FIGURE B-7.5a OVERVIEW OVER TUNNEL CONSTRUCTION IN GANDERTOBEL TUNNEL (PHOTO, W. STEINER, 1978)

Cat 977 Transporting Steel Sets

Convergence Measurement
Heading

Sidedrift
(not clearly visible)

Bench

FIGURE B-7.5b ACTIVITY IN THE HEADING OF THE GANDERTOBEL TUNNEL

TABLE B-7.1 EQUIPMENT USED AT GANDERTOBEL TUNNEL

Quantity	Make	Type
1	Broyt X-4EL	Excavator electro-hydraulic
1	Atlas Copco Boomer H 132	2-armed electro-hydraulic drill jumbo on wheels (For boltholes)
2	Cat 977	Tracked excavator
1	Cat 955, with platform	
1	Shotcrete plant unit, two shotcrete pumps, with storage silo, conveyor belt and accelerator mixer	Icoma (≈GM57)
1	Atlas Copco ZR5	Electric compressor, 50 m^3/min
1	Transformer	
2 to 3	Kiruna trucks K-250 (number depends on transport distance)	Dump trucks capacity 25 m^3
1	Concrete mix truck	
1	Supply truck, 5 tons	

TABLE B-7.2 PERSONNEL IN GANDERTOBEL TUNNEL

Number	Description
2 (+1)	Excavator operators (+ 1 reserve)
8 - 10	Miners who can operate most of the equipment (including mechanics)
2	Foremen
12 to 15	Persons

Tasks performed by the above personnel

2	Foremen
2	Excavator operators
10	Dump truck operators (number depends on transportation)
4	Nozzlemen
6	Shotcrete pump operators
3	Loader operators
6	Drillers
2	Blast foremen
2	Electricians
2	Mechanics for drill rig
2	Mechanics (pipefitters)
3	Operators for platform cars
44	total tasks

advance rates may in turn cause problems since, given critical stand-up time, it may become necessary to place more support, which again lowers the advance rate!!

The crew has to be versatile on an Austrian job; each member must be able to perform different tasks. A crew member must know how to place shotcrete, but he also may be required to operate a frontend loader, the hydraulic shovel or to drive a truck. This versatility, however, does not mean that safety is reduced. To keep safety standards high, the contractor for the Arlberg Eastern section is issuing internal operator's licenses to his staff specifying which type of equipment a person can operate.

The Flirsch Tunnel

The Flirsch Tunnel, which is located to the west of the Gandertobel Tunnel, is 820 m long and has a cross-section of 90 m. It passes through metamorphic rock of the Silvretta Nappe. The rock is a schistose gneiss with two major sets of persistent discontinuities striking primarily parallel to the tunnel axis. One set is steeply dipping; the second set is approximately horizontal. This second set is undulating as it follows folds and in some instances tends to dip towards the face. The two sets of discontinuities form a very unfavorable pattern favoring large scale roof failures.

Only one type of initial support system has been designed for the entire tunnel, i.e., only one ground class is provided. The support system approximately corresponds to a ground class III (see Einstein et al. 1977) of the Arlberg Tunnel; excavation is thus by heading and benching. The major difference from a regular class III is that, in addition to shotcrete, wire fabric and grouted bolts, steel sets are also required for support. They are spaced closely, i.e., 1.0 to 1.5 m, corresponding to one round length. The shotcrete thickness is 20 cm, and the rock bolts are 4 to 6 m long and placed every 1 to 1.5 m circumferentially. At the time of the visit, only the top heading had been advanced to station 150 m. The bench excavation had not yet started. Figure B-7.6 shows the face and the placement of wire fabric. Convergence measurements were not yet available. The equipment used for the excavation of the Flirsch Tunnel is listed in Table B-7.3. The crew size is the same as for the Gandertobel Tunnel (Table B-7.2).

3.3 The Arlberg Tunnel (Eastern Section)

3.31 General

At the time of the visit, construction of the Arlberg Tunnel was to a large extent completed. The interior liner had been placed, and the roof slab separation the ventilation channel from traffic space was nearing completion. The roadway had to be placed and the electro-mechanical and safety equipment had to be installed for the opening by December 1, 1978.

After holing through of the tunnel, Mr. Treichl has published a brochure describing the work performed by the Arlberg East Contractors, which provides the basis for this description. In addition, the special issue of Oesterreichische Ingenieur-Zeitschrift (Vol. 21, No. 11, 1978), published at the occasion of the opening, provides a wealth of information.

Figure B-7.7 shows construction schedules: the schedule in the owner's bid documents, the schedule which served as the basis of the contract, and the finally executed schedule.

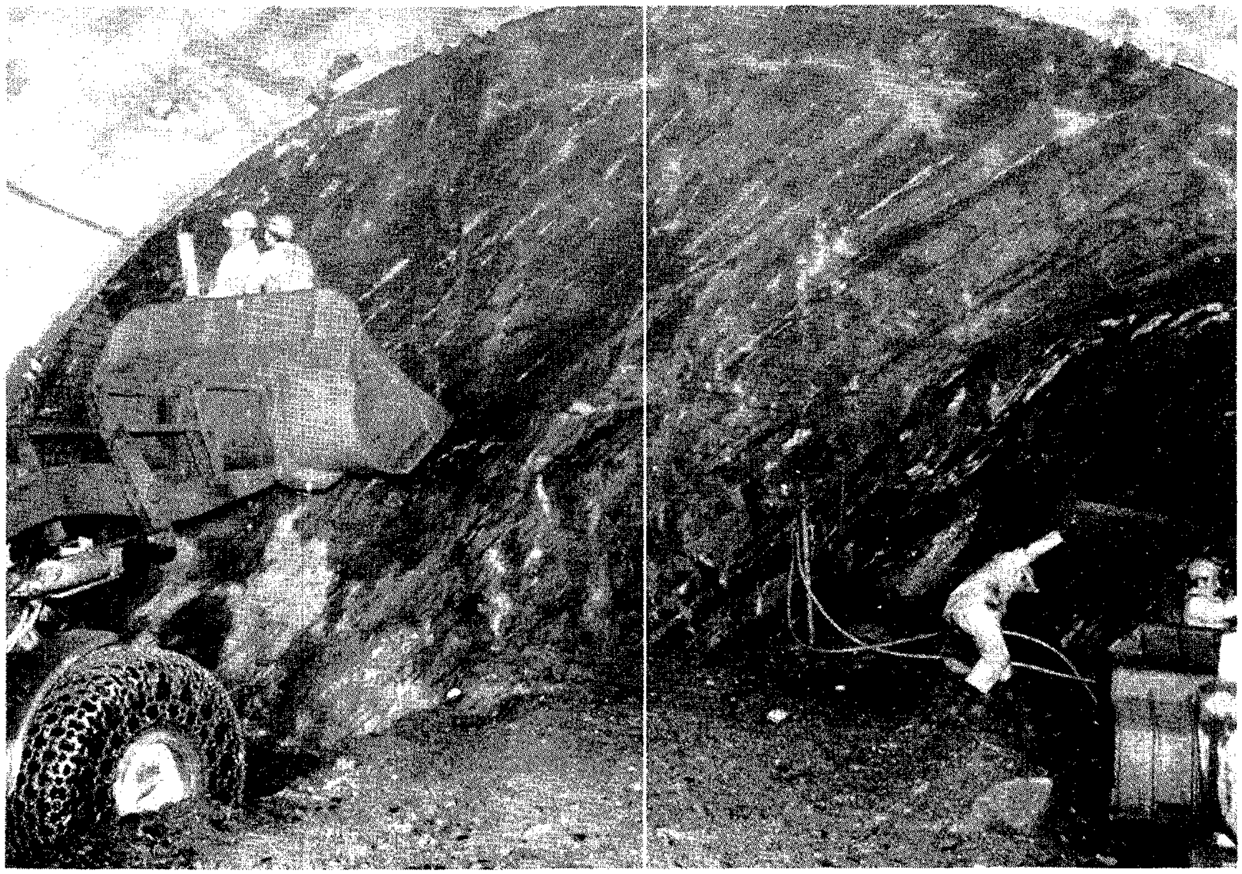


FIGURE B-7.6 SUPPORT AT THE FACE OF THE FLIRSCH TUNNEL (PHOTO, W. STEINER, 1978)

TABLE B-7.3 EQUIPMENT IN FLIRSCH TUNNEL

Quantity	Make	Type
1	Atlas Copco Promec	Drill jumbo 4-armed electro-hydraulic
1	Cat 966	Frontend loader
1	Broyt X-4EL	Excavator electro hydraulic
1	Shotcrete plant with two shotcrete pumps, storage silo, conveyor belt and accelerator mixer	
2	Cat 955	Platform cars
1	Atlas Copco ZR5	Compressor $50m^3/mm$
1	Transformer	
2 - 3	Kiruna Trucks K250	
1	Ventilator	

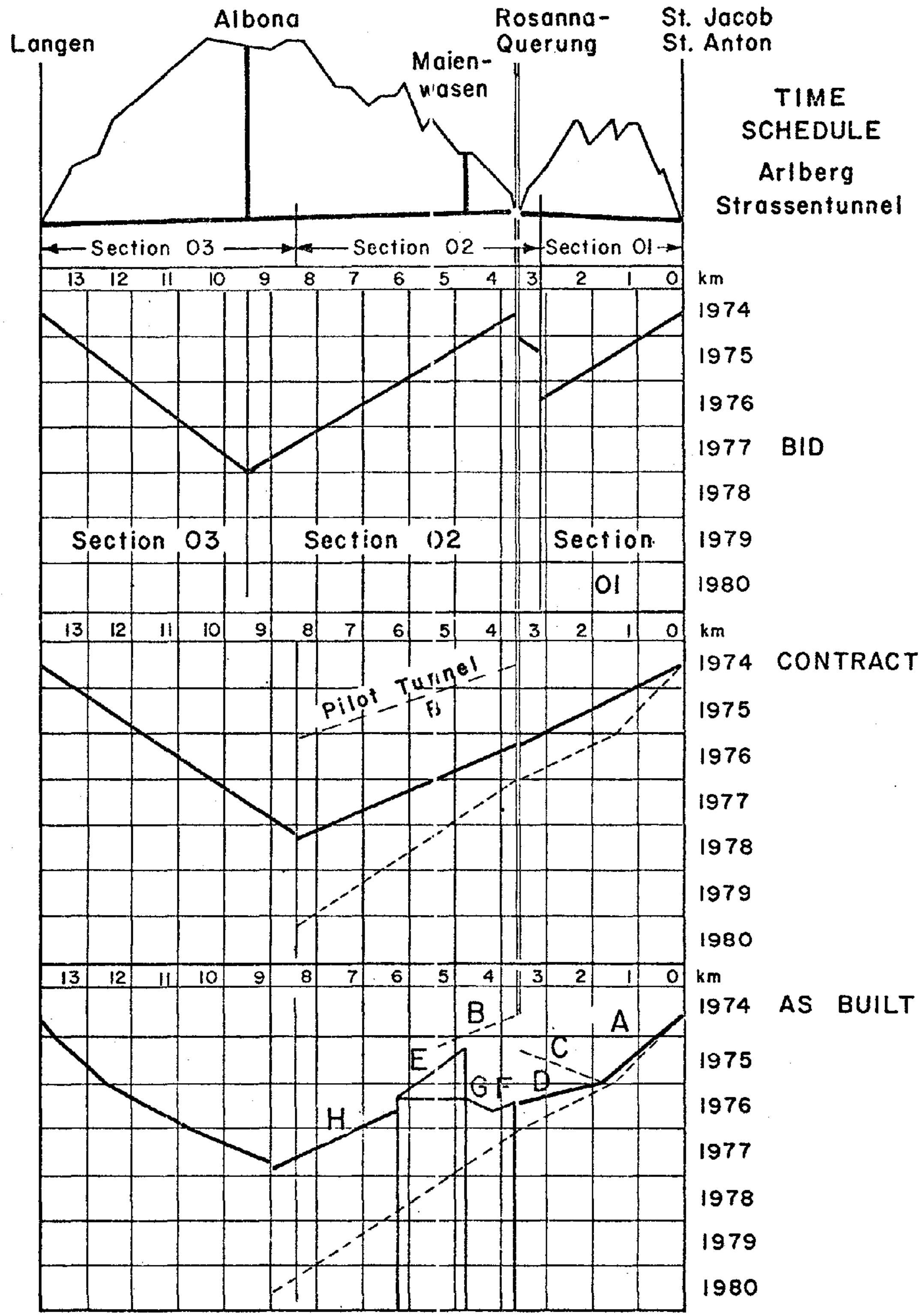


FIGURE B-7.7 CONSTRUCTION SCHEDULE OF ARLBERG TUNNEL (FROM TREICHL, 1977)

The contractor joint venture Arlberg East was awarded the construction contract for sections 01, 02 and 03, i.e., a total length of tunnel of 8.5 km and the shaft Maienwasen of 230 m depth.

The schedule adapted for the contract (Figure B-7.7) called for excavation of the tunnel from the eastern portal (Section 01) with a heading and bench excavation (A) and supported with shotcrete and rock bolts. At the same time (B), a pilot tunnel of 16 m^2 was started in Section 02 from the Rosanna Gorge (initially an 8 m^2 pilot tunnel was planned). This pilot tunnel should serve as an exploration drift for the tunnel and also make it possible to start the excavation of the Maienwasen shaft.

As the encountered ground conditions in section 01 and 02 were worse than anticipated, the schedule was changed. The pilot drift in section 02 (B) was stopped. Instead, a pilot tunnel of 16 m^2 in the crown was driven in section 01 eastbound from the Rosanna Gorge (C).

After holing through this pilot tunnel, it was widened from east to west to a full top heading (D). This top heading was supplied with electricity, air, water and support material (Steelsets, bolts, shotcrete, wirefabric) from the west, while mucking was eastbound. The muck was dumped on the edge of the bench and then transported out eastbound with the bench excavation which followed this heading also from east to west.

Essentially two independent construction sites were thus available. The advance rate for this section was 12 m/day on the average, with peak rates of 22 m/day. One problem developed in the pilot tunnel, as the ground (gneiss) was deteriorating rapdily due to equipment traffic, notably in areas where water was present. The invert of the pilot tunnel had to be protected with 20 cm of concrete.

The pilot tunnel in section 02 had been halted approximately 800 m west of the Maienwasen cavern. Instead of continuing a pilot drift in the base, the top heading of the tunnel was excavated and supported for a distance of approximately 1.5 km (E), starting at the base of the Maienwasen shaft simultaneously with the tasks C and D. After completion of section 01 (D), the tunnel in section 02 between the Rosanna Gorge and the shaft was widened to its full cross-section (F, G).

This excavation was performed from the east (F) and west (G). The muck from the west excavation (F) was temporarily deposited in the already excavated section (E) and the Maienwasen cavern. Once this section (G,F) was fully excavated, tunnel driving continued in H. However, in E only the heading had been excavated. The bench and invert excavation followed initially at a distance of 1.5 km; normally this distance should only be 200 m. The distance was reduced by bench excavations from intermediate points of attack within E (while excavation continued with heading and benching in H).

This adaptation of the construction sequence to the prevailing ground conditions was only possible because the design-construction method is very flexible and adaptable. The fact that trackless equipment was used is particularly important. During these described changes in operation, the pilot tunnel was sometimes in the lower part of the final tunnel, sometimes in the top. The trucks thus had to climb and descend some slopes.

If excavation had been in accordance with the guaranteed advance rates of the contract, the total time required for holing through would have been 2.5 years longer. Despite the considerably worse ground conditions the anticipated, schedule could be kept. This would not have been possible without adaptation of the construction procedure to the encountered ground conditions. A similar increase in advance rates for worse ground conditions has been achieved in the western section of the Arlberg Tunnel (see section 4.2 of this appendix).

3.3.2 Equipment Used at Arlberg East

The contractor had previously used some of its equipment in the southern section of the Katschberg Tunnel (1970-74). Additional equipment was purchased when construction started. Of note were new electrohydraulic drill jumbos. Other new purchases included Kiruna trucks; originally only smaller K-160 (16 m^3) were used, but these were later supplemented by the larger Kiruna K-250's with a capacity of 25 m^3. The exhaust of these diesel trucks is cleaned in scrubbers.

Ventilation in the tunnel was continually adapted to the type of excavation in progress. A minimum of 2700 m^3/min of fresh air was required at the face. Once the ventilation channels of the highway tunnel were completed, they were used for construction ventilation. The final ventilation channels were usually completed at a distance of approximately 1200 m from the face. Beyond the final ventilation channels, air was pumped through a pipe of 120 cm diameter with a ventilator installed at the end of the final channels.

A detailed description of the ventilation system, as well as other features of the site installation like concrete plant, and temporary housing, can be found in the brochure by Treichl (1978) and OIZ (1978).

3.3.3 Technical Problems

In the eastern section, the ground conditions deteriorated with advancing excavation. The worst ground conditions required a top heading with two benches.

The total length of bolts placed per lineal meter of tunnel is on the order of 100 to 120 m; total convergences reached a maximum of 75 to 80 cm. The interior liner with a theoretical thickness of 25 cm has a real thickness of 50 to 60 cm due to overbreak and deliberate overexcavation.

The drilling of holes for the rock bolts requires continuous flushing with water. For a 9 m long bolt the water required is in the order of a 1,000 liters per hole. Some of this water certainly penetrates the rock and leads to a deterioration of the rock mass, especially when the rock is originally dry. The drill water may actually produce unwanted pore pressures and softening of the ground. Thus, high density bolting may possibly not have a stabilizing effect.

3.3.4 Geological Problems

This part is based on the interview with Dr. F. Kunz, the site geologist of the owner, the Arlberg Highway Tunnel Authority. During the interview, Dr. Kunz explained the general geological problems of the Arlberg Tunnel and some details that were observed in the tunnel. Figure B-7.8 shows a geologic map of the Arlberg area and Figure B-7.9, of geologic section along the tunnel.

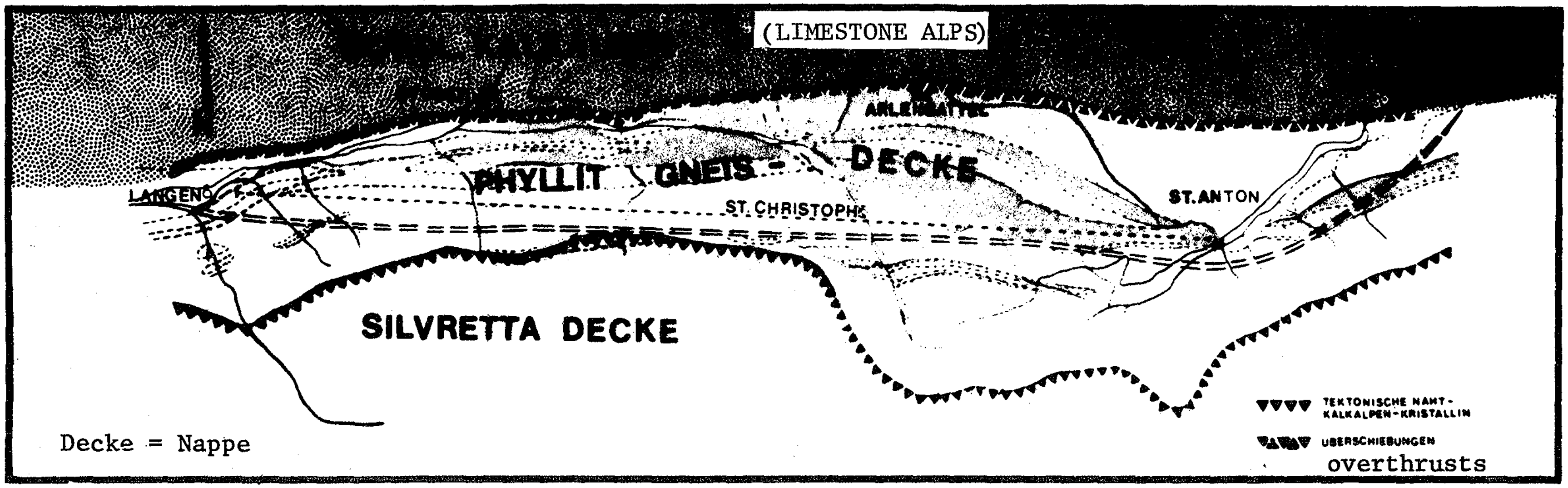


FIGURE B-7.8 GEOLOGIC MAP OF ARLBERG REGION (FROM TREICHL, 1978, AFTER W. RESCH)

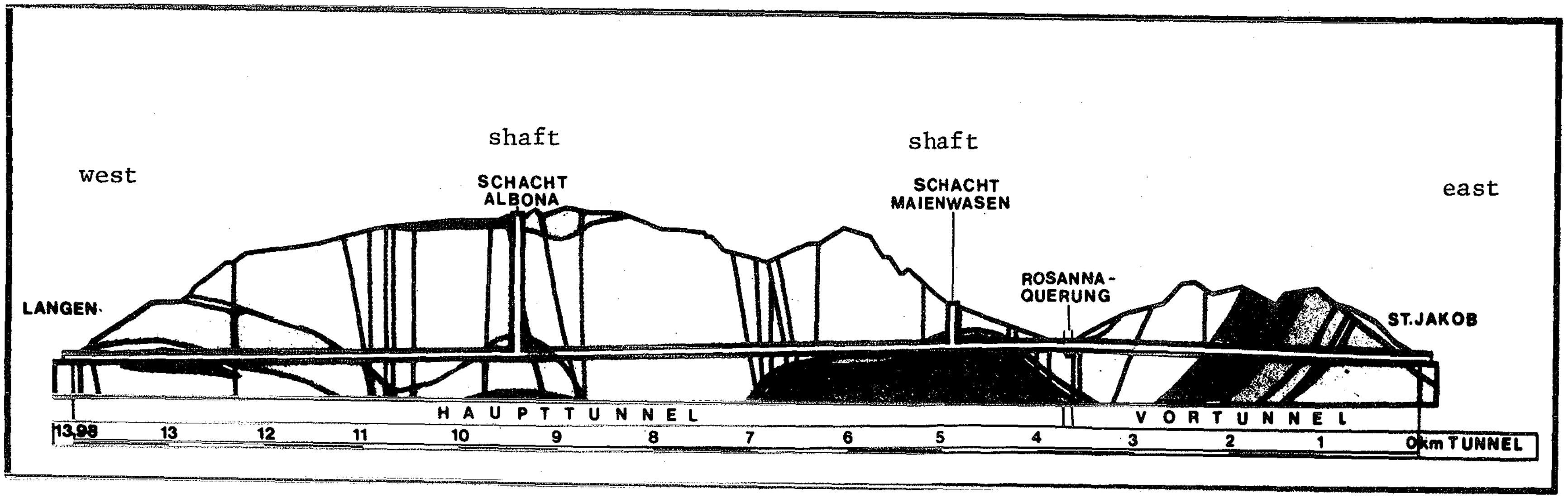


FIG. B-7.9 GEOLOGIC SECTION ALONG ARLBERG (FROM TREICHL, 1978)

The Arlberg area is the contact zone of the crystalline and limestone Alps. The existing railroad tunnel and the new highway tunnel are located in the gneiss-phyllite nappe of the crystalline. The geologic thrust direction has been from south to north, and faults and shear zones thus are not unexpectedly striking generally east-west, subparallel to parallel to the tunnel axis.

Earlier studies considered also tunnel alignments which would have passed to the north through the limestone Alps. This would have required at least one change from the crystalline to the limestone. Due to the potential problems in the cross-over from the limestone formation into the crystalline formation (fault zone) and due to the possibility of large water inflow in the limestone, this plan was abandoned in favor of an alignment parallel to the existing railroad tunnel where the ground conditions were better known.

Performance monitoring and support adaptations The convergence of the initially supported tunnel has been monitored by convergence measurements. In the eastern section of the Arlberg Tunnel, measurement points were placed and initial measurements were taken 12 to 24 hours after excavation of the round. (In contrast, in the western section, the first measurement was taken 6 hours after excavation). These differences between the two sections imply that the total deformation and the initial rates of deformation cannot be directly compared.

Criteria for support performance and adaptation In the eastern section of the Arlberg Tunnel, empirical criteria listed in Table B-7.4 were established for support and excavation adaptation. The first performance criteria to be used

TABLE B-7.4 CRITERIA USED AT THE ARLBERG EAST SECTION (AFTER KUNZ, 1978)

Rate of Deformation	Construction Criteria
Residual Rate in Heading < 0.5 cm/d	Spacing of stellsets (round length) is reduced
Initial Rates in Heading > 2 cm/d	Additional bolts are required for the next round
Initial Rate in Heading > 5 cm/d	Bench excavation starts only when rate of deformation is less

are the initial rates of convergences. (One day after placement of monitoring bolts). If the initial rate of convergence exceeds 2 cm/day, additional bolts are placed in the already excavated and supported sections and also in the following rounds. If the rate of convergence exceeds 5 cm/day, then the spacing of the steelsets for the following rounds is reduced. This results also in an increase in the density of bolting, since the same circumferential bolt pattern is used for each steelset.

The third criterion determines bench excavation, which is only allowed once the rate of convergence is less than 0.5 cm/day.

The criteria used in the eastern section are comparable to those in the western section, where support is considered sufficient when the total deformation after two days is less than 4 to 6 cm, which corresponds to a rate of 2 to 3 cm/day.

Additional Problems According to Dr. Kunz, squeezing did not occur under the greatest overburden. In this context, some other "inconsistencies" noted by Dr. Kunz are worth discussing. In one case, convergence measurements were available for the pilot and main tunnels. The convergence monitored in the pilot tunnel of 4 m diameter was 11 cm. For the main tunnel of 11 m diameter, the convergence monitored was only 8 cm (Figure B-7.10).

Around the circumference of the pilot tunnel, a loosened zone 20 to 50 centimeters thick was observed. This is insufficient to explain the large convergence. It may be hypothesized that the difference in convergence may be caused by the fact that deformations occur both ahead of the face of the pilot tunnel and ahead of the top heading. These "ahead of the face"

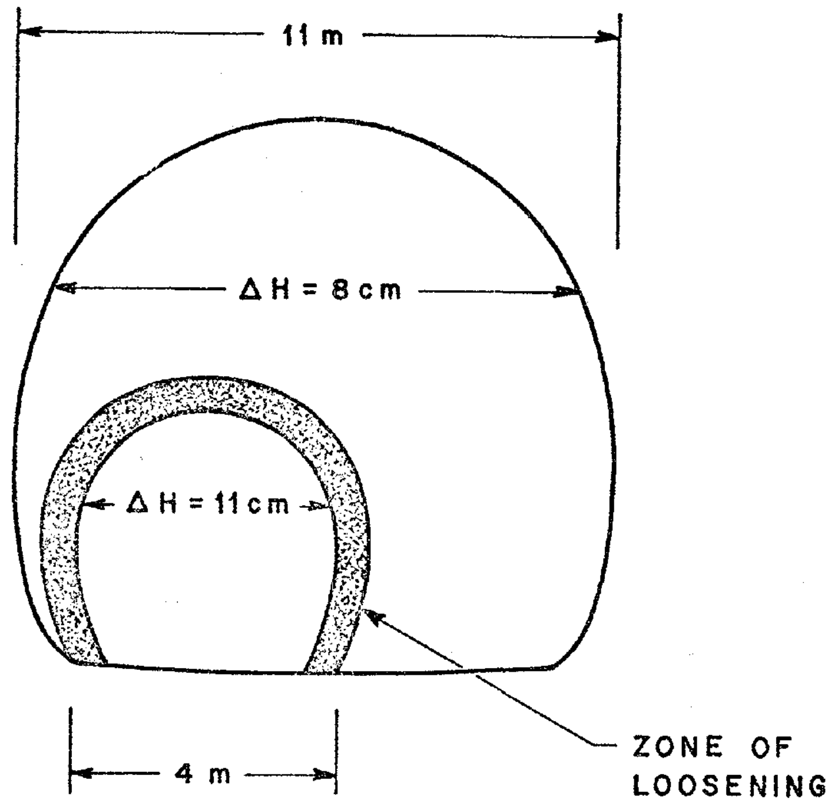


FIGURE B-7.10 DISCREPANCIES IN CONVERGENCES OBSERVED IN ARLBERG EASTERN SECTION (AFTER KUNZ, 1978) deformations are substantially greater in the top heading than in the pilot tunnel, and as consequence the residual deformations that can be observed as convergence are smaller in the top heading.

In the vicinity of the face, longitudinal movements behind the sidewalls have been found, as Dr. Kunz mentioned. Rock was sheared off at a depth of 0.5 to 1.0 m from the tunnel wall, as shown in Figure B-7.11. These movements were observed in bolt holes where no bolts had been placed and which became off-set a short time after drilling (as shown in Fig. B-7.11); sometimes bolts were even sheared off.

From his experience at the Tauern and Arlberg tunnels, Dr. Kunz concluded that not only the crew quality is important, but also the mood of the crew. He noted that performance of the tunnel support placed was not as good when the crews returned from their days-off or when the days-off approached as compared to the middle of a work period (decade). This effect has, however, not been quantified.

4. Arlberg Western Section

4.1 General

For the overall geologic conditions the reader is referred to Figures B-7.8 and B-7.9. This tunnel section was constructed by Arlberg West Contractor (ARGE Arlberg West) a joint venture of several Austrian firms (see page 372), that had previously built the Tauern Tunnel and the northern section of the Katschberg Tunnel. Equipment used initially at the Arlberg had been transferred from the Tauern.

The difficulties which had to be overcome, however, mainly as

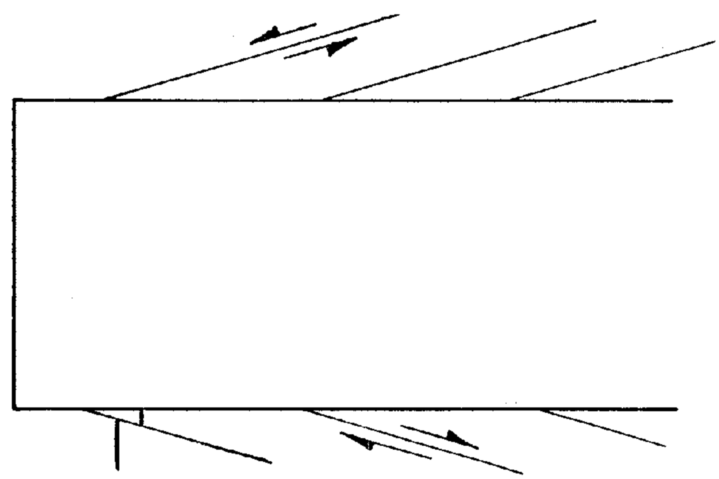


FIGURE B-7.11 LONGITUDINAL MOVEMENT IN THE SIDE-WALLS
(AFTER KUNZ, 1978)

a consequence of ground conditions, were much greater than at Tauern.

The description of the western section of the Arlberg Tunnel will follow a format similar to that for the eastern tunnel section. First, problems of scheduling and equipment are discussed, followed by technical problems and geological aspects.

At the time of W. Steiner's visit (January 1978), the tunnel had been holed through (10/10/77). Final construction work in progress included the placement of the final liner for 500 m near the section boundary and the ventilation channels for approximately 1 km. Simultaneously, work continued in the ventilation chamber at the base of the Albona shaft.

During W. Steiner's visit, he could talk to Mr. Mayrhauser, site manager of the contractor; Mr. Obst, section engineer; and Dr. Kaiser, the field geologist of the owner. Mr. Obst was the guide during the site visit.

4.2 Scheduling and Equipment

Since the joint venture, Arlberg West Contractors, had previously constructed the Tauern Tunnel, the bulk of the equipment was already available. This included tracks, locomotives and cars as well as compressors, pipelines, transformers, mucking equipment (Cat 966 and Cat 977). (Track haulage was also selected because the section is 5.1 km long and rail transport is advantageous over such great lengths). The excavation-support procedure is similar to the one used for the Tauern Tunnel. A top heading (Figure B-7.12) of 35 m^2 cross-section was followed by two benches of 30 m^2, which were excavated alternately on the right or left side; finally, the invert of 10 m^2 was excavated. The total excavated cross-

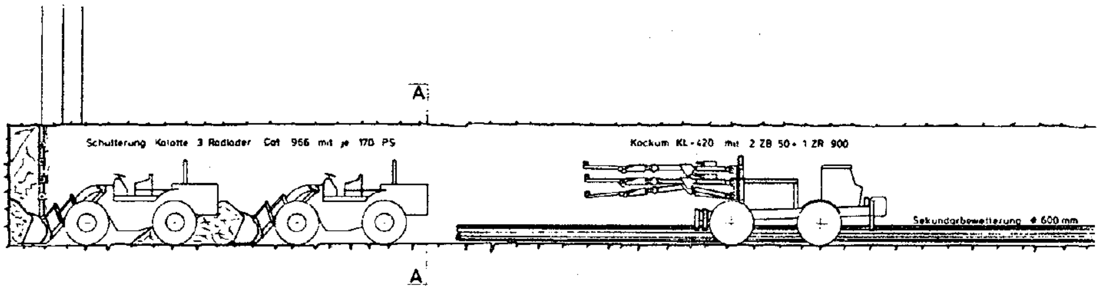


Abb. 1 Kalottenvortrieb. (Heading)

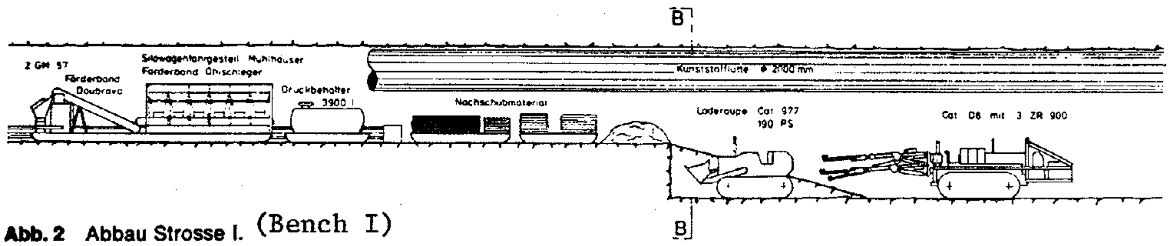


Abb. 2 Abbau Strosse I. (Bench I)

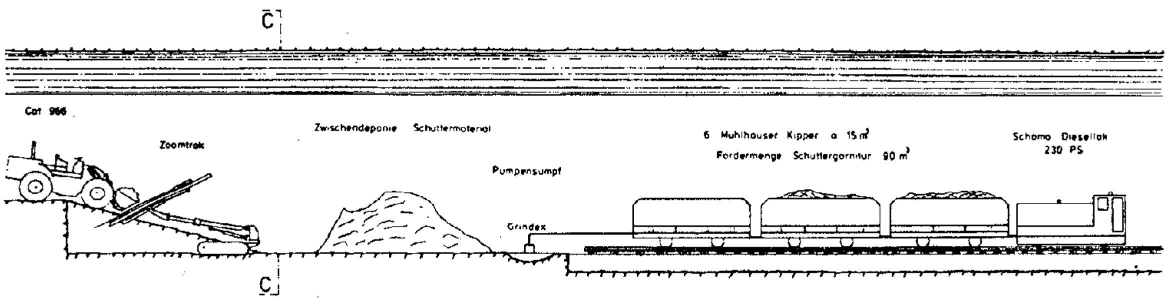


Abb. 3 Abbau Strosse II. (Bench II)

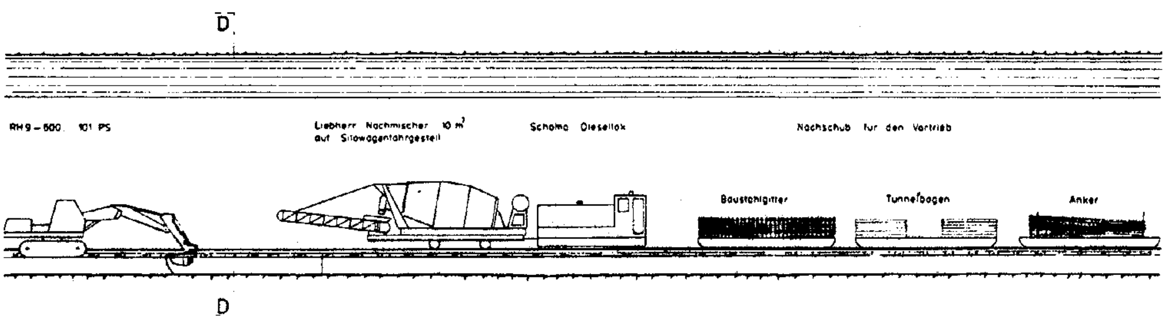


Abb. 4 Sohlaushub und -beton. (Invert)

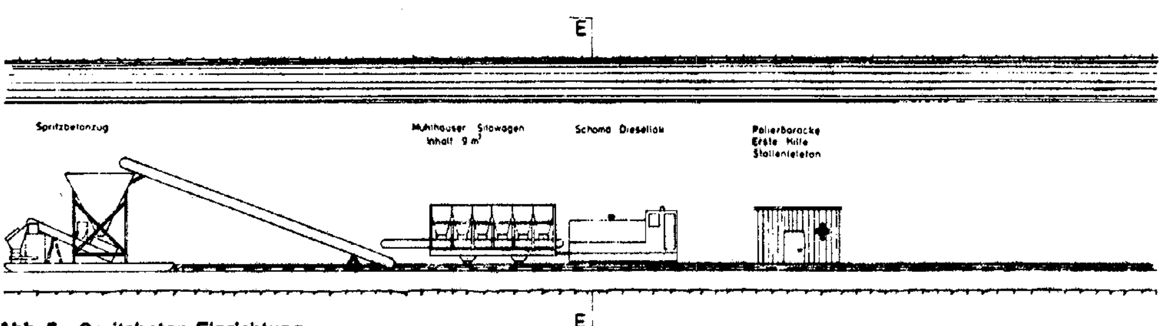


Abb. 5 Spritzbeton-Einrichtung. (Shotcrete-Equipment)

FIGURE B-7.12 EXCAVATION PROCEDURE FOR ARLBERG WEST SECTION (FROM KICHLER, 1976)

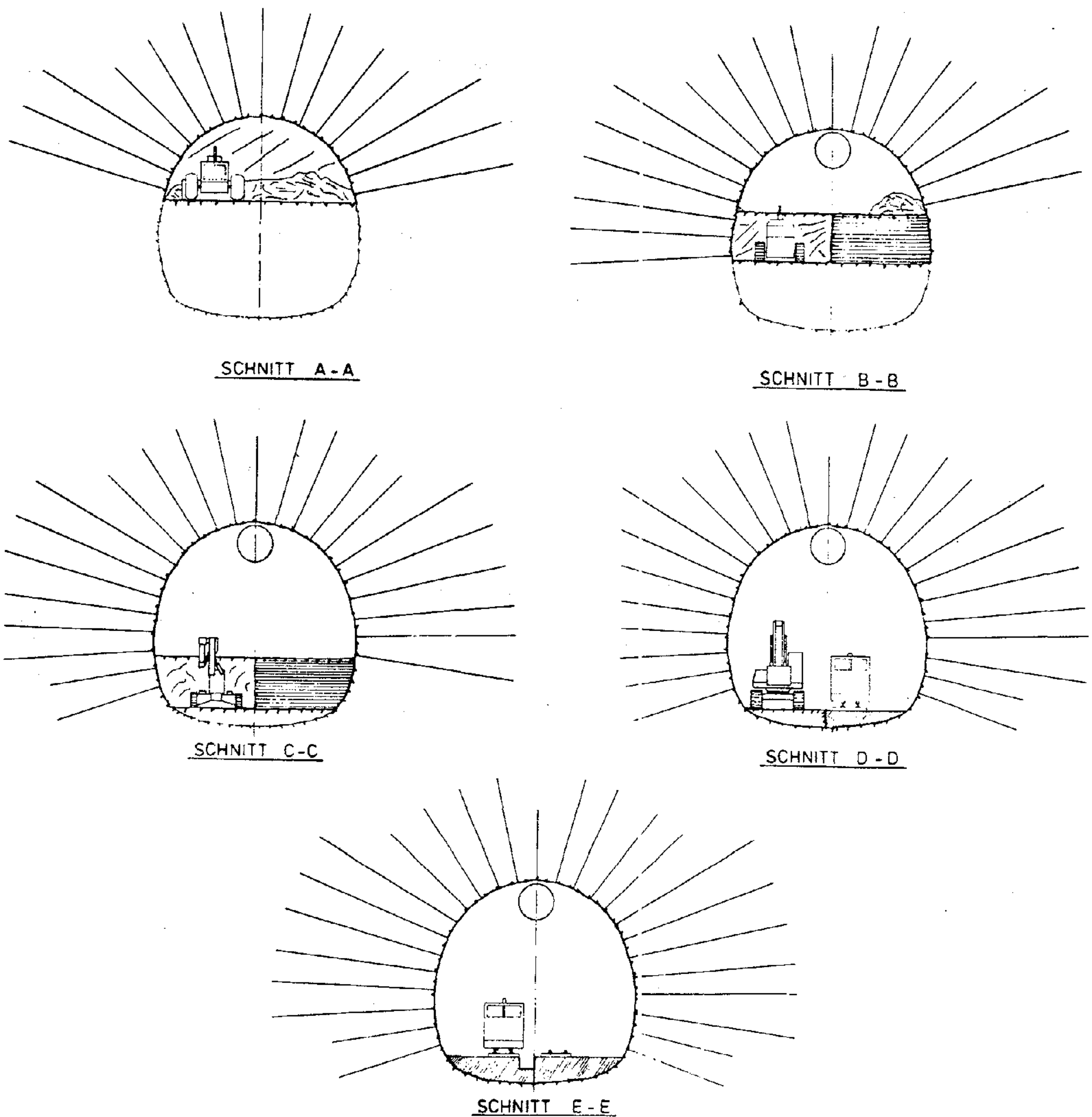


FIGURE B-7.12 EXCAVATION PROCEDURE FOR ARLBERG WEST SECTION (FROM KICHLER, 1976) (CONT.)

section thus is approximately 100 - 105 m^2.

The type of excavation and the equipment are shown on Figure B-7.12. The general as-built schedule is shown on Figure B-7.2 and a detailed schedule with advance rates for the first 2,000 m of tunnel in Figure B-7.13. The ground conditions were very unfavorable in the first part of the section and the advance rates were thus lower (The support requirements are described in section 4.3.2 below). Eastward, with improving ground conditions, the advanced rates increased to a maximum rate of 11.2 m/day, which was reached on one day in August, 1977.

Average daily rates over the last month of construction were on the order of 8 m/day. Figure B-7.2 shows that the advance rates improved considerably with advancing tunnel and a comparison with the advance rates of the eastern section reveals that for comparable conditions the western section eventually attained somewhat higher advance rates.

Originally, the advance for the western section was performed under a 3/3 operation (see Section 5 of the main text); a 4/3 operation was used once the advance rate dropped behind schedule. As mentioned in Section 5, main text, regular equipment maintenance becomes a problem in 4/3 operation, since most of the equipment is in constant use and cannot be serviced. Breakdowns of equipment might result in a reduced advance rate. A 4/3 operation may thus require additional equipment to allow proper maintenance and servicing. However, no data has been obtained whether there were more breakdowns of equipment and whether back-up equipment had been used.

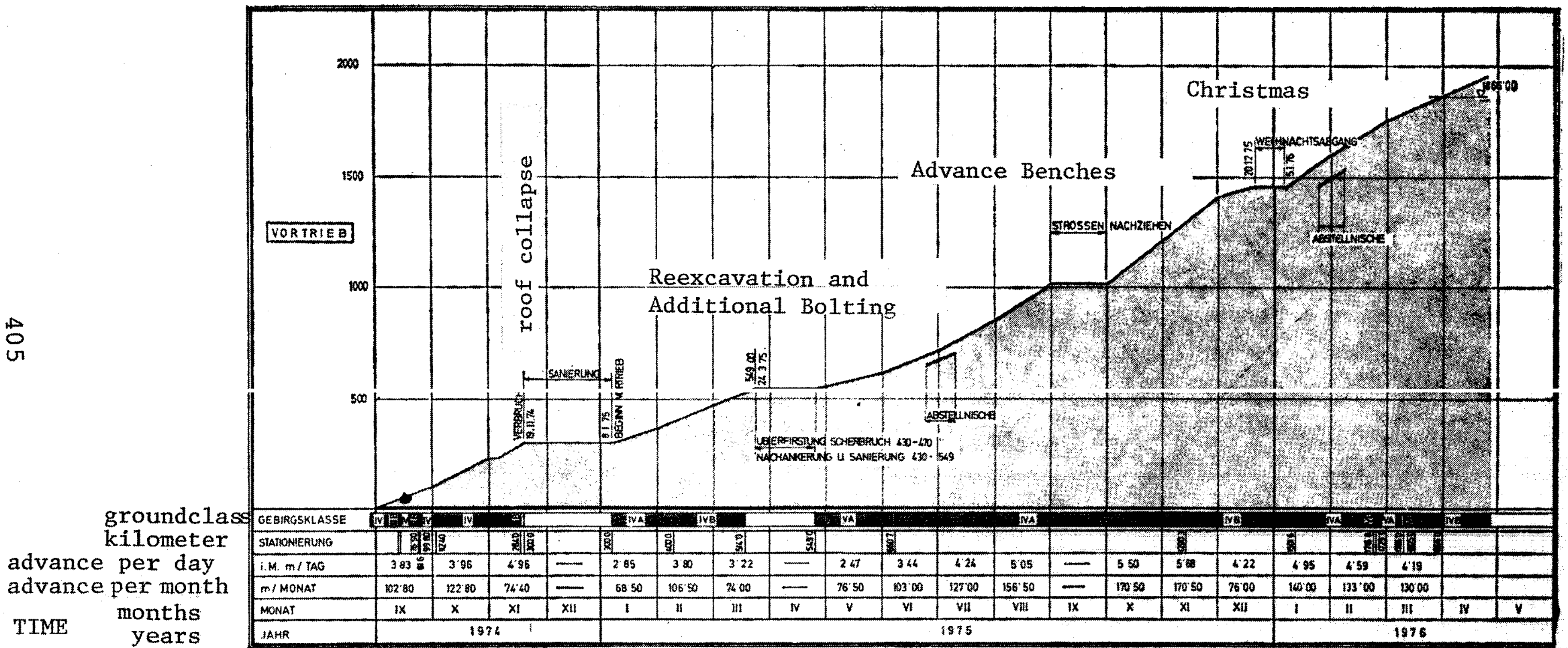


Bild 5: Fortschrittsdiagramm

FIGURE B-7.13 DETAILED SCHEDULE, ARLBERG WEST (FROM MAYRHAUSER, 1976)

4.3 Technical Problems

Problems regarding ground classification large deformations and support procedures had to be solved. Also, accidents due to unfavorable ground conditions are discussed.

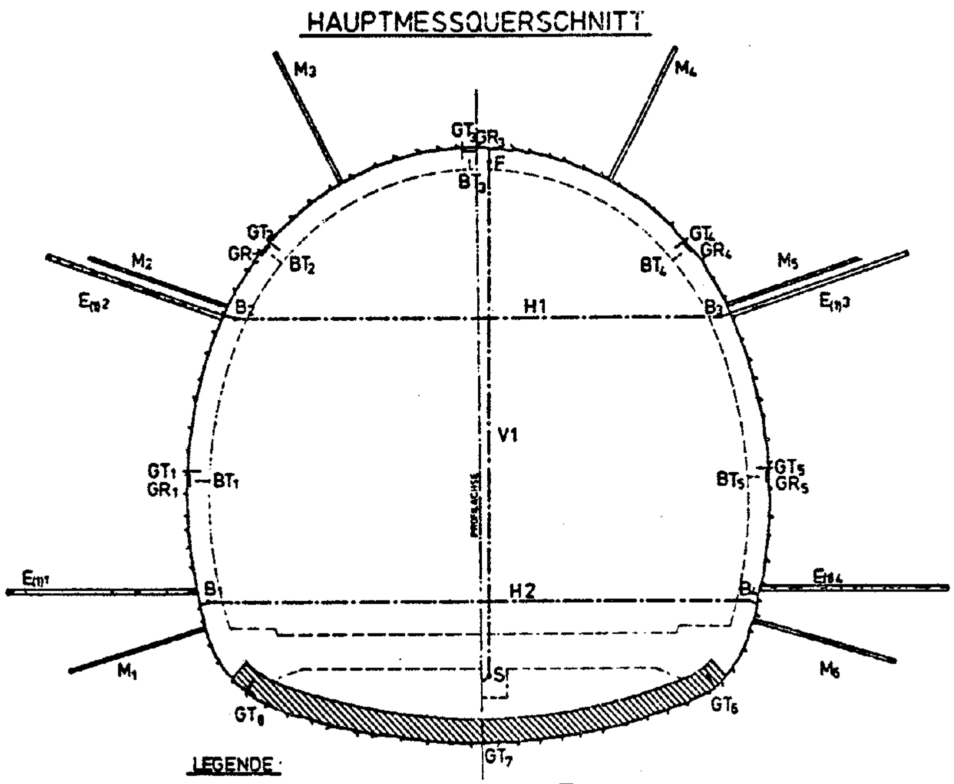
4.3.2 Problems of Ground Classification

Payment provisions and ground classification procedures are described in sections 4.5.3 and 5.4.3 of the main body of this volume.

The bonus-malus provision did not work properly in the Arlberg West, since the actually placed support nearly always exceeded the predicted quantities. In particular, the total placed bolts lengths often were 5 to 10 times the predicted ones (Fig. B-7.17b). The contractor was thus actually always in a malus position. Further, the bid had called only for bolts up to 6 m length, while actually 9 and 12 m bolts were placed. The contractor had to submit a supplemental bid. Longer bolts cost more on a per length basis because the drill steel has to be extended (normal length of drill rod = 4m) and because the extra long bolts also require more labor. The contractor used (or at least intended to use) the supplemental bid to compensate for the malus. The entire issue is still not completely solved; however, construction always continued and was never shut down due to contract disputes. Austrian Standard B2110 does not allow a contractor to shut down a site in case of disputes, except if his bills have not been paid.

After excavation of the heading, the tunnel experienced large convergences on the order of 50 to 70 cm. Thus, overexcavation is necessary to obtain the necessary cross-sectional area after convergence has stabilized. The required overexcavation has to be predicted reliably. Since no theoretical procedure is available, convergence has to be predicted based on experience gained over the first 1,000 meters of tunnel. Performance was monitored as follows: (i) in the principal monitoring cross-sections, spaced 500 m, convergence measurements, extensometers, force monitoring bolts, stress cells (Figure B-7.14); (ii) in regular monitoring cross-sections which are more closely spaced, often every 10 meters, convergence (H1) and crown settlement (F). Two typical convergence-time curves were established for the western section of the Arlberg Tunnel (Figures A-7.15 and A-7.16). The curves in Figure B-7.15 apply to the case where major shear zones exist outside the cross-section of the tunnel, while curves in Figure B-7.16 apply in case the shear zone intersects the the tunnel. In the first case, the convergence two days after excavation and support of the top heading is between 4 to 6 cm; in the second case it reaches 8 to 10 cm. Convergence two days after excavation was used as a performance criterion. If the actual observed convergence exceeded the above quoted values, the support was adapted: additional bolts were placed in this section and more support was placed from the outset in the next rounds. If convergence was less than these values, the support for the next round was reduced. These convergence criteria made it possible to limit the total convergence (H_1) to 50 cm on the average ($\pm$ 10 cm).

Figure B-7.17 is a summary drawing provided by the Arlberg West Contractors which illustrates the total length of bolts placed per meter of tunnel, the additionally placed bolts, observed convergences, overexcavation (types and numerical values), and ground classes (according to the contractor and the owner)



LEGENDE:

B, F, S } Measurement Bolts

H, V } Convergence Base Difference

E(t) ... Extensometer

M ... Bolt equipped with strain gauges (force monitoring)

GR (radial), GT (tangential), BT (tangential) } Stress Cells

FIGURE B-7.14 MONITORING CROSS-SECTIONS

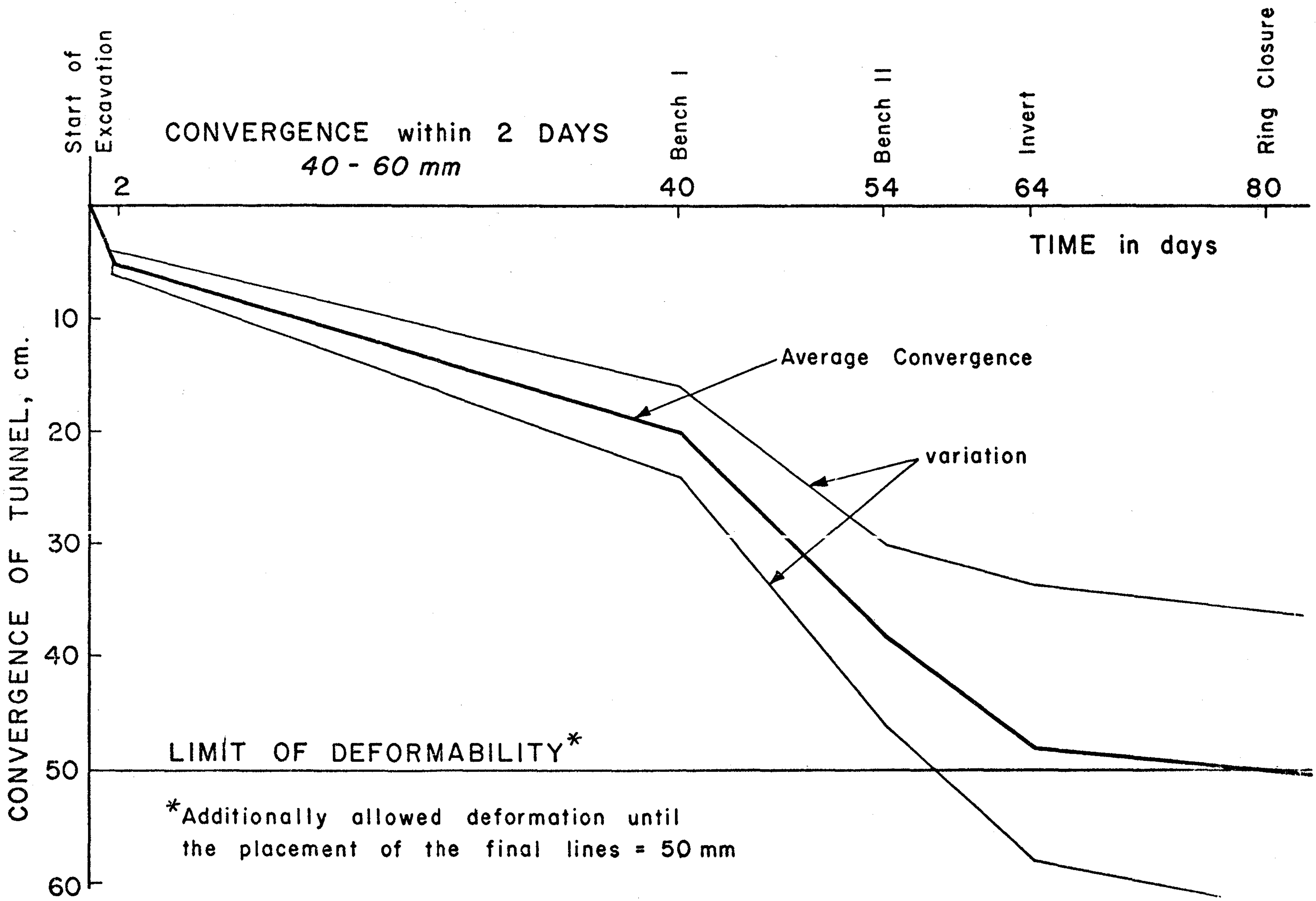


FIGURE B-7.15 TYPICAL TIME-CONVERGENCE CURVE FOR THE CASE MAJOR SHEAR ZONES ARE OUTSIDE THE TUNNEL (FROM ARGE WEST, 1978)

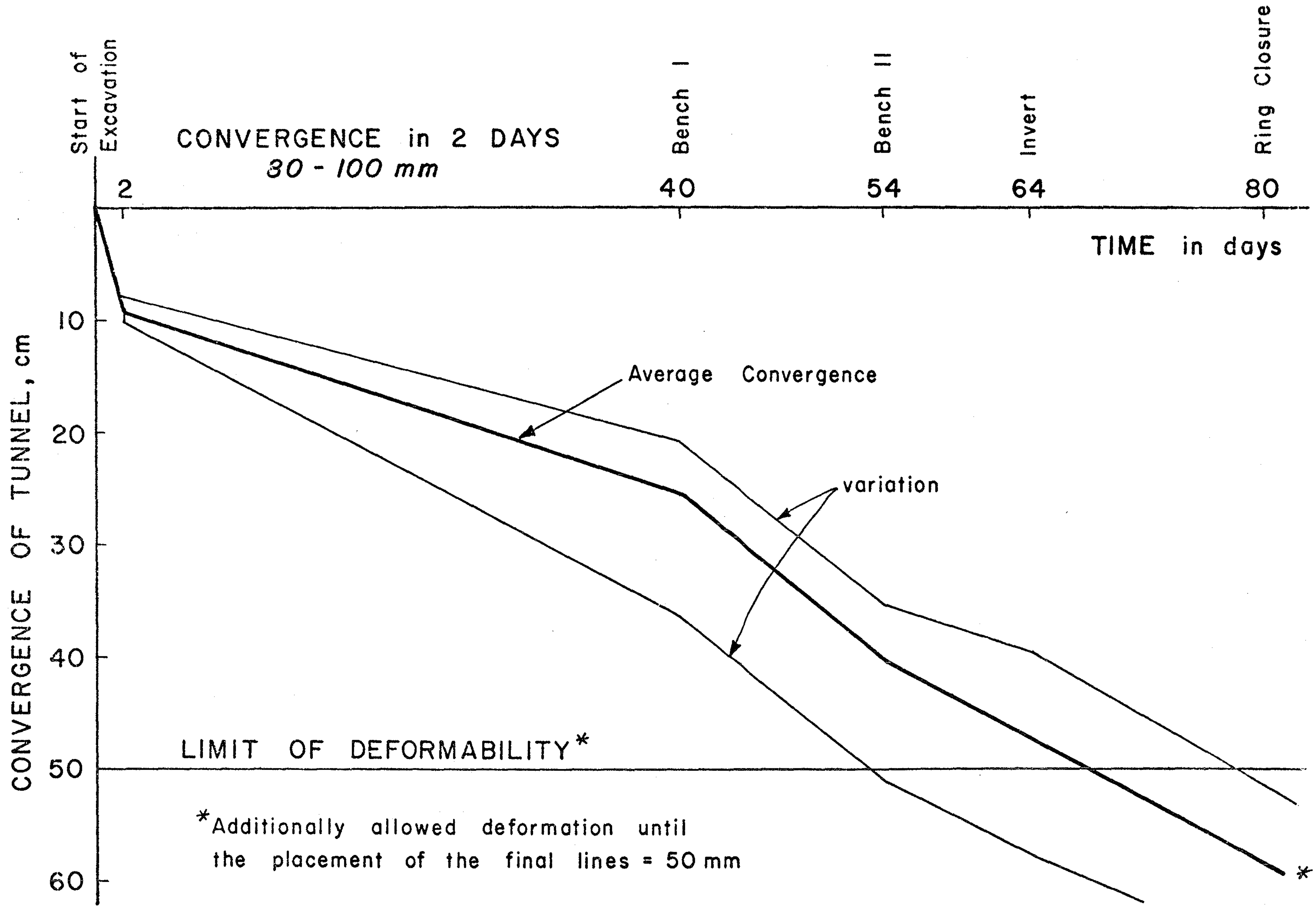
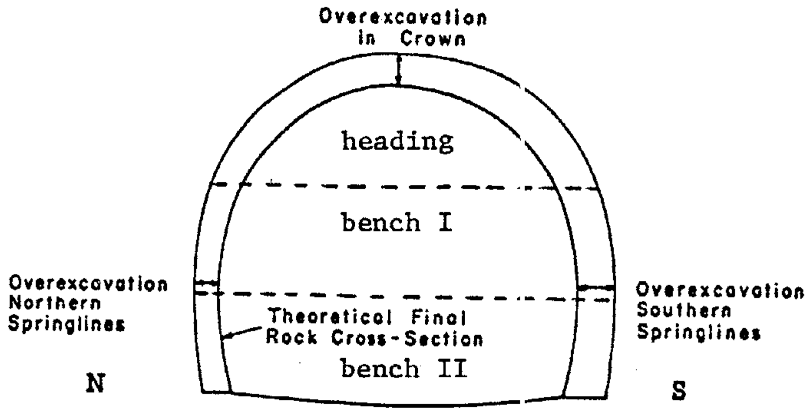


FIGURE B-7.16 TYPICAL TIME-CONVERGENCE CURVES IN CASE MAJOR SHEAR ZONES INTERSECT THE TUNNEL (FROM ARGE WEST, 1978)



OVEREXCAVATION (AS BUILT)			
TYPE	HEADING (cm)	BENCH I (cm)	BENCH II (cm)
1	20	20	20
2	30	30	30
3	30 ← 40 → 30	30	30 → 15
4	40	40 → 30	30 → 20
5	50	50	50 → 15
6	Circular Cross-Section	R = 6.21	
7	50	50	50
8	30	30	30
9	50	50	50 → 30
10	50	50	50 → 15
11	50	50	50 → 0
12	50	50	50
13	30	50 → 35	35 → 0
14	40 ← 40 → 70	No. 40 ← 0 So. 70 0	No. 40 ← 0 So. 70 0
15	40	40 0	40 0
1A	20	30	30
1B	30	40	40
1C	30	50	40
1D	30	50	50 → 0
2A	30	40	40
2B	40	50	50
2C	50	70	70
3B	50	70	70

FIGURE B-7.17a SUMMARY OF SUPPORT, CONVERGENCES AND GROUND CLASSIFICATION FOR ARLBERG WESTERN SECTION (FROM ARGE WEST)

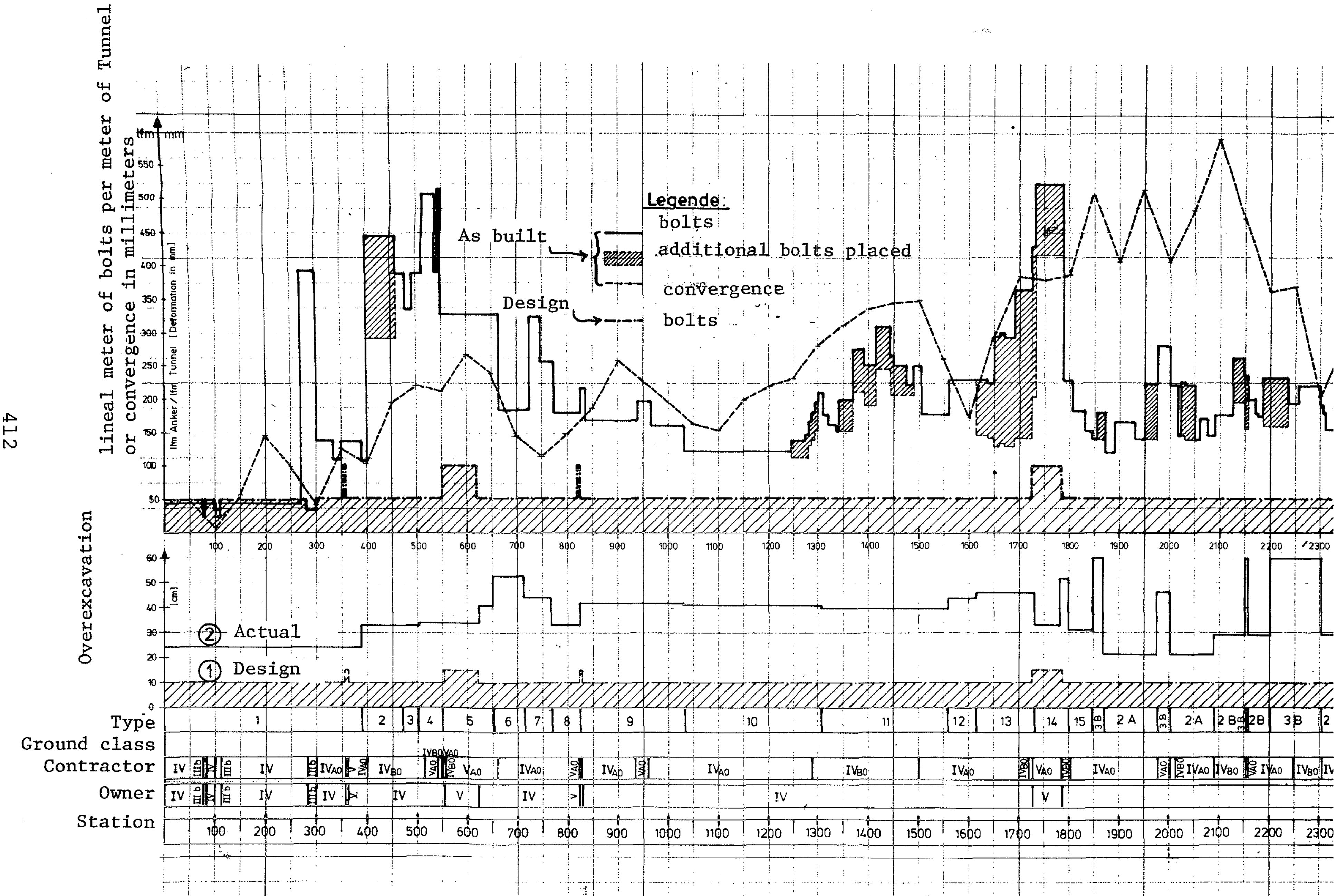


FIGURE B-7.17b SUMMARY OF SUPPORT, CONVERGENCE AND GROUND

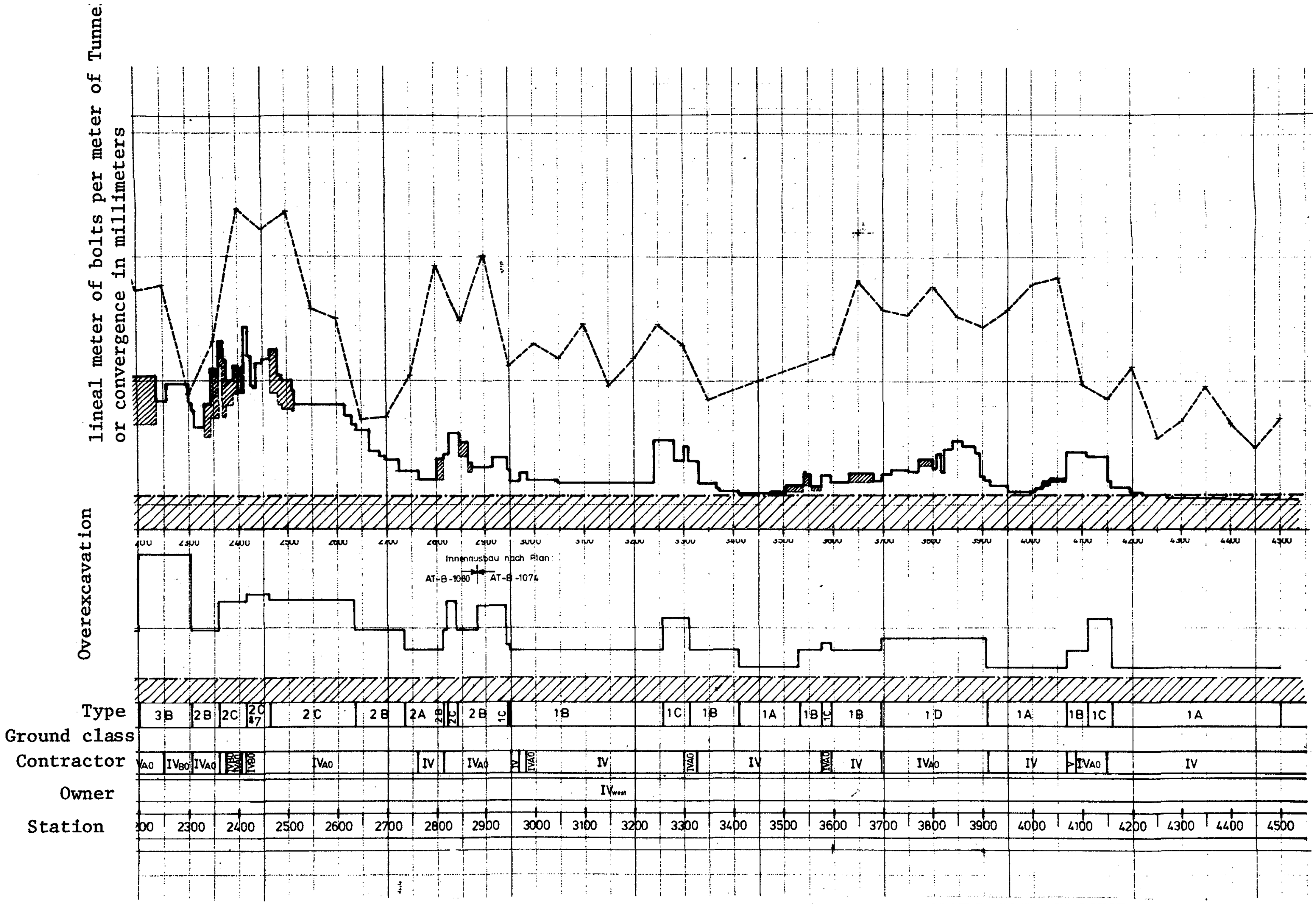


FIGURE B-7.17c SUMMARY OF SUPPORT, CONVERGENCES AND GROUND CLASSIFICATION FOR ARLBERG WESTERN SECTION (FROM ARGE WEST) for 4.5 km of tunnel. With increasing experience, the acceptable convergences increased and the support (bolts) was reduced. (Note, however, that ground conditions after station 2600 m improved.)

Different types of overexcavation were tried to accomodate the convergence; the types are listed in the table in Figure B-7.17a. In one section, a circular tunnel profile was used which, however, was abandoned because it required too much backfill concrete.

Figure B-7.18 is a geologic map of a section of the Arlberg Tunnel where shear zones strike at approximately 20° to the tunnel axis. Where distinct shear zones intersected the tunnel, more difficulties had to be expected, and thus an attempt was made to predict the zones of intersection ahead of the face. A horizontal boring was drilled from a niche which allowed more reliable prediction of the shear zones intersections. This procedure was, however, not continued, instead, the crews carefully observed the drill rates for the bolt holes. The drill rates increased in the shear zones; thus, a warning of an approaching shear zone was possible.

Not unexpectedly, the deformation of the tunnel was asymetrical due to the geologic structure. The deformations were largest when a shear zone intersected the tunnel wall. Two thirds of the deformation may occur on one side in absolute terms. With a symmetrical overexcavation this means that the final liner would have a different thickness at the southern and northern springlines. Thus asymetric overexcavation was chosen, see Figure B-7.17a.

Note also on Figure B-7.18 that the shear zones strike per-

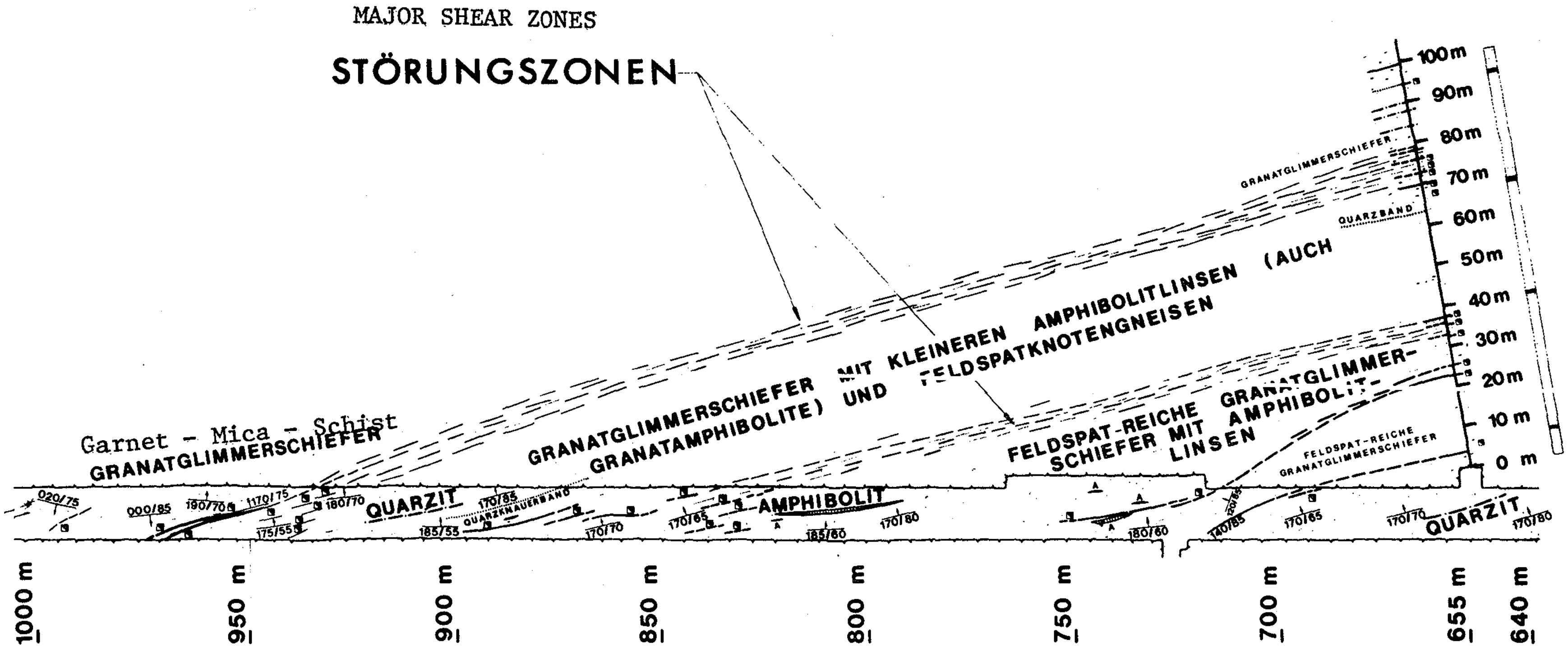
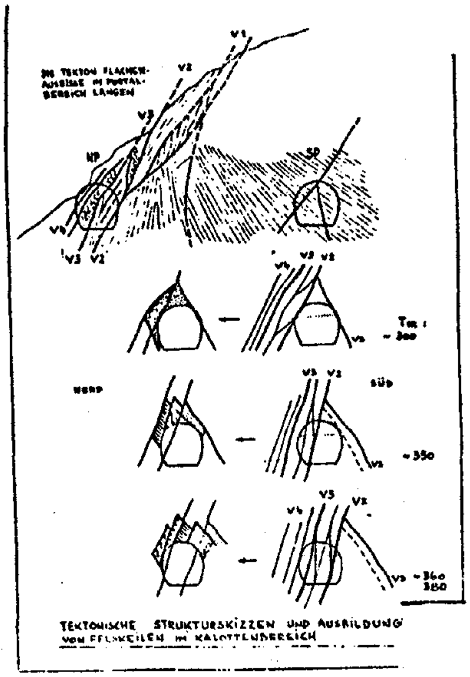


FIGURE B-7.18 MAP OF SECTION OF ARLBERG TUNNEL (FROM JUDTMANN, 1978) pendicular to the cross-cuts. This resulted in considerably smaller convergences in these cross cuts, where only a few centimeters were observed, an order of magnitude smaller than in the tunnel.

4.3.3 Accidents Due to Ground Conditions

In the western section of the Arlberg Tunnel, five fatalities were recorded. One resulted from a traffic accident; the four other fatalities were due to two accidents caused by rock conditions. One of these accidents was a roof collapse between stations 280-300 m. In this section, the two loader operators were buried when the roof collapsed in the top heading. Support was placed according to ground class IIIb, consisting of a nominal thickness of 15 cm shotcrete and rock bolts of 4 m length, spaced 1.5 meter longitudinally and 2 m circumferentially. A dispute between contractor and owner had occurred whether steelsets were to be placed or not. Shortly before the accident, the owner and his consultants decided that the rock conditions were ground class IIIb and that the support for this class (without steelsets) was sufficient. The contractor acknowledged that the rock encountered is a class III, but he wanted to place steelsets for safety purposes and wanted to be paid for them. The rock conditions have been described by Pacher (1975) (although his text does not explicitly state it, the headings on the figures clearly identify them as those from the Arlberg West). The geologic map and cross-sections shown on Figure B-7.19 and 7.20 indicate that two major shear zones intersect the top heading between stations 280 and 300 m. These shear zones formed a triangular wedge in the crown, which became more unstable with advancing excavation since both shear zones had a strike of



GURE B-7.19 GEOLOGIC CROSS-SECTIONS NEAR THE WESTERN PORTAL, AND STATION S 300 TO 380 m (FROM PACHER, 1975 AFTER WEISS)

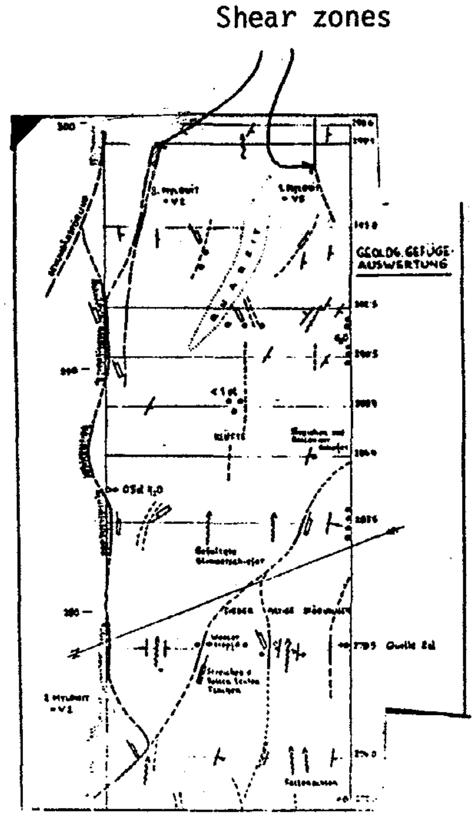


FIGURE B-7.20 GEOLOGIC MAP OF INVERT IN AREA OF ROOF COLLAPSE (FROM PACHER, 1975, AFTER WEISS)

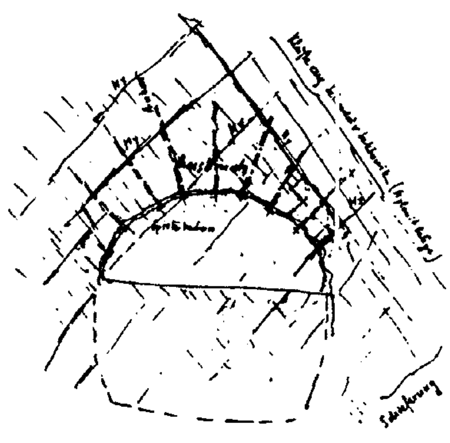


FIGURE B-7.21 SKETCH OF GEOLOGY AND SUPPORT (FROM PACHER, 1975) approximately 10° relative to the axis of the tunnel. The bolts of 4 m length (Figure B-7.21) were not sufficiently long and could not carry the whole weight of this wedge. Once it became possible for the wedge to drop out, a sudden unannounced failure occurred. After this roof collapse the ground was primarily classified as class IV, requiring steelsets.

The other accident related to ground conditions occurred in one of the cross-cuts of the Albona cavern. Rock popped out of the face of the heading during drilling and placement of the explosives. No detailed account of the geologic conditions is given. The popping of the rock must be associated with the geologic structure (Figure B-7.22); the bedding planes were dipping subparallel to the face, and a large stress concentration in one of these beds may have occurred once part of the bed was excavated.

4.3.4 Final Liner

The final liner was placed and in the initial design had a theoretical* thickness of 25 cm. However, to accommodate potentially large stresses due to large residual deformations, the owner increased the liner thickness to 35 and 45 cm in some sections. The performance of the final liner was monitored by means of radial and tangential stress cells as well as convergence measurements. The measurements indicated only small stresses in the final liner, on the order of a few kg/cm^2. As a consequence, the thickness of the final liner was reduced back to 25 cm

*Theoretical dimensions of the final liner according to specifications. Due to overbreak and deliberate overexcavation, the thickness of the final liner is 50 to 60 cm .

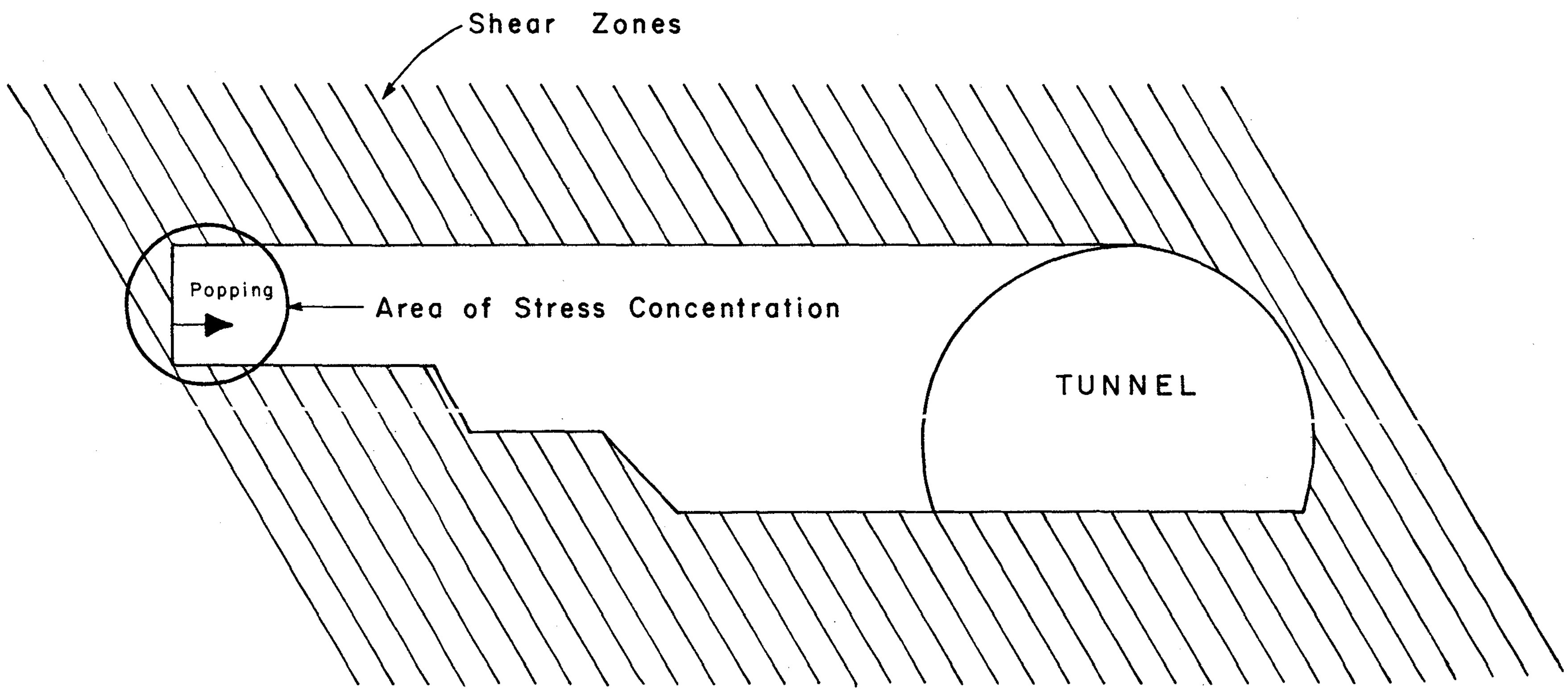


FIGURE B-7.22 POPPING ROCK AT THE FACE OF THE CROSS-CUT IN THE ALBONA CAVERN for the later sections of the tunnel.

Criteria for placement of the final liner had been established based on the residual rate of convergence of the tunnel. Typical criteria used in the Arlberg Western section are listed in Table B-7.6. It is more economical to place concrete with a higher strength rather than to wait until the rate of deformation subsided below the limit of 6 to 8 mm/month, because the work cycle and advance of the final liner would be reduced. It is interesting to note that the design of the final liner is thus adaptable through thickness, strength and time of placement variation.

As mentioned earlier, only nominal stresses of a few kg/cm^2 were observed in the final liner over a period of more than 2 years. The low stresses might also be attributed to the effect of an 8 mm thick felt layer between shotcrete and final liner which served as a "principal back packing" zone for residual deformations.

4.4 Geologic Conditions of the Western Section

The field geologist of the owner, Dr. J. Kaiser, is currently preparing a paper on the geologic conditions of the western section of the Arlberg. A summary of the geologic conditions is given here.

The gneiss nappe in which the tunnel is located was folded during the formation of the alps. Differential shearing took place and shear zones and faults developed. These shear zones strike subparallel to the tunnel and dip approximately 60° to 80° to the south. Minor shear zones are spaced in the order of 25 cm, whereas major shear zones are spaced approximately 3 to 5 m apart (Figure B-7.23).

TABLE B-7.5 CRITERIA USED FOR THE PLACEMENT OF FINAL LINER

Rate of Deformation (convergence)	Concrete
V < 6 to 8 mm/months	BN 225P
8 < V < 10 mm/month	BN 300P
10 < V < 12 mm/month	BN 350P
V > 12 mm/month	wait, do not place liner

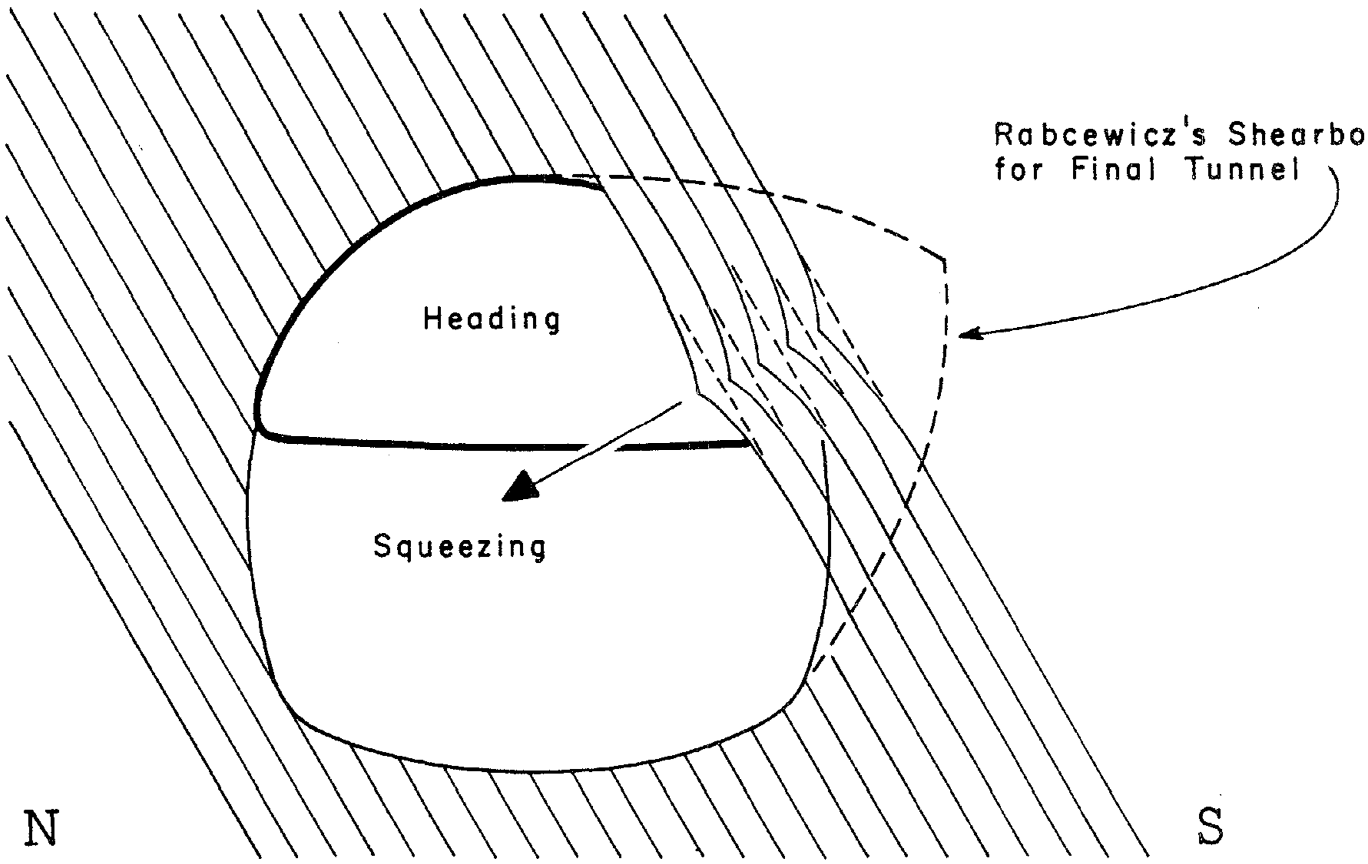


FIGURE B-7.23 SQUEEZING OF GROUND AND DEVELOPMENT OF SHEAR-BODY (AFTER KAISER, 1978)

4.5 Detailed Data

We have obtained detailed geological data for 2.3 km kilometers of the western section, which include convergence measurements and detailed geological maps. The data will be used for empirical tunnel design construction methods (Volume 5 of this series).

5. Dalaas Tunnel

The Dalaas Tunnel is a 1.6 km long tunnel at the western end of the road built under the jurisdiction of the Arlberg Highway Authority. The contract had been awarded by the end of 1977, and construction started in Spring 1978. The bid documents incorporate the experience gained at the Arlberg Tunnel. In particular, payment provisions for support have been changed, resulting in a modified ground classification procedure. Payment provisions no longer include a bonus-malus for bolt payment. Excavation and support are paid as entirely separate items. Excavation will be paid by cubic meter for each ground class. Support will be paid by unit (shotcrete is paid per square meter of nominal thickness, steelsets and wirefabric by nominal length, resp. area, bolts by unit of a certain length). Labor costs are included in the respective items. For each groundclass, ranges of support quantities are quoted and there are no standard support quantities per ground class. (The ranges of support overlap for adjacent ground classes).

The ground classes will still be determined in the field (see section 4.3.4 of the main body of this volume). Ground class assignment is primarily aimed at the excavation procedure (i.e., full face or heading and benching). Support is assigned separately, based on the monitored performance of already supported sections.

APPENDIX:	B-8
SITE:	Pfänder Tunnel
OWNER:	State of Vorarlberg
DESIGNER:	Ingenieurgemeinschaft Lässer-Feizlmayr
CONTRACTOR:	Joint Venture Pfänder Tunnel with: Beton-und Monierbau, Innsbruck E. Züblin, Wien, Hilti and Jehle
DATE OF VISIT:	3rd February, 1978
PERSONS MET:	Mr. Rucker, Site Manager, Contractor Mr. Wogrin, Site Engineer, ILF

1. Introduction

Bregenz, a resort city on the shores of Lake Constance, is squeezed between the lake and the Pfänder Massif. Traffic from southern Germany (Munich) to Switzerland (Zurich) and from southeastern Germany through Switzerland to Italy (Milan) has to pass through Bregenz. Relief from the traffic congestion is urgent, particularly during the summer months. After long debates, during which also a lakeshore highway and a viaduct along the Pfänder were studied, a tunnel under the Pfänder was finally selected (see Figure B-8.1). The tunnel, part of the Rhine Valley freeway, will be 6.75 km long and in its final form consist of two parallel two-lane tunnels; however, at the present time only one two-lane tunnel is being built. Information has been gathered during interviews on the site with Mr. Wogrin, the site engineer of the designer and construction supervisor; Mr. Rucker, the site manager for the contractor, Beton-und Monierbau, as well as during the inter-

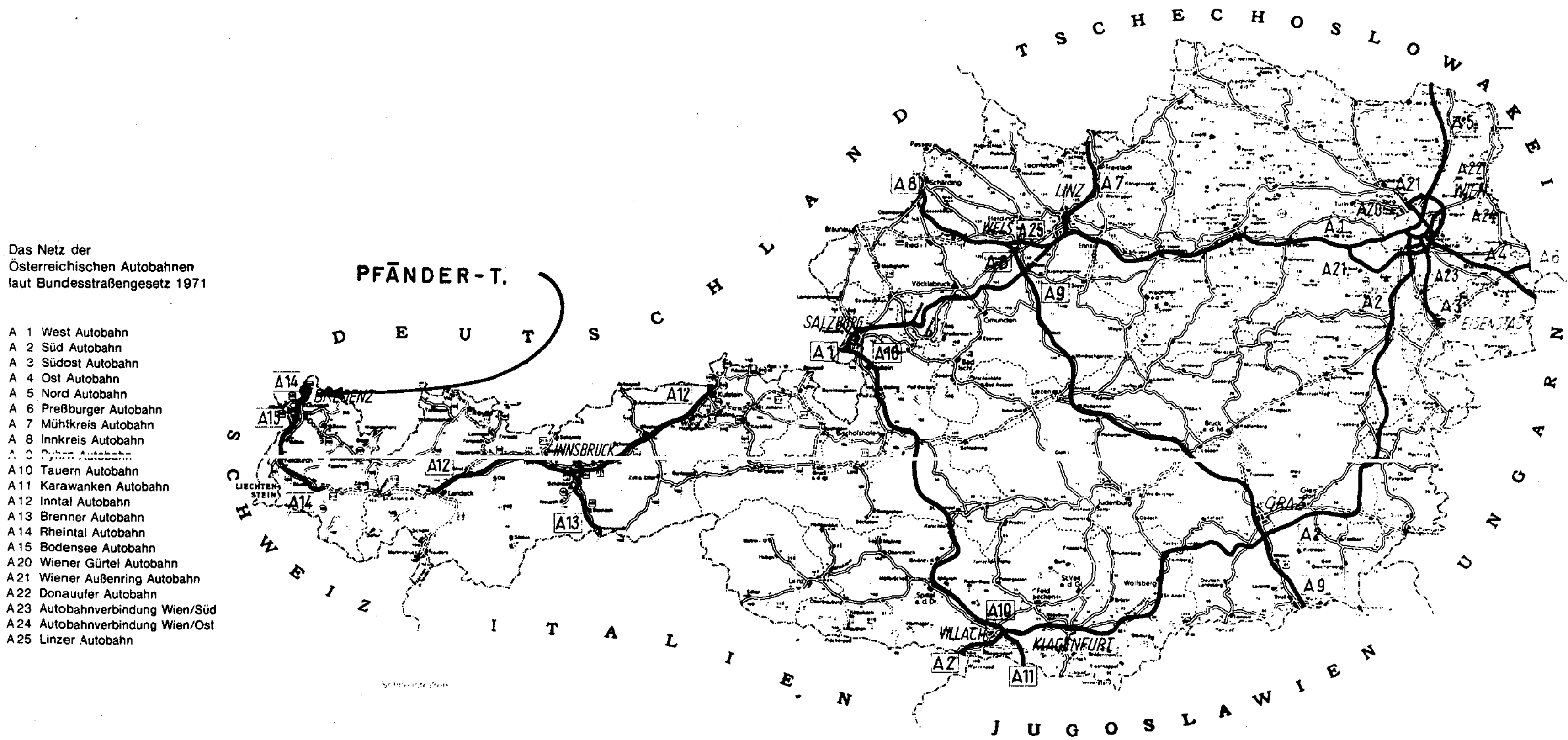


FIGURE B-8.1 LOCATION OF THE PFÄNDER TUNNEL views with Dr. John in the home office of ILF (Ingenieurgemeinschaft Lasser Feizlmayr) in Innsbruck and with representatives of Beton-und Monierbau's Innsbruck office.

In section 2 of this appendix, a technical description is given and the bidding process for the Pfander Tunnel is discussed, followed by geology and exploration (section 3), construction procedures (section 4), and contractual aspects (section 5) and finally a brief description of monitored performance.

2. Description of the Project and Bidding

2.1 General Description

The design of the Pfander tunnel had to take environmental and geologic aspects into consideration. A longitudinal section is shown in Figure B-8.2. The ventilation is from caverns at the base of the two shafts at the quarterpoints of the tunnel. This arrangement has been chosen primarily due to environmental reasons. The area often experiences temperature inversions with fog in the valley; the elevation of the shaft exits has been selected such that they are above the fog level in order to prevent formation of smog. Since geologic conditions were not known with sufficient accuracy, a pilot tunnel was driven (section 2.2). The design of the main tunnel was prepared for conventional (drill and blast) excavation (Figure B-8.3) and excavation by means of TBM (Figure B-8.4).

2.2 Pilot Tunnel

The pilot tunnel has a cross-section of 10.5 m^2 (diameter = 3.65 m); its location relative to the cross-section is shown in Figures B-8.3 and B-8.4 and was selected for TBM

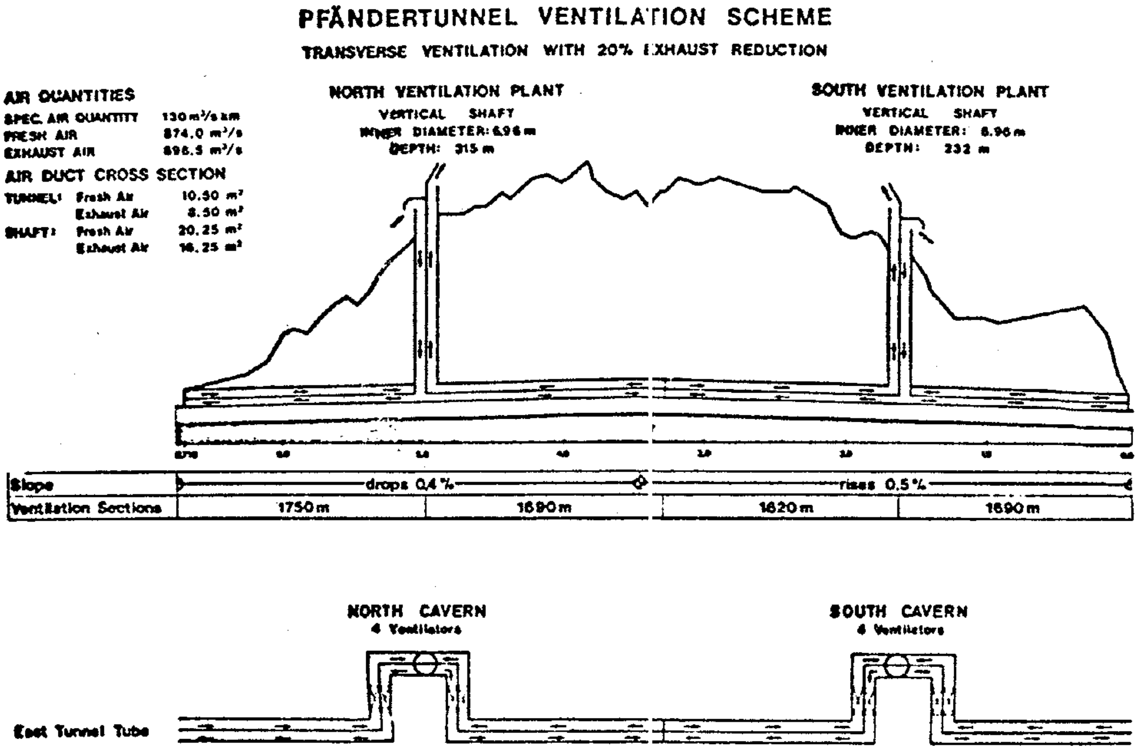


FIGURE B-8.2 LONGITUDINAL SECTION THROUGH PFÄNDER TUNNEL (WITH VENTILATION SCHEME) (FROM JOHN, 1978)

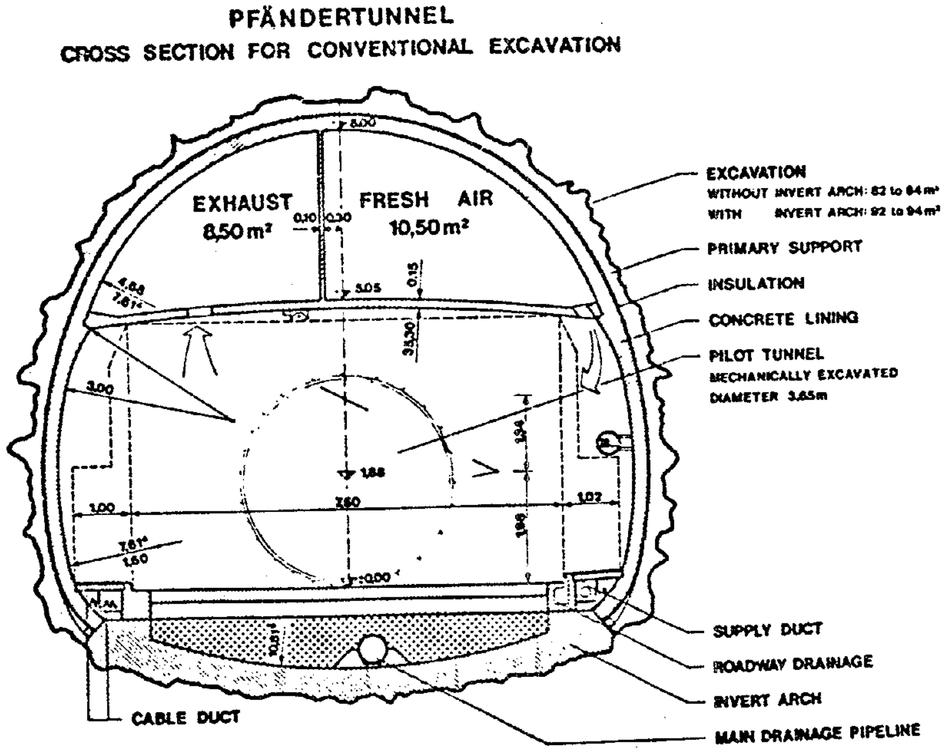


FIGURE B-8.3 CROSS-SECTION OF THE PFÄNDER TUNNEL FOR CONVENTIONAL EXCAVATION (FROM JOHN, 1978)

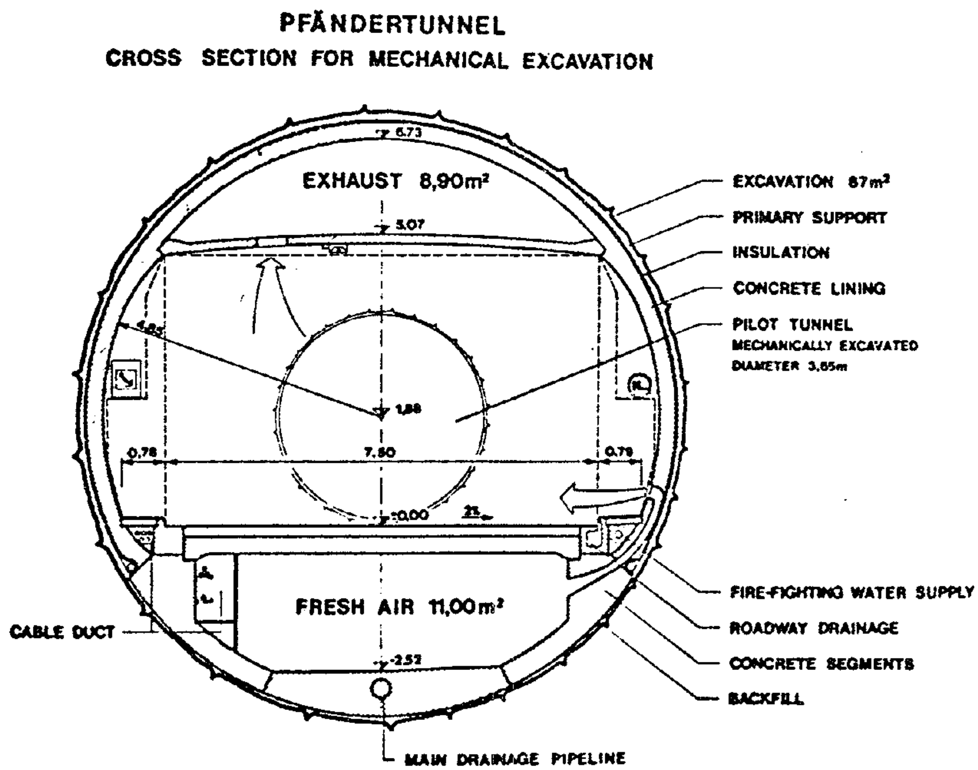


FIGURE B-8.4 CROSS-SECTION OF THE PFÄNDER TUNNEL MECHANICAL EXCAVATION (FROM JOHN, 1978) application, i.e., in the center of the circular cross-section. Some manufactueres (WIRTH) use a two or three stage excavation method with TBM's which are pulled forward using clamping mechanism in the pilot tunnel. Such a procedure had been used in the Sonnenberg Tunnel in Lucerne (Switzerland), and possibly the same machines might have been used at the Pfander. However, pilot tunnel location is not optimal for a conventionally excavated tunnel where a crown drift would have been preferable.

The pilot tunnel had the following beneficial effects:

(i) improved assessment of the ground conditions,

(ii) construction of the ventilation shafts was facilitated,

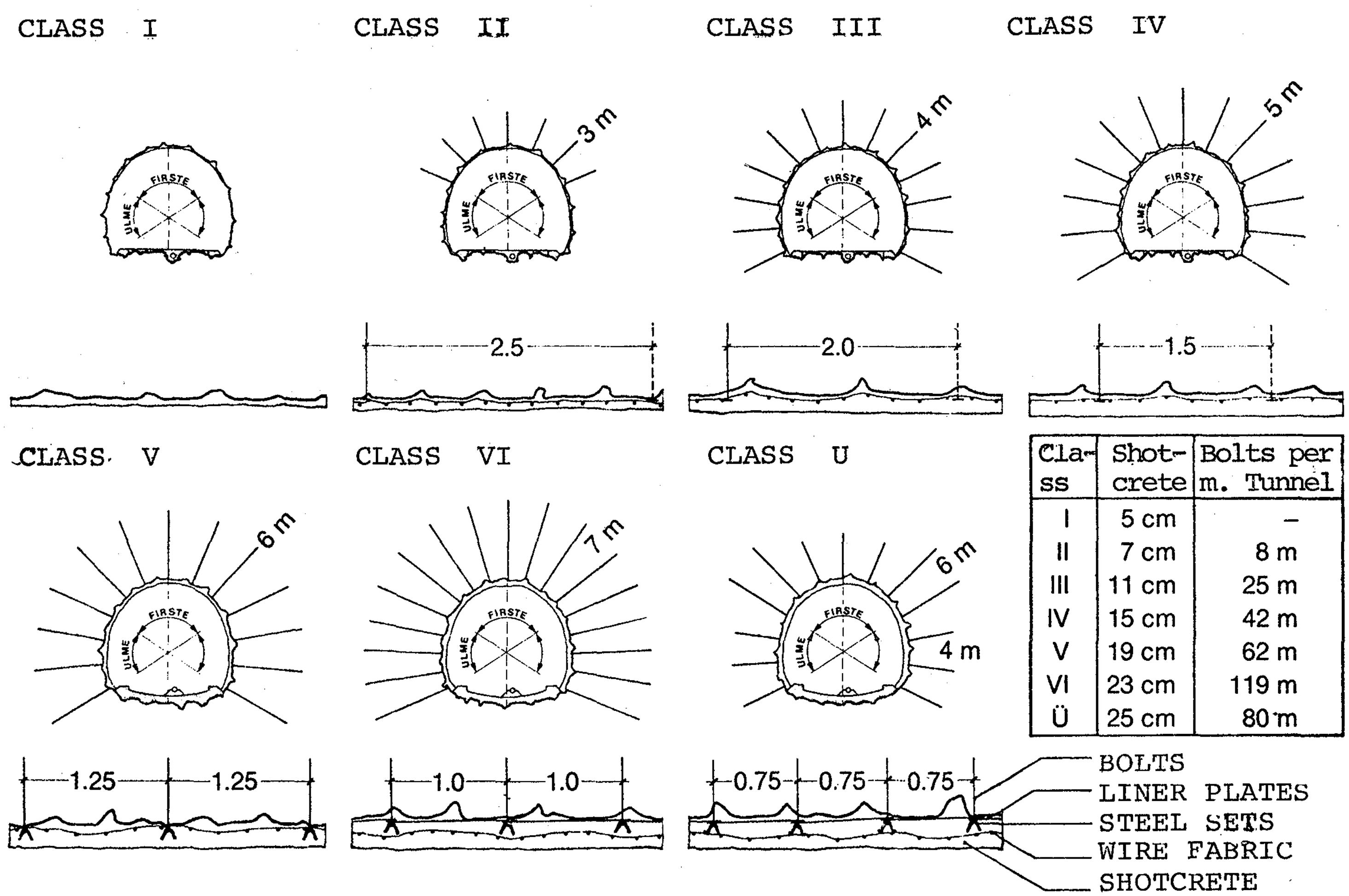
(iii) construction ventilation was greatly improved and simplified.

The pilot tunnel was driven with two TBM's (a Wirth TB II H, diam. 3.60m) in the northern sector, length = 1874 m, and a Robbins 122/123 with 3.65 m (12 ft) diameter from the south (length of section = 4650 m). The support in the pilot tunnel had to consider the possibility of the main tunnel excavation by TBM. Steel had to be avoided due to potential damage to the cutters of a TBM driving the main tunnel. Support was thus a combination of resin-fiber bolts, plywood and shotcrete. The bolt plates are steel, since they would readily fall off the wall of the pilot tunnel when it is widened and thus not damage the TBM. Support placement near the face of the pilot tunnel was hampered by the TBM, and shotcrete cannot be applied. To provide protection, plywood was placed and bolted to the roof. Shotcrete was used behind the pilot tunnel TBM. Since the rock was either covered by the support or was coated with mud, small chambers were excavated by drilling and blasting which allowed the geologist to obtain a three-dimensional picture and also allowed one to determine the behavior of the rock when blasted.

The contract for the excavation and support of the pilot tunnel was let a firm-fix price of 89 million AS = $6 million U.S. ($1 = 15 AS). In the final parts of each section, only the minimum support was placed by the contractor, such that the machines could be dissassembled and moved out of the tunnel. This zone coincides with shale layers which require support. Subsequently, rooffalls occurred, the shale in the invert swelled and the pilot tunnel was in danger of being destroyed. This had to be avoided, since the pilot tunnel serves as ventilation tunnel during the construction of the main tunnel. The contractor constructing the main tunnel thus placed additional support in the critical sections of the pilot tunnel in order to keep it open. These costs were recovered from the pilot tunnel contractor.

2.3 Bidding and Award of Contract of the Main Tunnel

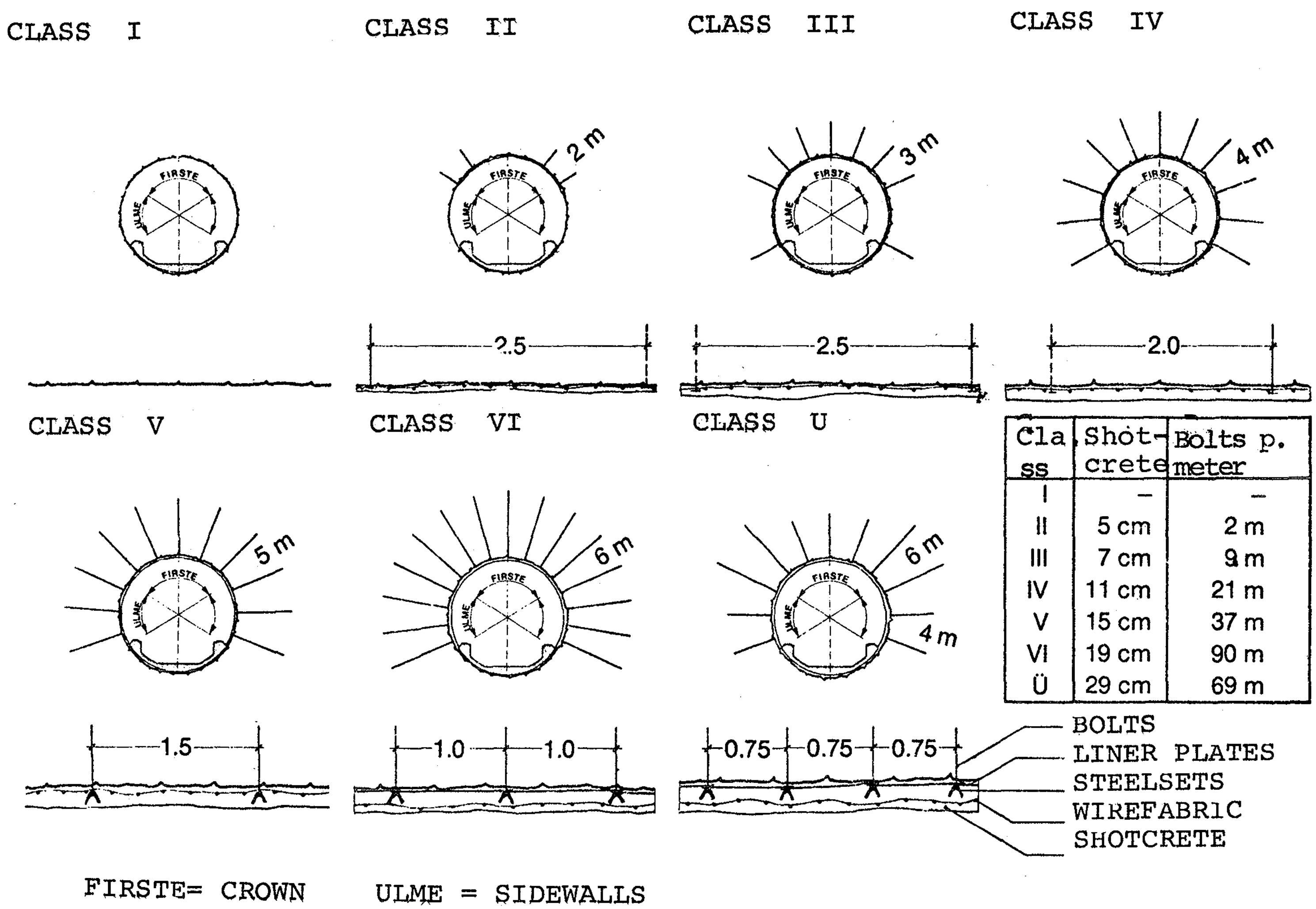
The bid documents for the main tunnel were prepared for excavation by drilling and blasting and TBM. The design cross-sections are shown in Figures B-8.3 and B-8.4 respectively. As is common practice in Austrian tunneling, a ground class-ification scheme was developed, which relates ground conditions with excavation procedures and support requirements. As expected, support requirements differ for drilling and blasting (Figure B-8.5) and TBM excavation (Figure B-8.6) (greater dis-turbance of surrounding rock by the drilling and blasting). Shotcrete thickness as well as the bolt pattern are reduced



Class	Shot-crete	Bolts per m. Tunnel
I	5 cm	–
II	7 cm	8 m
III	11 cm	25 m
IV	15 cm	42 m
V	19 cm	62 m
VI	23 cm	119 m
Ü	25 cm	80 m

FIRSTE = CROWN   ULME = SIDEWALLS

FIGURE B-8.5 SUPPORT MEASURES FOR THE PFÄNDER-TUNNEL FOR CONVENTIONAL EXCAVATION (FROM JOHN, 1978)



Class	Shot-crete	Bolts p. meter
I	–	–
II	5 cm	2 m
III	7 cm	9 m
IV	11 cm	21 m
V	15 cm	37 m
VI	19 cm	90 m
Ü	29 cm	69 m

FIGURE B-8.6 SUPPORT MEASURES FOR THE PFÄNDER TUNNEL FOR MECHANICAL EXCAVATION (FROM JOHN, 1978) for TBM excavation. The total length of bolts placed for TBM excavation is only 60% of that for drill and blast excavation. However, the predicted support quantities for the TBM case cannot be verified, since eventually drilling and blasting was selected.

The specifications limited the unsupported distance at the face. For the TBM, this unsupported length was specified irrespective of the length of the machine. This means that for some ground classes support would have had to be placed within the range of the machine (probably requiring a prefabricated segmented liner).

Payment provisions for the Pfänder tunnel incorporate the experience gained at the Arlberg tunnel. Payment of excavation and support is completely separate. Excavation is paid per cubic meter of excavation to the theoretical line of excavation (see Figure B-8.3). Overbreak this area has to be included in the excavation unit price; geologic overbreak is also included in the unit price, which basically means that geologic overbreak is not recognized. This is justified because the pilot tunnel provided an excellent way of judging rock quality prior to bidding. Support is paid by unit placed; all difficulties affecting excavation but related to placing support have to be included in the support unit price.

The results of the bidding were:

(i) of the 6 bidders, 5 submitted conventional excavation schemes (by drilling and blasting) which were considerably more economical than excavation by TBM. One contractor submitted a TBM bid which was lower than his drilling and blasting bid; however, in the general ranking, this TBM bid was one of the highest.

(ii) to achieve the same advance rates (to keep the prescribed schedule) two TBM headings would be required, whereas for drilling and blasting a single heading would be sufficient. With drill and blast excavation, work could start almost immediately after the contact was let, while TBM's would have a long delivery time (on the order of one to one and one-half years for new TBM's, somewhat shorter if two used TBM's that were available in Switzerland had been used). These conditions made TBM bids more expensive.

(iii) the contract was awarded to the low bidder with excavation by drilling and blasting.

(iv) the low bid had one heading from the southern portal, since essentially all the muck had to be moved to the south of the southern portal.

A heading from the north would have to consider the added transportation distance. Also, the transport through the City of Bregenz could only be performed with normal highway trucks, which have less capacity (12 m^3 vs. 25 m^2) than dump trucks and in addition would require reloading.

2.4 Costs

The Pfänder Tunnel was let for an average cost of 100,000 AS per meter of tunnel (6660 $/m) excluding shafts, caverns, and pilot tunnel. With the shafts, the average cost per meter is 120,000 AS = $8,000. The average cost of the pilot tunnel is 13,350 AS = $890 per meter. Thus, the total costs per meter of tunnel, including pilot tunnel, shafts and caverns is 133,350 AS = 8,890 $/m of tunnel.

3. Geology of the Pfander

The Pfander Massif belongs to the subalpine molasse, a tertiary sedimentary rock formation. It consists of sediments deposited prior to the alpine orogeny. In particular, it is believed that the Pfander Massif represents a tertiary alluvial fan. During the formation of the Alps this massif was tilted and the layers now dip approximately 15° to the north.

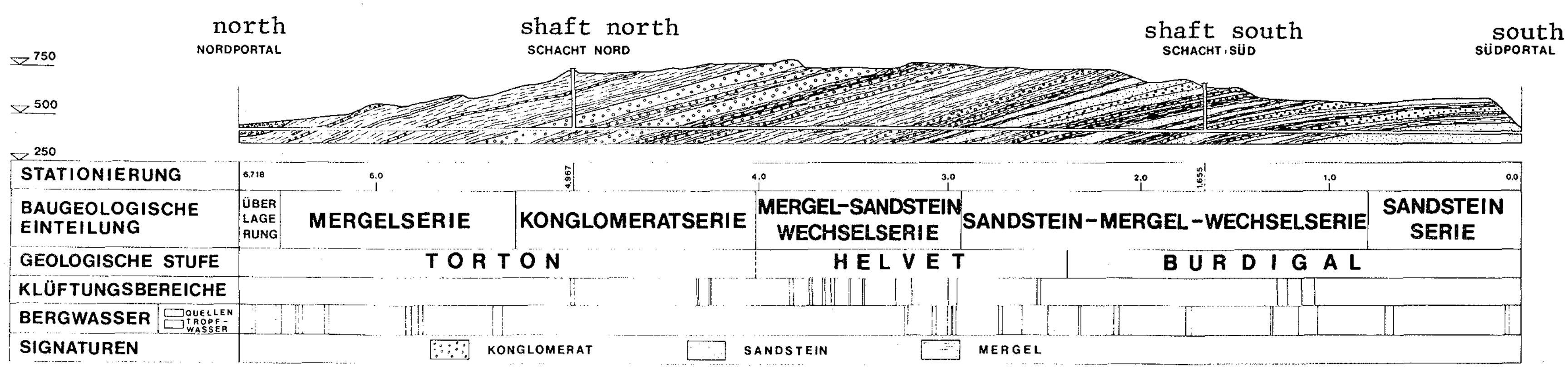
Fig. B-8.7 shows a geologic section along the axis of the Pfander Tunnel; rock types are sandstones, shales or marls* and conglomerates. (Alternating rock layers have been named after the predominant rock type.) Near the south portal, the pilot tunnel (km 0.0 to 0.8) encountered sandstones (predominant) and shales; interbedded are coal layers with thicknesses up to 20 cm and clay shale layers. The ground is thus overbreaking. Between kms 3 to 4 from the south portal, alternating shales (predominant) and sandstones were encountered followed by a conglomerate series. North of km 5.55, shales were found; finally, in the vicinity of the northern portal there is a soil-like overburden material for 150 m. The quality of the rock decreases from south to north. The thickness of the layers varies; in particular, the conglomerates show a decrease in thickness from south to north which is consistent with an origin as alluvial fan. The length of the different rock types in summary is: conglomerates, 20%; sandstones and marly sandstone 38%; shales 42%.

Figure B-8.7 also shows areas where significant jointing

*The correct translation of the German "Mergel" may be shale or marl, depending on the content of calcareous particles. However, the word is often inconsistently used in German; we thus use the term shale, hereafter.

Überlagerung = overburden
Mergelserie = marl series
Konglomeratserie = onglomerates
Mergel-Sandstein Wechselserie = marl (predominant), sandstones alternating
Sandstein-Mergel Wechselserie = sandstones (predominant) and marls alternating
Sandstein = sandstone

GEOLOLOGISCHER LÄNGENSCHNITT



Stationierung: Station
Geologische Stufe: Geologic Unit
Bergwasser: Water Inflow
Baugeologische Einteilung: Engineering Geologic Description
Klüftungsbereiche: Jointing

FIGURE B-8.7 GEOLOGIC SECTION ALONG PFANDER TUNNEL (FROM ILF, 1978) and water inflows in the pilot tunnel occurred. The bid documents included a detailed geologic map of the conditions encountered in the pilot tunnel, listing water inflow, joint spacing, behavior of the ground during and after the excavation and the support placed. The previously mentioned niches made it possible to obtain a three-dimensional picture (since the face of the pilot tunnel was not accessible (TBM)) and to know the behavior of rock when excavated by drilling and blasting.

Surface mapping complemented the geologic investigations. No deep borings were used, however. For the shafts, the correlation of outcrops with the geology in the tunnel was considered sufficient.

4. Construction of the Pfänder Tunnel

4.1 Introduction

Bids for the Pfänder Tunnel were submitted in April 1976. The contract was awarded to the low bidder, a joint venture of Beton-und Monierbau, E. Züblin and Hilti und Jehle in the fall of 1976, and construction started in January of 1977. Information presented in this section is based on the interviews with Mr. Rucker, the site manager of the contractor's joint venture, Mr. Kluibenschedl, head of the tunneling department of Beton-und Monierbau and Mr. Decker of B & M's equipment department. The description will be divided into the construction of the tunnel (section 4.2) and the shafts (section 4.3).

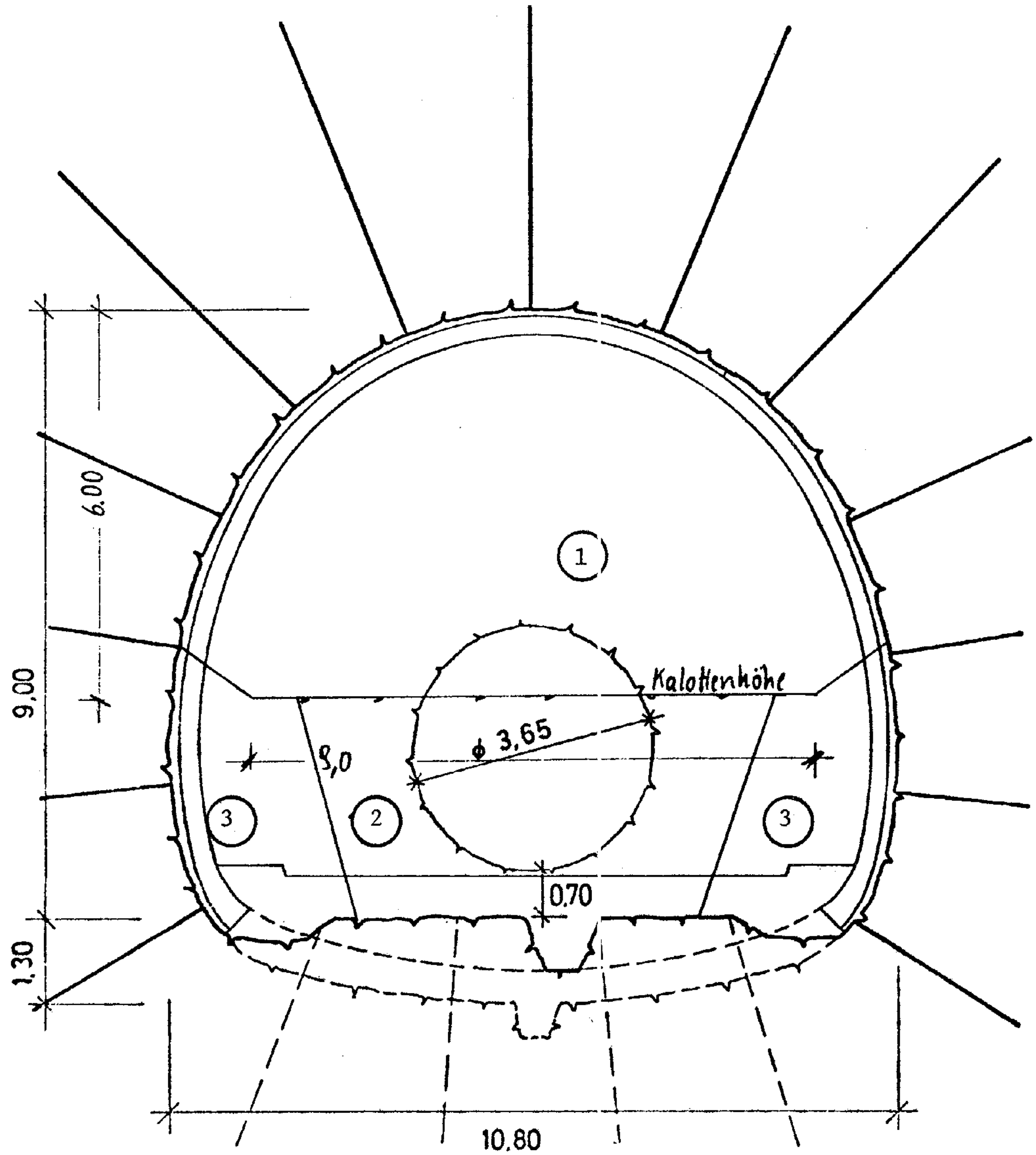
4.2 Construction of the tunnel

4.2.1 Method of Construction

From the detailed exploration, the contractor anticipated that approximately 60% of the tunnel would require excavation-support procedures by heading and benching. Since full face excavation and heading and benching would alternate frequently, the contractor selected a continuous heading and bench excavation rather than switching to full face and back. Figure B-8.8 shows a cross-section of the tunnel with the selected size of headings and bench (height = 6.0m). The length of the heading, which has been established through practical experience, varies from 120 to 180 m. The central ramp connecting heading to invert is 30 m long (10% slope).

Drilling of blast and bolt holes is performed with two Atlas Copco Boomer rigs with two electro-hydraulic drills, each on a multi-directionally adjustable arm. (The contractor also estimated costs for 2 rigs with three pneumatic hammers which were to achieve similar drill rates as two electrohydraulic hammers and concluded that excavation would be 30 AS/m^3 more expensive, which equals a total savings of 15 million AS = $1 million U.S., for the Pfänder Tunnel). Blast and boltholes are drilled simultaneously in the heading. The blasting pattern includes a wedge cut in the center of the face. Trench blasting with vertical drill holes is used for the excavation of the central bench (ramp = (2) on Figure B-8.8). For the sidewall bench ((3) on Figure B-8.8), drill holes are drilled horizontally and parellel to the tunnel axis. The excavation of the invert and drainage trench will be described later.

Initially, a partial face TBM (DEMAG H41) was used to trim the circumference; however, the rock proved to be too strong and abrasive, and the machine was subsequently removed from the tunnel.



1 = heading
2 = central bench (ramp)
3 = sidewall benches

FIGURE B-8.8 HEADING AND BENCHING IN PFÄNDER TUNNEL
(FROM BETON-UND MONIERBAU)

Mucking is performed with two CAT-D980 frontend loaders and transportation is by Kiruna trucks KL250 (capacity = 25 m^3). Figure B-8.9 shows how the space in the heading is utilized by this large equipment.

Note also on Figure B-8.9 that part of the pilot tunnel is backfilled; for drill and blast excavation, the pilot tunnel is lcoated unfavorably, but as mentioned the location was selected with respect to a possible excavation by TBM.

The pilot tunnel also provides an excellent ventilation system. The exhaust air is removed through the face into the pilot tunnel. Two ventilators are installed in a cavern widened in the pilot tunnel approximately 50 meters south of the future northern ventilation shaft. This location was chosen to minimize noise at the northern portal (populated area). During the excavation of the last section, however, the ventilators will be moved to the northern portal and noise protection measures will be necessary. The contractor also wants to use the natural draft in the northern ventilation shaft, once completed, to aid ventilation. However, as will be discussed in Section 4.3, the subcontractor for the shaft is behind schedule.

The capacity of the ventilators is 4800 m^3/min.; Mr. Rucker quoted a required air supply of 3500 m^3/min. With a blowing type ventilation through normal air chutes, only 2800 to 3000 m^3/min. could have been supplied. The air quality in the tunnel is excellent, since contaminated air does not flow along the working length of the tunnel. Although after each shot the loose muck piled in front of the face reduces the effectiveness of the ventilation, the voids in the muck are

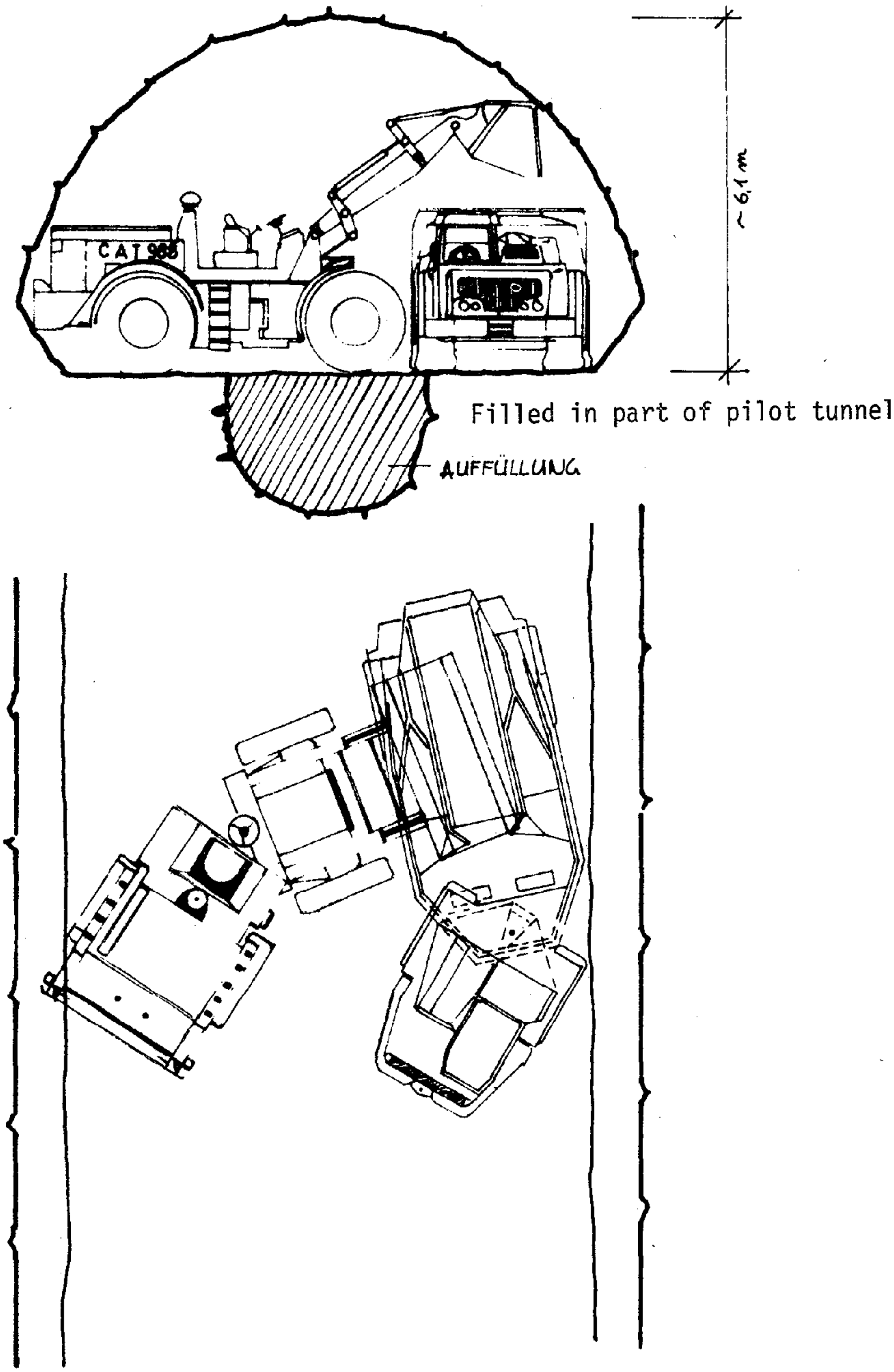


FIGURE B-8.9 USE OF HEADING OF LARGE EQUIPMENT (FROM BETON-UND MONIERBAU) continuous and ventilation is not cut off.

At a distance of 300 to 500 m from the face (variable), either the trench for the drainage pipe or, when an invert arch is required, the full invert is excavated and constructed. The rock is loosened by blasting and loaded onto the trucks by a hydraulic excavator; then concrete is placed. On the already-concreted invert, supply units on flatbed trailers follow the excavation. The supply unit consists of compressors for the shotcrete pumps (including pressure equalizer reservoir) and transformers. Thus, only electric power and low pressure water are supplied from the tunnel portal.

The final liner is placed completely independently of the excavation and approximately 1.5 to 2 km from the face. A first stage is the verification of the cross-section, which is performed by a frame and scaffolding moving on rails. If tights are found, the initial support and rock has to be trimmed or re-excavated; such work is done from the scaffolding. In a second stage, a subcontractor places the waterproofing of welded PVC layers; then the final cast-in-place liner is placed behind a collapsable steel formwork. Concrete is distributed with a swing arm that is installed on an intermediate deck and which can be coupled to different openings in the formwork. No hoses have to be moved. Concrete is supplied by transport mixer trucks to the pump and pumped into the formwork. At the present time, only one formwork set is used (10 m length). However, since the excavation progresses rapidly, a second formwork will be put into use. Finally, the slab separating ventilation ducts from the traffic space is poured, followed by the wall separating fresh and exhaust air ducts

(Figure B-8.3).

Underground work is supported by surface facilities at the south portal. In particular, a large repair shop, supplies and storage, canteen facilities, camp and offices are located there.

4.2.2 Crewsize and Wage Rates

Table B-8.1 presents crew distribution and size. For the heading and bench excavation, two crews are on site and the third one is off. Each shift works 11 hours per day. A crew works for 10 days followed by five days off; this is a so-called 3/3 operation (Figure B-8.10).

The invert excavation was initially conducted also with a 3/3 shift arrangement; however, this has been changed to a decade system in which two crews only are used, each working for 10 days followed by five days off (Figure B-8.10). A decade shift arrangement has been chosen for concrete work also; thus, during 5 days there are 2 crews working (complete day and night shifts) while during the following 10 days there is only one crew. This does not mean that concrete work is performed only during one shift, but rather that two shifts of smaller size are working. Table B-8.1 represents the crew arrangement as of February 2, 1978; however, the particular arrangement and crew size may be rearranged.

4.2.3 Wages

A worker has an average take-home pay of approximately 22,500 AS/month (= $1,500/month) for 220 workhours, corresponding to an average hourly rate of 102 AS = $6.82 U.S. However, the contractor's labor costs amount to approximately 210 AS/hour = $14/hour (These rates are roughly the same as those in the U.S.: take home pay is $7.50/hr, respective costs to

TABLE B-8.1 CREWS AT PFÄNDER TUNNEL (TUNNEL EXCAVATION)

	Men per shift	Shifts per day	Total crew
Heading and Bench			
Foreman 1			
Professionals (2 pipe-fitters, electrician, mechanic) 4			
Miners 10 - 12			
Operators 8 - 10	25	3*	75
Invert			
initally 8			
reduced to 5	5	2	10
Verification of Cross-section	3	1	3
Concrete			
Liner	9	2	18
Slab	4	2	8
Separating wall	3	2	6
Repair Shop and concrete transport			
Day shift (40)	40	1	43
Night shift (3)	3	1	
Total			163

* only 2 crews on site, 3rd crew is off

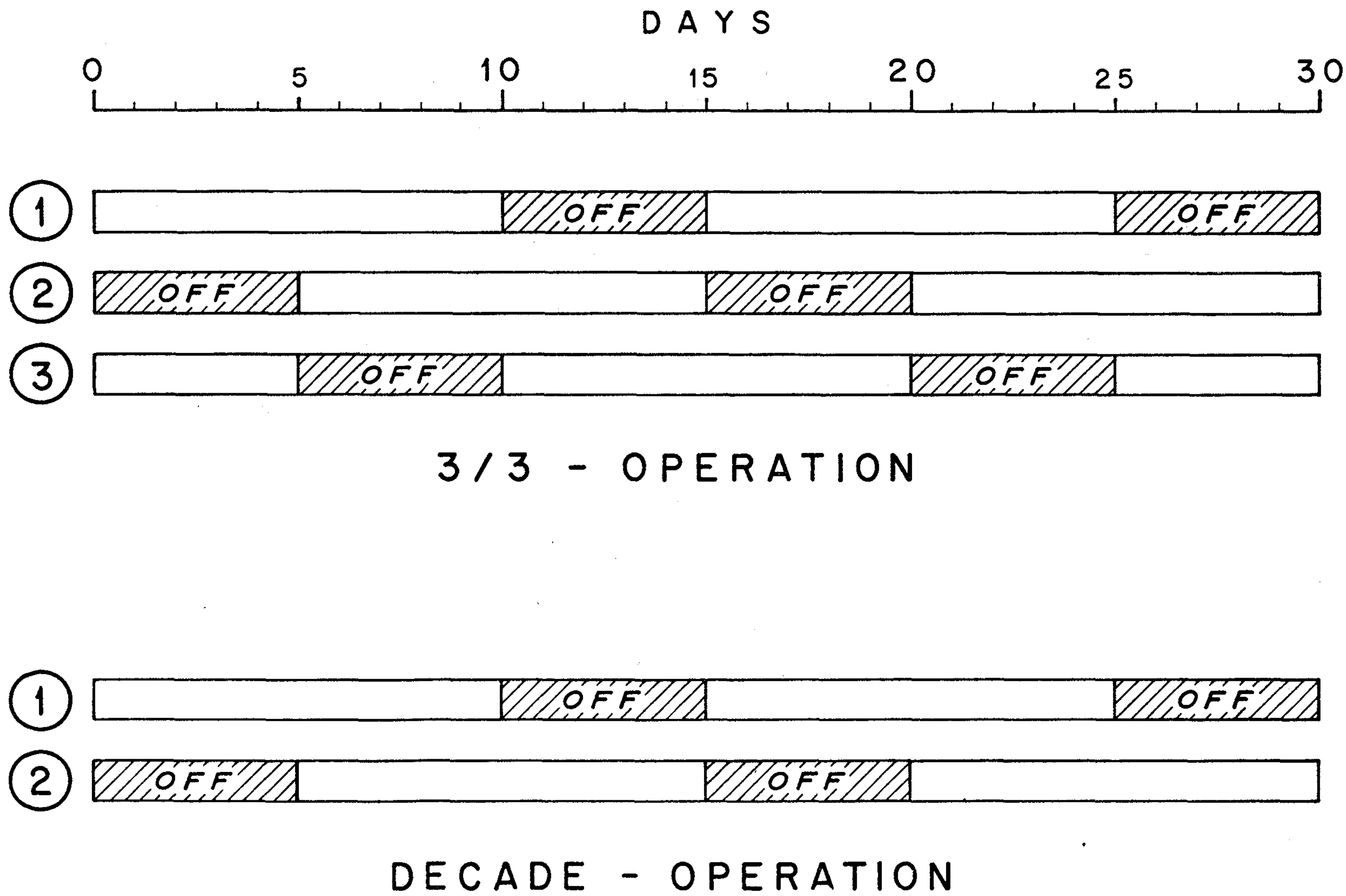


FIGURE B-8.10 DECADE AND THE THREE-THIRDS OPERATIONS contractor 11-14 \$/hr). The cost to the contractor tends to be higher in Europe because he has to carry a large share of the social payments; also, fringe benefits like pensions are greater in Europe.

4.2.4 Advance Rates

The Pfänder Tunnel advanced 3.1 km during the first year of construction. Monthly average advance rates of 400 m and maximum daily rates of 24 m were reached. As the tunnel advances deeper into the mountain, the determining factor for the advance is muck removal. At the Pfänder, this problem has been solved in that muck removed after each round is temporarily deposited in the southern ventilation cavern. The muck will then be transported from this intermediate storage to the southern portal during the period when the heading is supported and the next round is drilled. This leads to a maximum utilization of dump trucks in addition to ensuring high advance rates.

4.3 Construction of the Shafts

The two shafts in the quarter points of the tunnel are 232 m (south) and 315 m (north) deep. The final interior diameter will be 6.96 m, while the diameter of the excavation is approximately 8 m. The construction of the shafts has been subcontracted. First, an 11" diameter boring was lowered from the surface to the ventilation cavern; this drill hole was widened to 1.50 m diameter with a raise-drill. Then, the shaft will be widened downward to its final diameter; the muck is dropped through the raise hole into the cavern and transported out through the tunnel.

The south shaft is widened by drilling and blasting and is on schedule. For the north shaft, the subcontractor proposed widening by a boom excavator (partial face excavator) adapted to shaft construction. It seems that the machine was hastily constructed and did not perform too well. The excavator head required frequent changes because the conglomerate contains quartzite in abundance, leading to rapid wear of the cutters. Further, the shaft sinking machine had two platforms, the lower one for bolting and the upper one for shotcreting (no figure available). With this arrangement, it was impossible to place bolts when shotcreting was in progress, since rebounding shotcrete dropped onto the lower platform.

Another obstacle was the development of dust which flows upward by the natural draft and made it impossible for the operator to see the cutter head and also made a simultaneous support-excavation procedure impossible. In February, it was decided that the machine had to be removed and excavation had to continue by drilling and blasting as proposed in the official specification. Only 90 m of 315 m had been excavated at that time, and the shaft was thus behind schedule (the exact scheduled depth is unknown). The subcontractor not only faces the costs of changing equipment, but also claims by the general contractor (building the tunnel) for energy costs for the ventilation system, since the general contractor expected to use the natural draft in this shaft for construction ventilation of the main tunnel.

5. Contractual Aspects

As mentioned in the main body of this report (Section 4) and in section 2.3 of this appendix, contract and payment provisions had been changed based on the experience at the

Arlberg Tunnel. The excavation unit price only includes the costs of the excavation, while all the labor costs for the placement of the support have to be included in the unit price of the respective support items; also, support will be paid by quantity placed without bonus-malus provisions. The ground class descriptions, in addition to specifying the procedure, set guidelines for the support to be placed; the actual quantities, however, will be determined in the field. The procedure of ground classification has been described in Sections 4 and 5. The two representatives (of the owner and the contractor) not only have to agree on a ground class, they also determine the support to be placed since there are only guidelines, but no standard support quantities, for each ground class. A nominal ground class is determined for the entire cross-section based on the ground conditions encountered in the crown. In this particular sedimentary rock, ground conditions change within each cross-section. The support varies thus within a section, i.e., in the same cross-section, for strong layers, less support is placed than for weak ground.

The experience with these provisions seems to be good. The representative of the owner mentioned that there were essentially no changed conditions claims raised by the contractors. On the other hand, the representative of the contractor considers these contract conditions hard but fair. In particular, he prefers that labor costs are included in items where they actually occur(e.g., labor costs related to support placement have not to be included in the excavation costs). This complete separation of support and excavation items makes estimation easier and clearer for the contractor.

The contractor can thus also better estimate his risks.

The Pfänder Tunnel contractor estimated that the ground conditions were of better quality than anticipated by the design engineer, which means that fewer long bolts have to be placed than assumed in the bid documents. By reducing the profit margin on the items that he does not anticipate to place, he was able to reduce his actual bid by a few percent, which made him the low bidder. Until the time of the visit, the contractor's assessment seems to have been confirmed by the encountered ground conditions.

Contractual problems have developed, however, with the contractors for the pilot tunnel and the northern shaft. These problems have been mentioned earlier in the respective technical sections. The pilot tunnel was let as a firm-fix price contract. The contractor was required to place sufficient support, which he did not do. Since the pilot tunnel is used for the construction ventilation of the main tunnel, additional support had to be placed. The contractor for the pilot tunnel was not released from the contract until this support was placed by the contractor of the main tunnel, who required reimbursement from the pilot tunnel contractor before taking over the pilot tunnel for his purposes (the sum has not been disclosed). In turn, the contractor of the pilot tunnel is now suing the owner for additional reimbursement above the agreed firm-fix price. The legal process has started only recently and no decision has yet been reached.

In sinking of the northern shaft (section 4.3 of this appendix), the method of excavation had to be changed from a machine to drilling and blasting. The subcontractor for the shaft has again to reimburse the main contractor for additional costs, here related to ventilation. How this contractual relation will further develop cannot be predicted at the present time.

6. Monitoring

Performance is monitored primarily with convergence measurements and force measuring bolts (deformations are monitored with telltales inside a hollow bolt and forces are determined from the strains); in most sections, the observed convergence was only a few millimeters, and thus in the range of measurement accuracy, because the rock is of good quality. Convergences of a few centimeters were only monitored in a zone where a coal seam intersected the tunnel.

APPENDIX C

REPORT OF NEW TECHNOLOGY

The work performed under this contract has led to the development of improved practical design tools to provide more accurate representations of the ground-structure interaction in tunneling. This volume includes valuable information on the economical, contractual, and technical aspects of tunneling practices in Austria and Germany.

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