immersed tunnels

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editorial address:
Herengracht 507 – P.O.Box 10
AMSTERDAM
The Netherlands

Cover design: Otto Treumann
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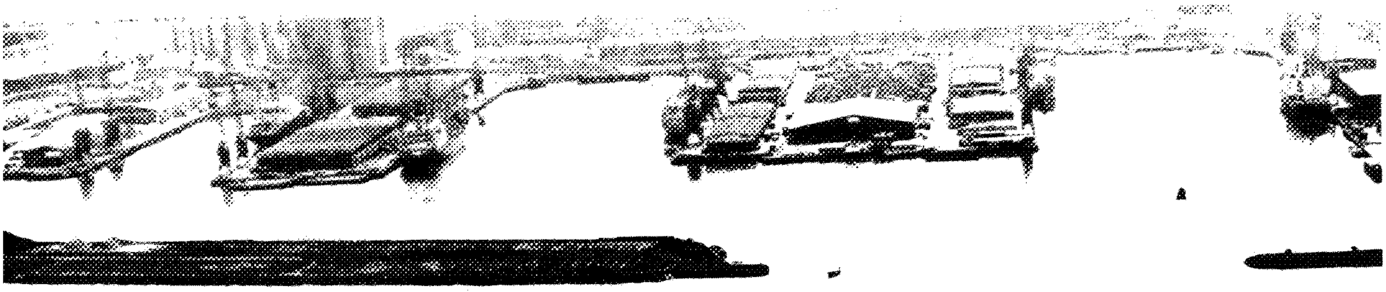
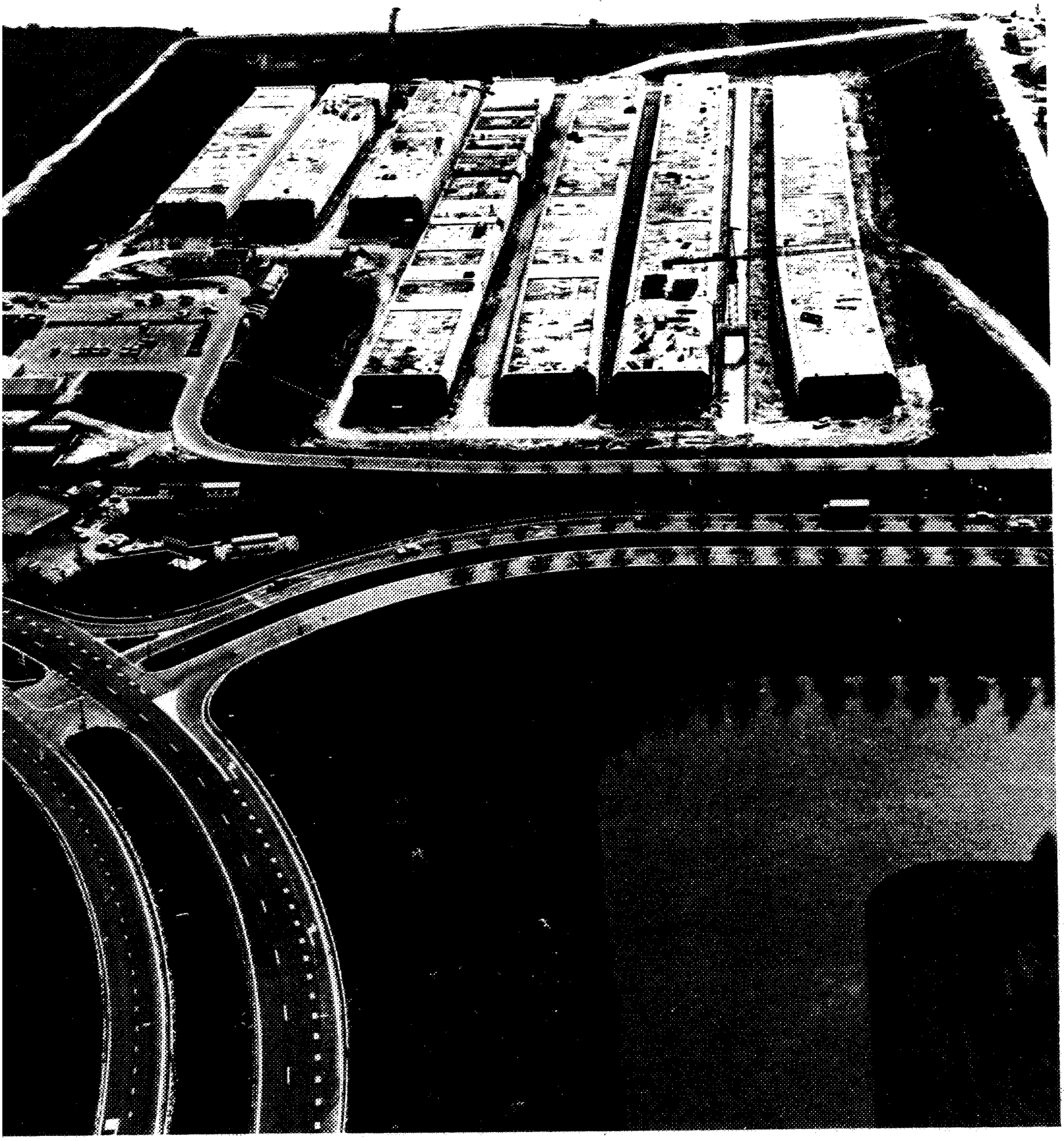
Delta Tunnelling

The purpose of the Delta Tunnelling Symposium is to give delegates an understanding of the problems in choosing between the various types of river and canal crossings (ferry, bridge, tunnel) and, should the choise be a tunnel, to provide background information on the immersed tube method.
On 16 November 1978 various speakers will discourse on these subjects on the basis of their own experience.
We realize that delegates will not be able to remember everything that is said, nor will this be necessary in most cases. Nevertheless they may wish to go through certain papers again at their leisure or to pass on information to colleagues who were not able to attend the Symposium.
We are grateful that the speakers have been willing to provide written versions of their papers, and we hope this book wil contribute to the aim of the Symposium.
Finally, we want to express our thanks to Mr.P.Kieft for his contribution to the organization of the symposium and the realization of this magazine.

The Board of the Tunnelling Section of the
Royal Institution of Engineers in the Netherlands





Ir.J.W.Tops
Director General of the National Public Works Department

Opening Speech

Techniques have been improved in many respects since then, as will be evident from the papers to be given this afternoon. I should like to mention a few features now: the method of linking the units in such a way as to provide a watertight seal, the omission of the waterproof lining, the sand flow method for foundations and the use of longer units.

Holland has influenced tunnel building in other countries, where rectangular cross-section reinforced concrete immersed tunnels are now also used. These countries include Belgium, West Germany, Canada, Cuba, France and Japan.

As I said earlier, this method can be used not only for road and rail tunnels but also for pipeline tunnels, tunnels to carry water and so on. Small-diameter tunnels of this kind can be round in cross-section, though still made of reinforced concrete.
The designs of immersed tunnels in Holland were produced by the Rotterdam and Amsterdam municipal authorities and the Locks and Weirs Division of the National Public Works Department. Although one would not think so from the name, the Locks and Weirs Divison was responsible for designing the majority.

The development of immersed tunnels – which incidentally is still in progress – has not been brought about only by the government bodies I have mentioned. Dutch contractors and manufacturers of certain components have also played an important part in developing and perfecting this type of tunnel construction. Representatives of five of the contractors will speak to you this afternoon, and of the manufacturers I should like to mention one firm – while not in any way detracting from the achievements of the others – :Vredestein in Loosduinen, who have not only provided effective sealing between the units with their rubber profiles but have also introduced the charm of more southern climes into hydraulic engineering by giving them the name 'Gina'.
Dutch laboratories, some internationally known, have also made an important contribution to this technique: Delft Hydraulics Laboratory, the Soil Mechanics Laboratory, also in Delft, and the TNO Applied Scientific Research Organization.

In this way a large group of public bodies and firms have jointly developed something tha can be used not only in Holland but also i many other countries where land and rive traffic have to cross and where the groun consists of sedimentary deposits as in delt areas.
Holland is ready and willing to pass on it know-how and to help with similar projects i other countries. Help can be obtained, for in stance, from Nedeco (Netherlands Enginee ring Consultants) or Amsterdam Publi Works. Nedeco draws on the know-how an experience of the Locks and Weirs Division o the National Public Works Department an Rotterdam Public Works. Help can also b obtained from the contractors, who do not onl build but also have their own design office This Symposium is being held to show you th potential of the immersed tube method. Si king in this case, as you will see, is not disaster but a deliberate operation aiming connect two river- or canal banks.

S.C.Hardy, C.Eng., B.Eng., FICE, FIStructE.
World Bank

The Financing of Infrastructure Projects by the World Bank in Developing Countries

Introduction

The World Bank is a multinational institution devoted to helping the poorest countries of the world realize their development goals. After careful study of proposed development projects and development strategies, the Bank lends funds for suitable projects of high economic priority. Commitments of funds to developing nations now exceed US $ 8,000 million annually. In the following remarks, I will attempt to give you a brief description of the World Bank's organization and operations, a summary of the recent trends in Bank lending, a review of the typical Bank project cycle from inception to completion, and an introduction to some current innovative Bank studies on construction technology and tender evaluation which may be of interest to you in your particular field.

History and Purpose of the Bank

The World Bank is now slightly over 30 years old. It was founded in 1944, along with the International Monetary Fund, at a conference of the allied nations of World War II in Bretton Woods, New Hampshire, USA. Its original purpose was to provide loans for the reconstruction and development of the war-torn allied nations of postwar Europe. Hence its original and formal name of 'The International Bank for Reconstruction and Development'. Around 1950 the Bank began to turn attention more to the activities it pursues today – making loans to promote development in underdeveloped nations.

The Bank is really a group of lending institutions each with slightly different focus. In 1956 and 1960 the Bank was enlarged by the establishment of 2 further development lending institutions, the International Finance Corporation (IFC) and the International Development Association (IDA). The IFC was established to make loans which would stimulate the growth of private enterprise in developing countries. IDA was founded to make loans to the poorest nations of the underdeveloped world on very liberal terms. The repayment of loans by the Bank and IDA must be guaranteed by the government concerned.

Organization of World Bank

The Bank has its headquarters and principal operating base in Washington DC, USA, and resident representatives with their supporting staff in some 30 other member countries. It is headed by a President who normally serves a fixed 5-year term; Vice-Presidents and other staff serve on regular appointments. The office of President is currently filled by Mr. Robert McNamara who was recently elected to his third term. Through an unwritten international understanding, the President of the Bank has always been an American, while the Presidency of the International Monetary Fund has always gone to a European.

The Bank is an international corporate organization controlled by its 131 member countries. The governments of these countries have delegated their powers to a more efficient decision making body – a Board of 20 Executive Directors. 5 Executive Directors are appointed singly by each of the Bank's 5 largest shareholders, and each of the remaining 15 represents and carries the voting rights of a group of countries. Each member country's voting rights is determined by its share of stock held in the Bank.

A staff of some 5,000, of which about half are professionals such as engineers, economists, financial analysts, agronomists, sociologists, etc., is drawn from over 100 different countries, and carry out the decisions made by the Executive Directors. The Bank is organized into 6 geographic, regional operating departments, and a central unit handling administration, research, and policy formulation. Each regional department carries specialists in the more important sectors of the Bank's operations, namely agriculture, power/energy, transportation, industrial development and finance, water supply and education.

Funding and Operations

A. Bank

The Bank is a specialized agency of the United Nations, and loan agreements with member countries are registered at the UN as international treaties, but its source of funding is not the UN. Funds for lending by the Bank come from 3 main sources. The first and most important is the bonds that the Bank floats in world capital markets. The second source is member governments' capital subscriptions which are determined upon entry and are based on the applicant country's economic and financial strength. Member governments pay in 10% of their capital subsuscriptions, while the remaining 90% remains on call to meet the Bank's obligations if necessary. Of the 10% that is paid in, 1% is freely usable for loans, and the other 9% may be used for loans with the contributing country's consent. The third source of funds for lending is the Bank's own profits, which with the agreement of its shareholders, are not paid out in dividends but are used to help in the development of member countries.

The Bank's charter spells out the basic rules which govern its loan operations. The borrowing government bears general liability for loans, and earnings from a particular project are not earmarked for loan repayment. The terms of loans vary, but on the average loans have a total term of about 20 years, which includes a grace period (or moratorium) of 3 to 4 years to enable the project to be completed before capital repayments commence. The lending rate also varies and is set at a level related to the rate of borrowing that the Bank must pay in the capital commercial market; it is fixed, however, for the life of any particular loan. The interest rate on Bank loans in 1978 was about 7$^1/_2$%.

B. IDA

IDA differs from the Bank in its main source of funds, qualifications for borrowers and the terms of its loans (or 'IDA credits' as they are sometimes called). The source of funds for IDA in order of importance are: contributions from richer members, members' subscriptions, and part of the Bank's net earnings. Rules for the payment of subscriptions to IDA differ from the Bank, and vary between 'developed' and 'underdeveloped' IDA members.

As to the difference in borrowers, IDA credits go only to the poorest nations. The criterion currently used is countries with a gross national product per capita below $550. Even within this category IDA concentrates on the poorest nations, and in fiscal 1977 85% of IDA loans went tot nations with per capita GNP less than $280.

The long terms of IDA credits – 50 years – are designed to meet the special needs of its borrowers. No interest is charged, and borrowers are required to pay only an annual $^3/_4$% service charge. A 10 year grace period is granted before repayment begins at the rate of 1% for 10 years and 3% for the following 30 years. IDA credits are often termed 'soft loans' because of the liberal terms of lending.

There is no difference from the Bank in the type of infrastructure projects for which IDA

Table 1
Projects financed by Bank/IDA, fiscal year 1977
(US $ millions)

	Total cost of Projects	Bank and/or IDA loan	% of total cost funded by Bank or IDA loan
Agriculture	5,360	2,308	43
Irrigation	1,895	760	40
Development Finance Companies	762	756	99
Education	630	289	46
Electric power	3,267	952	29
Industry	2,089	737	35
Nonproject	242	217	90
Population and Nutrition	94	47	51
Technical assistance	23	17	74
Telecommunications	583	140	24
Tourism	204	99	48
Transportation	3,084	1,048	34
Sector Urbanization	354	158	45
Water supply	802	301	38
Total	17,493	7,067	40

Note: Details may not add to totals due to rounding.

may lend; the appraisal and processing of projects is the same and is carried out by the same Bank/IDA staff.

Trends in World Bank Lending
A. Philosophy
Taking Bank and IDA operations together, the total volume of lending has undergone considerable growth and changes in composition over the years, partly due to a change in the Bank's development philosophy. The original concentration on large infrastructure projects, from which benefits were expected to diffuse throughout the whole of a nation's economy, has been shifted to an emphasis of more direct aid to the poorest 40% of a nation's population. The reason was, in Bank President Robert McNamara's words, 'we learned that the economist's trickle-down theory of growth is an insufficient basis on which to expect human needs to be met in a reasonable period of time'. Hence the need to identify, design and finance projects which directly benefit the rural and urban poor, in addition to 'old-style' traditional infrastructure projects. The practical effects of this change in Bank/IDA policy is the increasing number of projects such as the rehabilitation of urban slums, agricultural projects for small farmers, rural development, education and small scale industry, which are now being financed.

B. Trends in Total Volume
In fiscal year 1977, ending June 30, 1977, the Bank/IDA made commitments to 228 projects for a total of over US $ 7,000 million. This was an increase of close to US $ 400 million over the previous fiscal year. The projected total commitment for fiscal 1978 is close to US $ 8,500 million, of which IDA credits will account for about US $ 2,300 million.
We normally finance only the foreign exchange component of cost of a development project, and require the borrower to finance most of the local costs from domestic sources. Thus the total cost of projects we finance is far greater than the amounts we lend. Referring to table 1, you will see that the average Bank/IDA participation in project financing was 40% in fiscal 1977, and the total cost of projects was about $17.5 billion. In fiscal 1978, the total cost of projects is expected to be about US $ 20 billion.
The other major multinational lending institutions are the Asian Development Bank (ADB) and the Inter-American Development Bank (IADB). They are organized and operate along similar lines to the World Bank Group, but have fewer members and focus their lending on certain regional areas. Compared to World Bank lending of US $ 7 billion last fiscal year, ADB committed about US $ 0.9 billion and IADB about US $ 1.7 billion.

C. Trends by sectors
The Bank and IDA currently finance projects in the following principal sectors: agriculture, transportation, power/energy, development finance companies, industry, water supply, education, urbanization, telecommunications, population and nutrition. The sectors receiving the largest amount of financing are agriculture, transportation, power/energy, development finance companies and industry, which together absorb about 80% of total Bank/IDA lending. With the exception of development finance companies each of these sectors has undergone substantial changes in the amount of funding and their relative importance over the past 10 years *(see table 2)*.
The most dramatic change has been in the agricultural sector. In monetary terms the amount of loans to agricultural projects has increased more than tenfold from 1967 to 1977 to reach over US $ 2,300 million in 1977. Moreover, the sector almost doubled in relative importance as a percentage of total Bank/IDA loans – from 17% in 1967 to 33% in 1977.
The other major sector enjoying substantial growth from 1967-77 was industry, which includes such projects as the construction of fertilizer, cement, steel, aluminium plants, etc.

Table 2
Bank and IDA lending by principal sectors 1967-77
(US $ millions, fiscal years)

	1967-71 average	%	1972-76 average	%	1977 amount	%
Agriculture	$ 294	17	$ 1,163	25	$ 2,308	33
Development finance companies	179	11	434	9	756	11
Education	69	4	230	5	289	4
Industry	50	3	451	10	737	10
Nonproject	109	6	320	7	217	3
Population	2	–	28	1	47	1
Power/Energy	403	24	613	13	952	14
Technical assistance	2	–	12	–	17	–
Telecommunications	86	5	141	3	140	2
Tourism	6	–	32	1	99	1
Transportation	457	27	955	21	1,048	15
Urbanization	1	–	66	1	158	2
Water supply & sewerage	54	3	197	4	301	4
Total	$ 1,712	100	$ 4,643	100	$ 7,067	100

Note: Details may not add to totals due to rounding.

The monetary amount of loans going to this sector increased more than 10 times, and the proportion of total Bank/IDA lending it received rose from 3% to 10%.

The large civil works sectors (such as transportation, power/energy, and water supply) in which you will be primarily interested, have received a relatively decreasing share of Bank/IDA attention over the past 10 years, but still account for about one-third of total lending. Due to the substantial increase in total lending, the monetary amount commited to these works has continued to increase annually, from about US $ 800 million in 1967, to about US $ 2,300 million in 1977. Tunnels of varying shapes and sizes are often included in projects which the Bank finances in the transportation, power/energy and water supply sectors. Civil works projects financed last year by the Bank/IDA in those sectors amounted to about US $ 7,000 million in total cost. The following are a few examples of tunnels recently financed by Bank/IDA which may be of interest:

Highways: Yugoslavia, Peru, Korea – up to 800 m long;
Railways: Congo 4.5 km;
Power: Chile 9,0 km 13 km, Bolivia 3.5 km 0.8 km;
Water Supply: Colombia (Bogota) 28 km, 3 km, 2 km, Syria (Damascus) 15 km, Portugal (Lisbon) 5 km.

Reverting again to the trends in lending there have also been noteworthy changes over the past 10 year period in some of the minor sectors financed by the Bank/IDA. Urbanization projects have received a growing proportion of funds, mainly to help improve the lot of the urban poor by upgrading squatter settlements and installing essential water supply and sanitation facilities.

The World Bank has also turned attention to energy projects, after the sharp rise in international oil prices in 1973, and for the first time is making loans for oil and gas production. Emphasis is likewise being placed on developing the non-oil sources of energy generation – hydro, coal and geothermal power projects – in addition to traditional sources of energy, such as firewood, for rural areas in developing countries.

The Project Cycle

A. General

Despite the variation in the magnitude and the character of the projects financed by the Bank and IDA, all projects go through a common 'project cycle' which the Bank* has established as a matter of routine procedure. The cycle has 6 major stages, namely:

- Identification
- Preparation
- Appraisal
- Negotiations
- Supervision
- Evaluation

B. Identification

In the first stage of the cycle, the Bank applies the following 3 tests to prospective projects:

- is the sector of the project and the project itself of high priority for development?
- is the project technically and economically feasible?
- will the government support the project by financial and other means?

There are several sources from which project identification may spring. A project may be a repeater loan to an old borrower, may be the outcome of a study done as part of a previous loan, or may be suggested by a government to a World Bank representative or mission in the country. Once a project is identified tentatively, it enters the 'project pipeline' to await further action.

C. Preparation

The next stage of the project cycle is preparation, which can vary in length from a few months to a few years. The purpose of this stage is to reach decisions on the technical requirements of the project, to explore systematically alternatives and to analyze the financial position of revenue earning projects. The detailed preparatory work is usually completed by consultants and not by Bank staff. The Bank may offer guidance to borrowers on terms of reference and specialists in a particular field, but the borrower has the responsibility of selecting and hiring suitably qualified consultants.

The Bank initiates the selection of consultants when services are financed under UNDP (the United Nations Development Programme) with the Bank as the executing agency. The selection process then consists essentially of inciting proposals from 4 or 5 consulting engineers suitably qualified to undertake the project, and then negotiating a fee with the consultant who subsequently submits the best technical proposal. When borrowers are financing the services of consultants, the Bank suggests the use of its selection procedures, but cannot insist. There is a growing tendency amongst some borrowers – unfortunate as it may be for the consulting community – to carry out some form of competitive tendering and to award the consulting services to the lowest tenderer.

D. Appraisal

The preparation stage is followed by the appraisal stage, in which systematic and comprehensive examination of the project is undertaken. The study usually covers up to 6 aspects of a project. First, is the *technical* aspect. In this part of the study all features of project design, cost estimates, and the construction schedule are examined to ensure that the proper technical solutions have been found. The second aspect covers the *economic* analysis, where the focus is on the relationship between the project, the sector and the economy. The third aspect of an appraisal study is the *commercial* aspect, which reviews the buying and selling arrangements of the project, and evaluates the market demand for the project's output, the marketing mechanisms for the output, and the supply of inputs needed for the project. The fourth dimension of a project is *financial*. In this area the appraisal team pays particular attention to two concerns (i) whether there are sufficient funds for construction, and (ii) whether the enterprise will meet its financial obligations. The fifth aspect is the *managerial* aspect. This part of the study covers the adequacy of alle levels of staffing in a project. The last dimension is *organizational*. Issues of particular interest to the analysts covering this area are whether publicly owned enterprises have a sufficient degree of autonomy to administer their affairs, and whether the decision making process and implementation capability of an organization are efficient. The studies of these 6 aspects of the project are drawn together in an appraisal report by a team of Bank staff, and the report is then submitted to a Loan Committee of senior Bank management for consideration of lending.

E. Negotiations

In the fourth stage, Loan Committee approval of the proposed lending is sought. If approval is given, the borrower is invited to send a negotiating team to the Bank's headquarters to discuss the legal and financial obligations contained in a draft loan agreement, and the technical and organizational aspects of carrying out the project. The draft loan agreement and supporting documentation are then submitted to the Board of Executive Directors for consideration, and if approved, the loan agreement is signed and the commitment made.

F. Supervision

The project then moves into the stage of implementation involving the award of contracts for construction, goods and services. This stage is closely monitored by Bank staff and is termed 'project supervision'. The Bank has adopted a policy of international competitive bidding (tendering) for almost all procurement needs to ensure fair and equitable treatment of contractors and suppliers from all member countries. The procedure to be followed are set out in the Bank publication entitled 'Guidelines for Procurement under World Bank Loans and IDA Credits' to which reference is made in all loan agreements.

Tender advertizements for all projects financed by the Bank and IDA are published in a UN publication *'Development Forum'*, a copy of which is available for your perusal. Contractors and suppliers from all member countries and Switzerland are eligible to offer tenders. The borrower is responsible for evaluating the tenders, but a major contract award cannot normally be made without first seeking the Bank's concurrence.

In keeping with our development philosophy that the stimulation of local industrial enterprise is important for the growth of an underdeveloped country's economy, the Bank allows borrowers to give a margin of preference to Domestic manufacturers in the evaluation of tenders. Domestic manufacturers are given a preference of 15% or the prevailing import duty, whichever is lower, when comparing tenders from a foreign supplier. For civil works projects, domestic construction and building contractors are not generally given a margin of preference over foreign tenderers. Howe-

*the term 'Bank' in the subsequent text is deemed to include IDA also ver, the Bank's Executive Directors approved a policy in 1973 whereby in countries with per capita GNP less than US $ 280 in 1976 prices (36 in number), the government has an option to give a $7^1/_2$% margin of preference to domestic contractors when tendering in competition with foreign contractors. This policy is on a trial basis until January 1979, when it will be reviewed again.

Other important parts of the supervision stage are progress reporting, supervision missions, disbursement and accounting. Periodic progress reports are submitted by borrowers according to a schedule established at the negotiations stage. Supervision missions of Bank staff are sent periodically to the borrowing country to evaluate the progress of the project, and make adjustments in project details when changing circumstances dictate their necessity. Disbursements from the loan account are made only as certified expenditures for goods and services are incurred.

G. Evaluation

The final stage of the project cycle is evaluation. The Bank has recently established an internal auditing capacity independent from the units directly responsible for financial, administrative, lending and other activities. The Operations Evaluation Department (OED) makes a critical review of projects after completion to evaluate whether they have met their objectives, and whether the management and lending procedures of the Bank were applied in an efficient and effective manner. OED's annual review of project performance audits was published for the first time in 1978.

Innovate Bank Studies

A. Labor/Equipment Substitution

The Bank continually responds to the needs of developing countries, and seeks to adapt its policies and procedures to meet those needs. Among the innovative work being carried out at the Bank on its project procedures are studies of construction technology and bid evaluation. A research project, started some years ago, studied the possibilities for the substitution of labor and equipment in civil construction. It responded to the fact that many developing countries have a relatively abundant supply of labor which is available at low economic cost. The civil construction industry was an appropriate sector for study because it provides a large proportion of the employment opportunities in developing nations, and construction projects can usually be executed using a range of technologies, many of which are relatively labor-intensive.

The study found that the substitution of labor for equipment in a number of discrete construction tasks was practical, and economically competitive at daily wage rates of about US $ 1.00 to $ 2.00 equivalent in 1977 prices. The minimum daily wage for unskilled labor in a developing country may be higher than the $ 1.00 to $ 2.00 range, but in conditions of substantial rural unemployment the 'shadow price' or opportunity cost of labor (which is the price the input would command in its most remunerative alternative use) may be lower than this artificial ceiling, and may still justify labor substitution on economic grounds.

The study produced about 30 Bank technical memoranda, which are available on request. Some of the subjects which have been covered in the memoranda are the design and use of hand tools, haulage by manual, animal and motorised methods, and various techniques of excavation. The findings of the labor substitution study are being implemented practically on low cost road projects in Kenya, Rwanda, Lesotho, Benin, Chad, Mexico, Honduras and Colombia, using Bank-sponsored management teams.

Many developing countries have a political and social commitment to keeping unemployment low, and are carrying out large public works programs by departmental forces (force account) merely to absorb the surplus labor. These programs are often undertaken without proper planning or due regard to economy and efficiency. Competitive tendering on such public works projects would ensure more realistic prices, and the surplus labor problem could be alleviated if contractors, particularly domestic contractors, were encouraged to use more labor and less equipment. The World Bank is seeking to refine its project design and procurement methods so that contractors using widely different technologies over different construction periods might compete for the same project.

B. 'Neutralization'

Designs and contract specifications for civil works projects are normally biased in favor of contractors using equipment based technology, due mainly to their preparation in so-called 'developed' or mechanised countries, and the influence of those countries in overseas training and advertising. A recent Bank study examines the causes and possible solutions to this problem and suggests a process called 'neutralization' to identify and counter the bias towards the use of equipment. A spectrum of possible technology is considered, the objective of which is to provide alternative designs having a basic stuctural 'equivalence' so that contractors in developing countries, who traditionally tend to use more labor-intensive methods of construction than their foreign counterparts, might tender on an equitable basis with contractors using more equipment-intensive methods of construction. Neutralization would not be appropriate for some countries and projects, and a prior screening process is desirable to eliminate countries and/or projects which appear suited for construction by only one type of technology.

C. 'Present Value' Tender Evaluation

Another refinement of the procurement process which would reduce some of the bias against labor-intensive contractors is the use of a present value tender evaluation procedure. This method would be appropriate for any project in which tenders could be offered with different payment schedules resulting from different construction technologies or other factors. The current tender evaluation procedure customarily involves an examination of the total payment required by the contractor, and the rough 'balance' of those payments over the construction period. Award of contract is made on the basis of the lowest responsive *total* tender amount.

The present value procedure would require tenderers to give a detailed program of construction and a schedule of their expected progress payments throughout the projected construction period. The series of periodic payments would be discounted at an appropriate interest rate to yield an amount representing the present value of each contractor's tender. Labor-intensive techniques may have higher total financial costs but would involve lower payments than equipment based technology in the early periods of construction. A present value evaluation would appropriately weight these lower costs in the early periods, and would also take longer construction periods into account (see sketch and comments on table 3). Discussions with contractors in developed countries, all using equipment-intensive methods, have indicated a positive interest in a rational tender evaluation procedure which provides flexibility in construction periods and competition in mobilization advances.

A modification of the present value procedure would take into account the expected price rises in factor inputs likely to occur over the construction period. A procedure along these lines has been tried out recently on the evaluation of construction projects in South Africa. An assumed increase in the price of labor, equipment and materials is used to inflate or escalate the tender amount over the construction period. The escalated tender amount can then be used as a factor in evaluation, or more rationally, the stream of periodic escalated payments can be discounted back to present value for the comparison between tenders.

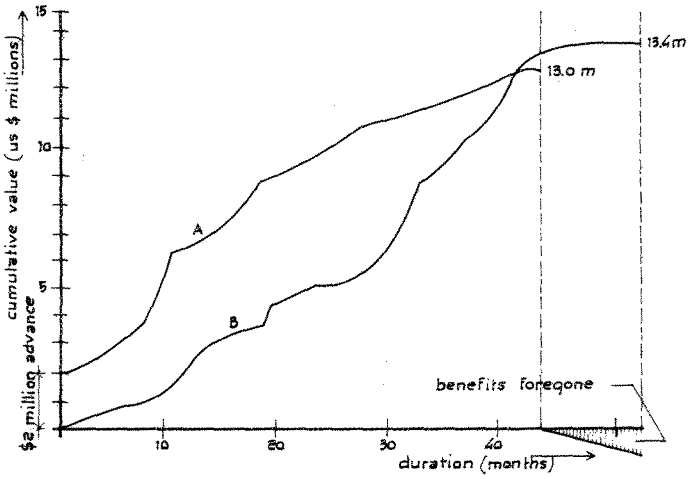
D. Discounted or 'Shadow' Wage Rates

Another procedure under study in the Bank would further remove some of the bias against contractors using labor-intensive construction technologies by shadow pricing the constituent inputs of construction in tender evaluation. Market prices in developing countries often do not represent the real economic or 'opportunity cost' of labor, equipment, or construction materials. The minimum wage rate for labor in developing countries often overstates the opportunity cost of labor because of high rural unemployment, while prices for imported equipment and materials often understate the opportunity cost of foreign exchange. Three possible alternatives of taking this into account in evaluating the economic cost of construction are the shadow pricing of:

- the foreign exchange component
- the equipment component, and/or
- the labor component.

Shadow pricing of the labor input presents the fewest conceptual and practical problems. In developing countries with surplus labor resources, tenderers would be given an appropriate discounted rate of shadow price for unskilled labor (less than the official minimum or urban market wage rate), and would price their tenders using this rate. Other inputs would be priced by them at perceived market rates. Tenderers offering more labor than equipment would have a lower tender price. Clients would award the contract on the basis of shadow tender prices, but contractors would be paid the full financial cost of the labor

Tabel 3
Simulated Examples from East African Highway Projects



Contractor	Tender Price	Duration	Present Value (PV) à 15% per annum	Total benefits lost	PV benefits lost	Effective Tender Price
A (Equipment-intensive)	13.0	44	10.67	0	0	10.67*
B (Labor-intensive)	13.4	52	9.93	0.6	0.38	10.31

Decision: Contractor B, with higher tender price, obtains award on the basis of his lower effective price.

*

Note: If contractor A bidding $13.0 million, had required only a $*1 million* mobilisation advance instead of the $*2 million* shown, his effective tender (assuming a similar payment curve) would have been $10.20 million and he would then have been the lowest effective tenderer.

and other inputs. A simple example of tender evaluation by shadow pricing is shown in the attached table 4. To discourage potential abuse of the procedure by contractors, a number of disincentives and sanctions can be introduced.

Conclusion

This presentation has attempted to give you a short introduction to the World Bank's history, activities and procedures. Trends in the volume and composition of the Bank's lending over the past 10 years have been examined, and the cycle of project identification, appraisal, execution, and post-completion evaluation has been described. I have also referred briefly to current innovative Bank studies which could result in some changes in tendering and evaluation procedures in the future.

Summary

The paper gives a brief introduction to the activities of the World Bank, more formally known as the 'International Bank for Reconstruction and Development', and its affiliated lending institutions the 'International Development Association, and the 'International Finance Corporation'.
The paper first describes the history, objectives, funding and operations of the World Bank group, and then surveys the trends in the total volume of lending for development projects, and the changes in the relative proportions commited to various sectors, over the past ten years.
The principal sectors for World Bank lending in developing countries are agriculture, transportation, power/energy, development finance companies, industry, water supply, education, urbanization, telecommunications, population and nutrition. Last fiscal year (to June 30, 1978) the Bank approved loans of about US $ 8,500 million to assist in financing development projects with a total cost of over US $ 20,000 million.
The paper also describes the various stages in the routine procedure for handling all projects in the Bank, through identification, appraisal, execution and supervision.
Finally, several innovative Bank studies relating to construction technology and tendering are discussed, including labor/equipment substitution, the evaluation of tenders by net present value, and the shadow pricing of construction inputs. Illustrative examples are provided.

Table 4
Bid Evaluation Using Discounted Wage Rates (Illustrative Example), Breakdown of Bid Price

Contractor	Construction Methods (Factor Mix)	Bid Price	Overhead & Margin	Materials	Equipment 1) (+ Fuel)	Labor	Skilled 2)	Unskilled at 50% discount	Discounted (Shadow) Bid Price	Incremental Wage Bill
A	Equipment-intensive	100	20	20	40	20	10	10 = (5) + 5	(95)	+ 5
B	Intermediate	100	20	20	30	30	10	20 = (10) + 10	(90)	+ 10
C	Labor-intensive	100	20	20	20	40	10	30 = (15) + 15	(85)	+ 15

1)
Usage only i.e. depreciation, servicing, repairs, etc.
2)
The skilled labor component will remain roughly constant for the different technologies, as drivers, mechanics, operators, etc. in the equipment-intensive operations are substituted by supervisors, foremen and middle echelon site managers in the labor based operations.

Ir.G.Plantema
Managing Director of the
Rotterdam Public Works

Planning and Organization of River Crossings

Through my work with the Public Works Department of Rotterdam I have been involved for more than thirty years in the growing development of the harbour, the city and the industrial area and in the design and construction of river crossings for road- and railway traffic.
This has been long enough for me to realise that the problems connected herewith in the technical field – difficult though they may be – are often easier to solve than those in the fields of planning, management and finance.
Still it is with pleasure that I comply with the request made by the organisors of this conference to restrict myself to the non-technical aspects of river crossings in my discourse.
I shall impose upon myself yet another restriction. For a river crossing the following solutions are possible:

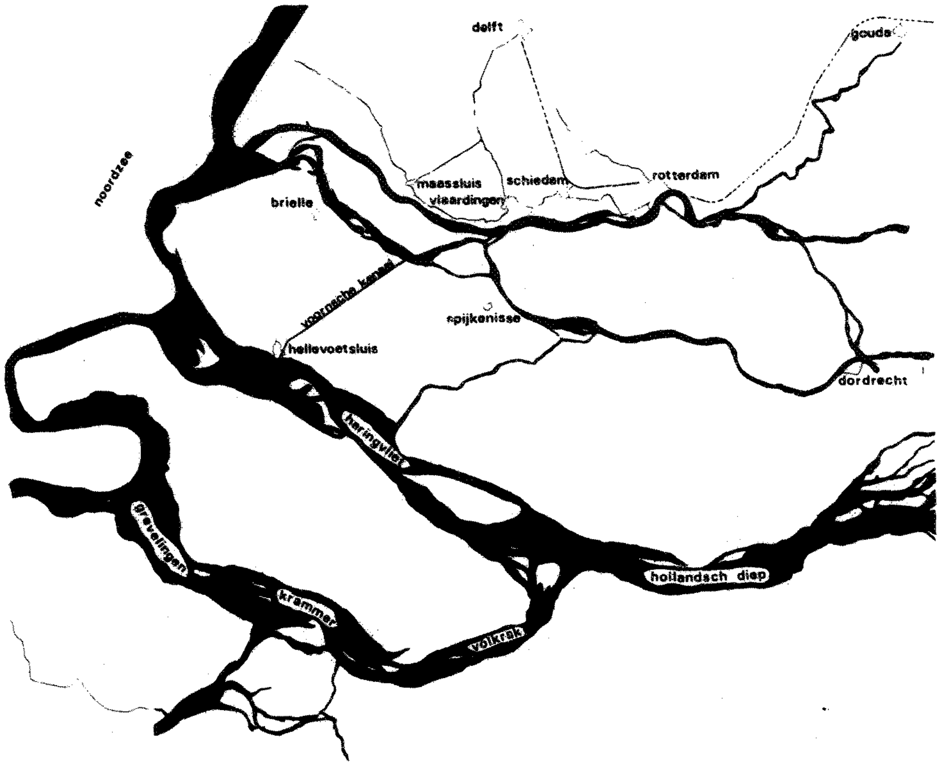
- a ferry connection
- a bridge, movable or fixed, and
- a tunnel

Ferry connections I shall not consider. . . . although I may mention them in so far as they enter into the historical developments. My story comprises short sketches concerning:

- the origin of the land around Rotterdam
- Rotterdam when permanent cross-river communications for road- and railway traffic came into existence (appr. 1875)
- Rotterdam in appr. 1975 and in particular the decision-making in connection with two river crossings which are now under construction: the Botlektunnel and the Willemsbrug. The first a road tunnel to the new harbour area and the latter a new bridge in the centre of the city.

The Land around Rotterdam
In dealing with this subject I may thus name the land where the rivers Rhine, Meuse and Scheldt run into the sea, without being suspected of Rotterdam chauvinism or nefarious plans for annexation.
The area where commerce, shipping and industry could blossom is also called the golden delta of Europe. These 'Lage Landen', the Low Lands, Pays Bas, are really 'hand-made', they were laid out according to a plan.
The soil on which we live has formed itself in the last millennia. Originally there was a lagoon, a stretch of salt water behind a line of dunes, which came into being about ten thousand years ago. The rivers which ran into the lagoon have slowly filled it with layers of clay in addition to which thick layers of peat were formed.
The bottom of the lagoon slowly sank, while the water-level of the seas rose. The layers of clay and peat increased to a twenty metres thick and often very soft stratum, on which our technicians have to create their public works.
About 2000 years ago people in this area started to defend themselves against inundation by the construction of dykes.
In a later stage the inhabitants began to make their land suitable for agriculture and cattle-breeding by digging drainage ditches.
In some places they dug off the peat to use as fuel. The stretches of water which resulted were later reclaimed by means of windmills amongst other things.
People began to pay organised attention to road- and waterways. There was yet a lack of technical means, necessary to prevent the river-estuaries from silting up and river-banks from being displaced.
The struggle between land and water continued. Every piece of land gained by man was challenged by nature. In his struggle man sometimes had to suffer hard blows.
The most recent major effort by man was the construction of the Delta-works, which closed off deep sea channels with large dams. In this region it was not before the 19th. century that the inhabitants thought it necessary to replace the ferry connection across the large rivers by a bridge *(fig. 1)*.
This development was stimulated by the arrival of the railway. Thus the first bridges were railway bridges.

Rotterdam 1875
In the 1st half of the 19th century Rotterdam had grown into a town of over 100.000 inhabitants. There was a need to extend the town and the other side of the river was taken into consideration.
In 1849 the first plan for a bridge was made. This plan did not result from a government initiative but from a competition run by private persons.



he Delta area of Rhine and Meuse around Rotterdam in appr. 1850; the shipping connection with the sea had a depth of about 3 metres. he banks of the river were by no means con-ected

And what did the Municipal Council say: it is a lousy plan! Shipping hindered by the bridge would try to avoid Rotterdam and prosperity would dwindle.

Rotterdam found itself so closely related and bound to the water that the Municipal Council even opposed to having Rotterdam connected by a railway and of course also against a railway bridge.

This opposition was of no availance, however; the Central Authorities put the plan through: the railway bridge was to be constructed!

With the same violence, as the question whether a bridge or a tunnel should be constructed is discussed nowadays, the question if a high or a low railway bridge ought to be built was discussed in those days.

The importance of extending the city to the Southern river bank won: the Government approved of the territorial extension of Rotterdam as a compensation for its demands with respect to the railway.

Once it was settled that a railway bridge would be built, the opposition to a road bridge was also broken. The financing of this bridge with the aid of private persons still had to be arranged. In those days the decision-making for a river-crossing still was so simple in our eyes.

Apart from the bridge in Rotterdam more large railway bridges were constructed over other branches of the river in this delta in those years. Railway trains can hardly be taken across by ferryboats.

In the North-South communication large bridges were built at Dordrecht and Moerdijk, but it was to be a long time before bridges for road traffic were taken into consideration.

Around 1920 no more than three road-traffic bridges could be counted in Delta area, namely the one in Rotterdam over the New Meuse and those in Spijkenisse and Barendrecht over the Old Meuse. The latter two were actually built for the steam-tram which ran there at the time.

Between 1914 and 1940 road-traffic bridges were constructed at Moerdijk, Dordrecht and Hendrik-Ido-Ambacht. In 1938 the construction of the first road-traffic tunnel began in Rotterdam.

After 1945 an important extension of the National Road Network followed *(fig. 2)*, including the following large bridges with movable sections:

A bridge at coastal navigation height (25 metres above water level) at Brienenoord: a tunnel was too expensive;

a bridge at Rhine navigation height at Papendrecht;

a motor/railway bridge to the western harbour area in the Botlek connection.

When the Maastunnel was completed in 1942, no more road traffic tunnels were built until 1965, namely those at Schiedam, Heinenoord and Dordrecht.

In Rotterdam also a metro-tunnel was constructed.

The tunnel at Heinenoord replaces the Barendrecht bridge, which has been demolished meanwhile.

The Spijkenisse bridge was replaced by a new bridge with two raisable spans, which allow the up- and down river shipping traffic to pass through simultaneously.

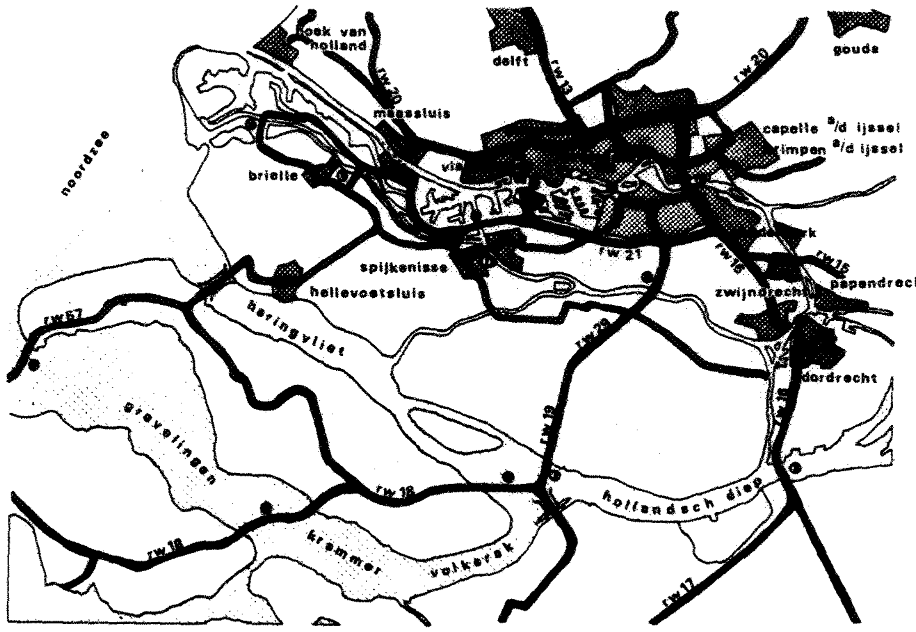
The Willemsbrug and the Botlektunnel are under construction.

The Choice between Bridge and Tunnel

Before I enter more specifically into the decision-making for the Botlektunnel and the Willemsbrug, I would like to mention a number of general aspects and to elucidate those which are important in the choice between a bridge and a tunnel.

The demands made on a river crossing are partially derived from legal regulations, often based on international conventions. In addition there are the demands of shipping traffic, road traffic and railway traffic.

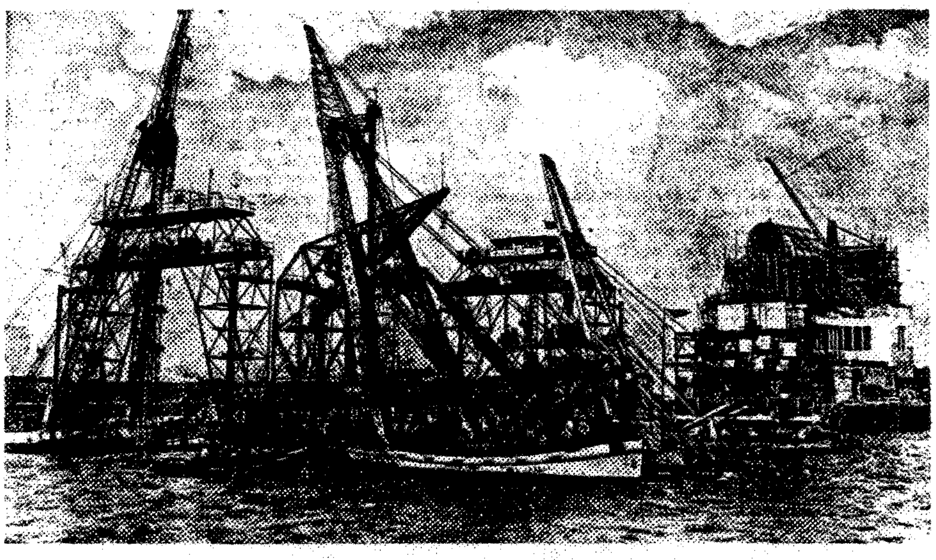
2
A network of motorroads crosses the shipping routes by means of bridges and tunnels. The large sea-arms were closed off from the North-sea with dams



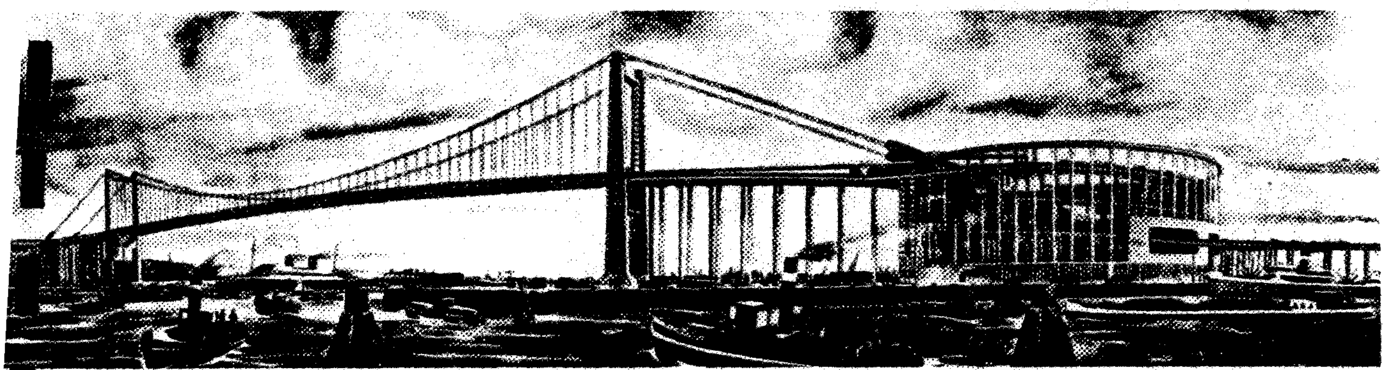
In the decision-making we are governed by the technical possibilities and aspects in the field of finance, planning and environment. These aspects cannot all be measured in terms of money. In decision-making the financial considerations and in particular the building costs usually weigh heavily.
The problems and inconveniences, which are sometimes called 'social costs', though not expressed in money, should nevertheless carry a considerable weight as well.
It may be premised that the question bridge or tunnel will only arise, when there is possible conflict of interests between water- and land-traffic during bridge openings.
When there is no shipping-traffic, the traffic over land may be provided with a low bridge or even a road over a dam.

The more important and intensive shipping-traffic is, the greater a conflict will arise with road-traffic.
It is obvious that this conflicts between water- and land-traffic did not arise until land-traffic began to demand bridges.
On the grounds of historical privileges water-traffic in Holland still holds priority over road-traffic.
Bridge-keepers are shipping men who receive their commissions from the controller of the waterway or harbour in question.
With the coming of the railway bridges a change occurred. Trains have to run according to schedule.
It follows that for international train services there are international agreements. The result of this is that movable railway bridges have fixed opening times; in other words: ships that are too high to pass under a closed bridge must sometimes wait. Furthermore this means that the train schedules must be adjusted to the fixed opening periods, which, in the case of seagoing vessels, are often lenghty.
Everyone will understand that this conflict of interests has grown considerably since it arose a century ago.
One only has to look at the change in the nature of inland shipping.
In the Fifties a large fleet of Rhine vessels was operating, most of them still in private hands.
We have seen a development whereby smaller, wage-intensive ships were replaced by larger units, which required a relatively smaller crew.
This led to push-boat navigation. Four large rectangular barges are coupled to form one unit, which is propelled by a large powerful pushboat.
One coupled unit is capable of transporting appr. 9000 tons, which is ten times as much as the cargo of a ship twenty years ago.
In the old days a shipper's family lived on board; during the day they sailed, during the night they rested.
Nowadays the shipper's wife lives on shore and the ship sails with the aid of radar day and night, even in fog.
The bulk of these pushboats forms, in case of collision, a danger to bridges, and if the bridges are high enough, to the bridge piers. For these reasons bridges are preferably constructed without piers in the river, even if this must raise the costs of bridge-construction.
In my brief sketch about the development of traffic in the Rhinedelta I have mentioned that road traffic outside the cities did not really require bridges over the large rivers until fifty years ago.
In that period – and in particular since 1945 – road traffic has increased to such a degree that the conflict with water traffic manifested itself clearly, for example, in the occurence of long traffic queues and serious delays due to bridge openings.
There are two means to put an end to this conflict: low bridges with movable openings must be replaced by high bridges, allowing passage to all ships; or tunnels, allowing the crossing of all ships.
Bridge or tunnel: that is the question *(fig. 3-4)!*

stunnel in Rotterdam under construc- ... – since 1960 the traffic intensity ... r. 100,000 vehicles per 24 hours



ge with a headway of 60 metres was ... eriously as an alternative solution for ... tunnel

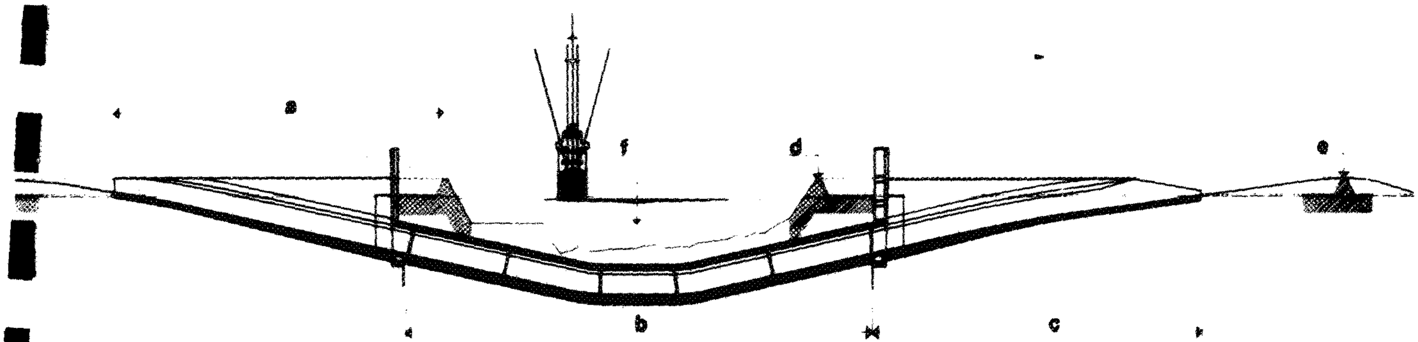


In any case present-day authorities have the advantage that, balancing one possibility against the other, they no longer have to consider the tunnel as a luxurious and more expensive solution.
The technical developments – upon which others will discourse – have had such a profound influence on the reduction of costs in the construction of traffic tunnels that the price difference between a high bridge and a tunnel no longer forms a restriction to the liberty of decision-making.
About the environmental aspect I would like to say briefly that in countryside areas high buildings above ground level are increasingly experienced as being unnatural elements. A tunnel has the advantage of being 'invisible' *(fig. 5)*. As to this last point: the ultimate in development has been reached by making use of the modern ventilation techniques, which require no longer ventilation towers.

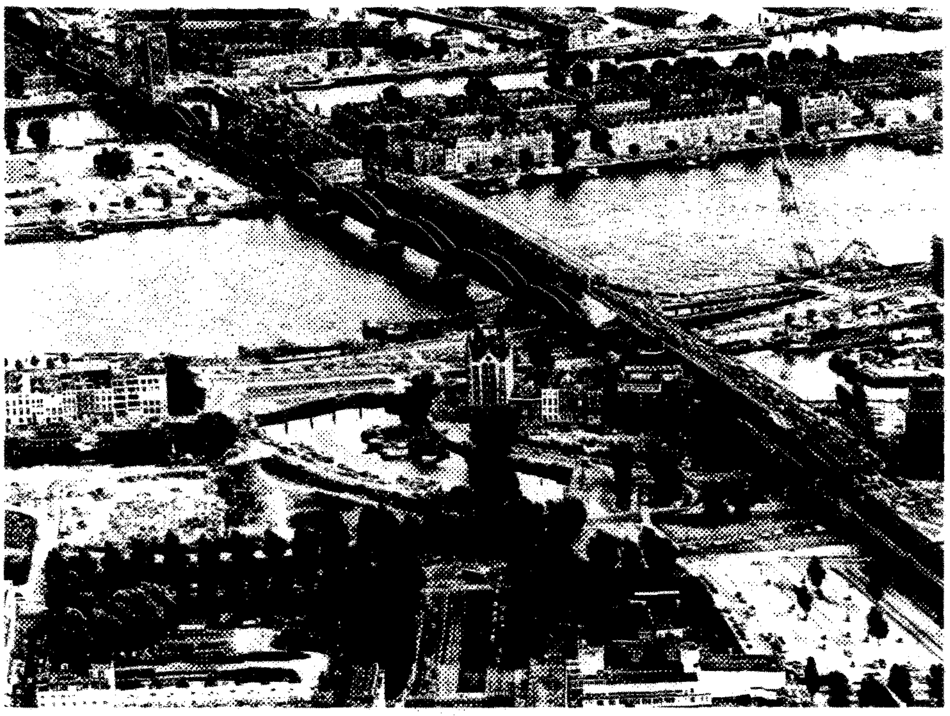
Finally a word, particularly concerning the choice between railroad tunnel and railroad bridge. Talking about a bridge I think of a bridge at Rhine navigation height (9.10 metres above waterlevel). A bridge at coastal navigation height (25 metres above water level) is in more than one respect unacceptable in this urban area.
Railway materials have been designed to be used on flat terrain, with a maximum inclination of 6.5% for bridges. Should this apply to tunnels, these would consequently be very long and expensive. A thorough analysis has now demonstrated that an inclination of 25% may be adopted for tunnels. A favourable development for tunnel builders, but no less for planners.

...nken tube under a river suited for seaships. ... dikes were resituated so that the tunnel is ...pletely outside the dikes

... = open trench, b = sunken part, c = land part, ... old weir, e = new weir, f = tidal river



6
The hundred years old traffic bridge and railroad bridge in Rotterdam; the traffic bridge will be replaced by a new rope-bridge under construction now; the railroad bridge will be replaced in the near future by a 4-lane tunnel.



The Willemsbrug

The choice between tunnel or bridge in the centre of the city of Rotterdam.

The one hundred years old bridges of Rotterdam are in need of replacement *(fig. 6)*.

When the railway bridge and the Willemsbrug were built, the Koningshaven was dug, parallel to the New Meuse, in order to enable the building of movable bridges in quiet. Once swing-bridges were constructed there, fixed bridges could be built over the wide river.

Road traffic to the Southern bank grew fast and soon was seriously hindered by the frequent bridge-openings. The road traffic bridge lay in fact only a few metres above water level, almost five metres lower than the railway bridge.

In the twenties the Willemsbrug was raised by 3.50 metres and the swing-bridge was replaced by a faster working bascule bridge. In the same time the swing-bridge in the railway line was replaced by a lift-bridge.

Even then these measures were considered as temporary, since the bridge would have to be replaced in the not too distant future. In 1956 a working group was formed, composed of representatives of governmental, provincial and municipal services and the railways, whose task it was to advise the Minister.

The advice given in 1959 was as follows:

instead of the two bridges, construct:

- a tunnel for road-traffic with 2 x 3 lanes and
- a railway tunnel for double track.

This advice was based on studies of what would be necessary in order to prevent conflict situations between interested parties that might arise in the long run.

Why was no decision taken upon this advice? The money required was not available.

But Rotterdam could not wait.

In the original plan the railway tunnel and the traffic tunnel were coupled in technical respect.

A method of de-coupling was looked for and found.

In 1969 the Municipal ordered the elaboration of plans for the traffic tunnel according to new insights.

A tunnel was designed with a capacity of a 100,000 cars per day. However, the course of events was different from what had been expected at the time.

In order to assess the situation in Rotterdam it is important to know that the 1,000,000 inhabitants of the agglomeration have a tendency to leave the urban centres in order to live in satellite towns. Since 1950 the city of Rotterdam saw its number of inhabitants decrease from 730,000 to 600,000.

However, employment remains in the city as well as in the harbour and in the industrial areas along the waterways.

Mass-motorisation due to the rise in the standard of living and the increasing distance between the places of dwelling and working areas. This increase becomes evident from the number of cars which cross the New Meuse daily.

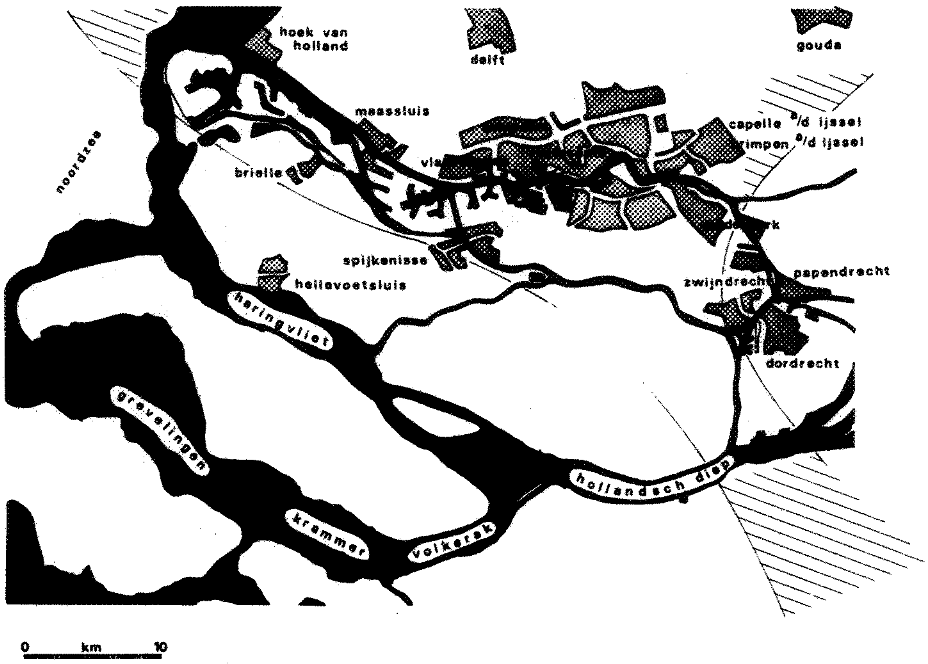
In 1950 their number was 30,000; in 1960 100,000; in 1970 200,000 and according to forcasts made in 1970, their number would increase to 600,000 per day by 1990.

This last forecast has meanwhile been abandoned because of the decrease of the population growth.

As regards the water traffic 'Rijnmond' is an exchange area between sea- and inland navigation *(fig. 7)*. More than 30,000 ships per year enter from the sea and more than 400,000 and 500,000 vessels, both large and small, sail to the hinterland across rivers and canals.

We see sea-navigation sailing landward and inland navigation bringing and fetching cargo to and from sea ports.

e Delta area of Rhine and Meuse around
otterdam in appr. 1975. By dredging to a wa-
rdepth of at least 8 metres with large basins,
e rivers around Rotterdam have been made
vigable for seaships



For an analysis of the conflict to be expected between shipping and road-traffic with a bridge or a tunnel in a particular location, it is necessary to investigate the draught and the headway.
Inland navigation requires a river-depth up to 5.50 metres and sea-going vessels up to 60,000 tons a river-depth of 13 metres. Vessels with a greater draught such as super tankers remain west of the city.
As to the headway, the Act of Mannheim is in force, with the condition recognized by all Rhine-bank States that new bridges over the Rhine must lie at least 9.10 metres above the highest known water-level.
The category of vessels higher than the Rhine headway is relatively small, but does determine the number of bridge openings in Rotterdam. To this category belong coasters – up to 25 metres high –, work-shipping such as dredgers, which maintain the required depth in the waterways and floating gantries and cranes to lift cargoes (up to 800 tons) for ship yards, for construction industries and for loading ships at ever changing places in the harbour area. Also sea-going vessels and off-shore installations are of special importance. They are built or repaired at yards which have a historical position east of Rotterdam.
Of the 200,000 ships passing Rotterdam yearly, appr. 5% – more than 10,000 ships – could not pass under the fixed bridge in 1970, which at that time was at 7.80 metres above Ordnance Datum. After the bridge was raised by another 1.50 metres, the number of these ships in the queue decreased to 7,000.
If the fixed part of a new bridge is laid at 11.50 metres above Ordnance Datum, their number will decrease to appr. 3,000, leaving still ten ships per day for which the bridge has to be opened.
Waiting costs money in our modern society. Especially for larger vessels waiting-hours are extremely expensive.
The repair-yards east of Rotterdam experience the harmful influence of this situation upon their chances to compete.
The fact that their complaints are taken seriously is evident from the decision of the Minister in May last year when, after a serious collision of a vessel with the railway lift-bridge he postponed the resumption of the railway traffic over a temporarily constructed bridge-section, giving precedence to the finishing and sailing of a ship which was being built at one of the yards. These facts seem to urge for measures which free road traffic from the hindrance caused by bridge openings.
From the manner in which Rotterdam was rebuilt after the war – with roads of ample proportions, one can see that the willingness to make the necessary provisions for the increasing traffic was there.
Since 1974, however, the policy of a new Municipal Council has taken clearly a different direction. They wish to call a halt to motorized traffic in the inner city.
It is the intension of the Municipal Council to enhance the quality of life in the centre of the city: expansion of the office area must be found on the outskirts, old quarters of the town must be renovated. Other measures which fit into this vision are the abolition of free parking places in the centre, the restriction of driving speeds, the provision of pedestrian zones and the construction of separate cycling paths.
Through and semi-through traffic must be eliminated from the city. Public transport must be fostered.
Symbolic of this policy was the replacement of the plans for a 2 x 3 lanes WillemsTunnel by a design for a WillemsBridge with one lane for either direction and a centre lane. The bridge will have two free bus lanes for Public Transport.
In this case the Rotterdam river will not lose its bridge – entirely according to the heart's desire of the modern city architect, who cried out that the umbilical cord which unites the two halves of our city must never be buried underground.

The Botlek Tunnel

The Botlek communication is the link with an area of appr. 4 x 25 square kilometres west of Rotterdam, which in the last thirty years has received a totally new appearance.

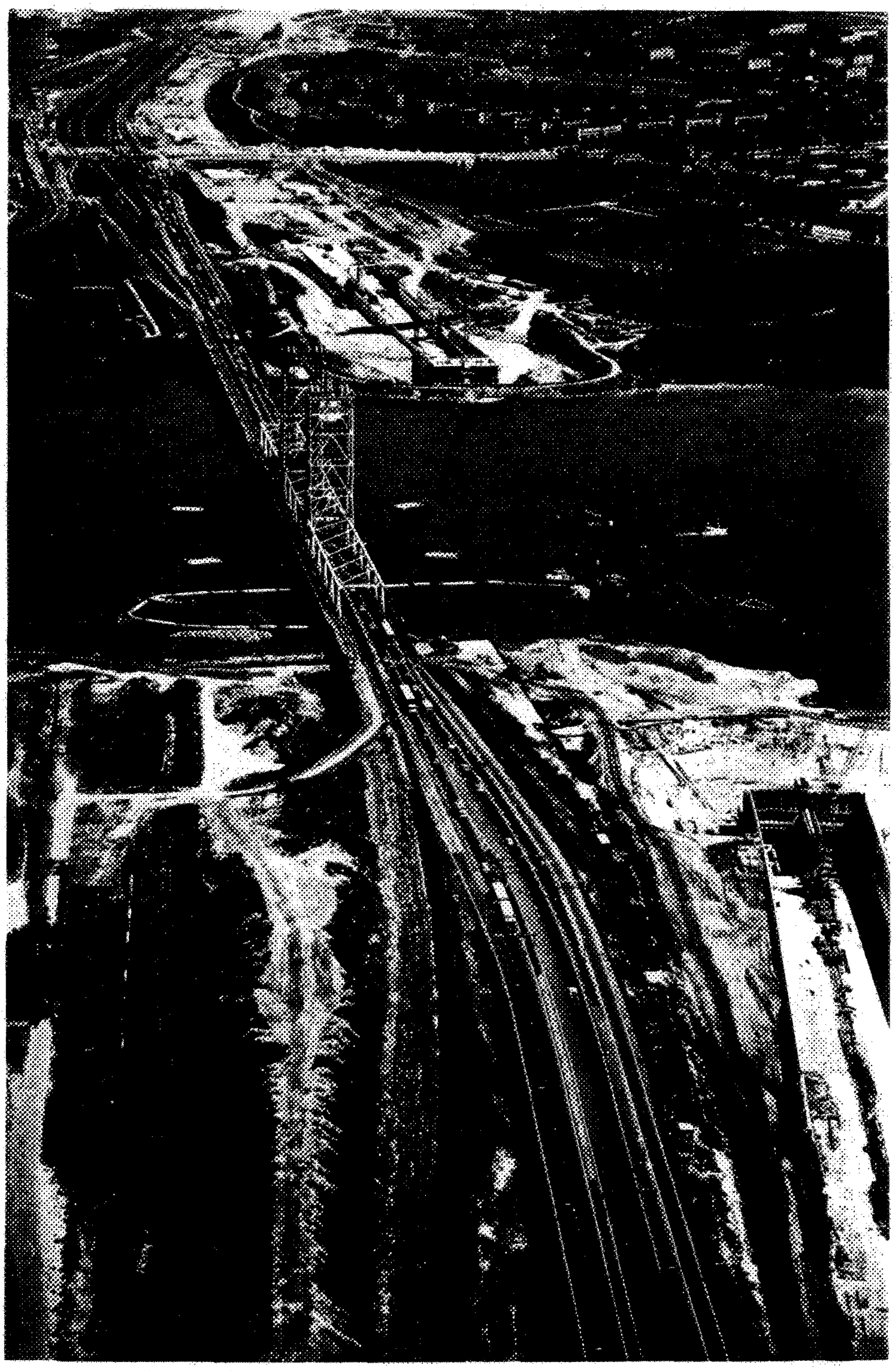
Where there lay an almost exclusively agricultural area in 1950, nearly every square metre is in use now as harbour- or industrial grounds.

This thorough transformation began in the Fifties when a number of deep-sea harbours were dug in the Botlek area with the object of offering possibilities there for the establishing of heavy and semi-heavy industries.

The purpose of this activity was to broaden the economic basis of the port of Rotterdam. During the economic depression of the Thirties it became more than evident how the existence of a harbour which serves principally as a transit and staple port is susceptible cyclical influences.

The trend towards industrialization existed even before World War II: at that time there was already an oil refinery on the Sluisjesdijk, which had to be removed later for safety reasons.

For the opening up of the new industrial area communications to the hinterland were necessary. There were sufficient waterways available in this Delta area. Road- and railway communications had to be created, part of which was the Botlek bridge, built in 1953 *(fig. 8)*.



8

Botlek tunnel under construction. Beside the 15 years old bridge, a traffic tunnel is under construction, because the bridge has to be opened much too often for sea-traffic

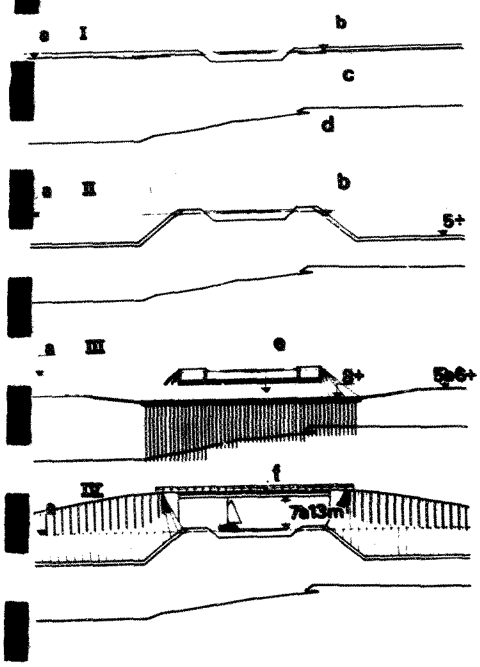
In anticipation of future developments of the new harbour area a bridge with a modest traffic capacity was designed. It was a lift-bridge with room between the main girders for a rail-track and a two-lane motorway. Outside the main girder, cycle- and pedestrian paths were constructed.
The expected industrialisation took place at a surprising pace. The harbour- and industrial areas were expanding continuously and were also taken into use.
For the increasing road traffic the Botlek bridge soon proved to be too small. In addition, the bridge had to be opened more and more frequently for increasingly larger ships.
Gradually a situation arose that road traffic had to put up with queue forming and consequently serious delays because of bridge openings.
In our harbour town where the reliability of the communications must be proverbial, this was unacceptable. Within ten years after the opening of the Botlek bridge studies began to replace this bridge.
If we cast our minds back to the manner in which a hundred years ago the decision was made about the construction of the first fixed cross-river communication in Rotterdam, we will see that today both authorities and technicians have considerably more difficulties to face than their predecessors.
First of all a forcast was made of the various future traffic streams: road traffic, shipping traffic and railway traffic. After that an investigation was carried concerning the necessary provisions to cope with these streams.
The result of this investigation showed that a choice had to be made between:

- a 2 x 3 lane tunnel;
- a 2 x 4 lane bridge at sea navigation height;
- a bridge at Rhine navigation height with 2 x 3 lanes.

From the beginning of the investigation up to the time that the decision to construct a tunnel was made, the demand for an optimal solution of the conflict between road- and railway interests on the one hand and navigation on the other, has run like a red ribbon through the process of decision-making.
To this end the interests in question were thoroughly analysed.
A decision about a new cross-river communication need not be made in haste, since in the meantime emergency arrangements had been made to keep the traffic streams on the bridge on the move.
In addition the railway track on the bridge was modified for use by road traffic during rush-hours; during these hours the bridge was not lifted for inland navigation: at a later date the bridge was jacked up to a greater height.
I shall now elucidate a number of aspects which have played an important role in the consultations.

- From an investigation it soon became clear that a bridge at Rhine navigation height with a movable section would not offer an acceptable solution for capacity problems, waiting times for road traffic and delays. The choice was now between a bridge at sea navigation height and a tunnel.
- The safety, vulnerability and capacity of a tunnel show up to advantage in comparison with a bridge. If the cross-river communication consists of a tunnel, shipping has no need to reckon with local narrow passages and obstacles which exist with bridges. There is no danger of collision.
- In case of a bridge at sea navigation height, the long approach slopes will reduce the traffic capacity because of the large amount of cargo transport *(fig. 9)*.
  In case of a tunnel this may be easily avoided by the construction of an extra crawling lane on the approach slope.
- No exceptionally tall transport may pass through a tunnel and no dangerous substances may be transported.
  In connection with the latter I would like to make a point. Although in Holland, in comparison with abroad, relatively many tunnels have been built, the tunnel here has to contend with the handicap impassed upon it by national legislation that no dangerous substances may be transported through the tunnel. Since precisely these transports occur very often in the Rotterdam harbour area, it was very important to know if in a tunnel one tube with one lane could be constructed so strongly that the rest of the tunnel traffic would run no danger in case of explosion or fire. The fact is that in our country it is easier to build a tunnel than it is to change a law.
  The Botlek tunnel will not have such a tube. As long as the old bridge exists dangerous transports may use it.
- Finally I mention the costs as a last aspect. It is certainly not the least important aspect – on the contrary!!

When calculations showed that in this case a tunnel construction would be cheaper than the construction of a bridge, this argument was added to all the others which plead in favour of the construction of a tunnel: this was – as it were – a starting signal to commence building.
In the beginning of my discourse I have already said that the organisors of this conference had imposed upon me restrictions with respect to the contents of this narrative. Now, at the end, I am very well aware of the fact that I have had to impose upon myself even greater restrictions.
There still remains so much which has *not* been said.



9
The flat Dutch country consists of a peat layer of appr. 18 metres on top of a diluvial sand-layer (section I)
Locally this peat was removed to be used as fuel; the canals remained on the old level (Section II)
A typically Dutch landscape in which tunnels and/or aqueducts (section III) fit better than high bridges (section IV)
a = Normal Amsterdam Level, b = black soil, c = peat, d = diluvial sand, e = aqueduct, f = bridge

Summary
Rotterdam is a classical example of a harbour that developed due to its position in a river delta. Cargo imported by sea is distributed to the hinterland by a system of inland waterways, canals and natural waterways such as the Rhine and the Meuse. Roads and railways also serve to distribute the cargoes, but these inevitably cross the waterways.
These intersections lead to conflicts of interest, for the available land and funds, and traffic movements. The approach used by the authorities in solving these conflicting interests will be presented.

Drs.G.R.Otten* and **G.A. de Vries**
The Netherlands Economic Institute, Rotterdam

The Economics of River Crossings

Introduction

In July 1972, the 'Centrale Bond van Scheepsbouwmeesters in Nederland' (CEBOSINE; Central Association of Shipwrights in the Netherlands) requested the Netherlands Economic Institute to investigate how the realisation of a new connection between the two river banks in Rotterdam would affect the shipyards east of it. The new connection would replace the existing bridges across the Meuse, viz. a railroad bridge and a road bridge, both consisting of a fixed and a movable part.

The commission to the Netherlands Economic Institute was actuated by a Memorandum issued by the Department of Public Works in Rotterdam about the choice between bridges and tunnels. This Memorandum held a plea for railway and road bridges of the Rhine-shipping level, viz. approx. above O.D. (Ordnance Datum) to be raised for the sake of shipping between 0.00 and 6.00 hours only. It was the latter part of the proposal that caused concern to the shipyards situated east of Rotterdam, because it would cut the present number of raisings to about one third.

To analyse the problems involved, the method of the social cost/benefit analysis was chosen. In cost/benefit analyses, a form of policy analysis, the advantages and disadvantages of alternative policy measures are reviewed as much as possible in quantitative terms, over a considerable length of time.

The usual procedure in cost/benefit analyses is to compare a project with the so-called zero alternative, i.e. the situation that would exist if the project were not carried out (the do-nothing alternative). In principle the same procedure can be applied to cases with several alternative projects; each of these is then compared to the zero alternative.

In the present situation in Rotterdam the zero alternative is not considered realistic, however. The road bridge is so old that it must be replaced anyhow. Moreover, a reconnaissance into the future growth of road-, shipping- and especially rail transport has made it abundantly clear that the existing bridges and the present bridge-raising regime cannot be maintained without increasing difficulties. For that reason it was assumed in the analysis that it would not be realistic to let the present situation continue and the cost/benefit analysis was adjusted accordingly. The alternative solutions envisaged were not compared to a zero alternative, but *ranked* by means of the cost/benefit analysis. To that effect the alternatives were compared pair-wise, and benefit/cost ratios derived from the differences in costs and benefits. The question is not whether or not a connection of the river banks is desirable, but *which* of the alternative types of connections is socially preferable.

The Situation at the Time of the Investigation

The Bridges

At present the *Meuse* is spanned by a double-track railroad bridge and a three-lane car bridge. The car bridge is also used by pedestrians, cyclists and motorscooters. The height of the (fixed) railroad bridge is 9.15 m above O.D. and that of the (fixed) car bridge 7.80 m above O.D. Each day the bridges are raised 14 times (if necessary) at fixed hours. The road bridge is about one hundred years old.

Shipping

The ships passing under the Meuse bridges form a mixed batch. Local counts have registered seaships (> 500 GRT), coasters (< 500 GRT), derricks, floating cranes, and other floating equipment, as well as barges, yachts, and tugs. In 1971 12,600 passing objects were counted, 60 per cent of which were barges, 20 per cent coasters, 10 per cent derricks and cranes, and 10 per cent other objects.

Most of the seagoing vessels sail to or from the shipyards east of Rotterdam on the New Meuse, The Hollandse IJssel and the Noord, for repair or demolition, or being newly constructed. The coasters passing have various motives; their passage may be due, as with the seaships, to repair, demolition or new construction, but they may also carry goods to or from places in Germany and the Netherlands. The derricks and cranes are mostly used for construction and repair at the shipyards east of Rotterdam.

It can be observed that, as far as seaships and coasters are concerned, most vessels pass when the bridge is up from 18.40-19.00 hours, while few of them use the opportunities during the night. Derricks and cranes pass mostly in the course of the morning or afternoon, in connection with the work hours at the yards.

The Shipyards Involved

At the time of the investigation there were 19 shipyards in the area that would be affected by the choice of connection. Their turnover amounted to some *f* 91 mln in 1971; the yards employ 1,600 workers and indirectly provide employment to another 1,400 men in supplying industries. Ships stay but a short time at the yards, 6-9 days being the average length.

Road Traffic

Road traffic is not hampered too much by the Meuse bridges. For one thing the traffic flow is modest: at the time of the study 40,000 private car units were crossing the bridges in 24 hours. For another, the bridge does not always stay up for the full 20 minutes. Moreover, the times of bridge-raising are widely published, so that it is not unrealistic to assume that drivers anticipate, choosing an alternative route or start their trip somewhat earlier or later than they would if there were no impediment.

Rail Traffic

The railroad connection crossing the New Meuse is one of the busiest in the Netherlands railway network. It is also one of the most important, used not only by commuters between Dordrecht and Rotterdam, but also a large portion of the interregional and international rail traffic. Apart from that, lots of goods trains roll across every day.

The problems of Netherlands Railways spring from the system of guaranteed opening times. The guaranteed duration (approx. 20 minutes) need by no means be the real duration; actually that is often shorter. But the time tables have to be drawn up a long time in advance and cannot be based on daily fluctuating limits the frequency of trains on the link Dordrecht-Rotterdam, which is the more frustrating as this link is very much like a metropolitan line.

The unfortunate accident in May 1978, when the bridge was seriously damaged by collision proved how vulnerable a railroad connection is when there is a lift bridge to be crossed.

So naturally the Railways Administration assumed that the present rail bridge was to be replaced with a tunnel. Rail and water traffic would then no longer bother each other at this important crossing of various traffic routes. The tunnel solution, though still thought optimum, is not the only one considered, an alternative being the construction of a new bridge and the introduction of a new regime, the bridge to be raised only

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Thanks are due to Mrs. A.C.A.Elderson-De Boer for her translation of the original Dutch text.

during night hours. That would leave the passage free for trains in the daytime, and improve the quality of the railroad connection.
Quite probably, however, there would be losses to other sectors of the economy (shipping, shipyard activities), and they would have to be imputed to this solution.

The Growing Conflict

From the studies available in 1973 (Integral Traffic and Transport Study; Freeman – Fox – Wilbur Smith Study), and our own analyses of shipping traffic it could be inferred that cars, ships and trains, would to an increasing degree make contradictory demands on the bridges' raising regime and that serious conflicts were inevitable. A few solutions had already been suggested, some others could be envisaged, and finally, the NEI-study set out to identify all alternatives, to find out how they would affect the parties concerned, and rank them the help of a cost/benefit analysis.

The Alternatives Retained

To connect the river banks in Rotterdam, four combinations of tunnels and/or bridges and various regimes of bridge-lifting can be envisaged, viz.:

- rail bridge – road bridge
- rail bridge – road tunnel
- rail tunnel – road bridge
- rail tunnel – road tunnel.

Four variants bridge-raising regimes were chosen for the analysis, viz.:

- the existing raising regime (14 times per 24 hours);
- 9 times per 24 hours (6 at night and 3 in the daytime);
- 6 times per 24 hours (during the night);
- raising by request (a variant only applicable to the combination road bridge – railroad tunnel).

The number of alternatives involved in the analysis thus came to 11, symbolised as follows:
(1) $T_1 + T_2$
(2) $T_1 + B_2$ (14)
(3) $T_1 + B_2$ (9)
(4) $T_1 + B_2$ (6)
(5) $B_1 + B_2$ (14)
(6) $B_1 + B_2$ (9)
(7) $B_1 + B_2$ (6)
(8) $T_2 + B_1$ (14)
(9) $T_2 + B_1$ (9)
(10) $T_2 + B_1$ (6)
(11) $T_1 + B_2$ (raised on request)
Legenda:
T_1 = railroad tunnel
T_2 = road tunnel
B_1 = rail bridge
B_2 = road bridge
The figures between brackets behind the combination symbols refer to the number of raisings per 24 hours.
The technical specification of bridges and tunnels was that of the plans in existance at the time. These plans assumed that ships which can pass the Baanhoek Bridge near Sliedrecht will not be hampered by the Meuse Bridges. For road bridges and road tunnels 2 x 3 lanes were envisaged, for the railroad bridges or tunnels four tracks, dimensions that followed for the road connection from the study by Freeman, Fox, and W. Smith, for the railroad connection from the NEI's Integral Traffic and Transportation Study in 1972.

The Costs of the various River-Bank Connections

In accordance with the guidelines laid down for a cost/benefit analysis of the COBA (interdepartmental committee for the development of policy analysis), costs are understood to represent the costs made for the express purpose of the project; benefits are all remaining effects; they may be positive or negative. In the cost/benefit analysis discussed here the costs are composed of two elements:

- costs of investment in the road and railroad connections;
- the exploitation costs, consisting of maintenance costs for bridges and tunnels and operation costs for the bridge.

Investment and operating costs are presented in the table below. It may be recalled that the amounts given are based on estimations at the time of the study, i.e. the year 1972. The costs of the road connections are derived from the Synthesis Report drawn up by the Department of Public Works in Rotterdam, those of the railroad connections from a Memorandum of the Netherlands Railways of May 1972. All amounts are expressed in Dutch guilders of 1972 and have been discounted to the 1st of January 1973 with the help of the respective investment schemes. They take into account the costs of access and egress connections, including traffic provisions, on both banks of the river Meuse.
In table 1 the 11 alternatives of section 4 could be compressed into four, because the bridge-raising regime does not affect the costs.
N.B. The building of the road connection (bridge or tunnel) has been synchronised with that of the railroad connection. Because a railroad tunnel takes a year longer to construct than a railroad bridge the effect of the discount factor leads to different outcomes for the road connection in combination with a railroad tunnel or a railroad bridge.

The Benefits

As pointed out earlier the benefits include all 'other' effects. Because in our analysis benefits were defined as differences in cost between the alternatives, it was necessary just to trace the costs of each alternative in absolute figures.
In the study the following cost elements (i.e. all remaining effects apart from investment and operation costs) were identified; naturally they are closely associated with the users (direct and indirect) of the bridge or tunnel:

- losses to road traffic;
- losses to railroad traffic;
- losses to shipping;
- losses to shipyards.

Below we shall discuss in some more detail how each of these categories of losses was measured for the individual alternatives.

Losses to Road Traffic

It should be pointed out first of all that for alternatives featuring either a road tunnel, or a road bridge raised only at night, the losses to road traffic can be put at zero. That applies to six alternatives, viz. (1), (4), (7), (8), (9), and (10) (see paragraph 4). In the other four alternatives with raising regimes there *are* losses, due to private cars and lorries being compelled to wait; the waiting times in terms of money can be expressed as an economic loss.
The following elements were taken into account in computing the loss:

- daily pattern of river-crossing road traffic;
- average total time the bridge was closed to road traffic per 24 hours; this period varies with the number of raisings;
- average waiting time;
- travel motives; their composition varies through the day (home-to-work, business, shopping, other);
- travel-time valuation by motive in guilders per hour.

Table 1
Investment and operation costs, discounted to January 1, 1973, for possible variants of river-spanning connections, expressed in mln guilders of 1972

Homogeneous tunnel variant					
Road tunnel	:	investment	142,6		
		exploitation costs	7,2	149,8	
railroad tunnel	:	investment		253,9	**403,7**
Heterogeneous variants					
A. road tunnel	:	investment	153,9		
		exploitation costs	7,8	161,7	
railroad bridge	:	investment	210,5		
		exploitation costs	2,2	212,7	**374,4**
B. railroad tunnel	:	investment		253,9	
road bridge	:	investment	89,4		
		exploitation costs	2,9	92,3	**346,2**
Homogeneous bridges variant					
road bridge	:	investment	96,5		
		exploitation costs	3,1	99,6	
railroad bridge	:	investment	210,5		
		exploitation costs	2,2	212,7	**312,3**

Thus the present value per January 1, 1973 of the waiting-time losses over 30 years could be determined for two bridge-raising regimes:

- at 14 raisings per 24 hours: Dfl. 7.5 mln
- at 9 raisings per 24 hours: Dfl. 6.2 mln.

It may be recalled that the variant with 6 raisings per 24 hours does not cause any losses to road traffics, because all the raisings would be at night.

Losses to Railroad Traffic

Railroad losses refer mainly to passenger traffic, which is impeded by bridge raisings in the daytime. The losses incurred in the bridge variant with only (6) raisings at night have been put at zero.

In the same way as for road traffic, the losses calculated represent money valuations of passengers' time losses. When the bridge has to be raised in the daytime, trains will be dropped from the time table on the link Rotterdam-Dordrecht in both directions. The waiting times have been calculated on the basis of the time elapsing between the desired moment of departure (= the moment of departure that would be chosen if the bridge were kept down) and the actual moment of departure (= the moment of departure delayed by the bridge being up).

With the help of passenger-transport forecasts for the relation Rotterdam-Dordrecht valid at the time, and/or the assumption of a certain distribution of travellers by motive and a certain value of time relevant to each motive, the present value per January 1, 1973 of waiting-time losses were computed. They amount to:

- Dfl. 34.0 mln for 14 raisings (of which 8 in the daytime)
- Dfl. 12.7 mln for 9 raisings (of which 3 in the daytime).

Losses to Shipping

Economic losses to shipping are defined as the money valuation of time lost in waiting under the various raising regimes of a bridge. The following objects may have to wait before a raised bridge.

First, newly constructed seagoing vessels launched from the shipyard.

Second, seagoing vessels which are sailing to the shipyards for repair, or leaving the shipyards after repair. Third, the category of so-called loading and unloading coasters with various destinations and origins. Finally, derricks and cranes recruited by the eastern shipyards for repair or construction activities, or passing under the bridge for other reasons.

For the four categories together the following waiting-time losses (discounted to January 1, 1973), were found:

- 14 raisings: Dfl. 8.3 mln
- 9 raisings: Dfl. 5.3 mln
- 6 raisings: Dfl. 21.2 mln.

That with the transition from 14 to 9 raisings the money value of time losses goes down rather than up is due to the fact that fewer raisings lead to a considerable portion of the repair work being lost to the shipyards; in the next section this aspect will be highlighted.

The Impact of the River-spanning System on the Costs and Benefits of the Upstream Shipyards

The shipyards established upstream from the bridges across the Meuse in Rotterdam are at a disadvantage compared with their competitors downstream, as is argued extensively in (2). For one thing, seaships destined for those yards for repair have a longer stretch to sail, and for another, they have to pass three obstacles, viz. the old road- and railroad bridges (both raised 14 times in every 24 hours to let ships pass) and the modern Brienenoord bridge (raised on request). The longer route and the obstacles make the accessibility of the upstream shipyards extremely vulnerable. The old road- and railroad bridges are due for replacement; the various combinations of bridges and tunnels that can be envisaged have, of course, different impacts on the business results of the upstream shipyards. In fact, by the system they will choose for spanning the river, the public authorities control directly the prospects of these shipyards. Their turnover would expand if tunnels were to replace the bridges, or if the 14 raisings were replaced with a system of raising on request; it would contract if the raising regime were to be tightened to 9 or even 6 raisings per 24 hours. The public authorities will have to base their investment decision on a cost/benefit analysis, comparing the relative merits of various river-spanning systems.

If a combination of tunnels or a combination of a railroad tunnel and a bridge raised on request is taken as the base for comparison, both the *actual decreases* in turnover due to a tighter regime and the *increases foregone* because a system more favourable to the upstream shipyards is not chosen, should be represented in the cost/benefit analysis in terms of national-economic costs.

On the other hand, when the merits of new systems are considered in relation to the present system of bridges raised 14 times a day, actual turnover increases have to be represented by national-economic benefits.

In (2) a simulation model was presented to estimate the gains and losses in turnover that are to be imputed to the various river-spanning systems. To that end, a kind of 'price'-variable was introduced, indicating how much the competitive strength of the upstream shipyards is weakened or strengthened as a consequence of additional time losses or decreases in time losses due to different river-spanning systems. The price variable is in fact used as a variable of economic resistance: a tighter regime raises the 'price', raising on request or a set of tunnels lowers it. Analogously, the price-elasticity coefficient is to be interpreted as a structural coefficient of economic resistance. The value of this coefficient could only be derived for the present situation of 14 raisings, by comparing the present frequency distribution at upstream and downstream yards according to duration of repairs. How that was done and how the price variable was constructed is explained in detail in (2).

The structural coefficient was found to have a high (minimum) value: 40.32; it reflects, apart from the calculable time losses already referred to, additional delays that are impredictable and, therefore, incalculable. Such delays, due to vertical and horizontal movements, fog, and strong winds, confront shipowners who want to have their ships repaired at upstream shipyards, and cannot be compensated for by the shipyards. They spell uncertainty to shipowners, and thus weaken again the competitive strength of the upstream shipyards. If the structural coefficient of economic resistance is high under present circumstances, it can be expected to go up if the bridges were to be raised only 9 times a day (which means 4 hours' waiting instead of 1.5), and will certainly soar dramatically if bridge raisings are to be restricted to only 6, and that during night hours; shipowners will no doubt anticipate on the additional burden and thus help the coefficient to rise. In the quantification of the actual decrease in turnover to be expected under a six-raisings regime the deterioration in the upstream shipyards' position was expressed by raising the structural coefficient to 60.48.

Let us now consider what happens when tunnels are built to replace the bridges. In table 2 only the elemination of waiting before the bridges and the corresponding reduction of sailing time are taken into account; because the value of the structural coefficient has not been reduced, the increase in turnover assigned to the indroduction of tunnels represents a minimum.

Tunnels would substantially improve the accessibility of the upstream shipyards; for one thing they can be reached faster, and for another the connection becomes less vulnerable with only one obstacle to pass instead of three. It is, moreover, a navigational advantage that ships would no longer have to pass through the Koningshaven, which is the part of the river that is spanned by movable bridges.

The competitive position of the upstream shipyards would become almost equal to that of the downstream ones if the last obstacle – the Brienenoord bridge, were also replaced with a tunnel. Unhampered by obstacles in the river, they could hope to realise the potential turnover corresponding to their capacity and skill.

From the estimated actual turnover changes can now be derived the turnover developments to be imputed to each river-spanning system such as they are required for a cost/benefit analysis, on which the investment decision is to be based *(table 3, page 22).*

Table 2
Estimation of actual changes in turnover in 1983

River-spanning systems in 1983	**mln guilders of 1972**	
tunnels	at least	16.1
bridge raisings on request		9.7
14 bridge raising		–
9 bridge raisings	–	19.3
6 bridge raisings	–	86.9

Table 3
Changes in turnover dependent on the investment decision of the public authorities

River-spanning systems in 1983	mln guilders of 1972
tunnels in relation to 14 bridge raisings	at least 16.1
14 bridge raisings in relation to tunnels	– at least 16.1
9 bridge raisings in relation to tunnels	– at least 35.4
6 bridge raisings in relation to tunnels	– at least 103.0
bridge raising on request i.r.t. 14 bridge raisings	9.7
14 bridge raisings i.r.t. bridge raising o.r.	– 9.7
9 bridge raisings i.r.t. bridge raising o.r.	– 29.0
6 bridge raisings i.r.t. bridge raising o.r.	– 96.6

On the assumption that an improvement in the competitve position of the upstream shipyards means an equal improvement in the competitive position of the Netherlands as a whole, as far as ships' repair works are concerned, it can be said that the benefits gained by the upstream shipyards owing to the elimination of obstacles on the river count as national-economic benefits. If the greater share in the European regional ships' repair market can be maintained for, say, ten years, the total national-economic benefits to be reaped will add up to ten times the yearly turnover growth of the upstream shipyards.
By maintaining the present regime of 14 bridge raisings per 24 hours the national economy foregoes those benefits. Tightening of the regime to 9 or 6 raisings moreover costs the economy the losses in turnover the upstream shipyards are bound to sustain. In (2) the loss in turnover is fixed at one year. To it are added the loss of capital ensuing from the under-utilisation of capital goods due to the loss in turnover.
Table 4 shows what the national economy stands to gain or lose in the way of shipyards' benefits or losses according to whether the government decides to invest in tunnels or in a bridge that will be raised on request, or in bridges that are up to let ships pass 14, 9 and 6 times per 24 hours, respectively.

Ranking the Alternatives according to Benefit/Cost Ratio
From the figures now obtained a tentative ranking can be derived. It is tentative because it rests only on effects in terms of money, while qualitative aspects, which ought to be taken into account as well, might alter the ranking.
It will be remembered that 11 possible alternatives were distinguished (cf. section 4). These alternatives can be arranged as in the table below, which is based on the figures presented in the previous sections. In table 5, alternative 1 is compared to each of the other alternatives.

The figures in the bottom row, comparing the combination of two tunnels with that of a railroad tunnel and a road bridge raised on request, call for some comment. As benefit difference for road traffic, the amount found for 14 raisings has been filled in. The benefits to shipping have been put at zero, but will probably a bit of an underestimation.
From the benefit/cost ratios found, the alternatives may tentatively be ranked as follows (> = better than):
11 > 1 > 6 > 5 > 2 > 3 > 7 > 4 > 9 > 8 > 10
The ranking implies that from the effects computed so far, the combination railroad tunnel – road bridge raised on request, appears preferable to a combination of tunnels for both road and railroad traffic. For a road tunnel an additional investment of Dfl. 57.5 mln is required, against benefits to road traffic and shipyards of Dfl. 27.5 (at a minimum), the deficit being Dfl. 30 mln. It could be argued, however, that the risk of damage to the bridge due to navigational errors should be taken into account as well, a risk that corresponds with the costs of repair plus the loss in turnover suffered by the shipyards. The amount of Dfl. 30 mln could then be looked upon as a risk premium for undisturbed dispatch of all traffic for many years to come.

References
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Summary
Bridges, so utterly indispensable in a country so rich in rivers and canals as the Netherlands, yet have an unfortunate capacity for being in the way. Up, they obstruct land traffic, down, they obstruct navigation. When, as in Rotterdam, the river is a major shipping route and its crossings are vital links in important highways and railroad connections, while moreover the interests of shipyards upstream are at stake, the situation is rife with conflict, and the government has to tread wearily in changing regimes or making new investments.
Tunnels, of course, can bring relief, but they take a lot of time and money to construct.
In Rotterdam, the 100-year-old road bridges and the old-fashioned narrow railroad bridges are overdue for replacement. In making an investment decision, the government will have to take into account all the effects on the various sectors involved as well as the cost of the alternatives from which it can choose.
This paper presents the alternatives with their advantages and drawbacks, and explains how a cost/benefit analysis has provided the government with the data on which to base its investment decision.

Table 4
Shipyards' costs and benefits (in millions of Dutch guilders of 1972) depending on the government's investment decision

Shipyards' benefits and national-economic benefits			
River-spanning systems	in 1983	1983 up to 1992 inclusive	present value on January 1, 1973
tunnels		at least 161	at least 50.0
bridge raising on request		97	30.0
Shipyards' costs in relation to tunnels as national-economic costs			
14 bridge raisings		at least 161	at least 50.0
9 bridge raisings	45.1	at least 161	at least 69.4
6 bridge raisings	202.8	at least 161	at least 137.0

Table 5

alternative	cost difference (mln Dfl.)	benefit difference (mln Dfl.) for road	railroad	shipping	shipyards	$\frac{B}{C}$
(1)–(2)	56.5	7.5	0	8.3	50	1.14
(1)–(3)	57.5	6.2	0	5.3	69.4	1.41
(1)–(4)	57.5	0	0	21.2	137.0	2.75
(1)–(5)	91.4	7.5	34	8.3	50	1.09
(1)–(6)	91.4	6.2	12.7	5.3	69.4	1.02
(1)–(7)	91.4	0	0	21.2	137	1.73
(1)–(8)	29.3	0	> 34.0	8.3	50	3.15
(1)–(9)	29.3	0	> 12.7	5.3	69.4	2.98
(1)–(10)	29.3	0	> 0	21.2	137	5.40
(1)–(11)	57.5	7.5	0	0	20	0.48

Ir.A.Glerum
National Public Works Department,
Locks and Weirs Division

The Design of Immersed Tunnels

Introduction

Previous speakers have already dwelled upon the fact that the immersed tube method can be used for *all* kinds of sub-aqueous crossings. These include road tunnels, railway tunnels and metro tunnels, but also pipeline tunnels and tunnels to carry water and so on.

In the Netherlands, a delta region, fifteen of the 22 sub-aqueous crossings completed or under construction have been constructed using the immersed tube method. These are: 10 road-, 1 railway- and 1 metro tunnel, 2 pipeline tunnels and 1 tunnel to carry water.

However, the immersed tube method need not be restricted to the construction of river- and canal crossings. For instance, in Rotterdam it was found to be an attractive financial proposition to use the immersed tube method for the construction of a 1,600 metre-long metro tunnel on land. A temporary canal was dug for the purpose, protected by steel sheet piling.

I have too little time to talk about all the different types of tunnels, so I shall speak only about road tunnels.

Horizontal Alignment

A tunnel may also have to be curved in the horizontal plane if the location so demands. The dimensions of the horizontal radii have to correspond with those of the road, and in the tunnel the motorist has to have a sufficient view of the road ahead, in order to be able to pull up safely. In the Drecht Tunnel (the horizontal radius is 800 m) which was built by the immersed tube method, it was necessary to increase the lateral clearance at one side of the carriageway. If this had not been done the motorists' view would have been too restricted.

navigation channel, and with a shield-driven tunnel it would be about 20 metres below it. This difference of 12.5 metres means the tunnel would have to be no less than 555 metres longer (if the longitudal gradient is kept at 4.5%). With an average tunnel length in Holland of 1130 m (including the trough structures) this would mean an extra 49% in length.
If we take these factors into account, we find that a shield-driven tunnel, in spite of having a better shape for withstanding the water pressure, will be more expensive than an immersed tunnel. It only becomes competitive if the tunnel diameter is smaller. However, even a shield-driven tunnel with an inside diameter of 4 metres as a river crossing was found to be more expensive than an immersed tunnel likewise with a round cross-section.
In general the immersed tube method is economically the most attractive method, especially where there is sedimentary soil (sand, clay, silt etc). Another point in favour of the immersed tunnel is that during construction the water flow is not affected and shipping only very little so.

In this connection, the pressure head of the groundwater below the tunnel base may lag behind the water level in the river. At low tide this may result in an additional upward force. To compensate effects of this kind the design criterion is often adopted that at this stage the weight of the tunnel must exceed the water displacement by about 600 kgf per m² tunnel base, corresponding to an additional allowance of 7.5% for an external tunnel height of 8 m. The safety margin is later increased because the dredged trench in which the tunnel is immersed is filled in again. This results in the first place in friction on the walls but also in a load on the roof. Depending on the longitudinal section of the tunnel and the cross-section of the waterway the latter factor may be very slight in places or even disappear altogether due to erosion of the river bed.
Assuming, in cross-sectional terms, that the area of the required 'hollow' space is H m², that of the structural concrete S m² and that of the ballast concrete B m², we arrive at the following two equations *(fig. 5)*:
At the transport stage:
Weight = 0.99 maximum water displacement or

$$2.46\,S + 3.0 = 0.99\,(B + H + S) \quad \dots\dots (1)$$

In the final phase:
Weight = 1.075 water displacement, or

$$2.42\,S + 2.25\,B = 1.075\,(B + H + S) \quad \dots\dots (2)$$

N.B.
• A value of 2.46 has been taken as the specific weight of reinforced concrete in equation (1), and a value of 2.42 in equation (2). Experience in the Netherlands has shown that safe working limits can be obtained if it is assumed that the specific weight of concrete (without reinforcement) varies between 2.34 and 2.37 tf/m³. This dispersal range not only allows for variations in the specific weight of concrete (which generally fluctuates around a mean value of 2.355 tf/m³) but also for dimensional deviations in the concrete structure (eg. thickness of structural components) in relation to the theoretical dimensions shown on the drawings. The high specific weight has been taken at the floating stage and the lower weight in the final phase. The calculation also allows for 120 kg structural and reinforcing steel per cubic metre of concrete.

• The surface area of the ballast concrete has been multiplied by a specific weight of 2.25 (the compaction of ballast concrete is generally less good than that of structural concrete).
• A weight of 3.0 tf/m[1] has been used in equation (1) for the immersion equipment and bulkheads.
• In the equations it has been assumed that the specific weight of water is 1.0. This must be checked in each individual case. In a tidal river it is quite likely that a value of 1.0 must be taken for equation (1) and a higher value for equation (2) as a result of the salt strip penetrating along the river bed.
• A number of mean values have been used in the equations. This means that in specific cases more accurate data must be used, e.g. for the quantity of reinforcing and structural steel, the dimensions of recesses, the shape of the unit, the immersion equipment, bulkheads etc. Furthermore, specific weights have been used that relate to concrete manufactured in the Netherlands.
Solution of the equations gives the following results (in m²):

$S = 0.728\,H - 1.154$ ....(3)

$B = 0.081\,H + 1.319$ ....(4)

Solution (3) is particularly important because it shows the relationship between the area available for constructional purposes and the 'hollow' space which must be taken as the starting point for the design. The area H is the sum of the required traffic clearance profile and the areas needed for equipment and ventilation as described earlier. This also includes the area of the road surface, about 0.07 m thick which is to be applied later.
The structure must be strong enough in the final phase to withstand the loads due to water and ground pressure and its own weight, taking into account the stresses caused by temperature changes. In the case of tunnels built up to now in the Netherlands, it has been found that a reinforced concrete structure of sufficient strength can be built with the area calculated from (3). The deepest units of the 2 x 2 lane Benelux Tunnel were an exception to this rule; here allowance had to be made for a pressure of almost 21 m of water on the roof. In this case the floor and roof sections were partially prestressed transversely.
If a greater area is required than suggested by equation (3) owing to the water pressure on the tunnel or the width of the traffic tubes, a choice must be made between the following solutions:

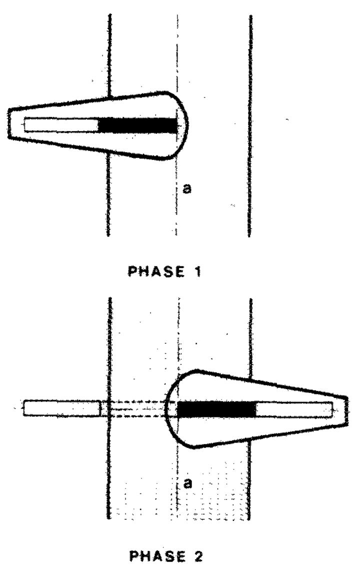
a. Full or partial transverse prestressing of the tunnel; in this way a lower structural height than with reinforced concrete will be sufficient;
b. Use of lightweight concrete for the entire cross-section or parts thereof (e.g. roof and floor sections);
c. Increase in the hollow space H.

The final choice must of course be made on the basis of an economic study.

Rectangular or Circular Cross-section?

In the above I have worked on the basis of a rectangular cross-section. But is this correct? Older than the rectangular reinforced concrete design first used for the Maas Tunnel in Rotterdam, which was opened in 1942, is the circular design developed in the USA, consisting (working outwards) of an inner shell of reinforced concrete, a cylindrical tube of thin steel plate (e.g. 8 mm thick), an outer jacket of reinforced concrete and an outer membrane of thin steel plate (e.g. 6.3 mm thick) usually almost octagonal in shape. The steel structure is built on a slipway.
The concrete is applied in the consecutive stages of construction. The steel structure acts as waterproofing and as formwork for the concrete. As with the shield-driven tunnel, owing to the shape there is a relatively large amount of unused space, which can of course be used for air ducts if a transverse or semi-transverse ventilation system is included, but very little of which is required for modern longitudinal ventilation with booster fans.
It will be clear from the above that the extra empty space will have to be compensated for by extra weight (in the shape of concrete) to prevent the tunnel from floating in the final stage.
A cost estimate based on Dutch prices for a two-lane road tunnel (one bi-directional tube) has shown that a circular cross-section is 40% more expensive than a rectangular one, taking the 'hollow' space as shown in figure 4 and in the left tube of figure 1 and an immersed tunnel length of 600 m.

3
Tunnel built in-situ
a = waterway



In addition to these extra costs we should note those caused by the lower position of the road; owing to the circular shape it will be 1.1 m lower than if a rectangular cross-section were used. With a gradient of 4.5% this means that the tunnel (in particular the open trough structures) will be about 49 metres longer.
The difference will be greater if it is a 2 x 2 lane tunnel (the entire cross-section of fig. 1) and considerably greater with a 2 x 3 lane tunnel (the type commonly being built on Dutch motorways).
The entirely reinforced concrete design has also been found advantageous for smaller cross-sections: for a relatively small internal diameter of 4 m a reinforced concrete tube was found to be cheaper than a steel tube weighted with concrete. This was the case with two pipeline tunnels built under the Hollands Diep and the Oude Maas (1770 and 566 m long respectively).

The Length of the Immersed Part
It has already been pointed out that the closed part of the tunnel joins up with a trough structure somewhere on the bank and that the location of the join is generally determined by economic considerations unless it is necessary to continue the closed structure in order to simplify the way roads or railways can cross the tunnel. The trough structure is constructed together with a short closed section (min. 15-20 m long) in an open excavation. The closed section is needed so that the immersed section has something to join on to. In all, then, we have an immersed part and a closed part with a trough structure on each bank. Where then does the immersed tunnel join the closed part built in situ? It is difficult to give a general rule, but it is often worthwhile to commence the open excavation immediately behind the dike. The location of the closed part built in situ is then determined by the gradient the slope *(see broken line in fig. 2)* can have without endangering the stability of the dike. If the join between the immersed part and the runnel built in situ were to be closer tot the river, the open excavation would have to be strengthened with steel sheet piling walls at the end, which is often expensive. Once the location of the parts built in situ is decided, the length of the immersed tunnel is also known. All this means, however, that the dredged trench into which the units are sunk has to be continued beyond the dike, which therefore has to be temporarily removed, and so another temporary dike has to be constructed round the end of the trench. It may be worthwhile to continue the immersed part further inland, for instance if a well-point system were to cause damage to buildings. A well-point system is not needed for the immersed part (the dock is generally located elsewhere) but one is needed for the open excavation. If the immersed part is extended the excavation can be made shorter and shallower, which means that the well-point system can have a much smaller capacity. For this and other reasons the immersed part of the Hem Railway Tunnel was continued well beyond the dike.

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Summary
The immersed tube method can be applied to all kinds of sub-aqueous crossings (railways, subways, pipelines etc.) but by way of example the paper will deal with road tunnels. The main aspects are: the influences of the traffic and the navigation on the geometry of the tunnel, the choice of the construction method (built in-situ or immersed tube method) and the outlines of the structural design.
The influence of the floating transport on the dimensions of the cross-section will also be reviewed.

Ir.Ch.J.Vos
HBM Hollandsche Beton Maatschappij

General Outlines of the Construction of Immersed Tunnels

Immersion as a Construction Method

Considering the immersed tunnel concept purely as a construction method, two major aspects are decisive for the attractivity to the constructor; it is the programme and the work that has to be implemented apart from the tunnel itself in its final shape, such as temporary works and special equipment.

The construction programme of an immersed tunnel is more or less an ideal one, because most of the main construction activities run parallel and contain a fair degree of repetition. A programme with construction activities running in parallel means that the job is relatively simple to control and is of low intricacy. There is in fact less chance that something getting out of hand would cause major spiral type escalations of repercussions in other parts of the construction programme. Repetition in construction activities offers the facility of making full use of the most advantageous learning curve as regards efficiency.

It further allows the acceleration of certain jobs on the critical path when there is the threat of the original planning of such a job proving to be wrong and possibly leading to escalation of the schedule.

Figure 1 illustrates this paragraph. The dry dock construction activities and both approaches constructed in situ show the main streams of construction jobs. The repetition in the segmental construction of the tunnel in the graving dock and the sequential construction of walls, bottom and foundation of the approaches may be clear.

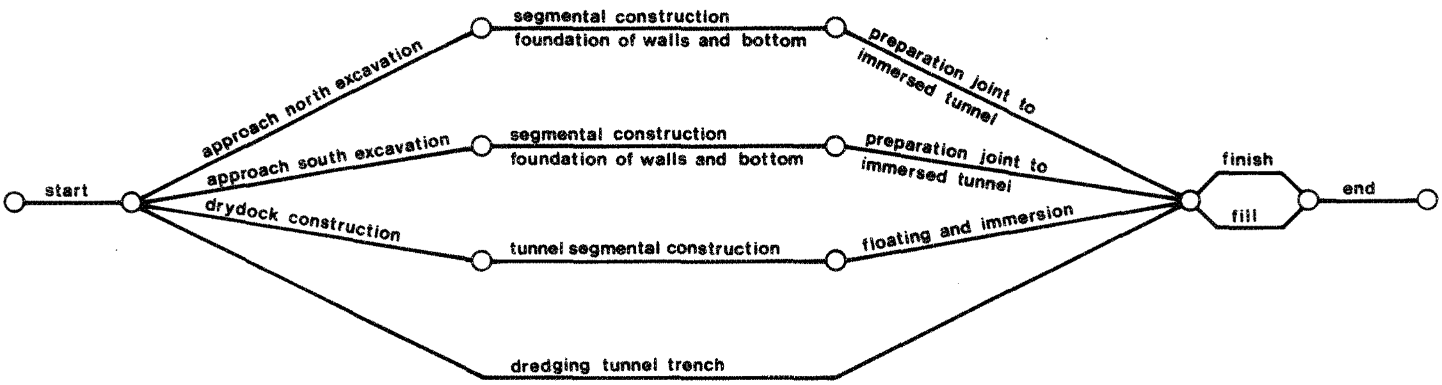
Temporary works, special equipment and all other cost types affecting aspects that do not serve the tunnel in its final function are influencing the total costs of the construction method. Next to the programme it is the conclusive aspect for the selection of a construction method.

The immersed tunnels as constructed in the Netherlands do not use many materials whereas overdimension is not required for their structural function, i.e. to resist hydrostatic pressure and other loads they have to carry. This is e.g. different from a tunnel constructed of pneumatic caissons where the whole pneumatic chamber is in fact subsidiary work needed for construction only.

Another example is e.g. the shield driven tunnel. The final structural hull remaining in the ground shows the expensive traces of the construction, with small segments bolted together, to ensure watertightness resulting in far more space than is functionally required because of the circular shape dictated by the shield.

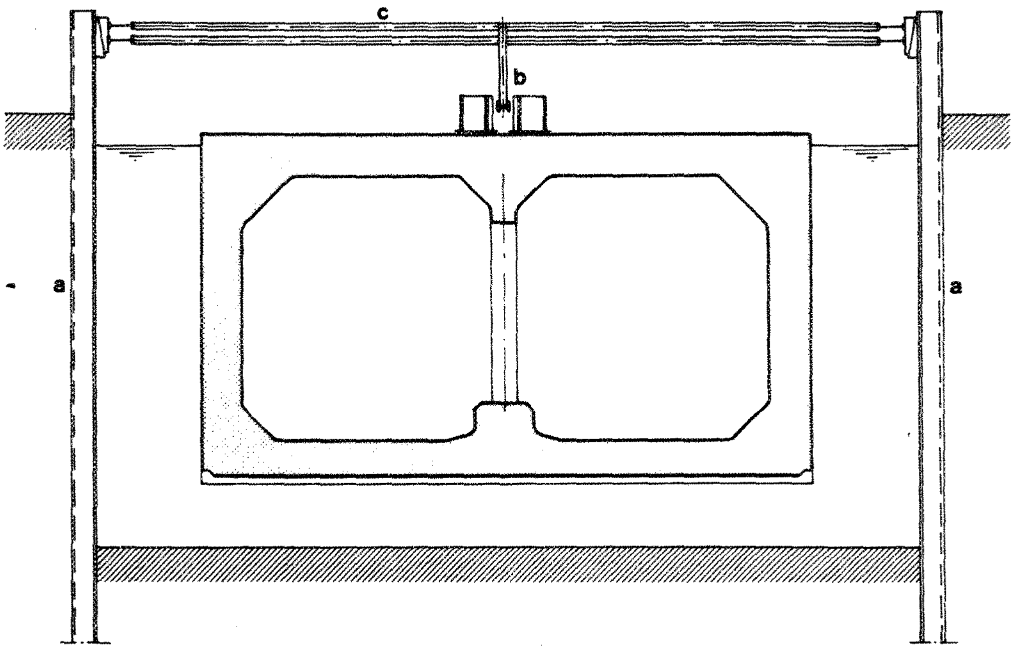
The immersed tunnel only requires a graving dock for which existing facilities can sometimes be used as a major cost-affecting facility. In most cases in the Netherlands graving docks are used for the construction of several tunnels. The Heinenoord dock, originally constructed for the Heinenoord tunnel in 1965 has since then been used for four other tunnels. The remaining special investments for the construction of an immersed tunnel are comparitively low to the rest of the costs.

They consist of the temporary bulkheads, temporary access shafts, water ballast tanks including pumps and the immersion equipment such as the flotation pontoons, winches and guide- and controle towers. Part of these provisions can be used for further tunnels and may be composed of standard equipment and materials.



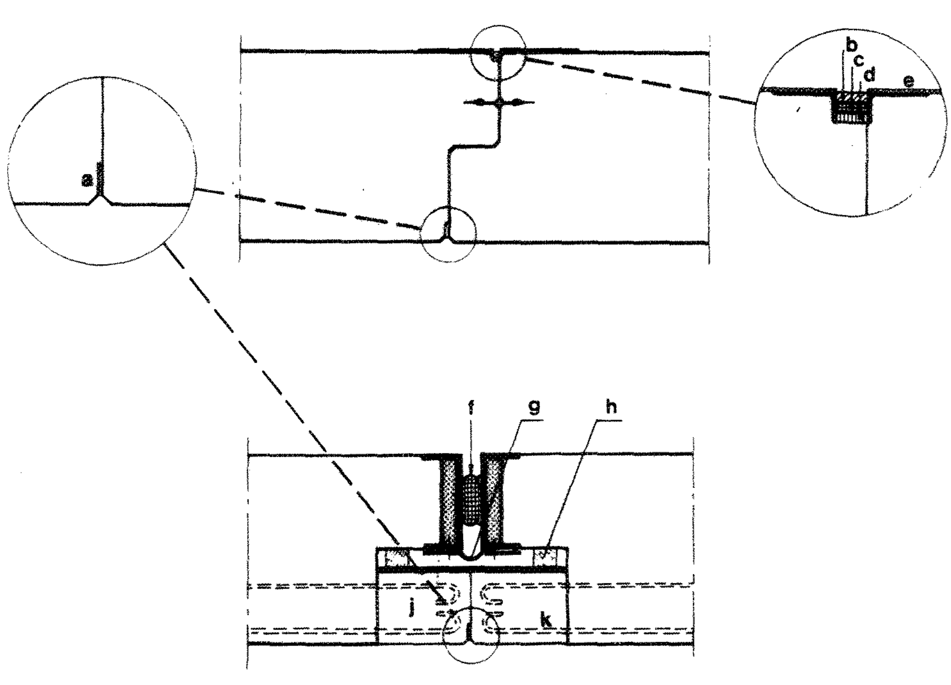
1
Main construction streams for immersed tunnel

2
Rotterdam subway construction; tunnel unit floating in sheetpile strengthered trench
a. sheetpiles; b. guiderail; c. struts

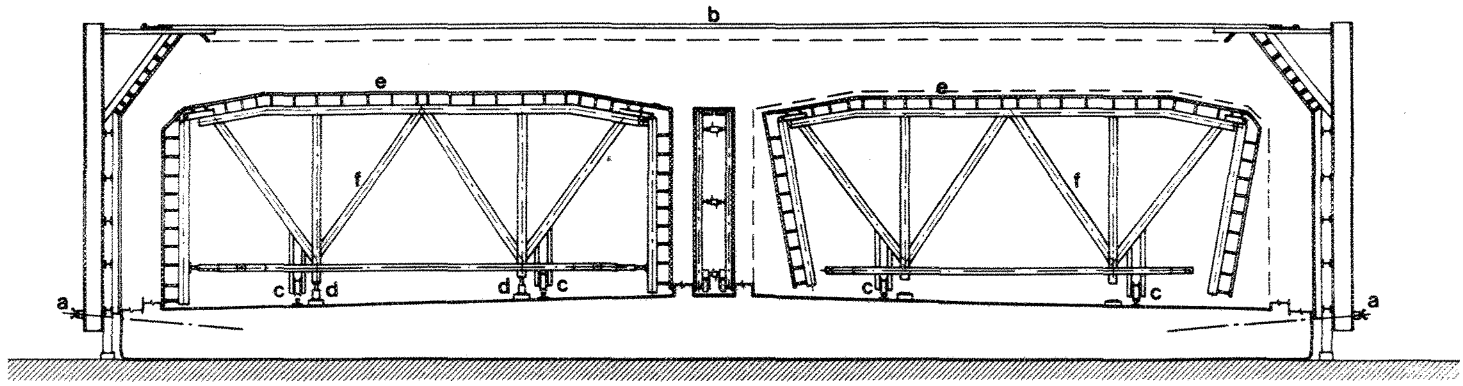


All this makes the immersion concept for the construction of a tunnel an ideal one. Especially for longer tunnels and for areas where more underwater tunnels have to be constructed one could not think a cheaper, less intricate or faster way of construction.
For this reason the method is not restricted to sub-aqueous crossings but can economically be used on land as for instance in the case of the first Rotterdam subway line, where a canal in between sheet piles was dredged first for transport and subsequent immersion of the tunnel units *(fig. 2).*

The Dry Dock Construction Activities
Most immersed tunnels in the Netherlands have been constructed in a graving dock, where all units for the tunnel under construction could be constructed simultaneously. In the case of comparitively long tunnels, such as for the Rotterdam subway and the Amsterdam IJ-tunnel, specially constructed dry docks with doors were used offering space for the simultaneous construction of one or two units only.
Most aspects of the construction of the graving dock constitute a straight-forward engineering job. The bottom construction however needs some attention, as it has to allow for equal distribution of the water pressure to develop underneath the tunnel unit bottom so as to allow controlled floating up of the unit to take place.
Several methods have been used. For tunnels with a permanent watertight skin, a gravel bed was used on top of which the watertight bottom skin was being assembled from 6 mm thick steelplate panels. Where no waterproof skin was applied, use has been made of a 150 cm thick 'no-fines' concrete layer on top of a gravel bed to provide the required stiffness of the temporary foundation; also a 0.3 mm thick steel plate was used to separate the bottom concrete which was to be cast on the

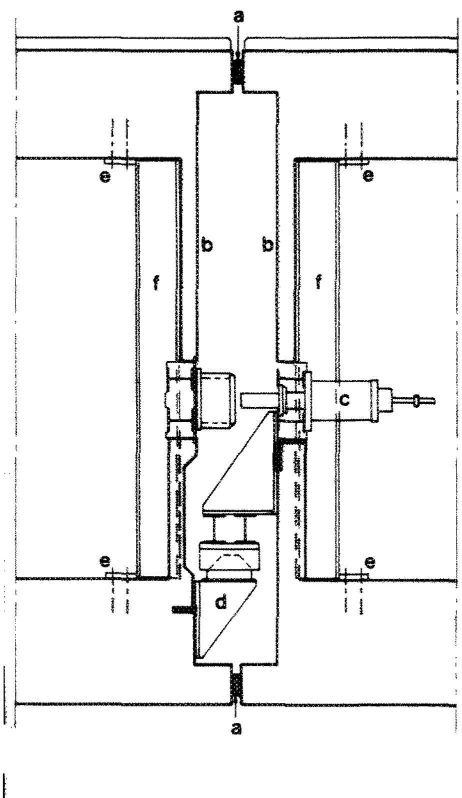


3
Walljoint between two tunnel segments and between two tunnel units at Vlake Tunnel
a. airex 1 x 8 cm; b. polyurethane mastic; c. polyethylene foam; d. asbestos cement; e. buthyl rubber; f. rubber sealing strip (Gina); g. permanent rubber seal; f. wooden shuttering; j. concrete fill; k. reinforcement



4
Formwork for Vlake immersed tunnel roof and walls
a. anchor in floor; b. anchoring wall shuttering; c. wheel with rail; d. jacks; e. shuttering of board; f. steel frames with peri-supports

5
Tunnel bulkhead
a. rubber sealing (Gina); b. temporary concrete wall; c. pull-in construction; d. bracket; e. supports; f. support beams



dry dock bottom. Although no serious difficulties were ever encountered in the Netherlands during float-up of tunnel units it is always assumed that at least 100 kg/sq.m adhesion may develop. This may be due to silting up of the gravel bed for instance.

Most of the immersed tunnels in the Netherlands have no pile foundations. This neccesitates a flexibility in the longitudinal direction to allow for uneven settlement, because otherwise very high stresses would develop. This flexibility is created by waterproof hinged joints every 15 to 30 m *(fig. 3)*. A number of such segments are assembled with prestressing cables to one floating unit of usually over 100 m length. After immersion the prestressing cables are released. The length of the segments is structurally limited to about 40 to 50 m for reasons explained above. The construction programme however, the practicability of concrete pours of a certain size and the construction capacity, costs and handling of the formwork to be used in sequence for the walls and roof dictate a far shorter segment of about 15 to 20 m length *(fig. 4)*.

The construction of the tunnel units does distinguish itself from conventional work by the required accuracy of the dimensions and the attention to be paid to the watertightness.

The small tolerances in dimensions are required to assure freeboard and to minimize trimming. The bottom slab is usually poured first. Walls and roof are cast later simultaneous. The formwork used is tunnel shaped and has a timber face on a very stiff steel support frame.

The formwork can be skidded after jack-controlled releasing of the timberface from the poured concrete across the pre-poured tunnel bottom to the next segment wall and roof to be poured. Measures to be taken in order to obtain watertightness with or without an extra watertight skin will be explained in Mr. Jansen's paper.

Before the flooding of the dry dock, watertight bulkheads are installed *(fig. 5)*. Only in a few cases steel bulkheads are used. The same applies to the multiple use of bulkheads; this was the case where one drydock was used several times for tunnel construction, consequently allowing the installation of used bulkheads on newly constructed units. The most common bulkhead type used is a composite structure of a 15-20 cm thick reinforced concrete wall supported by steel girders.

Before the flooding of the drydock the remaining equipment required for controlled immersion, such as the access shafts, ballast tanks, provisions for the temporary foundations and of course the bollards and other equipment for the marine operations are installed. The ballast tanks are then loaded to prevent early float-up during the flooding of the drydock.

The Marine Chapter of the Construction

Simultaneously with the construction of the tunnel units and the approaches, a trench has been dredged under the original channel mudline. Especially in deep access channels for modern ports this causes problems as the usual high-production standard dredging equipment is of course designed to dredge to draft levels of, say, 60-70 feet only. During the Eighth World Dredging Conference a lecture will deal with the special techniques being developped for this type of dredging work and also with the problem of early silting up of such trenches in a river bed.

Tests were performed in the Delft Hydraulics Laboratory on this silting up problem, especially where in tidal sequence salt- and fresh water passes the trench. The tunnel units, consisting of a certain number of segments are floated up in the drydock by de-ballasting. The total length and consequently the number of segments in one such unit chosen is as high as possible. This is governed by the drydock dimensions and the local possibilities of manoeuvring the units from the drydock to the installation site.

It may be clear that a shorter length, and consequently more segments involve more costs, because of the additional number of bulkheads and other immersion provisions as well as additional time, because of more immersion operations. After float-up a tunnel is usually moored to a jetty where it is trimmed and equiped with immersion towers, pontoons and other repeatedly used equipment.

The papers by Mr. van Loenen and Mr. Molenaar will deal with the specialities of towing and immersion; therefore only a rough sketch of these operations will follow here.

Figure 6 illustrates a tunnel unit ready for immersion. The towers and immersion pontoons are installed on top of the unit after they have been used for the previously immersed unit. This operation governs the time involved in between two immersion operations, which can be reduced to one less than a week.

From this position the unit to be placed is towed by tugs to the site of immersion. Here control is taken over by a winch-controlled wires. These winches are usually placed and controlled in one of the towers on the unit. After immersions position control is taken over by jacks in the tunnel base. These

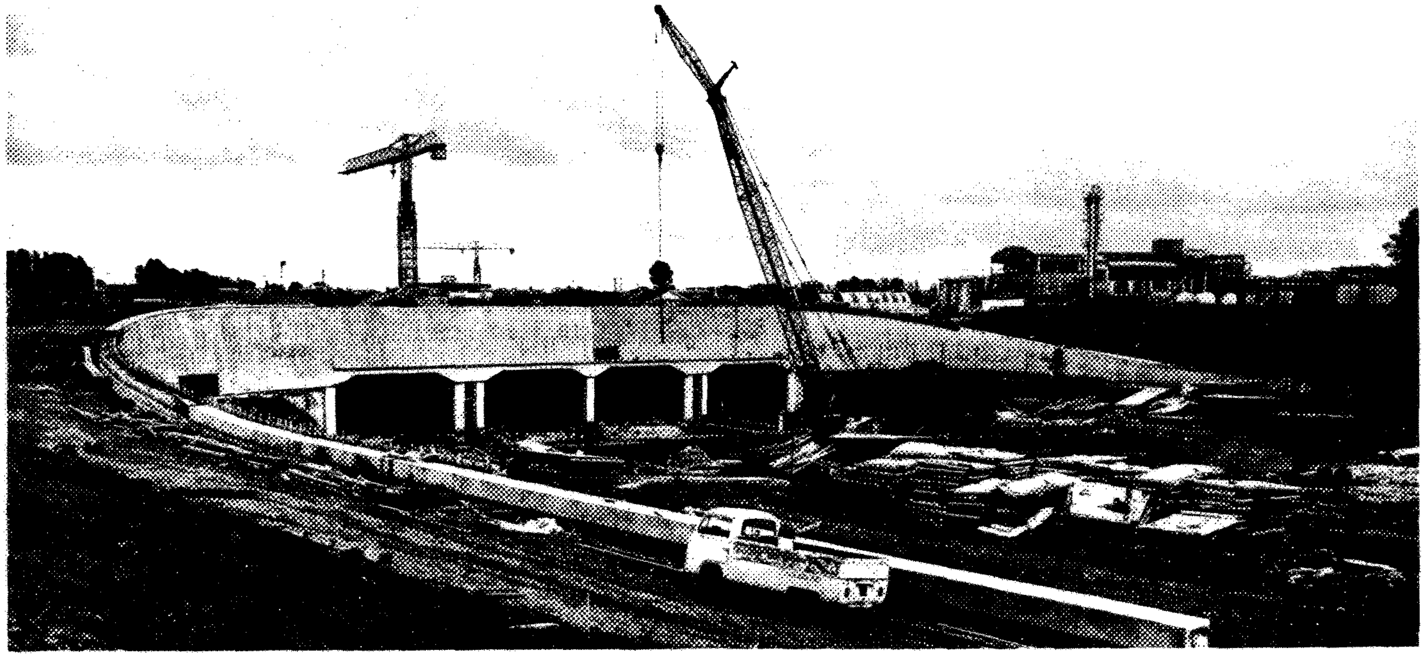
jacks can control the distance relative to an L-shaped temporary support plate on both sides of the tunnel unit front. These L-shaped support plates are usually suspended from the tunnel as shown in figure 6. On the rear side, where a previous unit has been installed, the new unit is supported by a bearing. For these three steps of control which call for ever greater handling accuracy, tugs, winches and jacks are essential.
The actual immersion usually takes place by the intake of ballast into tanks in the unit, when the immersion pontoons are fixed with by tensioned wires on the tunnel roof. After the tunnel roof has been fully immersed and the pontoons have reached more than their own draft, the immersion winches on the pontoons are released for further immersion of the unit.
This operation is continued until the unit can be supported and controlled by the above-mentioned system of bearings and jacks.
The mating with the previously installed unit is a very simple one. The newly installed unit is pulled by jacks towards the previous one. The mating surface consists of a soft rubber nose on a solid rubber strip to function as a gasket all round the perimeter of the tunnel. The space created by both bulkheads and the cantilevering walls of the tunnel, imparted a watertight connection by the rubber strip known as GINA-profile, so called by their inventors and manufacturers 'Vredestein', is subsequently drained. This causes the pressure to be atmospheric on the rear bulkhead of the newly installed unit, where the full waterpressure is still acting upon the front bulkhead. The difference in force increases the pressure on the rubber strip so far that a safe watertight joint is created in between the two units.
Now the finishing of the joints, the permanent foundation and the backfill of tunnel trench can start. Mr. van Tongeren's paper will deal in detail with the foundation aspects.

Conclusion

You may get the impression that these papers about design and construction of immersed tube tunnels are presenting about everything that can be said about it. One should however realize that in engineering about only 1 % of the problems can be detected behind your desk, 10% of the snags may be encountered during detailed design behind the drawing board and in the laboratory, but the remaining 89% of possible questions that have to be solved are encountered and overcome in the practice of construction.
Only experience counts here; here is the area where engineering really proves to be an Art. An Art, as we in the Netherlands know, because we had the opportunity of constructing so many immersed tunnels ourselves.

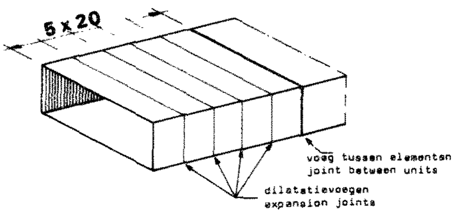
Summary

This paper will deal in general with those aspects of immersed tube tunnel construction that distinguish this method from conventional civil engineering construction. The construction of the tunnel units in the dry dock, with the specific details to obtain watertightness and the provisions for floating transport (bulkheads), will be emphasized.



Ir. W. Janssen
OGEM Building Division

Waterproofing of the Tunnel Structure



1
Scheme of tunnel

Introduction

A tunnel to be built by the 'immersed tunnelling' method is made up of 'sub assemblies': the immersion units, with a length of 70-120 metres. These units each consist of sections, approx. 20 metres long, between which expansion joints have been designed *(fig. 1)*.
The use of these joints prevents the generation of large tensile stresses due to shrinkage after concreting and uneven settlements in the subsoil after immersion. They allow for minor rotations, due to settlements, and a small longitudinal displacement, due to shrinkage.

A tunnel must be watertight. This implies that no water may penetrate through the concrete, the working- and expansion joints, or the seals between the tunnel units. The impermeability of the concrete is so very important because corrosion of the reinforcement must be prevented.
In this article we will discuss in brief how the impermeability of concrete and joints can be ensured.

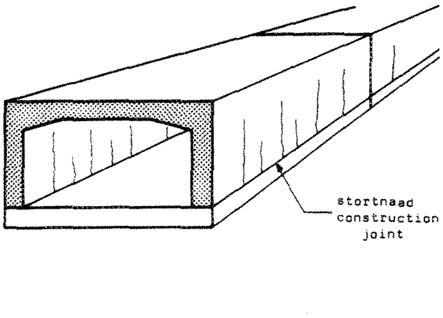
IMPERMEABILITY OF THE CONCRETE

Causes of Cracking

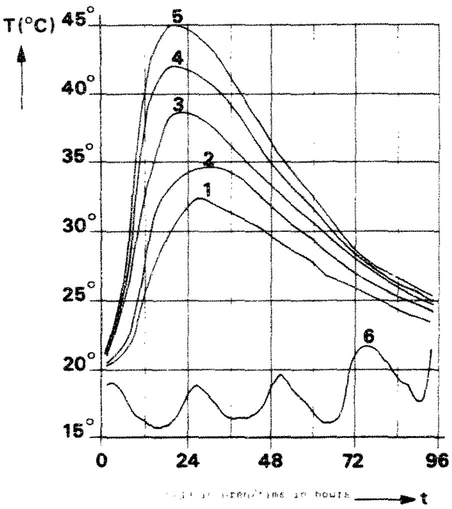
The most important cause of cracking within the concrete is uneven shrinking.
The shrinkage can originate from:
a. Drying-out
When the tunnel units are still in the dry dock, micro-cracks may appear on the outside caused by drying-out of the concrete. After the immersion and placing of the units these cracks will be closed again by the swelling of the concrete in water.
b. Climatic changes in temperature
During a change in ambient temperature, a temperature gradient will occur in the walls, the roof and the floor. In general this gradient is too low to cause large stresses.
c. Temperature changes due to hydration of cement
Hydration heat is the heat that evolves from chemical reactions which take place during the setting of newly-poured concrete. As a result of this process, a temperature increase occurs in the core of a heavy concrete structure. In relatively slender structures the heat dissipation to the environment is so extensive that a temperature increase scarcely develops at all.

During the construction of a tunnel, the hydration heat evolved can cause two kinds of problems:

- A higher temperature will arise in the core of the floor, the walls and the roof than in the outer layers of these parts. As a result of the temperature gradient, the cooling down of the outer layers will cause the concrete on the outside to shrink at a higher rate than the concrete in the core. Therefore tensile stresses will occur, with the danger of cracking.
When the formwork is removed from a concrete wall while it still has a high temperature a thermal shock will result. Especially during the night, the surface temperature of the concrete can decrease considerably in such cases. From calculations it follows that in concrete structures such as tunnels, a temperature difference of 15-20 °C causes cracking. This has been confirmed by practical experience. As the stress zone expands during cooling down, a typical crack will progress further inward. When the temperature in the core decreases, the crack will close.
- During the pouring of the walls upon the floor, difficulties arise because the fresh concrete will not behave in the same way as the older concrete of the floor. The deformation of the new mass of concrete is interfered with by the floor. In the wall, hydration will increase the temperature. The heat dissipation to the floor is only limited and the floor temperature will lag behind. At first the wall can freely expand but in the process of hardening of the concrete, the cohesion with the floor will come into play. With a further increase of the temperature in the wall by hydration, compressive stresses will occur in the wall and tensile stresses in the floor. During cooling down, the opposite takes place: compressive stresses will arise in the floor and tensile stresses in the wall and the latter will begin to crack *(fig. 2)*. In this case the stress will occur at a later stage than in the previous example of the interaction between the core and the outer layer, because the whole concrete wall must cool down first. In the meantime, the tensile strength has increased. At the ends of the section wall the stress is nil; but it increases in excess of the ultimate strength and there cracking sets in. This process repeats itself lengthwise.



2
Cracks in the tunnel walls caused by temperature influences



3
Hydration heat of different types of cement
1 = blast furnace cement A – 325 kg
2 = blast furnace cement B – 325 kg
3 = portland cement A – 325 kg
4 = portland cement B – 325 kg
5 = portland cement C – 325 kg
6 = temperature of environment

requirements with regard to strength and impermeability of concrete demand a minimum limit of the cement content. In many cases a quantity of 250-300 kg of cement per m³ of concrete is used.
c. The use of coarse aggregates
When using coarser aggregates, less fine grained material (cement etc.) can be used, if the particle size distribution is adequate. For a satisfactorily dense concrete around the reinforcing rods, a mortar with a limited maximum particle size is needed, while on the other hand, reasonable workability requires a certain amount of fine-grained material. Beside cement, stone meal, trass, glacial sand etc. can be used for this purpose.
d. Decreasing the amount of mixing water
A lower water/cement ratio results in a reduced amount of hydration heat per unit of time; however the workability will decrease. The additional use of a plastifier and air-entraining agents can improve the workability.

Decreasing differences in temperature
Various methods can be applied to reduce the temperature differences between the core and the outer layers of the walls, the floor and the roof and on both sides of the working joints.
a. Reducing initial temperature of mortar by cooling
Cooling the mortar has two effects:
- the maximum temperature in the mortar is decreased.
- the setting of the concrete takes more time, by which the evolution of heat is spread over a longer period of time.

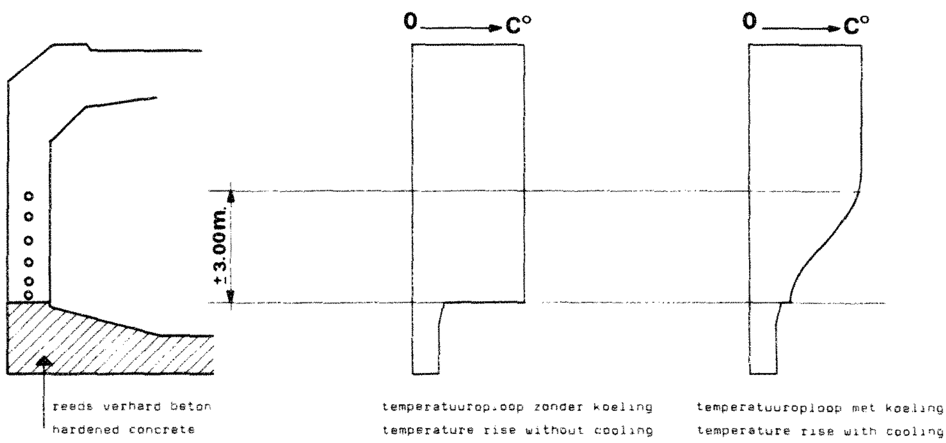
The temperature decrease can be attained by cooling the raw materials and by using ice water as mixing water. In tunnel construction this method is seldom used, since it calls for extensive installations.
b. Cooling the newly-poured concrete in the walls
This cooling method has proved to be succesful. Its purpose is: to obtain a gradual temperature plot between the relatively cold floor and the roof. (*fig. 4*). An automatic cooling system is used, by which cooling water is pumped through a concreted-in piping system. See flow scheme in figure 5. The water flows through the wall from the bottom to the top and is heated up during circulation. As a consequence, the cooling capacity of the water decreases as it flows upwards.
By means of thermocouples the temperatures of ambient air, inlet water, outlet water, concrete at various levels, concrete below working joints are measured. A recorder records the temperature plot.

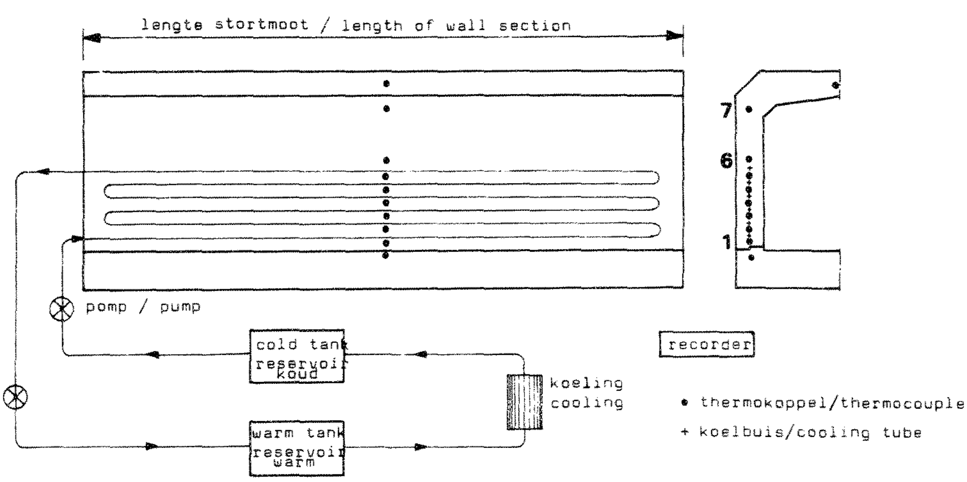
In the upper part of the wall and in the roof, a zone of compressive stresses may develop. As a result, the shrinkage cracks will not extend to the roof. Research has proved that the placing of longitudinal reinforcement near the floor cannot prevent cracking. In this way, however, the crack-width can be limited and the occurrence of cracks distributed more evenly along the length of the tunnel.

Measures against Cracking.
There are many remedies against cracking. Not one single method is effective enough, so that a combination of measures is necessary. The most important methods will be explained below. They refer to the composition of the concrete mixture, the decrease of the sharp rise in temperature difference between floor and walls, and special measures during construction.

Composition of concrete
The following measures are considered:
a. Selection of a suitable cement
The chemical composition of cement has a large influence on the hydration heat. Especially cements with a latent hydraulic binding agent, such as puzzolane, fly ash, fine, ground volcanic stone or slag (blast furnace slag) develop relatively little hydration heat. In figure 3 the hydration heat data of some cements widely used in Holland are compared.
b. Decreasing the cement content
A decrease of cement content is attended by a decrease of hydration heat. However,

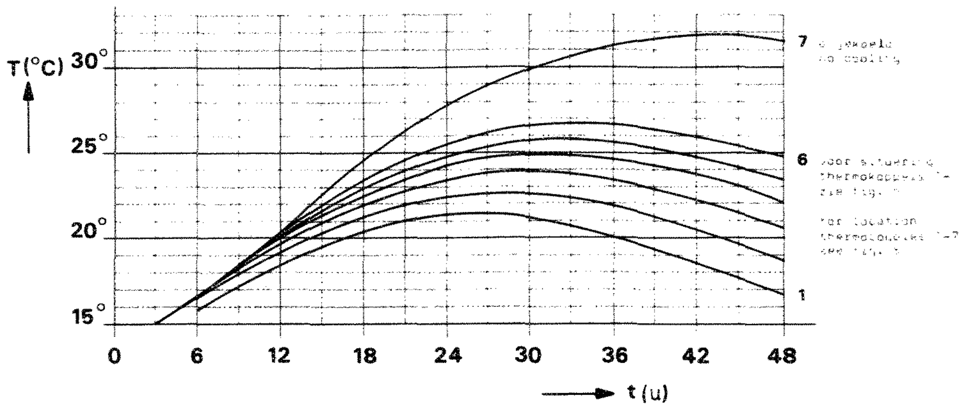
4
Temperature rise without and with cooling





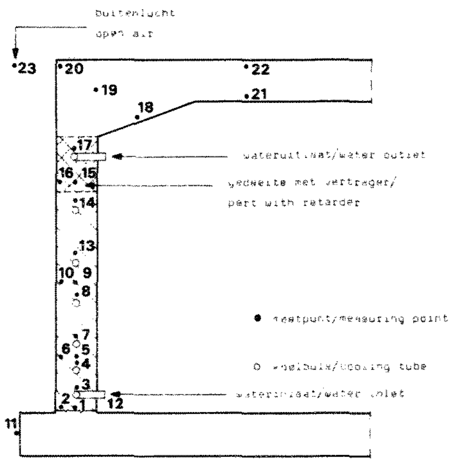
5
Cooling scheme

7
Temperature range in the concrete, with and without cooling



The degree of cooling depends on:
- diameter, length and spacing of cooling tubes;
- temperature difference between inlet water and concrete;
- flow capacity of water;
- the dimensions of the tunnel section;
- the properties of the concrete mix.

Especially the distance between the pipes is an important factor.
For the determination of the position of the cooling pipes and of the flow capacity, given a certain initial water temperature, usually the calculation method of W.Mandry is used. This calculation is based on numerous simplifications; marginal conditions such as heat insulation capacity of formwork, changes in ambient temperature, length of tunnel section, etc. are left aside.
The method provides average concrete temperatures without and with cooling *(fig. 7)*. As a consequence previous experience plays a very important role in every new project.



8
Cross section tunnel unit no. 1, location of cooling tubes and measuring points

When setting up the automatic cooling system, the following criteria apply:
- with an increasing concrete temperature, the cooling system remains in operation;
- should a temperature decrease occur, cooling continues;
- should the temperature decrease continue over a certain period of time (e.g. 5 hours), cooling is stopped;
- if after shut-down the temperature increases again, cooling is recommenced.

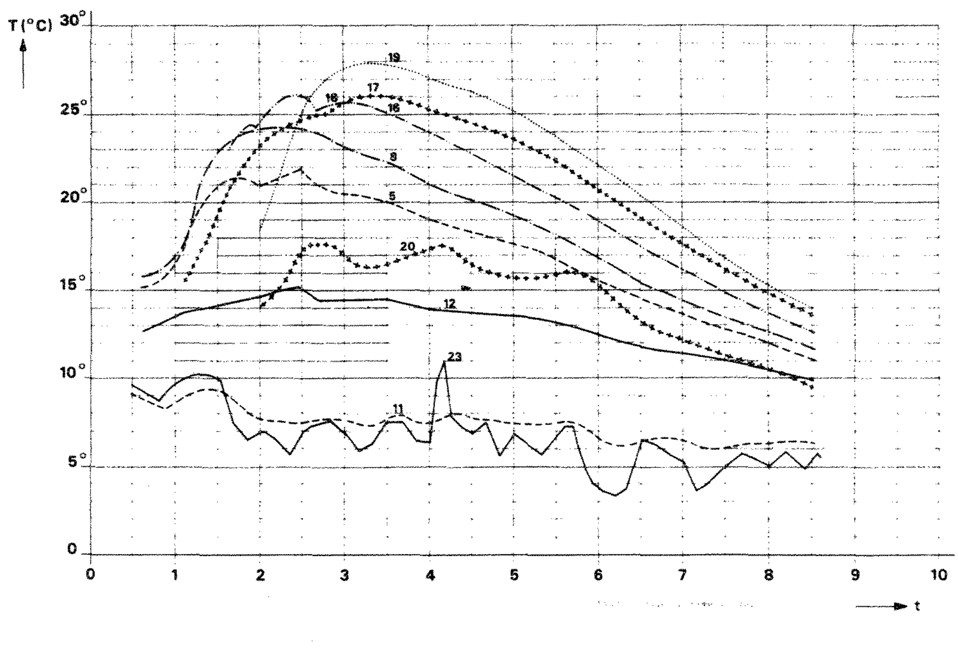
For every concrete pour the following preconditions are determined:
- the position of the measuring points;
- time delay of cooling start-up after switch-on of automatic system;
- discharge of cooling water;
- time delay between set-in of temperature decrease and cooling shut-down;
- the maximum cooling time (approx. 50-60 hours).

The correct operation of the equipment and the temperature plot within the concrete are checked at regular intervals. In illustration, the following figures show the cooling arrangement for No. 1 tunnel section of the Amsterdam Airport tunnel:
figure 8 – Sectional view of tunnel wall and position of cooling tubes and measuring points;
figure 9 – The recorded temperature plot.

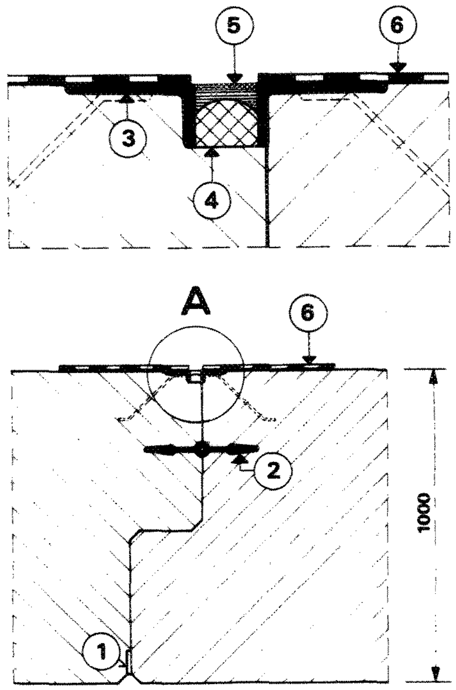
It can clearly be seen that the objective: a more gradual change of temperature, has been attained; it is also remarkable that fluctuations in ambient temperature have no influence on the concrete temperature. When no cooling is used, this effect does occur however.
The total costs of the cooling system are rather high, but they should be compared to the significantly higher costs of a waterproof outer lining. Moreover, steel cooling tubes can permanently serve as shrinkage reinforcement.

c. Heating of the floor
A decrease of the temperature difference between the floor and a wall can also be effected by heating the floor, e.g. by circulating hot water through concreted-in tubes. This method is seldom used. In practice, one example is known: the construction of the Underground Tunnel in Stockholm, in which case the longitudinal cable ducts for the prestressing rods were used to this end.

The method of heating the floor may be combined with the cooling of the wall method: the heated up water from the upper part of the wall is circulated through tubes in the floor. Thereafter, the water is cooled.



9
Recorded temperature plot, time in days (for location of measuring points see fig. 8)



10
Expansion joint with rubber/metal waterstop and polyurethane putty
1 = airex strip, 2 = rubber/metal waterstop, 3 = steel angle, 4 = polyethylene foam, 5 = polyurethane putty, 6 = glued butyl strip

Measures during construction
a. Postponing removal of formwork; insulating the top of the roof. By using formwork with a reasonable heat insulation capacity, the temperature difference between the core and the outer layer of a wall is decreased, while the temperature gradient becomes less abrupt. Since wood insulates better than steel, wooden formwork is prefered.
Removal of formwork must be postponed until the whole section has cooled down to an acceptable temperature. As a consequence, the formwork should not be too thick-walled. The newly-poured concrete of the roof may be covered with a layer of insulating material.
b. Continuous pouring
An elegant solution for the problems encountered in pouring fresh concrete onto older concrete is obtained by continuous pouring of the whole tunnel section. In general this can easily be done with comparatively small tunnels, but not with the large and wide tunnels for road traffic.
In Holland this method has been used in the construction of a siphon under the Amsterdam-Rhine Canal, a pipeline-tunnel through the Hollandsch Diep Estuary and the Rotterdam METRO-politan railway TUNNEL under the Nieuwe Maas River.
The siphon and the pipeline-tunnel were built up of sections with a length of 3-6 m, poured in upright position. The sections of the Metro-tunnel could be poured in one operation thanks to the compact spectacles-shaped profile and with the use of very ingeniously designed formwork.

Additional impervious layer
Until recently the methods described above were not considered to be adequate and every large tunnel was provided with an additional waterproof layer. Such a layer consists of several layers of fibreglass or another non-organic material, impregnated with asphaltic bitumen; the layer can bridge crack widths of approx. 0,2 mm. At the expansion joints, the layer has an extra reinforcement, while it is not pasted to the concrete over an area stretching for several centimeters at both sides of the joints. In this way longer lengths of this material can take up the expansion that occurs.
For the upper part of the walls and the roof, this bitumen waterproofing is protected against damage (e.g. falling anchors), by means of a reinforced concrete slab which is secured to the tunnel structure with watertight fasteners.
The covering of tunnels with an expensive additional impervious layer has recently gone out of use. Modern methods to prevent cracking, (mainly cooling of concrete) are so satisfactory that such a layer is no longer needed.

WATERTIGHTNESS OF JOINTS

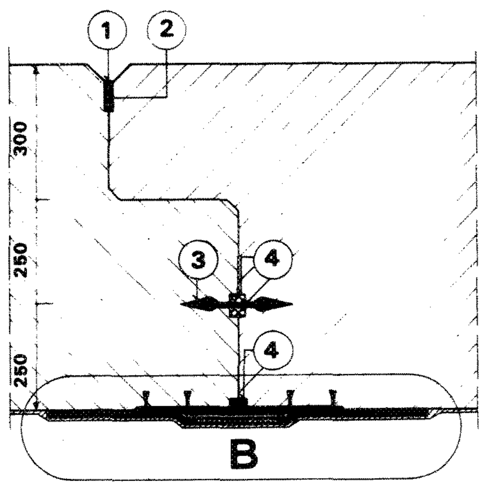
Working Joints

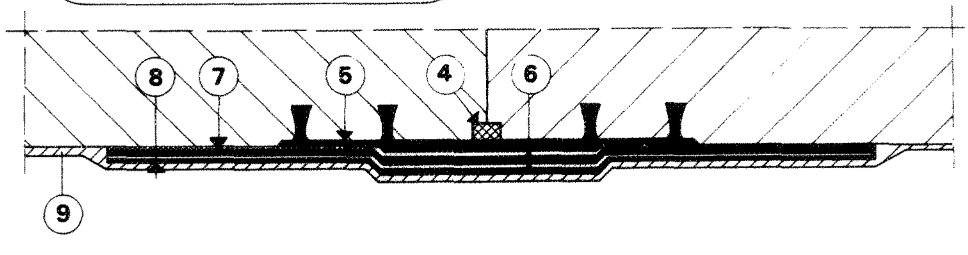
The working joint between a floor and a wall is a potential source of leakage; its construction calls for careful supervision. Mostly the joint is provided with an iron strip to ensure an impermeable bond.
Beside the normal measures to treat a working joint, such as rough chipping and grouting with cement mortar the shape of the joint is a matter to be studied carefully. The working joint may be flush with the top of the floor or extend by about 60 to 100 mm above it.

Expansion Joints

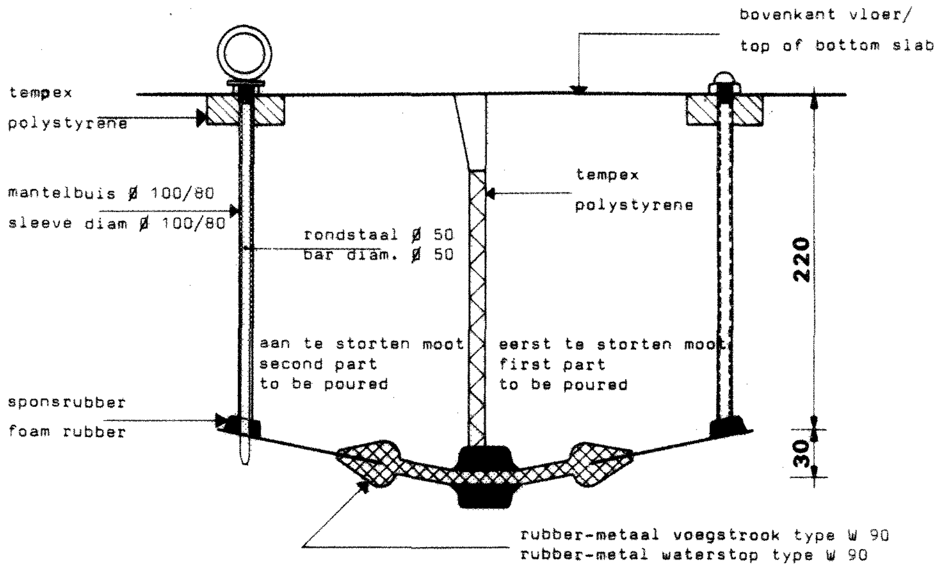
The sealing of expansion joints generally consists of two devices:
a. an inner joint, consisting of a rubber-metal joining strip *(fig. 10)*;
b. an outer joint covering consisting of polyurethane putty *(fig. 10)* or the so called 'Dubbeldam' rubber strip *(fig. 11)*.

11
Expansion joint with 'Dubbeldam' rubber-strip
1 = Thiokol, 2 = Airex, 3 = rubber/metal waterstop, 4 = foam rubber, 5 = Dubbeldam rubber-strip, 6 = heat resisting foam plastic, 7 = PVC sheet, 8 = Ruberoid vitrix, 9 = watertight cover



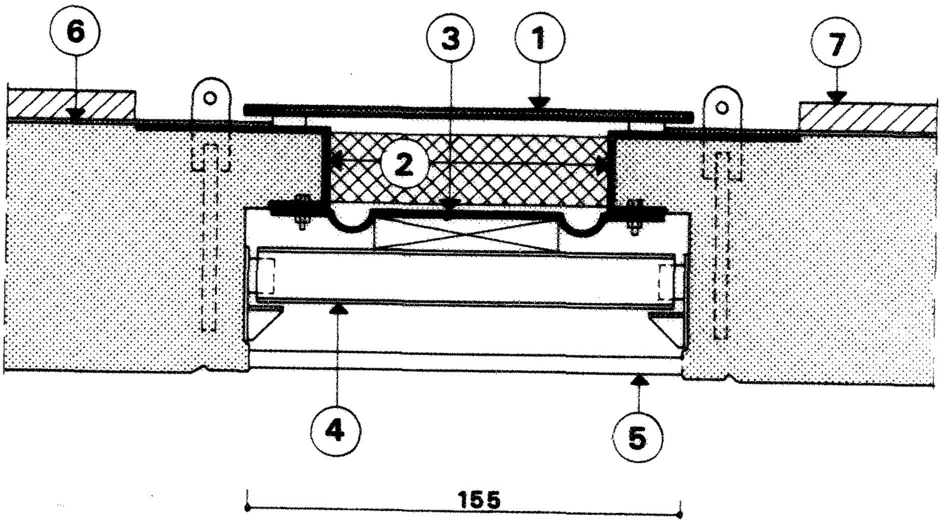


12
Detail injection tubes near rubber-metal waterstop



The second seal was used because the concrete around the rubber-metal joining strip often contained gravel pockets caused by seperation of the coarse aggregate from the concrete.
In the construction of the Hem railway-tunnel, an improved version of the rubber-metal joining strip has been applied *(fig. 12)*.
A foamrubber strip was bonded to the ends of the metal strips; through steel tubes, pressed against this strip, the concrete around the rubber-metal strips can be injected with epoxy resin. This method turned out to be so satisfactory that the outer sealing joint could be omitted.

14
Detail final joint in the wall
1 = primary sealing, 2 = folded steel sheets, 3 = permanent sealing (rubber), 4 = steel support, 5 = fireproof cover, 6 = watertight cover, 7 = concrete protection



Summary
This paper describes the measures to be taken to ensure the watertightness of a tunnel. These measures are:
- *manufacturing a dense concrete without cracks,*
- *reducing the effects of shrinkage and settlements by designing expansion joints requiring special double water stops,*
- *closing the joints between the tunnel units by applying specially designed rubber gaskets.*

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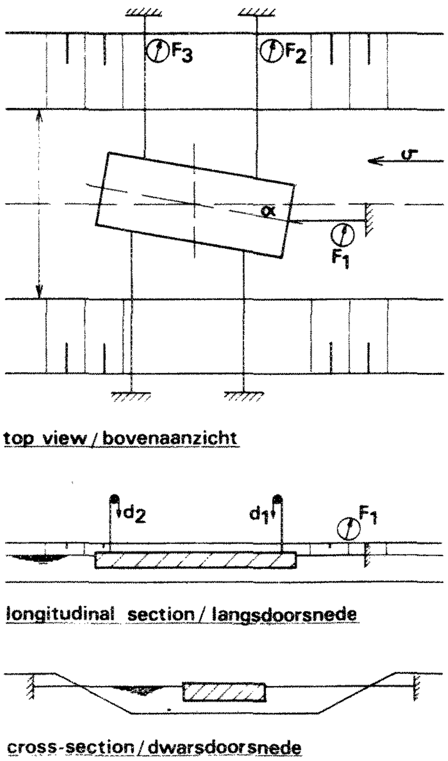
Ir.J. van Loenen
Stevin Group Holland

The Floating Transport of the Tunnel Units



1
Laboratory equipment for measuring of forces
F = force, d = displacement

Introduction

After the units have been cast in the building dock and prior to immersion at their final position, the units have to be transported.

Some aspects of this *transport* operation are:

a. testing of the behaviour of the tunnel element as a ship;
b. the towing forces;
c. the set-up of the towing operation;
d. the navigation;
e. the procedure of decision making during the operation.

Laboratory Tests

In the building dock the units are provided with temporary watertight bulkheads at both ends allowing them to float during transport.

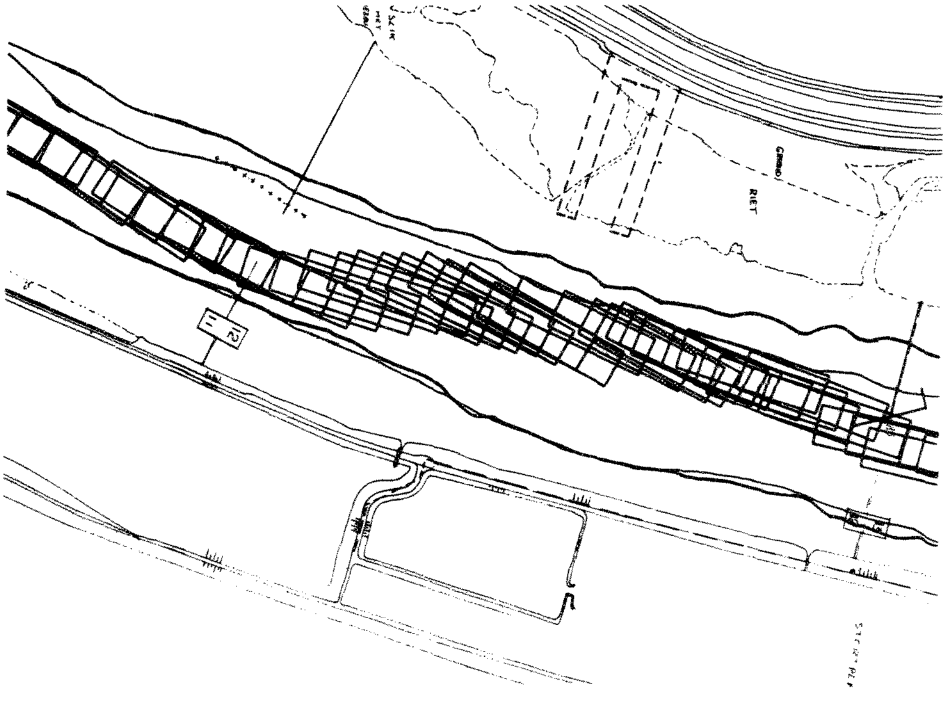
Other provisions for the submersion are already being installed during concreting of the units in the construction stage. These include facilities for temporary foundations, matching with other units and sandstreaming the units, access to units in various stages, systems for water-ballasting, communication, energy, surveying, anchoring, suspension, etc.

As soon as the construction of the units is finished the ballast tanks in the units are filled with water to prevent them from floating once the building dock is filled with water.

Before the tow-out of the units laboratory tests are made in order to determine the forces exerted on the units during the towing *(fig. 1)*.

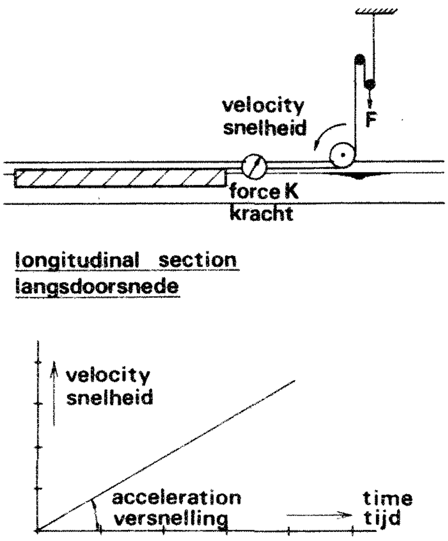
These forces are given by the formula $F = \frac{1}{2} A.r.V^2.R$.

A = the exposed section of the unit perpendicular to the current
r = specific volume of the water
V = relative velocity
R = friction coefficient



Part of a plotter chart

Laboratory tests

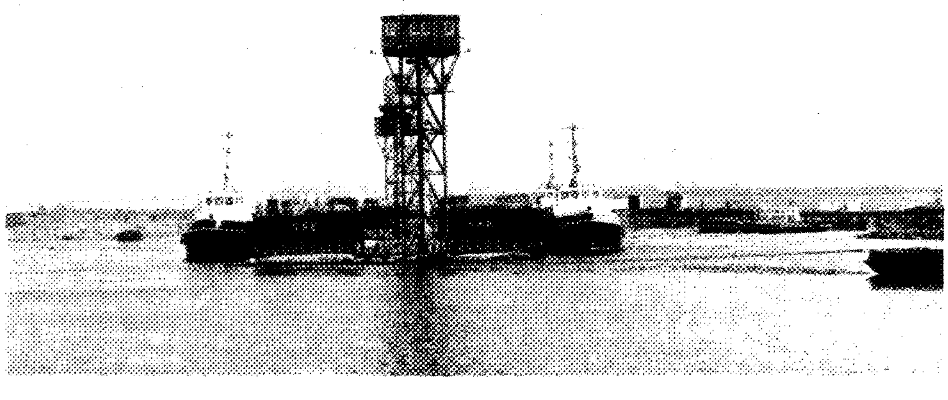


2
Measuring of additional mass: $K = m \cdot a$

The relative velocity is the difference between the speed of the unit and the velocity of the water with respect to the bank in an undisturbed cross section.
The friction coefficient depends on the shape of the unit. Apart from that, factors such as relation between draught and water depth and relation between the cross section of the flow have influence. This friction coefficient varies from about 1.5 under very favourable circumstances to about 3 or 4 normally and becomes yet higher under unfavourable conditions.
Special attention has to be given to special circumstances during the tow-out such as conditions at the building dock, tributary rivers, bends and the anchoring at the immersion trench. These situations will in general determine the minimum power required during the towing operation. In these circumstances the flow is perpendicular to the unit axis. The friction coefficient then increases and can reach a value of 6 or higher.
The coefficient can be so high that some activities may have to be planned at special moments with respect to the tide and this may govern the whole time schedule of the towing operation.
A model study will be made in a hydraulic laboratory including the immersion trench, tributary rivers and other difficult stretches of the transport channel. In the laboratory tests the stream velocity, water level and unit position will be subject to variation.
Furthermore it is possible to study the influence of changes in shape of some of the cross sections e.g. by dredging narrow passages or shaping sharp bends. The friction coefficient is also determined in the model. The result can also be used for identical circumstances existing along the transport channel. The model tests allow the study of how to manipulate the units along the difficult spots. Currents can have unexpected effects. In that case the way of manipulating has to be determined by tests. This implies that constraints have to be imposed regarding speed or position of the unit and the moment of passing dif-ficult spots with respect to the tidal flows. In general the entire time schedule for the transport can only be made on the basis of the outcome of these tests.
The model tests always include the most difficult part of the whole transport, i.e. the arrival of the unit at the immersion trench *(fig. 2)*. Upon arrival at the trench, the unit has to be turned to coincide with the tunnel axis which is often perpendicular to the centre-line of the river. There the unit has to be anchored into an anchoring system. The model test furnishes an excellent insight into the fixing of the unit to its anchors and into the type of forces to be expected during anchoring.

Beside the forces caused by the current there is another aspect to be taken into account in the design of the anchoring system. Units have a big mass. The surrounding water adds to the mass. This means that it takes time to impart speed to the unit, but also to slow it down. At the immersion trench, in the beginning the unit is initially held in position by tugs with an inaccurancy of many decades. Then it is gradually anchored with heavy cables. Experience has shown that in that case substantial cable forces arise as a consequence of the unit being caught by the cables.
Fig. 2 shows the laboratory equipment used in determining the overall mass of the unit including the added mass of water. The calculation is based on Newton's formula: $K = M.a$.
A constant force K pulls the unit. Due to this force the unit is accelerated and from the relation between time, speed and force the total mass can be calculated.
The added mass depends upon the direction of the movement, parallel or perpendicular to the axis of the unit. The mass of the unit and the additional mass determine the forces necessary for accelerating or slowing down the unit.

Towing Force/Width of the Transport Channel
The installed numbers of horsepower required to tow the unit is influenced by several factors.
First of all there is the friction coefficient in relation to the relative velocity of the current which can be expected during the programmed time schedule. Then the fluctuation of current velocity due to changes in the normal tide has to be taken into account. Apart from the horsepower required for towing the units, additional capacity is needed to steer the units. These steering requirements depend upon the width of the waterway and the problems arising in specific stretches of the transport like bends, tributary rivers etc. The mass also influences the required steering power.
It must also be possible to hold the unit with the tugs or with an anchor during one tide in case of emergency.



Transport of a unit

Unit near the immersion trench

There is a certain relation between the installed horsepower of a tug and the maximum pull (± 15 kg for each HP), though this is influenced by the type of propulsion, etc. Generally the tugs cannot work constantly under full power. Furthermore the River Authorities may make their permisson for such a transport dependent upon all safety aspects thereof including a safety margin on the installed horsepower. To the Authorities the loss of a unit is less important than the consequences of blocking the waterway, specifically in the case of an important waterway and/or where no alternative routes are available.

All these additional requirements like steering, efficiency, safety etc. result in a tug boat configuration having 2 or 3 times the theoretical horsepower computed from the test.

Before the units are towed, a transport channel may have to be dredged.

An optimization study shows that the center line of the transport need not coincide with the middle of the river. The total amount of dredging work involved in the preparation of the transport channel may be reduced by following the deepest gully, or alternative routes.

In general the units have very little freeboard.

The depth of water must be sufficient for the units to be towed even at low tide. In case of great relative velocity with respect to the water, the front of the unit dips down. This has influence on the draught required. To check the depth of water an intensive echo sounding programme has to be carried out. Special attention has to be paid to the shallow areas of the river and to obstacles on the river bed. The obstacles may damage the units and especially the rubber profile at the front which serves to join the units may be affected. Echo sounding has to be repeated prior to the transport of each unit. Before towing, the areas may have to be dredged to the required level and obstacles removed.

The width of the transport channel has to be determined by, first of all, the width of the element. Swinging during transport requires extra width of the channel.

This swinging is due to the mass of the units and factors like over-corrections, misunderstandings during communication, and so on. In certain places the transport channel may have to be wider if circumstances are more complex due to bends, tributary rivers, and so on.

Anchoring of a unit

Filling an excavation pit

Survey towers, equipped with winches

Time Schedule for the Towing Operation

As a rule the towing operation is not set up to work under any tidal conditions. For this reason an intensive study has to be made of a whole tidal cycle.

The relation between different tidal conditions and the frequency of their occurance may be important. The normal tide may be influenced by astronomical and meteorological aspects and by the discharge of the rivers. Economic motives may result in the operation not being based on the most extreme conditions. Optimization of costs and other aspects determine the conditions – with frequency distribution relative thereto – during which the operation may still proceed.

Based on the average tidal situation a time schedule is prepared which serves as a provisional plan for further elaboration. The whole transport is related to the tide. The difficult stretches of the transport get special attention. These parts have to be passed at favourable moments as to the tide.

Just before the start of each operation the last adjustments to the time schedule are made in accordance with information on astronomical, meteorological and river discharge forecasts.

It must be possible to reach positions in time with the available tugboat capacity.

In order to bring flexibility into the towing programme, places are included in the time schedule where the transport can wait for a shorter or a longer period. In this way corrections can be made with regard to the tide.

During transport data are collected by survey boats, stations, etc. In a control centre experts receive these data by radio and use them for updating the forecast programme.

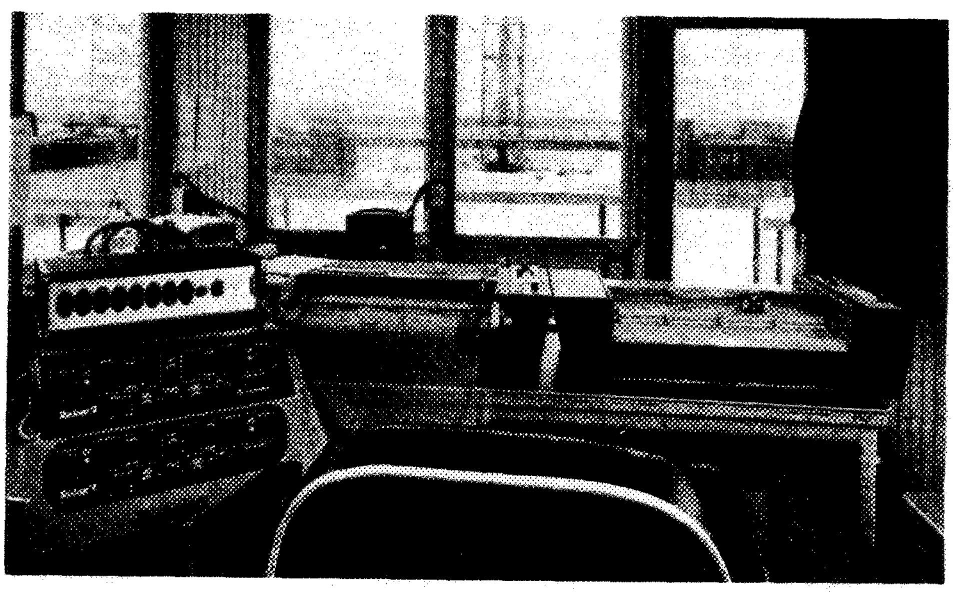
At the end of the transport operation the unit has to be moved into an anchor system. A script for anchoring spells out this procedure in detail.

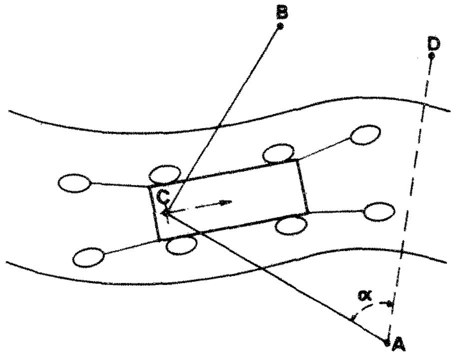
Handling has to be done carefully as 'being caught by the cables' of the units results in excessive anchor forces. The anchoring operation requires special attention because of the fact that many boats, pontoons etc. are assembled in a small area, whilst cables are fixed in the water from unit to anchor.

Navigation

In general the channel will follow the deep stretches of the river wherever possible. Where it is not possible to mark the transport channel in an optical way by means of buoys, an automated navigation system may present a solution.

Navigation equipment





3
Navigation
System I: measuring of length CA/CB
System II: measuring of length AC, angle ∝

Modern equipment allows the locating of positions over long distances with great accuracy.
Fig. 3 shows two navigation systems. System I measures radiographically the distances to two fixed beacons.
The transmitter is on board the unit. The stations on the river bank reflect the signal to the receiver and the position is worked out.
In system II the station on the bank determines the distance from, and angle with the unit as indicated in the above-mentioned figure.
In both systems the position of the axis of the unit can be observed by a gyro-compass. This system is more expensive than an optical survey, but radio systems still function when there is a light fog.
The data of one of these systems are put into a computer. This computer controls a display or a plotter. The computer shows the position of the unit in the river. The display, or the plotter must give a continuous registration of the location.
Experimental tows may test the system and indicate which additional safeguards and devices have to be observed or introduced.

Conditions during the Towing Operation
During preparation of the tow, the tidal, the current and the weather conditions have to be monitored continuously. The whole transport system is designed for a specific tidal situation.
As soon as it is known with sufficient certainty that the tide and the current will go beyond the admissible conditions, the operation has to be cancelled. Storm, heavy fog or low temperatures may likewise prevent the operation from being carried out.
In general reliable forecasts as to wind and temperature can be made; however, forecasts related to visibility seem to be more difficult.
During the transport and immersion operation, shipping may be obstructed. The time schedule of this traffic obstruction has to be announced in advance. In the interest of the public in general and the parties involved in such an operation a cancellation has to be announced as early als possible, preferably together with an indication of a new date for the operation.

Summary
Afther the construction of the units they have to be fitted with facilities for transport and immersion.
Before tow-out begins laboratory tests are to be made in order to determine the behaviour of the elements during the towing operation.
On the basis thereof a calculation can be made for the required horsepower during the towing operation.
Extra horsepower is required to overcome special conditions for steering purposes and to serve as a safety margin.
Apart from the forces caused by the stream there is another factor to be taken into account. The water around the unit increases the mass of the element. Laboratory tests determine the additional mass based on Newton's formula.
Before towing the units, a transport channel may have to be dredged. In general the units have very little freeboard. In order to check the depth of the water intensive echo sounding is applied to show up the shallow parts of the water and obstacles in the river bed. The obstacles may damage the unit, the rubber profile at the edge is very important, and even the temporary bulkheads may be holed, with catastrophic consequences.
An intensive study is made if the area of towing is a tidal.
Economic motives may result in a towing plan that is not focussed on the most extreme tidal conditions. A time schedule is prepared with the average tidal conditions as a guideline. Just before the operation starts the last adjustments to the time schedule are made according to the influences of river flow, astronomical and meteorological forecasts.
In order to reduce the quantity of material to be dredged the transport channel need not coincide with the centre line of the river. Where it is not possible to mark the deep transport channel by means of buoys, a solution may be found by an automated navigation system.
The whole system is prepared for a specific tidal situation and for specific weather conditions. Once it is known with sufficient certainty that prevailing conditions will go beyond the admissible conditions, the operation has to be cancelled as soon as possible in order to save costs and reduce inconvenience to the general public.

Ir.V.L.Molenaar
Royal Bos Kalis Westminster Group

The Immersing of the Tunnel Units

Introduction

An immersed tunnel is a cross-river communication which consists of individually immersed units which are connected under water and finally – together – form a tunnel.
This sort of cross-river connection can serve for all kinds of transport like road-traffic, railways and pipelines. The techniques applied are not necessarily restricted to cross-river connections. Outfalls can also be constructed in a similar way.
In the following elucidation it is not possible to discuss all techniques in detail as the time available is too short. However, you will find a summing up of all possibilities of immersing in relation to the circumstances under which they take place. The first thing to do will therefore be an overview to determine outside influences which may affect the progress of the job.

Survey

The following factors and elements will have to be investigated.

Current

The direction and strength of the energy generated by water currents are determinative for the heaviness of the equipment used. Extensive laboratory tests will have to be done. In the laboratory the various stages of immersing are simulated and tested under different circumstances such as: changing water-levels, flow direction and velocity. The forces measured on the model will have to be 'translated' – by using scales – into the forces which will occur in reality.
The velocity of flow can change over a period of time. With ebb and flow the velocity of the current will vary within a very short time and even change its direction. It is obvious that this can greatly affect the total job.
It is necessary to follow these movements for a long periode and to record them very accurately.
Except for the tide, the flow force can also vary as a result of seasonal influences. Higher velocities quadratically increase the flow forces.
When determining the equipment to be used, it is not necessary to cope with the highest flow forces. In each case the optimum will have to be calculated. When the dimensioning of the equipment is determined it will be necessary to watch its manageability.
The velocity of flow is also determinative for the dimensions of the tunnel elements. For example it is possible – which happened when the Hemspoor tunnel was constructed – to manoeuvre units of about 280 m long. The Hemspoor tunnel is immersed in a channel with no current at all. The recently built Drecht tunnel and the Kil tunnel, however, were immersed in swift flowing rivers. Consequently the units used here were only 120 m long.

Cross-profile of the River

In addition to the flow velocity the cross-section of a river is of great influence. For, as the river is blocked by the tunnel units, the flow force on them will increase. During the constructions of the Kil tunnel, e.g., it was necessary to transport the unit which was to be positioned in the middle of the river, by way of a trench that ran *below* the current.
The model tests allow us to find a factor which is the drag coefficient of the dimensions of the river cross-section and of the tunnel unit. In this case it is essential that the tests be equivalent to reality. A scale of 1 : 50 and a river length of two kilometers has proved to be adequate.

Specific Gravity of Water

The tunnel elements can be floated or – dependent on the weight of the element – be suspended from a transport system.
In the first instance the necessary weight to sink the element will be obtained by pumping water into the ballast tanks of the tunnel unit before it can be immersed. In both cases the resulting weight under water will have to be sufficient so as to allow the tunnel units to sink to the bottom of the trench despite an increasing gravity of the water. Particularly in case of large dimensions of the tunnel units the necessary ballast system can be very voluminous.

Waves

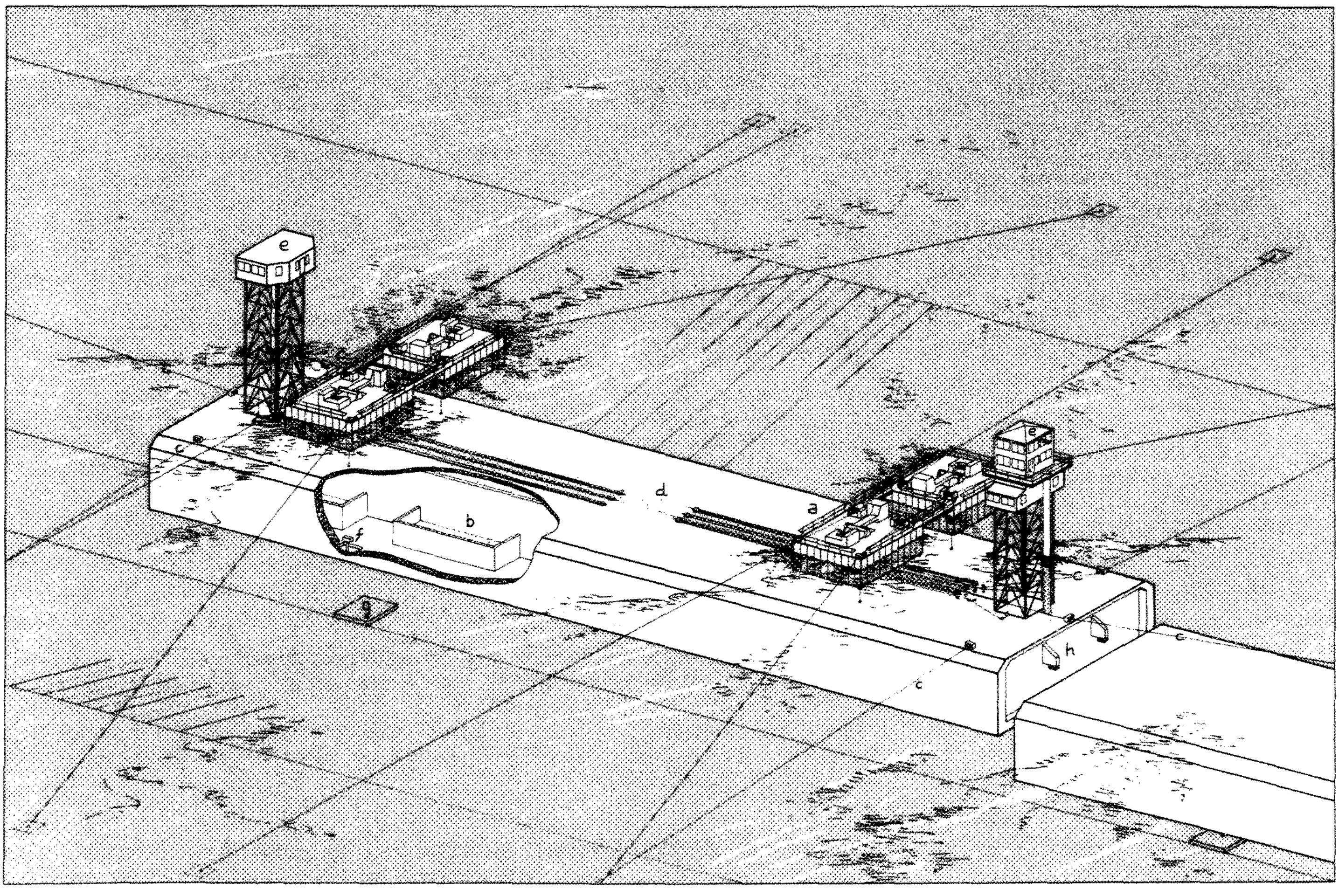
In rivers waves and swells wilp hardly be noticeable and no or hardly any consideration will have to be given to these phenomena. However, when building takes place in a river mouth/estuary or at sea (for example outfalls in the strip before the coast) things are different. Because of the fact that the greater part of the tunnel unit is under water, it will hardly move. However, when it is suspended from floating bodies (e.g. pontoons) and is dependent on the proportions of its masses, it can begin to move or else the forces in the suspension system will become intolerable. Immersion by means of jack-up platforms or semi-submersible pontoons will then be the only way.

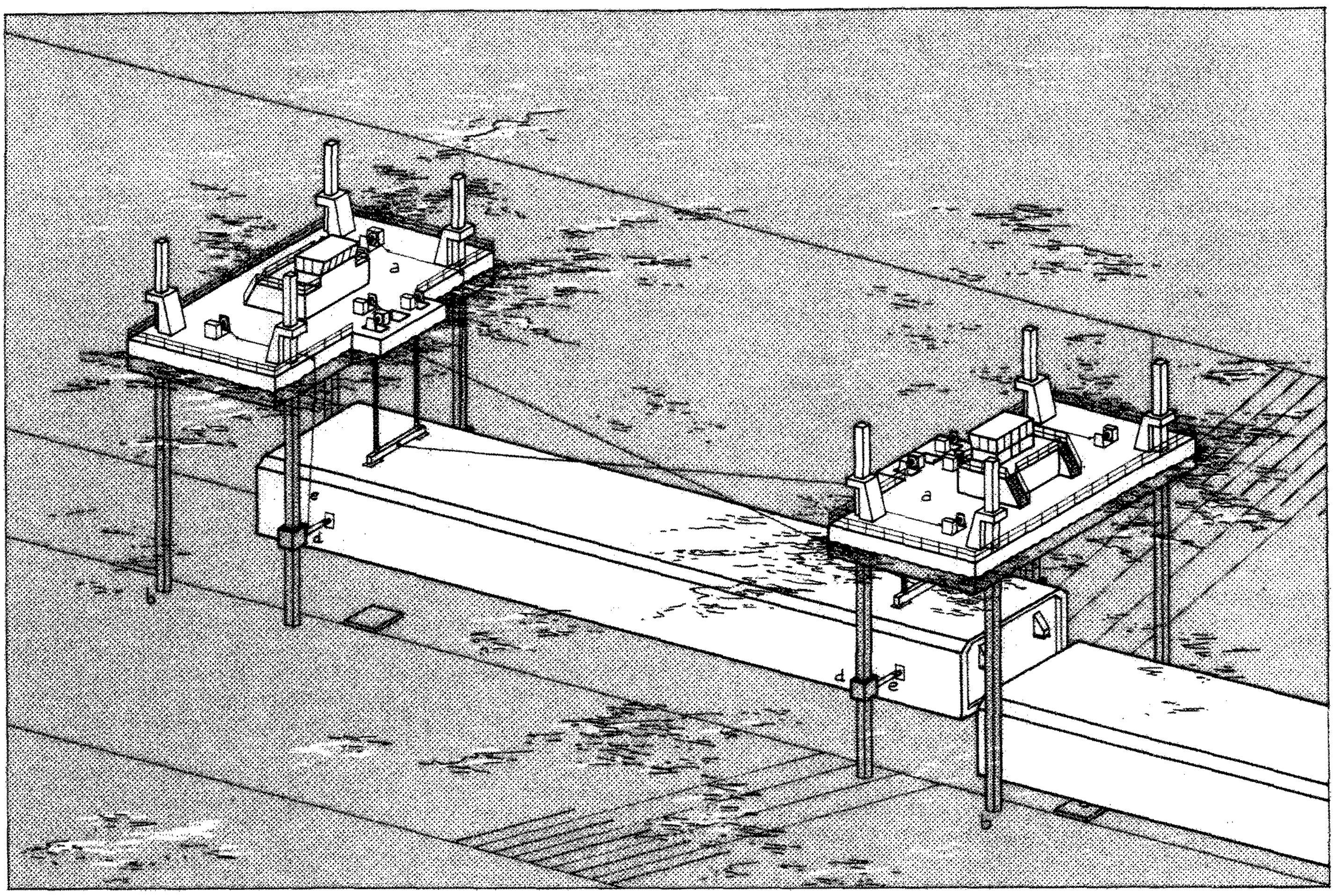
Depth for Immersing
When immersing a tunnel unit the depth of the trench is an important factor. Until recently it was thought essential to have the unit within reach during the operation. This implied that it was not possible to immerse tunnels in waters deeper than 20 m. Meanwhile techniques have been developed, also by the offshore industry, which enable us to immerse at depths which are multiples of those recently worked on.
In particular these techniques are directed to diving, coupling, rubber joint profiles, and so on.

Dimensions for Tunnel Elements
Up to now we have discussed outside influences on the choice of the immersion equipment. An important factor in this choice is of course the shape and the dimensions of the tunnel units.
We have already mentioned that the dimensions of the unit affect the flow force. Evidently, when using a larger unit, we are restricted in the choice of our system. With an increased mass of the element, for instance, the use of a jack-up platform will become increasingly difficult. One can imagine that the forces occuring in e.g. a unit for the pipeline tunnel with a water displacement of approximately 3,000 tons and an element for the Drecht tunnel with a water displacement of approx. 45,000 tons will differ.

The System
Since we want to carry out a radical and rather risky operation within as short a time as possible, and since much of the operation takes place *under* water, it is essential that we adopt the highest possible simplicity of operation. There are several reasons for this short period. The major one being that we do not wish to disturb shipping traffic longer than necessary, particularly in larger ports and harbours.
It is also possible that tidal influences necessitate a limited schedule. In all following systems the simplicity of the operation will be recognized.
After the tunnel unit has been taken over from the tugs onto the system which will take it to its final location, the immersing operation can be divided into a number of steps.

1
The system delineated above shows the tunnel unit suspending from the pontoons (a). The ballast tanks (b) are filled with water. For the horizontal movements of the tunnel unit 6 moored cables (c) run to winches on both ends of the tunnel unit and via a system of pulley-blocks and pulley-sheaves (d) these cables are connected with the winches at the top of the towers (e).
The tunnel unit is temporarily placed on jacks (f) on concrete slabs (g) placed in the trench and on supports (h) on the previous unit





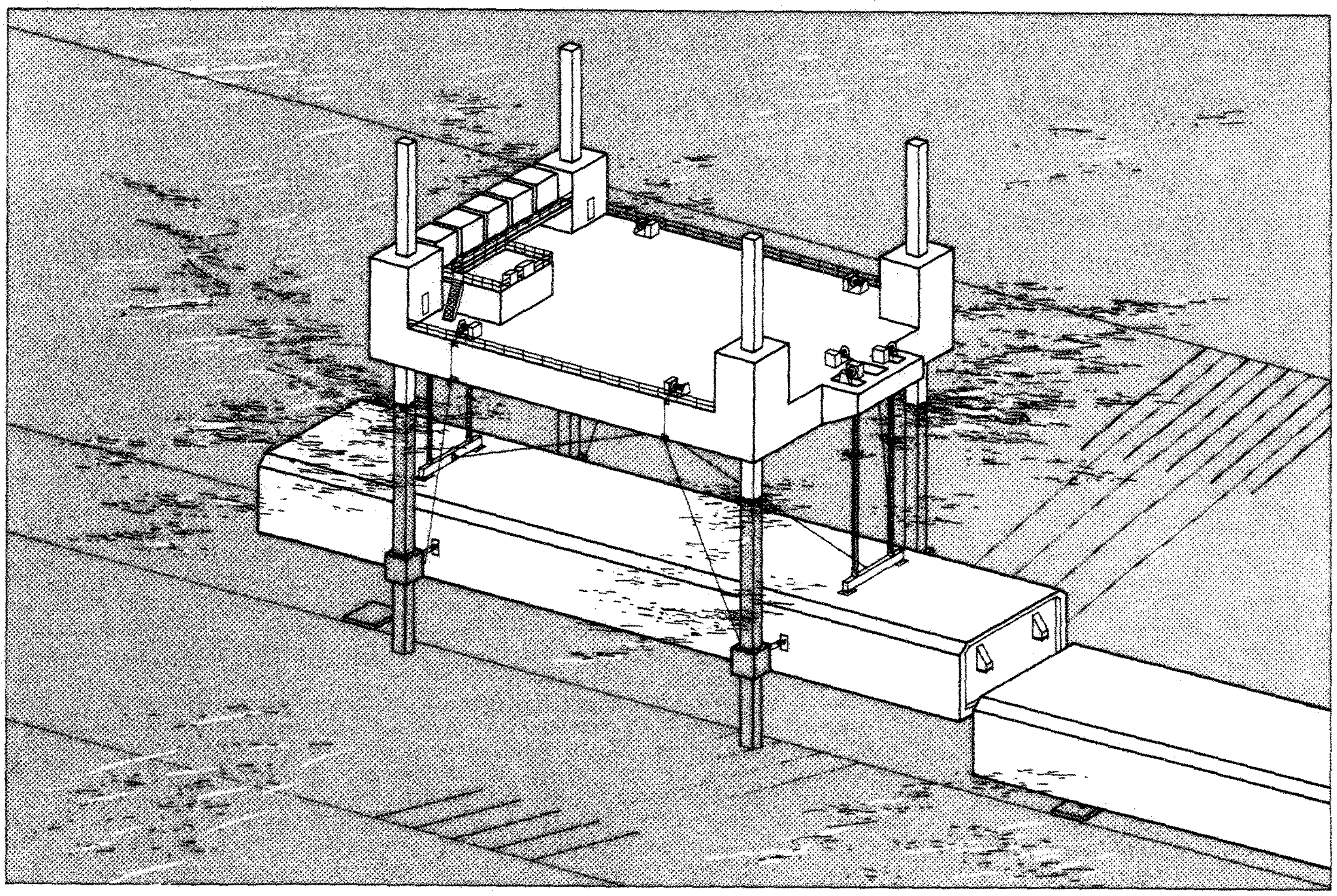
2
The tunnel suspends from floating pontoons (a) which are secured in the trench by spud-poles (b).
The horizontal movement is effected by winch cables (e) which are connected to the tunnel units via sheaves (d) on the poles. Furthermore the system is identical to that of figure 1. The direct connections have a highly increasing effect on the job. With small tunnel sections the immersion of one unit per day has appeared possible

Vertical Positioning
The tunnel unit which is suspended from a number of pontoons, or from a jack-up platform, will sink because of a built-in 'overweight' (small cross-sections) or because of the fact that the unit has been filled with ballast, usually water.
This overweight will have to be accurately calculated by measuring the gravity of the water. During the construction of the unit the tolerances in wall thickness and the establishment of the specific gravity of the concrete used will have to be minutely followed.
The vertical positioning of the element takes place in a number of steps, becoming increasingly smaller until the element has been positioned onto its supports. This positioning as a rule is done by a construction in which the tunnel unit rests on the previously immersed unit and by temporary supports with jacks allowing height corrections.
The jacks are positioned at the free end of the unit and supported by piles mounted on slabs placed in the trench.

Horizontal Positioning
When applying this method, cables are secured at the sides and the ends of the element. These cables run to fixed points at the bottom of the river or at the pontoons or jack-up platform. Winches placed on the pontoons or the jack-up platform allow for a very accurate horizontal movement of the tunnel unit.
Also in this case the manoeuvring is done in steps, becoming smaller as the previous unit or the land sections are approached. The strength of the cables is determined by calculations in which the factors mentioned before – such as flow force, dimensions and so on – are taken into account. Should, however, the forces on the anchorage system become such under influence of these factors, that the system becomes inoperable, we can resort to special measures.

As an example we take the immersion of the central tunnel unit of the Kil tunnel which was immersed in a sheltered part of the trench and then transported *underneath* the current to the place where it had to be finally positioned.
Once the tunnel unit has been manoeuvred towards the previous one and placed on a support system on this unit and a separate support system at the free end, the last horizontal movement will take place for the final connection.

Joint Construction
The joint constructions mostly applied are those based on the principle of the Gina profile. A Gina profile is fitted to the free end of the unit to be immersed which is to be connected to the previous unit.



3
In order to bring the influence of the waves on the manoeuvre to a minimum, a jack-up platform is used.
The rest of the manoeuvre is similar to that in figure 2. When the dimension of the tunnel unit are too big in proportion to the jackup platform and waves, semi-submersible equipment is used.

This Gina profile consists of a rubber ring with a soft rubber tip at the fore-end (approx. 25° shore hardness). The large mass of the Gina profile is hard rubber (shore hardness approx. 45°), but the hardness and the profile may vary depending on the cross section and the depth at which the unit has to be immersed.

By drawing up the unit against the previous unit by means of a special jack construction or cable, the first compression is obtained by the soft rubber tip of the Gina profile. It is useless to try and tighten it now because the water in the chamber thus created will not allow this.
Subsequently the water is pumped out of this chamber, but the water pressure remains at the free outer end. The pressure in the joint ceases to exist, but the hydrostatic pressure is so great that the unit is pressed further against the previous unit.

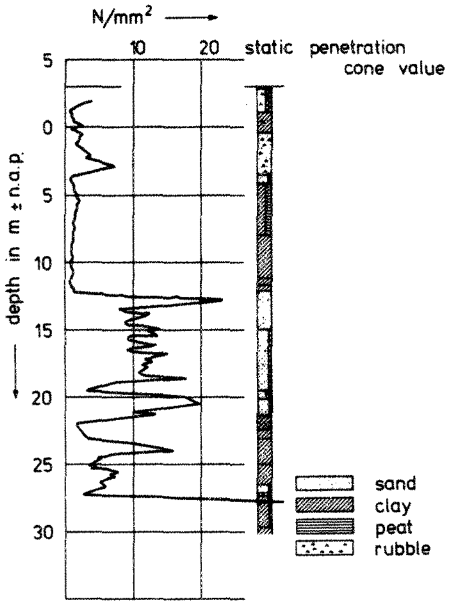
The Sealing Joint
After the final tunnel unit has been immersed, a space of approximately 1 m width exists between the last two units. This space is fixed by the placing of a number of wedges in the joint. Without these wedges the two units would be moving towards each other as a result of the pressure in the Gina profiles in the previous joints. Thus opening these joints.
Subsequently, watertight bulkheads are connected onto the sealing joint. The space thus created is pumped dry and the tunnel connection between the last two immersed units can be completed.

Summary
The immersing of tunnel units depends on the circumstances on the spot where they will be immersed. Model tests have to be made in order to ascertain the forces to be expected on the tunnel units under different circumstances. These forces can change with the water level, the current velocity and the specific gravity of the water and the depth at which the tunnel is immersed.
The equipment to be chosen also varies with the dimensions of the tunnel units. Up to now the following equipment has been used:
- *floating pontoons;*
- *spud pontoons;*
- *semi-submersible pontoons;*
- *jack-up platforms.*

The way in which the tunnel units are connected under water has been developed in such a manner that nearly no force is necessary using special rubber profiles and the under-water pressure.

Ir.H. van Tongeren
Ballast-Nedam Group BV, Amstelveen

The Foundation of Immersed Tunnels



1
Typical soil profile for the western part of the Netherlands

Introduction – Foundation Characteristics – General Soil Conditions

Accordingly in most cases the bearing capacity of the subsoil will be sufficient when the tunnel is founded directly on the bottom of the excavated trench. The preceding remarks apply to the situation in the middle of the waterway but even in the embankment areas where excavations may be deeper, the bearing capacity will in general not present any problem.

It has become general practice in the Netherlands to design immersed tunnels with expansion joints each 20-25 m. These joints are designed in such way they will prevent differential vertical movements of the adjacent sections and allow for rotation at the same time. As a consequence the tunnel will as a linked chain follow the settlements of the subsoil without introducing bending moments.

Where in most cases the subsoil conditions and the loads do not change very abrubtly along the tunnel settlements will fluctuate regularly. Hence, as regards to settlements as well as bearing capacity a direct foundation on the subsoil will in practice be possible in most cases. However, sometimes local conditions or special requirements can force the designer to another solution which in most cases will result in a pile foundation.

In the Netherlands most of the immersed tunnels have sofar been constructed in the western part of the country.

Soil conditions for this part of the country may in general be described as follows *(fig. 1)*:

The upper layers down to a depth of 10 to 20 m below O.D. consists of mixed sand and clay layers of low resistance. When measured with the Dutch static penetration cone a resistance varying from 0 to 5 N/mm² will be found.

Below this depth a sand layer is found with cone resistances of about 10 N/mm², in some cases rising to 30 N/mm².*)

As most immersed tunnels will have a height of 8-10 m and as the level of the roof will be situated between O.D. and 20 m below, it is obvious that soil characteristics may vary considerably along the tunnel.

In conclusion it can be stated that the choice of foundation for an immersed tunnel in the Netherlands will be determined by the purpose of the tunnel (rail or road traffic) and fluctuations in the magnitude of the settlements to expect along the tunnel.

In most cases a direct foundation is possible. Due to structural principle of a linked chain, the character of the subsoil will exert no further influence upon the structural design of a tunnel directly founded on the subsoil.

Development of Foundation Methods for Immersed Tunnels in the Netherlands.

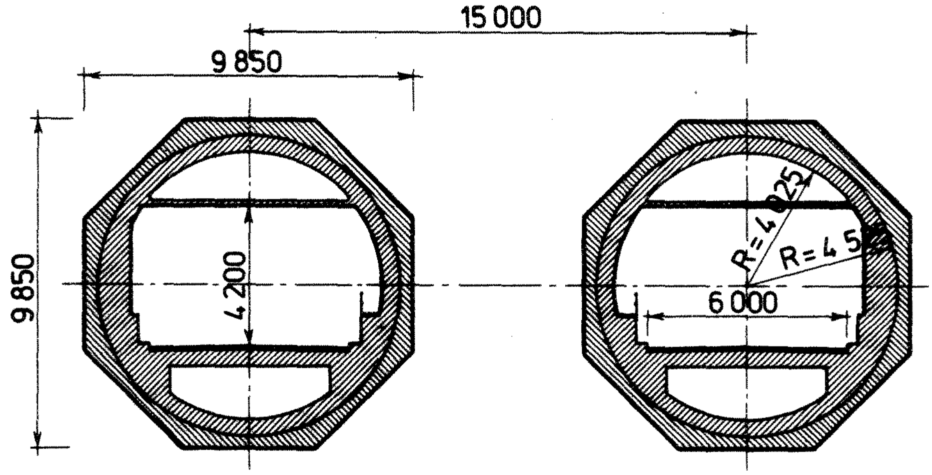
The first immersed tunnel in the Netherlands, the Maas Tunnel at Rotterdam, was built some forty years ago.

It is an interesting fact, that the construction of this tunnel led to the introduction of a foundation system which since then in principle has been used for numerous tunnels throughout the world, i.e. the foundation on a sand bed cast in situ after the temporary installation of the tunnel in a dredged trench.

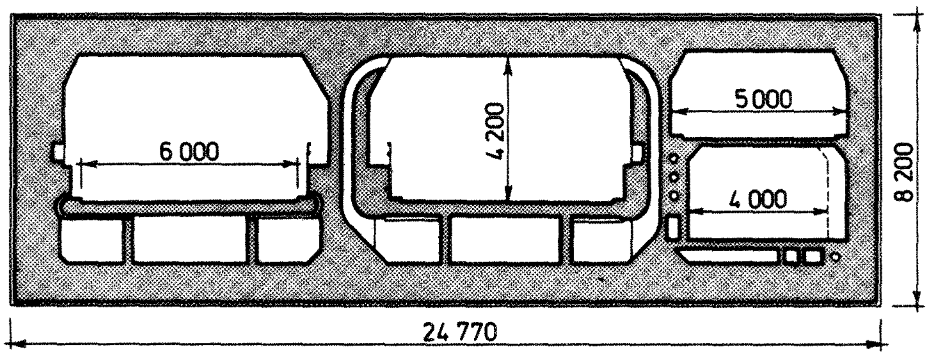
The design for the Maas Tunnel specified by the authorities provided for two traffic ducts situated in two independent tubes *(fig. 2)*. The contractors, however, proposed alternatively that the two traffic ducts should be combined in one rectangular cross section, which was then easily completed with tubes for cyclists and pedestrians *(fig. 3)*.

*) As a guideline cone sounding values (in N/mm²) may be converted to the N-value of the standard penetration test (SPT) by multiplying them with a factor of 2.5.

2
Maas Tunnel,
cross-section according to specifications



3
Maas Tunnel,
cross-section as built



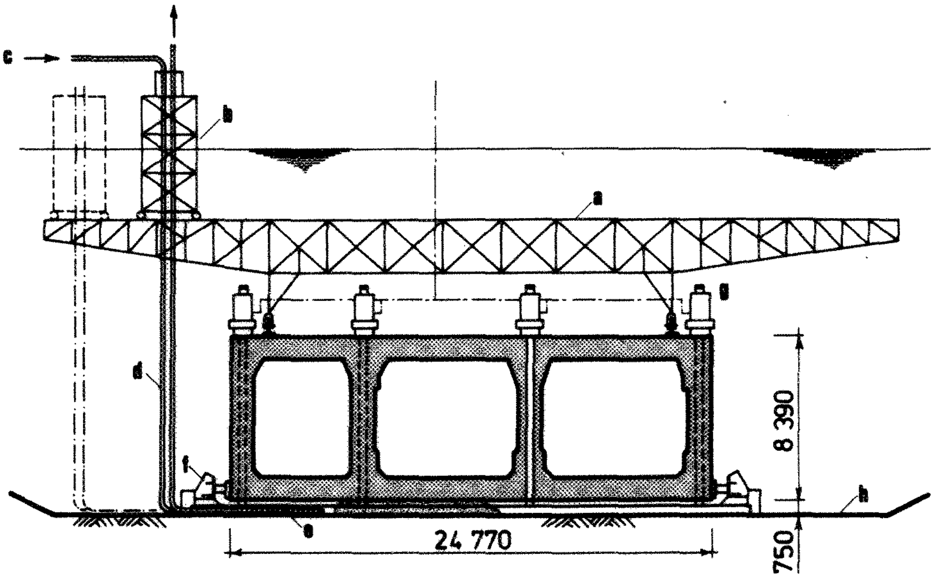
This solution was quite logical. In fact identical solutions have been considered many times before, but nobody did find a proper solution for the foundation problem.
For the Maas Tunnel this problem was solved in the following way:
The tunnel was placed in the dredged trench on temporary foundations leaving a clearance between the bottom of the tunnel and the trench. By using a gantry which did ride on the roof of the tunnel a sand water mixture was jetted into the open space to form a foundation layer.
After completion of the Maas Tunnel there was an interval of many years in which no immersed tunnels were built in this country.
In the nineteen-sixties immersed tunnel building started up again with the Coen Tunnel in Amsterdam.
Here a sand foundation was provided similar to the Maas Tunnel using a somewhat modified sand jetting gantry. Construction of other tunnels followed.
Around nineteen-seventy the design of a crossing through the Westerschelde river was iniated. This design includes a tunnel at greater depths than executed hitherto. Besides it has to be immersed in the river with intensive shipping traffic to Antwerp and current velocities up to 2 m/sec.
It was obvious that the idea of a sand jetting gantry riding across the river on the roof of the tunnel would present many difficulties. A new system comprising a sand-water mix injection through the bottom of the tunnel structure was developed. This so called sand flow method has provided to be successful in a great number of tunnel projects during the last 5 years.
Meanwhile, other methods such as foundation on driven and on bored piles and foundations on pre-made gravel beds have been applied in special cases.

The following summary indicates the number of various foundation methods applied for immersed tunnels in the Netherlands since the construction of the Maas Tunnel.

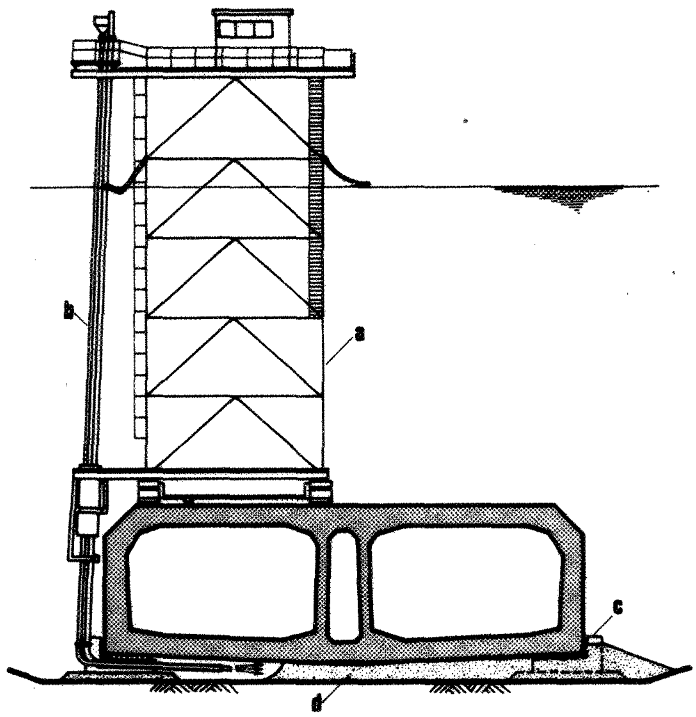
Tunnels founded on sand according to the sand injection (sand jetting) method	4
Tunnels founded on sand according to sand injection (sand flow) method	4
Tunnels founded on bored piles	1
Tunnels founded on driven piles	1
Tunnels founded on gravel beds	1
Other immersed tunnels	1
Total number of immersed tunnels built in the Netherlands	12

Sand Jetting Method

4
Maas Tunnel, sand jetting gantry
a. bridge, movable along tunnel axis
b. tower, movable along bridge axis
c. sand supply
d. jetting pipe vertical part
e. jetting pipe horizontal part
f. temporary support
g. vertical jacking device
h. dredged trench



5
Benelux Tunnel, sand jetting gantry
a. gantry movable along tunnel axis
b. pipe system for sand supply
c. temporary foundation
d. jetted sand



space under the tunnel completely. By adding suction pipes to both sides of the supply pipe a regular flow pattern is created, which will cause the sand to settle in a regular pattern. Moreover the filling can be controlled by observing the presence of sand in the return water in the suction pipes. As soon as the presence of sand is noticed the pipesystem will be retracted and shifted into a new position.
The whole pipesystem is connected to a gantry which is able to ride on the roof of the tunnel.
In case of the Maas Tunnel the connection to the gantry was in such a way that the pipe system could be moved in the direction perpendicular to the tunnel axis only *(fig. 4)*.
For later tunnels this was changed in such a way that the system was able to revolve around the vertical pipe *(fig. 5)*. Any jetting position could be reached by riding along the tunnel axis and by revolving the pipe system.

Sand arriving in barges is supplied into the pipe system by a barge suction dredger.
The grain size of the sand is limited to mean diameters above app. 0.5 mm. In all cases a quality was used as is normal for a concrete aggregate.
The reach of the jetted sand may be adjusted by a nozzle on the jetting pipe changing the exit velocity of the sand.
On average the concentration of the sand-water mixture will amount to 10 per cent of volume and has in some cases been increased to 20 per cent for shorter periods.
The packing of the sand will be rather loose with avoid ratio of 40-42%.

In general the thickness of the sand bed will be about 1 m as it is determined by the space necessary for operating the sand jetting pipe, and a certain amount of overdredging. By releasing the tunnel unit from its temporary foundation after the sand jetting is finished, the tunnel will settle about 5-10 mm. The final settlement of the tunnel will to a great extent be determined by the settlement of the subsoil due to the backfill of the trench.
In one particular case a 3 m thick layer of bad soil had to be replaced locally. After construction settlement at this place did not deviate much from the rest of the tunnel.

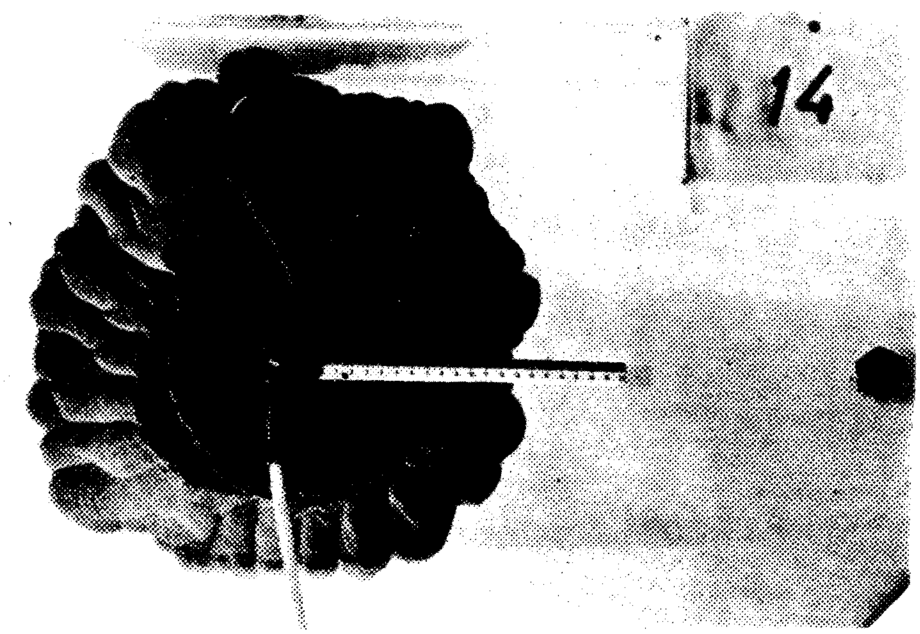
The sand jetting gantry can be used as well for the removal of silt in the clearance under the tunnel. By jetting with only water the silt will be brought into suspension and can be removed through the suction pipes.
In order to avoid as much as possible the silting up of the trench, it is advantageous to jet under each tunnel unit immediately after having it placed temporarily. However, this implies as a disadvantage that the sand jetting equipment as well as the immersion equipment will operate on the water at the same time and thus produce more hindrance to the shipping.
Therefore it is, in rivers with low contents of silt, preferable to start jetting first after all units have been placed.

Though the jetting method has proved its practical usefulness for a number of tunnels, in later years the method was in the Netherlands replaced by the sand flow method for the following principal reasons:

1. Most Dutch tunnels are situated in waterways with heavy shipping traffic, to which the sand jetting gantry presents a hindrance.
2. The use of a floating crane for transporting the gantry from one side of the tunnel to the other makes the system expensive.
3. A relatively coarse sand is needed which is rather expensive.

Sand Flow Method
As mentioned before, the sand flow method was developed when a crossing through the river Western Scheldt was designed.
The first idea was to provide the bottomslab of the tunnel with revolving jetting nozzles for the injection of a sand water mixture. However, during initial model tests it became obvious that mere jetting itself through an opening in the floor did already result in an almost complete filling up of the clearance under the tunnel.
The physical principle of this process of filling up may be described as follows (*fig. 6*).
In the first phase of the process, the sand after leaving the discharge opening, will be transported horizontally in all directions in the clearance under the tunnel.
At a certain distance from the opening the horizontal velocity of the current will have decreased so much that the sand will settle. This results in the building up of a circular shaped kind of dike. After some time a crater will have been created within this dike. The current is of such turbulence that no sand is able to settle here. Sedimention will now take place at the outer slope of the circular dike.
At a certain moment the top of the dike will reach the bottom of the tunnel. At this moment a new phase of the process will start. At a moment a sort of 'river' will break through and start to flow transporting the sand to the outer slope of the dike. There, the sand will settle and widen the dike.
Due to the lengthening of the river the friction resistance builds up so high that after some time the river will seek another bed of shorter length and less resistance *(fig. 7)*.

7
Small scale test of sand flow system, 'River' clearly visible
Photo: Ballast-Nedam/Dredging Division



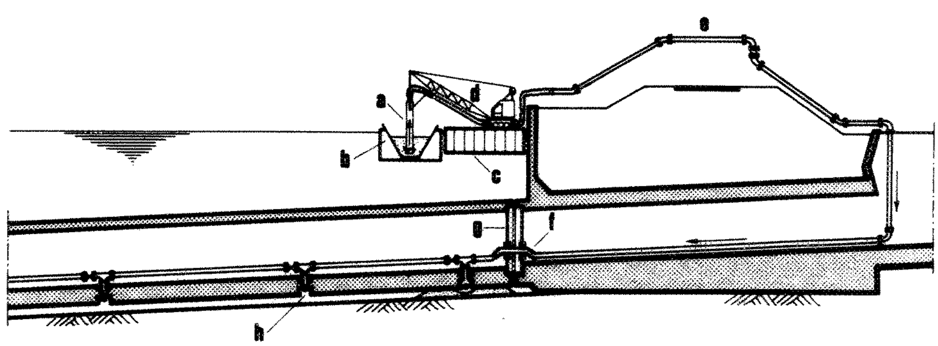
This process will be repeated again and again. By means of this revolving river the diameter of the deposit will get greater and greater. In this way the entire clearance under the tunnel will be filled up.
To maintain the current in the river, a gradient is necessary which means that the water pressure in the crater must be higher than at the edge of the deposit.
In the river itself the pressure drop is linear. On the remainder of the contact surface of deposit and tunnel bottom, the decrease in water pressure is determined by Darcy's law.
For a certain type of sand the depth of the river appears to be approx. constant for different discharges if the concentration of sand is kept constant. In most cases the depth amounts to about 0.05 m.

The width of the river *(b)* may be found from the formula:
$b = K.Q^{Y}$ (Q = discharge)
where the exponent will be nearly equal to unity. The factor K depends on the type of sand and on the concentration of the mixture.
The diameter of the crater (d_c) appears to be proportional to the square root of the discharge:
$d_c = d_o + K_1\sqrt{Q}$
where d_o denotes the diameter of the discharge opening.
The variation of water pressure in the contact surface between deposit and tunnel bottom at a radius r is determined by:

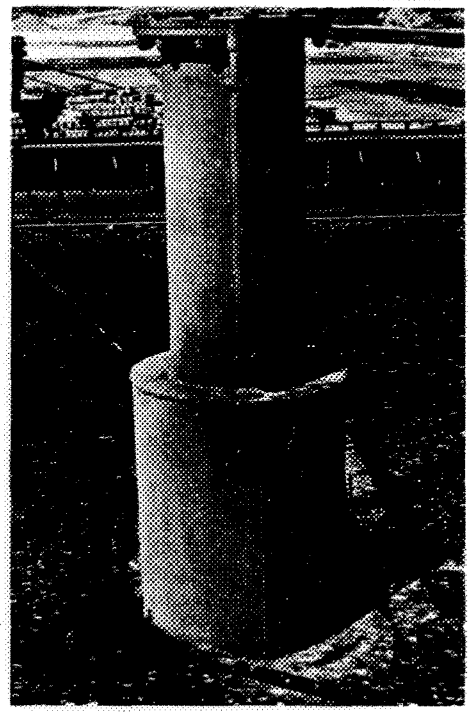
$$H = \Delta H \frac{\ln r_{max} - \ln r}{\ln r_{max} - \ln r_c}$$

where r_{max} denotes the radius of the deposit and r_c the radius of the crater.
The water pressure in the crater, ΔH, which will depend on the length of the river, has to be found experimentally. As a result of the higher water pressure in the crater, a vertical force will act at the bottom of the tunnel in an upward direction. Normally the upward force is limited to a few thousand (1000 to 3000)kN by limiting the diameter of the deposit. A greater upward force means more ballast water in the tunnel which leads to higher costs for the ballast tanks. On the other hand limiting the diameter of the deposit means more discharge openings.
An optimization procedure will yield the most economic combination.
In principle all types of sand may be used for this process. There is no evidence that a coarse sand will result in a better foundation layer. Besides, an execution with the use of coarse sand can include some doubts. Coarse sand will tend to have a larger permeability, which means that the discharge has to be rather high. All the discharged water could be lost by ground water flow through the deposit and as a consequence, no 'river' will originate.
The sand flow process will give results of the same quality as mentioned for the sand jetting system. The minor drawback of ending up with a crater and a river, which is not filled up, may be eliminated by diminishing the discharge at the end of the process. This means that the width of the river will be reduced. Theoretically it will be possible to fill up the whole surface.
The practical application of the process may be illustrated by reporting the first application for the Tunnel of Vlake. Later applications do not differ much. The amount of instrumentation has been reduced somewhat as a result of the observations made during this first application.

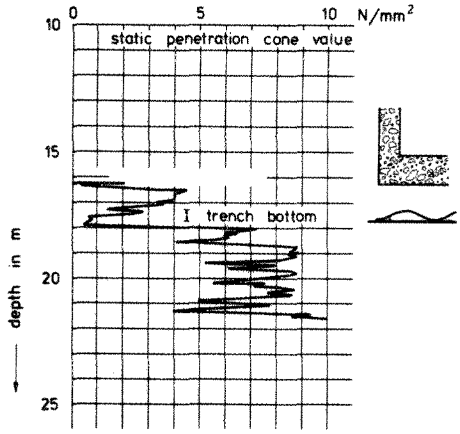
The Vlake Tunnel consists of two immersed units of a length of 125 m each. The tunnel contains two ducts and has an overall width of app. 30 m. Pumping from one end of the tunnel, the maximum total length of the pipeline was about 300 m. The pumping unit, moored to the bank of the river did consist of a pontoon mounted with pumps and a crane to handle the suction pipe *(fig. 8)*.
The pipelines, one in each tunnel duct, were 200 mm steel pipes. The pump had a capacity of 300 m^3 per hour at 28 m water column. The velocity in the pipeline amounted to about 3 m per sec. max. At distances of 20 meters c.c. discharge openings were situated in the floor, 13 in each duct.
The radius of the deposit was chosen at 12 m *(fig. 9)*.
This discharge openings were provided with ball valves fitted with rubber coated steel balls. When pumping water through the opening, the ball will be forced downwards thus allowing the sand-water mixture to pass. As the flow stops, the ball is forced bakc to its seat by the water pressure *(fig. 10 and 11)*. Finally all inflow openings will be blocked this way.
In each traffic duct the supply line was connected to three discharge openings at a time. When the area through one opening had been filled up, the area under the preceding opening was re-injected.



8
Vlake Tunnel, sand flow equipment
a. suction unit
b. sand barge
c. pontoon
d. dragline
e. pipeline connections
g. watertight bulkheads
h. ball valve in tunnel bottom



11
Ball valve before casting of tunnel bottom
Photo by courtesy of Van Hattum & Blankevoort



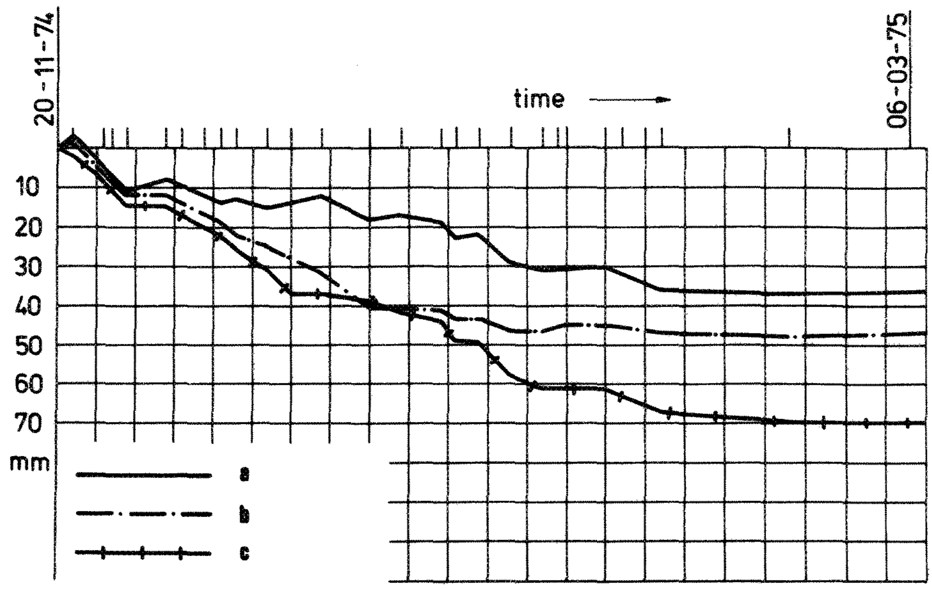
12
Vlake Tunnel, cone sounding through tunnel floor

This was done in order to fill up any small interstices which might have formed due to settlement of the sand foundation.
When the process had been completed for the three openings in one traffic duct, the injection unit was transferred to the next three openings. Meanwhile the injection process was started in the other traffic duct.
In order to check the process and also to verify the theories which had been developed, the sand injection was monitored by an extensive amount of instrumentation.

The following data were recorded:
1. Water pressure at discharge opening.
2. Jack pressure of temporary foundations.
3. Concentration of the mixture.
4. The amount of sand supplied.

To a certain extent the process was automated. Thus, e.g. the registration of the concentration was used to keep the concentration constant by adding water through nozzles at the suction mouth.
The sand-water supply pump would be automatically switched off as soon as the upward pressure against the tunnel bottom should exceed a certain limit.
Further, to control the filling of the clearance under the tunnel, a number of small openings is provided to give information about the degree of filling simply by opening them to see what comes in. This system did not work very well. Depth soundings by hand and by echo sounding along the outer wall of the tunnel appeared to be the best method to control the filling.
Furthermore to control the quality of a foundation layer, openings were provided through which it was possible to make cone soundings.
It appeared that in most cases the cone value of the foundation was better than could be expected when taking into consideration the low stress level. At the transition between foundation layer and the original subsoil low cone valves were found, possibly due to an insignificant thin layer of silt *(fig. 12)*.
After the tunnel has been released from its temporary foundations, settlements were controlled during a long period. The maximum settlement of about 70 mm appeared at the location of the last injection *(fig. 13)*.
This may be due to accumulation of silt at this place.
The magnitude of the settlement may seem considerable. However, as previously pointed out, the construction principle allows for differential settlements along the tunnel.

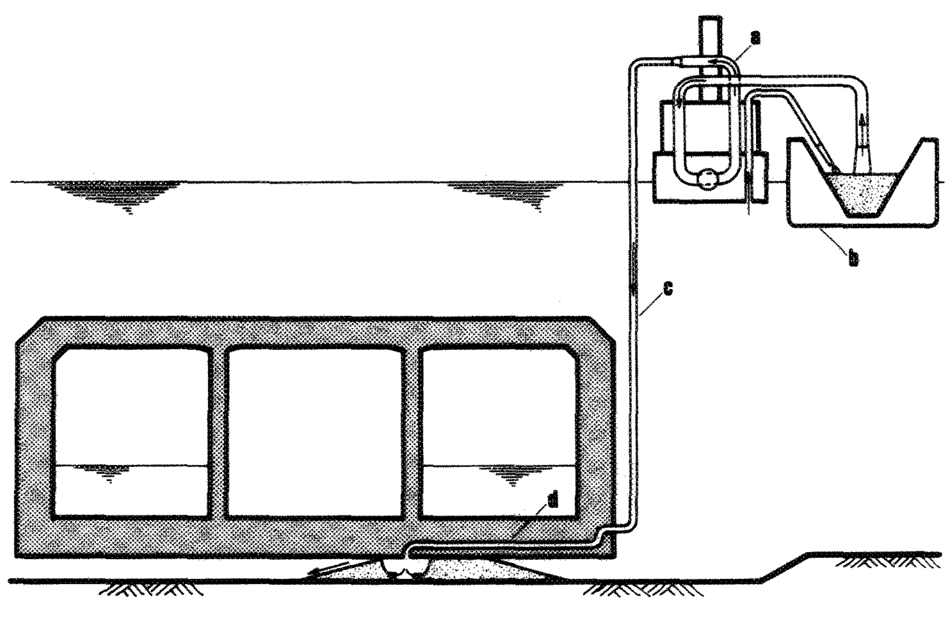
13
Vlake Tunnel, settlements of tunnel elements
a. settlement of joint between TE1 and TE2
b. settlement of end joint west
c. settlement of central part TE2

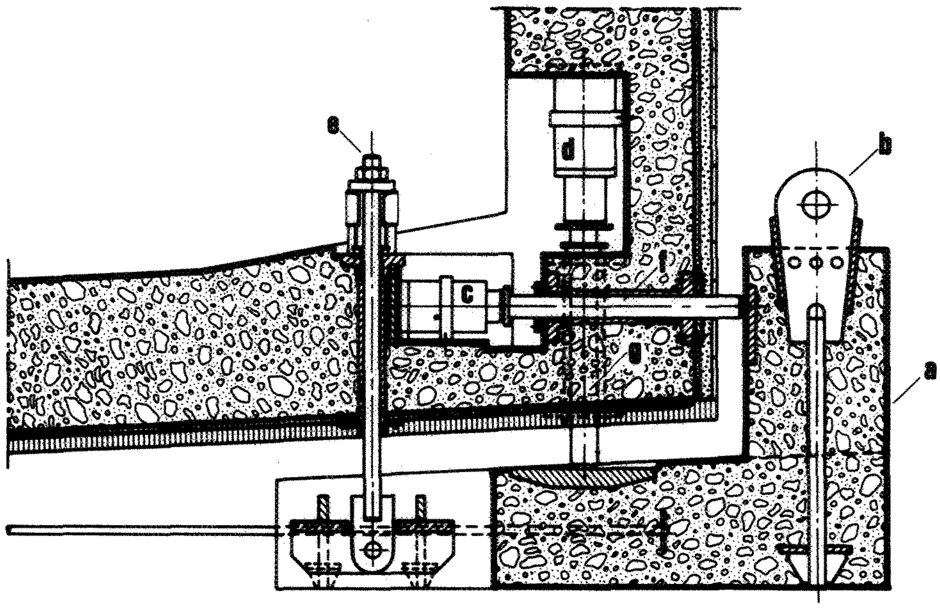


After completion of the work an attempt was made to correlate the water pressure at the discharge point with the upward force excerted on the tunnel and with the concentration of the mixture. This proved impossible due to the complicated static system of the tunnel units influenced by the sand already injected, the temporary foundations, and the flexible rubber gasket joints between the elements. In general it may be said, that the upward forces as calculated by the theory were not exceeded.
The whole tunnel was injected in about 200 working hours during 30 days. Theoretically about 11 000 m³ of sand would have been necessary. Actually about 13 000 m³ were used mainly due to the accumulation of more sand than expected along the outer wall of the tunnel.
The main grain diameter of the sand used varied between 160 and 270 μm.
To conclude this chapter, it can be mentioned that a major tunnel under construction, the Hem Tunnel for rail traffic in Amsterdam, will also be founded on injected sand.
In order to avoid the application of ball valves as waterstops in the tunnel floor, it was decided not to supply the sand from inside but from outside the tunnel through cast-in tubes in the floor (*fig. 14*). This solution is closely connected with local circumstances such as tidal fluctuation, local currents and shipping traffic because it is introducing the use of floating equipment again.

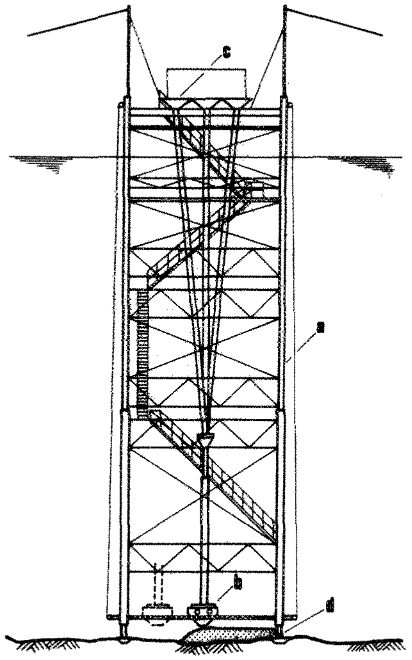
Temporary Foundations
Obviously the method of jetting or injecting a sand-water mixture under an immersed tunnel can only be used when the tunnel is temporarily kept in a position with its bottom above the bottom of the dredged trench. This is done by means of temporary foundations on which the tunnel is supported by vertical and horizontal rams thrusting out of the concrete structure of the tunnel.
Inside the tunnel the rams are provided with hydraulic jacks so as to keep the position of the tunnel adjustable after the immersing operation. The proper foundation consists of a concrete slab with dimensions of about 5 x 5 m, and 1 m thickness (*fig. 15*).

14
Hem Tunnel, sand flow system
a. sand pumping device
b. sand supply barge
c. pipeline
d. cast-in sand supply pipe





15
Benelux Tunnel, temporary foundation
a. concrete foundation block
b. suspension point
c. horizontal jack
d. vertical jack
e. suspension rod
f. horizontal ram
g. vertical ram

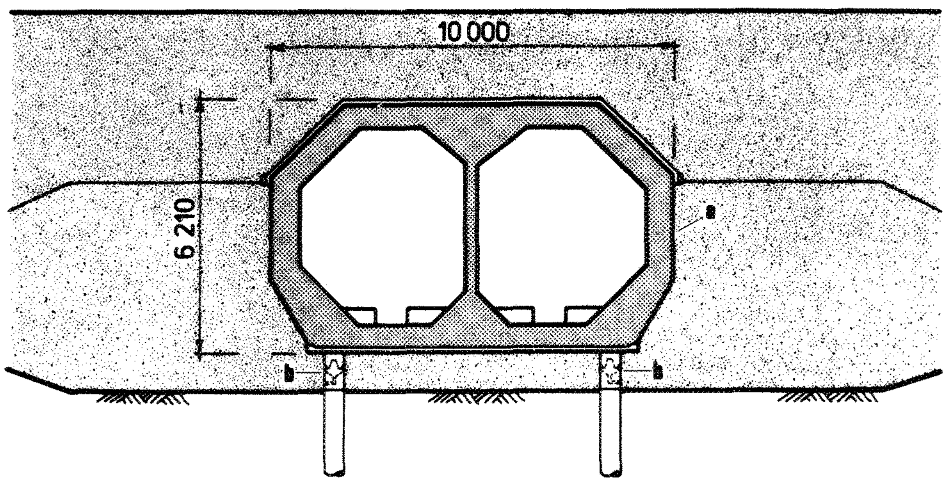


16
Screeding gantry for gravel beds
a. tower structure
b. movable gravel supply
c tunnels for gravel supply
d. adjustable support

The temporary foundation system has developed from a system with 4 pieces per tunnel unit into a system with the tunnel unit resting at one end by means of consoles on the preceding unit and at the other end on two foundation blocks.
The method of applying horizontal corrections by means of horizontal jacks has also been changed: this is now done by jacks in the joint between the units.
While the foundation blocks for the earlier tunnels were cast on the dry dock bottom and attached to the tunnel, the blocks are nowadays placed on the bottom of the trench separately before the tunnel unit arrives. In this way the draught of the floating tunnel unit could be reduced which meant that a smaller depth of the dry dock was possible and a smaller capacity of the equipment could be used to lower the ground water.
The temporary foundation blocks were placed on gravel beds in the trench because of the limited stroke of the vertical jacks. The gravel beds were finished at the exact level by means of a screeding gantry placed on hydraulic legs in the trench (*fig. 16*).
However, in later years it has proved more economic to invest in jacks with longer strokes and stronger foundation blocks and to place the bloks directly on the dredged bottom of the trench without any gravel bed.

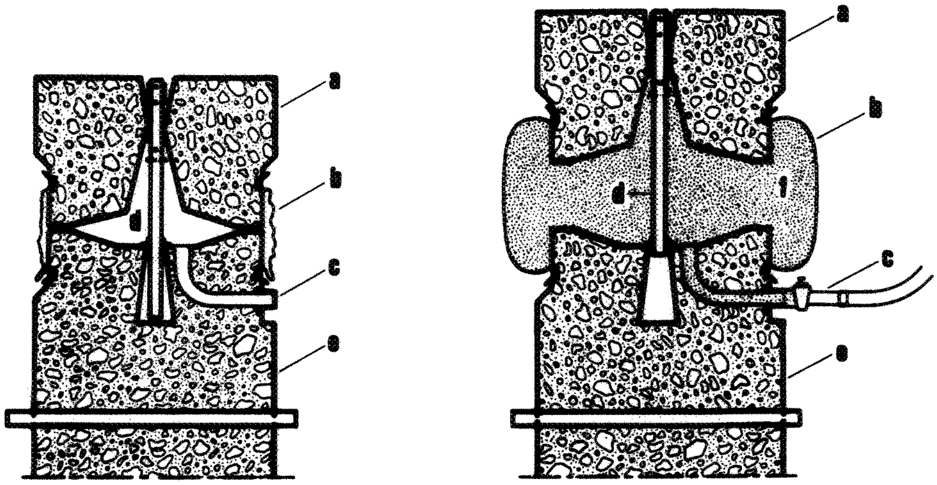
Other Foundation Methods – Pile Foundation – Gravel Beds
In the previous text the foundation of tunnels on a sand bed has been treated extensively, this method being the most commonly used in the Netherlands. However, as mentioned, other methods have also been used in special cases.
For the construction of a tunnel under the River Maas at Rotterdam for the underground railway, a special pile foundation was developed (*fig. 17*).
The reasons for a pile foundation were the following:

a. The stiffness of the subsoil varied considerably along the tunnel line, so that differential settlements could be unacceptable.
b. The mass of the tunnel is rather small in relation to the dynamic loads of the train passing the tunnel. It could therefore be feared that the tunnel might be subject to vibrations, which again would be transferred to the subsoil and effect unacceptable settlements had the tunnel been founded on the sand directly.



17
Rotterdam Metro Tunnel, cross-section
a. concrete tunnel structure
b. foundation piles with adjustable head

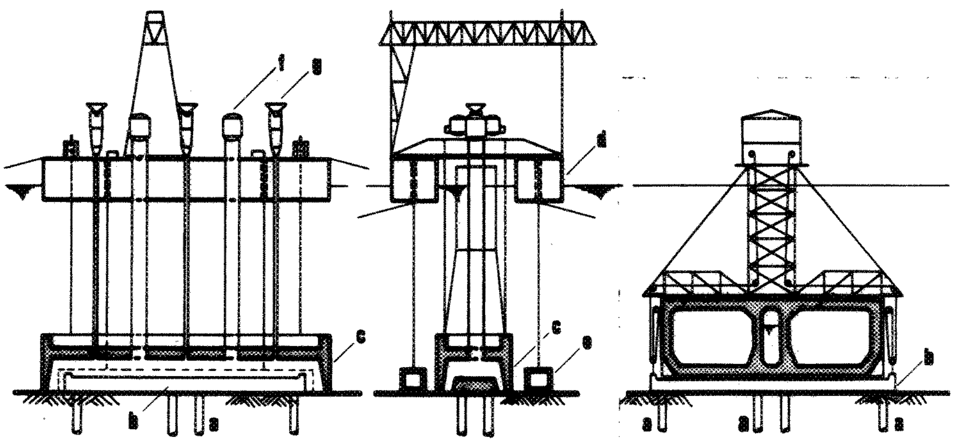
Rotterdam Metro Tunnel, adjustable pile head
a. pile head
b. nylon sleeve
c. grout pipe connection
d. guide rod
e. prefab pile section
f. grout injection



As it would be impossible to drive the piles to exactly the required level, a special type of pile with an adjustable head was developed. The piles were constructed in the following way:
First a steel tube, outer diameter 0.62 m, with a cast iron toe was driven into the soil.
At the bottom of the tube 0.5 m³ grout was cast on the cast iron toe. Now a prefabricated concrete pile was lowered into the steel tube pushing aside the grout thus forming a good connection between the pile and the cast iron toe.
The steel tube was withdrawn afterwards and as a result the enlarged pile toe contributes considerably to the bearing capacity of the pile.
The prefabricated piles were provided with an adjustable pile head consisting of a separate concrete part connected to the rest of the pile by a nylon sleeve (*fig. 18*). The tunnel units were immersed and temporarily placed on an alignment beam, fixed to four special piles.
After placement of the tunnel unit on the temporary structure and ensuring an accurate position, the adjustable pile heads were pressed up against the tunnel bottom by means of a cement grout injection into the sleeve, carried out by divers. Slight deviations of the pile from the vertical were taken up by the rotation capacity of the pile head. A felt layer on the head of the pile provided for uniform load transfer.

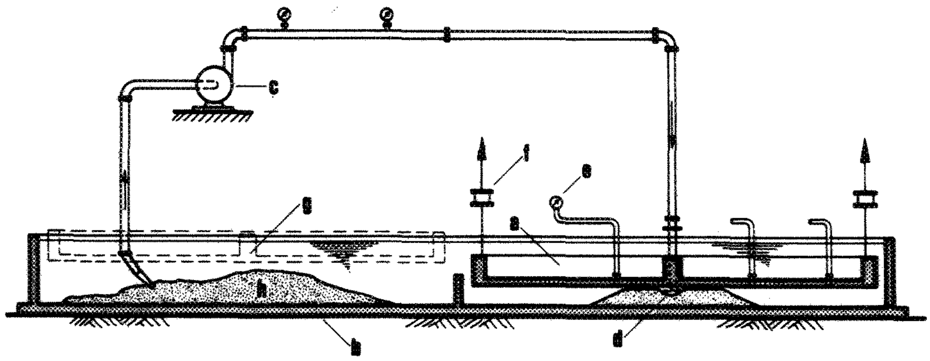
Another example of a tunnel founded on piles is found in the road tunnel under the river IJ at Amsterdam. The poor soil conditions at this place led to the choice of a pile foundation.
As most suitable system bored piles were chosen with a diameter of 1.08 m and a maximum pile toe depth of 90 m below O.D. The piles were produced by using a jack-up platform.
Each tunnel unit is supported by capping beams which interconnect a group of 8 or 10 piles. The capping beam was constructed in a reinforced concrete diving bell suspended from a floating double pontoon anchored to hollow concrete blocks on the bottom of the river (*fig. 19*).
After the tunnel unit had been immersed to rest on the capping beams by three temporary footings, the definite foundation of the tunnel was realized by grouting 24 rubber slab footings to make contact with the capping beam. The footings did rest on a sliding support made up of chromium-plated steel plates, with teflon as sliding medium. This was introduced in order to prevent transfer of horizontal forces to the piles due to movements of the tunnel as a result of temperature changes.

As a last example of the foundation of a tunnel structure by other means than jetted or injected sand the siphon under the Amsterdam-Rijnkanaal at Jutphaas has to be mentioned.
This siphon as a whole did consist of only one immersed unit with a length of 122 m and a width of 8.89 m. In such a case it is advantageous to look for a method which does not involve expensive equipment.

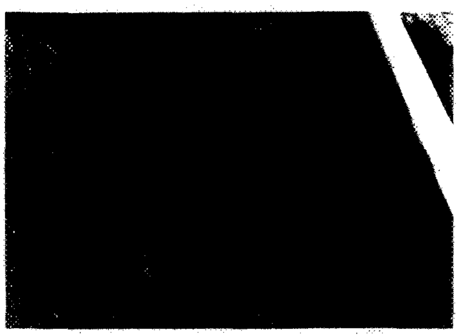


19
IJ Tunnel, foundation method
a. foundation piles
b. capping beam
c. reinforced concrete diving bell
d. pontoon
e. hollow concrete block anchores
f. air lock (persons)
g. air lock (material)

20
Sand flow system, large scale test
a. concrete slab 10 x 10 m
b. concrete basin
c. sand-water pump
d. discharge opening
e. water pressure meter
f. electric load cell
g. concrete slab removed after test
h. sand depot



The solution was found by applying a fixed screeding gantry which had previously been used for making gravel beds for temporary foundations of road tunnels. With this gantry gravel beds were produced with a finishing level tolerance of + and – 1 cm.
The beds were arranged in some sort of chess-board pattern so that a part of the lower surface of the siphon would not be supported directly. This non-uniform support was taken into consideration in the calculation of the siphon structure.



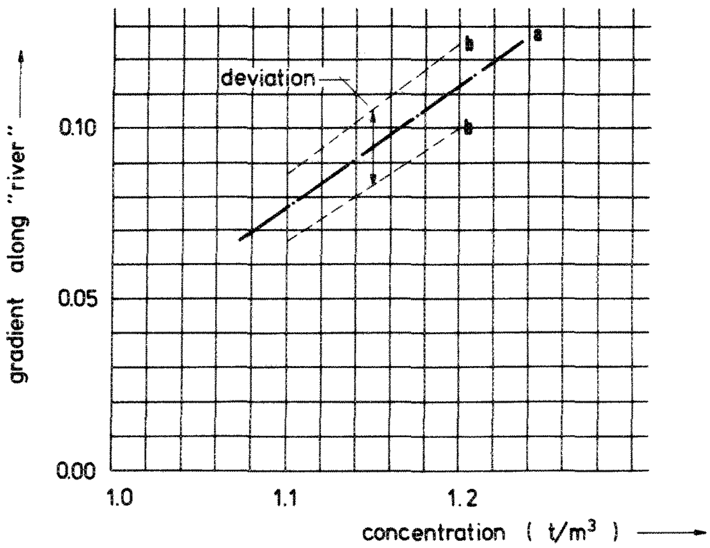
21
Result of large scale test, imprints of formwork used for test slab are clearly seen in the sand deposit.
Photo: Ballast-Nedam/Dredging Division

Research in the Field of Foundation Methods
The development of tunnel foundation systems in the Netherlands during the past 15 years involved the carrying out of a large number of experiments, many of them to full-scale, in order to ensure that the foundation would work out as intended.
As an example of this research, a short outline of the research program concerning the sand flow method will be given.
In order to understand the mechanism of the process small-scale tests were carried out at first. The test models were built from plexiglass, so that the whole process could be clearly observed. In the small-scale tests no attempt was made to measure concentration or upward pressures, as it was considered that the scale laws would be rather complicated. After completion of the small-scale tests a full-scale test was carried out.

For that purpose a large concrete model was built. A concrete slab of 10 x 10 m was suspended in a slightly larger concrete basin in such a way that a sand-water mixture could be injected through a discharge opening in the middle of the slab (*fig. 20*).
The slab was supported on the four corners by electrical load cells. Further it was provided with observation windows, water pressure meters, soil pressure meters and openings for making cone soundings. Discharge and concentration of the sand-water mixture were recorded as well.
After completion of one experiment, the slab was floated in order to enable inspection (*fig. 21*). In this way a large number of experiments was carried out using different types of sand and varying discharges and concentrations.
As a result of the tests it was possible to quantify the process, particularly the relation between the concentration of the mixture and the gradient along the sand 'river' (*fig. 22*).

Summary
The article is dealing with various aspects of the research, design and construction of foundations for immersed tunnels in the Netherlands. Emphasis is given to the foundation type where the tunnel is bearing directly on sand which is placed in situ by special methods. Attention is given to the development in the field of these methods, the theoretical and physical properties as well as the practical applications. Besides, the article describes shortly the conditions leading sometimes to application of other types of foundations and principles of the way they were actually constructed.

Later on, small size tests were executed on the effect of bottomslabs being spherical instead of flat. Furthermore there has been done theoretical research for the appliance of the sand flow system for the foundation of sea platforms of the gravity type. Other structures seem to offer an interesting field for the appliance of this method.



22
Relation between concentration of sand-water mixture and gradient along 'river'
a. mean values
b. limits of test results

Prof.Ir.H.P.S.van Lohuizen
Delft University of Technology

Research done by Tunnelling Section of the Royal Institution of Engineers in the Netherlands

Introduction

Since Dutch tunnelling specialists were being confronted with quite numerous unsolved questions despite the knowledge and experience that could be developed in this country with its specific soil conditions, various study groups were inaugurated in recent years.
I will try to give you an impression of the results obtained by seven study groups up till now. Since, however, the work performed covers a much wider range than the subject of this symposium, I will not be able to give you all the results.
For those who are interested in going further into the material produced, the complete reports, in as much as they are still available, will furnish the desired information.
One further remark should be made at the outset however. The study results shown in these seven reports do not cover all that has been done in the Netherlands, of which only a part was published up till now. Partly the reports may be out of date since further studies may have been performed since.
A point stressed by several of the study groups which I would like to strongly support, is that much research work and many results of measurements were never published neither in this country, nor in other countries.
I think it is very important that every one having data of interest at his disposal should publish them somehow. In this respect it might be a good suggestion to have research work better organised and coordinated between all our countries. Perhaps the International Tunnelling Association could take an initiative in this respect as we have done in our local situation.
The subjects being studied are the following:

- Watertightness of immersed structures (study group no. I);
- Measures taken to prevent immersed structures from floating (study group no. II);
- The influence of temperature differences on tunnel structures (study group no. III);
- Recharge well pointing (study group no. VII);
- Cost aspects of tunnelling (study group no. V);
- Ventilation of tunnels
  rail tunnels (study group no. IV);
  road tunnels (study group no. VI).

The reports were published in the period from 1973 to 1978.

Watertightness of Immersed Structures

The study group reviewed the requirements concerning watertightness of tunnel structures. These requirements can be formulated from the point of view of the use that is going to be made of the tunnel but there are also requirements as to the maintenance and preservation of the tunnel structure itself. During tunnel construction several measures can be taken to meet these requirements.
The first thing that is necessary is to determine what degree of watertightness will be required from the user's point of view. Does it concern a tunnel structure that has got to be dry under all circumstances, is some leakage now and then permissible or may the structure have so much water leaking through that the water can be removed economically by pumping.
These requirements together with the water pressure outside the structure mainly decide on how the tunnel units as well as the joints have to be constructed.
Watertightness can be obtained by an extra cover around the tunnel structure that is often made of steel plating, metal foil, or by several layers of bitumen.
Another way which is becoming more and more popular today is to take special measures during construction to prevent cracking and make the concrete as a construction material more waterproof.
The period over which the structure has to maintain its watertightness may also be of importance. An auxiliary construction like a sheet pile wall on the one hand, or a permanent construction like an immersed tunnel tube on the other, ask for quite a different approach.
The report gives an inventory of the different construction systems in use;

- the cut- and cover method (open trench);
- the under-the-roof method;
- pneumatic caissons;
- the immersed tube method;
- shield driven tunnels.

Since the immersed tube method is the issue of this symposium I will give some of the conclusions concerning this construction system only.
Since tunnel units as used for the immersed tube method are generally built in an open trench, both ways of waterproofing are applicable.
When a watertight cover is made special measures must be taken as to the transportation of the units. The bottom cover, for instance, may not be damaged when the tunnel element is detached from its supports during floating. Also the cover of walls and roof have to be very well protected.
Waterproofing of the joints is a very important factor that largely characterises the immersed tube method.
The report then deals extensively with the different construction methods and joint constructions.
The next chapter is written of tunnel constructions without a separate waterproof cover. Attention is given to all factors that may lead to cracking and how the latter might be prevented.
A further chapter is dedicated to tunnels with a waterproof cover furnished either by steel plating, by aluminium or copper foil, or by different layers of bitumen or synthetic materials.
Coping with possible leakages might prove to be problematic because it is often very difficult to trace the spot where the waterproof cover is damaged.

In tunnels without waterproof cover good results can often be achieved by local injections.
The study group concludes that a tunnel structure can be made sufficiently waterproof without a special cover.
They have found that frequently the designer of the tunnel knows too little of waterproofing problems and relies too much on the knowledge of the supplier of the waterproofing material.
Designers should generally work out the system of waterproofing together with supplier and laboratories to get a better knowledge of the technological characteristics and applicability of synthetic materials.
The study group further advises that new materials should be tested that seem to have good chances on smaller projects where watertightness is not so critically important.
It is of great importance that experience obtained with new materials becomes known as soon as possible.
The study group stresses another very important point of which very little seems to be known, i.e. the subject of stray currents and the danger they represent for concrete structures.
Especially the reinforcement steel can be very badly affected by them.
In the meantime a Dutch FIP study group has done some research on this point which yielded some interesting results.

Measures taken to prevent Immersed Structures from Floating

When an immersed tube and its connecting abutments and open approaches are build, one of the very important points is to make sure that the structure is not going to float.
There are several possibilities to make a construction stable when immersed. It is, however,

considered unattractive to use more material in order to add weight because that makes the structure generally more expensive.
The study group working on this problem only concentrated on Dutch soil conditions; rock formations have therefore not been taken into account. Furthermore vibrations in the subsoil caused by earthquakes were not considered. The cost aspects have not been evaluated since local situations influence this factor so heavily that a general conclusion was not possible.

The report first reviews the measure taken with the immersed tube itself to prevent floating.
These comprise, e.q. adding more weight to the tunnel unit by placing an overburden on the roof.
In addition the side walls of the tunnel can be made rough to add more friction resistance and even a little oblique so that the tunnel cross section becomes trapezoid, which imparts to the tunnel additional resistance against upward forces.
Measures can also be taken underneath the tunnel by mobilizing the weight of the subsoil through tension elements.
Since not enough was known of this subject quite an extensive research programme has been carried out on piles, anchor rods, anchor rods together with injection, anchors screwed into the ground, etc.
Another group of measures that can be taken is to reduce the upward forces developed by the groundwater by lowering the groundwater pressure. This is generally not applicable where there is any danger to adjoining structures. This may, however, be compensated by recharge well pointing, by creating a waterproof membrane, by injection, by applying a sheet pile wall, by making a narrow slit filled with bentonite as is also used for diaphragm walls etc.
The main point to be considered here is that underneath the structure the soil must be very permeable so that no water pressure can ever build up again. Drainage of these underlying layers must be done on a permanent basis. Injections that were performed and will have a permanent function must remain intact.
The report then goes in more detail concerning the question of how much ground can be mobilised to act as a counter weight, by the different kinds of tension elements mentioned *(fig. 1)*.

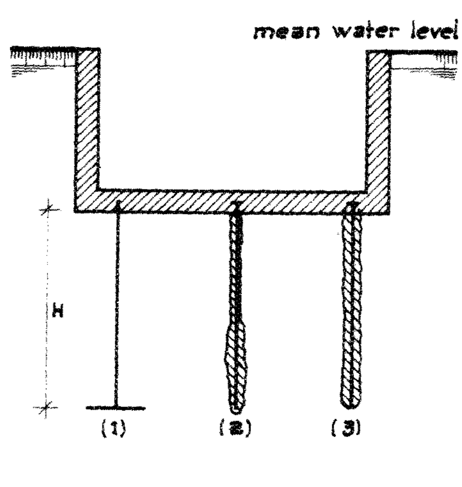
1. Tension elements with anchor plates perpendicular to the anchor rods at the foot, screwed into the ground or inserted by vibrating it into the ground (the so called 'rüttel verfahren').
2. Tension elements anchored to the lower side of the rod, called injection- or grout anchors.
3. Tension structures where the anchoring capacity is mobilised along the whole length of the unit as for instance prefabricated piles or piles cast in situ in the ground.

Vibrated or screwed anchors
Vibrated or screwed anchors are realised by inserting into the ground steel plates to which the structure that is to be anchored against floating is tied with rods. When the anchor plate is not located very deep, it is just the cylinder of ground above the slab that provides the counter weight. When the anchor slab is situated much deeper then the cylinder is not only taller but also spreads more beyond the diameter of the anchor slab. When tension elements are applied as a group, each element will influence the others. Then again practically the whole block of soil between the bottom of the structure and the foot of the anchor system may be taken into account.
When screw anchors are aplied we should consider that the whole cylinder of ground has been disrupted by the insertion of the anchor plate into its position. A strongly reduced tensile force will then result. On the other hand in the case of vibration anchors, especially in sandy soil, very good density is achieved because of the vibration of the soil when the anchor slab passes. As a result these anchors can absorb much larger tensile forces than the screwed anchors.
In the Netherlands screw anchors, like vibrated anchors, are not often used.

Injection- or grout anchors
Here the forces are carried by friction along the rod where high pressure injection has been applied.
There are some well known systems for the production of these anchors.
Since the succes of injecting the grout largely depends on local soil conditions, much care has to be taken in assessing the allowable tensile force.
It was found from experiments with oblique anchors for sheet pile walls that very large tensile forces can be coped with succesfully.
On vertical anchors, however, very few tests have been done up to now.
Generally speaking, it is advisable to test-load each anchor after it has been placed.

Tension piles
These are used much more extensively in this country. Many systems have been developed over the years. The report deals extensively with prefabricated and made in situ concrete piles, steel piles, and wooden piles.
Not only the well known systems are being dealt with but also some newly developed systems are mentioned.
Since a tension pile has to derive its resistance against pulling mainly from friction within the surrounding ground, the method of producing the piles is of very great importance.

Figure 1



Extensive experiments have taken place in recent years of which the report gives many detailed results. The study group concludes that knowledge of, and experience based on these and other results have grown to such a degree that permissible tensile forces can reasonably be predicted for different pile systems in given soil conditions.
In the report two calculation methods are given. The first method was introduced by the Delft Soil Mechanics Laboratory in 1969 and is based on mantle friction results. The second method uses the friction characteristics of the ground itself. The first method is mainly based on the total friction between ground and pile shaft which is found by integration of the local friction over the total length of the pile. The second method states that most of the tensile force is given by the friction of the ground on the pile shaft in the lower part of the pile only.
Neither calculation method however is based on a theoretical analysis of the problems. Some coefficients had to be introduced which were specially derived for Dutch soil conditions. In soil conditions differing from ours the formulae should therefore only be applied with great care until new coefficients have been elaborated.
Another point to be considered is whether the pile is being pulled at the bottom or at the top. In the first case only compressive stress is exerted on the pile shaft. When pulled at the top tensile stresses will be the result which can easily lead to cracks. The deformation of the pile will rapidly increase when cracks are starting to form.
There is no opportunity now to go into the details of the extensive information given in the report. However, I would like to draw your attention to the good results from experiments with wooden piles on which many of our structures have been built for ages.
In conclusion the report yet stresses some important points to be considered.
A very considerable advantage of tension piles over the other anchoring systems is a much more rigid foundation.
Tensioning the rods of both vibration- and grou ted anchors before the required tensile force i exerted on the structure will not only result i deformations taking place, but also the ancho will not be able to absorb any compressive fo ces. Piled foundations can take both tensile ar compressive forces, which is of considerab importance in Dutch soil conditions.
Further the report states that since nowada more and more underwater concrete is appli there is a tendency again toward structu where the deadweight is much higher. This be necessary often because we are no lon allowed to lower the ground water table du construction. However, ballasting material der water has only a limited effect; moreov requires deeper excavation with longer s pile – or diaphragm walls. Mostly it will there turn out to be a more expensive solution. advantage however is that these construc are much more safe, especially under ext circumstances when for instance liquifacti sand may be caused by an earthquake.
The advantage of prestressing or post- oning of anchor rods on the other hand i each anchor can easily be test-loaded.

Permanent anchors must be protected a stray currents that may occur around me railway tunnels.

The Influence of Temperature Differences on Tunnel Structures

The study group was asked to report on the following items:

- what kind of temperature changes may have an effect on tunnel structures;
- to design a method for calculating the temperature effects;
- to provide for measures that can be taken in the structure to prevent excessive stresses.

Since there was only about a year available to compile a report, the study was limited to the behaviour of completed tunnel structures under temperature effects coming from outside. Effects caused by the hardening process were therefore not studied although the study group stresses that this has to be done also.

The work that has been done gives a state of the art of the knowledge and experience gathered in the Netherlands.

There is a striking statement namely that little on this subject could be found in publications from abroad except three extensive German articles on the Rendsburg tunnel. Since most of the US immersed tube tunnels are built with a double steel mantle, temperature cracks will not have much influence if they occur at all. This does not apply however to other tunnels with a rectangular cross section. Often even these will have to operate under extreme temperature fluctuations.

The report is divided into two main parts.

1. Basic information for static calculations. In this chapter temperature data are given for concrete designers unfamiliar with tunnelling techniques.
2. Detailing of the structural design; a further explanation is given for the concrete designer how to use the acquired data in his calculations.

The report then goes into details on how to assess the temperature gradient through the wall of a tunnel in different situations and how to calculate both in transverse as well as in longitudinal direction the stresses caused by temperature differences.

In this respect tunnels differ from each other to some extent according to the purpose for which they are used.

Road tunnels are generally very intensively ventilated, hence no gradual building up of temperature occurs. A rail tunnel however is often much longer and the cross section is much smaller in comparison to the trains passing through. A forced ventilation system is not applied often. Because of these conditions in a rail tunnel, gradual heating up can take place more easily.

The tube might be compared to either a cooling or a heating pipe, depending on the temperature of the surrounding ground water.

The way in which the immersed tube rests in the ground and the distances between joints indicate what movements can be made in longitudinal direction.

After some examples showing how to calculate the effects of temperature fluctuations on tensile stresses in the concrete structure some very interesting conclusions are drawn.

When data of the thermic behaviour of the materials used in the stucture are determined, the temperature drop throughout a concrete section can be assessed rather easily.

The thermal characteristics of the materials are generally sufficiently known. The boundary conditions inside the tunnel are predictable from existing data or can be determined from calculations. At a given temperature of the surrounding groundwater the temperature gradient in the tunnel wall can be determined. Only little is known of variations in groundwater temperature, although it never will be more than a few centigrades.

Once the temperature fluctuations and the resistance of the structure against transformation have been determined, the resulting forces and tensions can be calculated. The role that temperature effects play in the total bearing capacity of the tunnel structure is, however, generally not so great.

There are acceptable theoretical calculations on the resistance of a tunnel structure against temperature effects in the longitudinal direction; these, however, have not been sufficiently borne out by test results so far.

With the trend towards ever wider tunnel sections, 50 m and more, the ground resistance perpendicular to the axis should also be taken into account.

Tunnel elements lying in a gradient have the tendency to creep gradually downward because of temperature fluctuations. No sufficiently clear observations exist regarding this phenomenon. Only measurements on joints have become available.

Forces caused by temperature have no influence on the load condition at failure and hence neither on the safety of the cross section of a tunnel, assuming that the rotation capacity of the determining sections is sufficient to allow for redistribution of the forces. Temperature tensions however do influence the normal load situation. Cracking of the concrete and sometimes even bending, especially in combination with tensions caused by other loads and by shrinkage may grow worse when temperature effects are added as well.

One would expect that tensions caused by yearly temperature cycles would be largely reduced by relaxation; this, however, is only partly true.

Attention should also be paid to transformations caused by temperature changes that can hardly, if at all, be prevented. These transformations might influence the width of cracks and thereby the watertightness.

Trying to prevent crack formation by applying more reinforcement steel generally does not work; only the width, and the distance of cracks from each other could perhaps be influenced to some extent.

When too little reinforcement is applied in the longitudinal direction, the steel will yield as soon as the first cracks appear. The forces caused by the load will not be redistributed because the structure is statically determined. A tunnel cross section however is usually statically indeterminate; hence redistribution of the forces is adequately possible; when cracking occurs, failure is not expected.

When a tunnel and two adjoining ventilation buildings are joined together tensile stresses may result.

In long tunnels, also in cases when they are not connected to ventilation buildings, and composed of units fixed to each other, a temperature drop will cause movements. Ground friction will be the result; this will again cause tensile forces to arise in the longitudinal direction, increasing towards the centre of the tunnel.

To prevent cracks as described above expansion joints must be made.

Knowledge has so far progressed by now that a reinforcement can be designed which will not yield for bending moments imposed by temperature differences which lead to exceeding the permissible tensile stresses. However, very little is known about the crack pattern that will occur. When these stresses have to be combined with tensile stresses of other origin (other temperature influences, shrinkage, hydration heat, settlements etc.), no advice can as yet be given regarding the necessary reinforcement.

The temperature gradient through the concrete structure of a tunnel can be favorably influenced by adequate insulation on the side where the largest temperature differences occur.

In the building of tunnel units attention should be given not only to the effects of hydration heat but also to direct heating by the sun, followed by a rain shower or frost at night which can cause large temperature gradients. There are however hardly any data on the extent and the depth to which these temperature influences penetrate into the structure and in how far hair cracks caused by this effect may be harmful.

It seems worthwhile to study whether the concrete cover that is anchored to the structure by rather light anchors only is influenced by temperature changes and what extent the forces assume that work on the anchors. Subsequently we should consider whether the concrete cover should also be divided by expansion joints.

Recharge Well Pointing

One of these days research results on recharge well pointing will be published. The study group considered mainly three items:

- an inventory of projects implemented;
- an inventory of the circumstances under which recharge well pointing is a realistic possibility;
- to draw up a set of requirements to which an installation of recharge well points must answer.

Inventory of projects implemented

To answer this item an inquiry was set up. Twenty-one questionnaires were sent out of which fifteen came back.

It turned out that the following facts had given rise to difficulties:

- oxygen was shown to react with iron that was present in the ground water forming iron oxyde which is insoluble. In this way recharge wells were choked up;
- gas originally being dissolved in the water may form bubbles when the water pressure is going down and as a result might choke up the recharge well;
- sulphate reduction by which reaction iron sulphate might be changed into iron sulphide by the action of bacteria. This again might clog both the well and the recharge well. Moreover the bacteria grow and may choke the wells also;
- sand sometimes penetrated into the wells because insufficient attention had been paid to the necessity of designing a filter bed round the well that was in accordance with the local soil information;
- because of the over-pressure in the recharge well it is inclined to rise, and ground water will consequently creeping upwards along the well.

It was not possible nonetheless to get a clear impression of interdependence of the results of a project and the materials being used, the soil conditions that were met, and the output of the wells.
The results of long term projects varied from 'not so good' to 'very good'. There were only four projects that ran into really severe difficulties. In three of these cases the problems were however solved after some time when the expected results were reached.
One project, an underpass in Zeist, was unsuccessful. This was the only drainage project in phreatic water where the groundwater table had to be kept intact because of a near-by park. Through porous upper layers rainwater containing much oxygen could reach the groundwater that was originally rich in iron. In the pumping wells an intensive mixing of both types of water took place which led to the forming of the already mentioned insoluble iron oxyde with subsequent choking up of the recharge wells.
Possibly aeration of water in the wells took also place because of the forming of a seepage zone. Mixing with air in the pump may have led to extra aeration. Aeration via the filter bed round the well points is however most likely because this filter bed was continued upward into the top layers, also dewatering these.
The study group thinks that also in this case it should have been possible to find ways to render the method successful. For this purpose an experiment with recharge well pointing will be started in November 1978.

Inventory of the circumstances where recharge well pointing will be realistic
The study group has come to the conclusion that in the case of upper layers of low permeability it will be technically possible to recharge groundwater sufficiently just the same.
In each new situation however a cost-benefit analysis will have to be made in comparison with other possibilities such as underwater concrete, injection of the subsoil, pneumatic caissons, ect.
Situations comparable to the Zeist project give as yet rise to doubt whether they can be succesful. The study group hopes that the experiments will provide an adequate answer so as to achieve good results also in similar cases like this in phreatic groundwater.

A set of regulations which an installation of recharge well pointing must satisfy
The results of the inquiry and some theoretical studies have led to considerable requirements for a project where recharge well pointing can be sussessfully used.

Cost Aspects of Tunnelling
The study group investigated all possible methods of tunnel construction in use in the Netherlands. A description of each method is given; they are divided into three main groups according to the 1970 OECD conference at Washington DC:
- Immersed tunnelling;
- Cut- and cover constructions;
- Soft ground tunneling.

The fourth group, rock tunnelling, does not apply to Dutch soil conditions.
In order to give a realistic cost comparison all designs were made for the same double track rapid transit system tunnel. The soil- and groundwater conditions were standardised to a Dutch situation as we might find in the West of the country.
For each design the cost per linear metre of tunnel was calculated based on total lengths of 400, 800 and 1200 m and an overburden as cover of 4.5 m and 1.5 m and where that was realistic, even 0 m. The costs, given in Dutch currency, include only the civil engineering part and are based on a cost level of 1972. Taxes and design costs are not included either.
The final selection of a construction method can never be solely based on costs. Attention has to be given to the following items as well:
The effect on the environment of the construction system can be of great importance especially in densely populated areas and streets with heavy traffic congestion. Settlements by well point drainage caused by lowering of the groundwater table may affect foundations of buildings and may cause damage to streets and public service mains. Noise production by ramming sheet piling or concrete foundation piles, by power stations and by pouring and vibrating concrete must be taken into account.
The report gives an interesting example of how a cost analysis can be made for a given situation and where the main aspects to be considered can be included.
It is worthwhile also for those who want to see at what point immersed tunnelling is becoming of interest. There are however many factors that were not taken into account, such as the underground station construction which often will be the most expensive part of an underground railway system. No attention has been given to comparing construction methods for road tunnels either. Moreover for present-day use it gives price indications only, comparing different construction methods since costs have gone up considerably after the report was released.
Because of the relative value of this report today and the fact that it does not specifically apply to the immersed tube method I will not go into more detail now.

Ventilation of Tunnels
Rail tunnels
The study group was asked to review problems of ventilation of rail tunnels both for metro- and railway use, and to try and find practical solutions where necessary.
A study of available literature comprised: theoretical calculations of air movement; heat production in tunnels and stations; thermo-dynamic problems; measurements of speed of air movement; as well as some specific problems of rail tunnel ventilation.
In underground rail transport, requirements as to heat production and draught play an important role for the public. Diesel engines will produce exhaust gasses.
The theoretical approach is much more complicated than for road tunnels, because of the much more complicated flow pattern of the air in a tunnel profile fitting much closer round the train profile.
Heat production is mainly caused by the train and can be calculated rather easily. It takes place only periodically just before and in a station during braking or the pulling out of trains.
The bulk of the heat disappears via ventilation shafts and staircases, a relatively small amount of heat is taken up by the walls.
Local climatological circumstances together with the type of trains, the way they are run, the cross section of the tunnel and stations and whether the tunnel is situated very deep under the ground or not are main factors to be taken into consideration. A lot of data are given concerning these items.

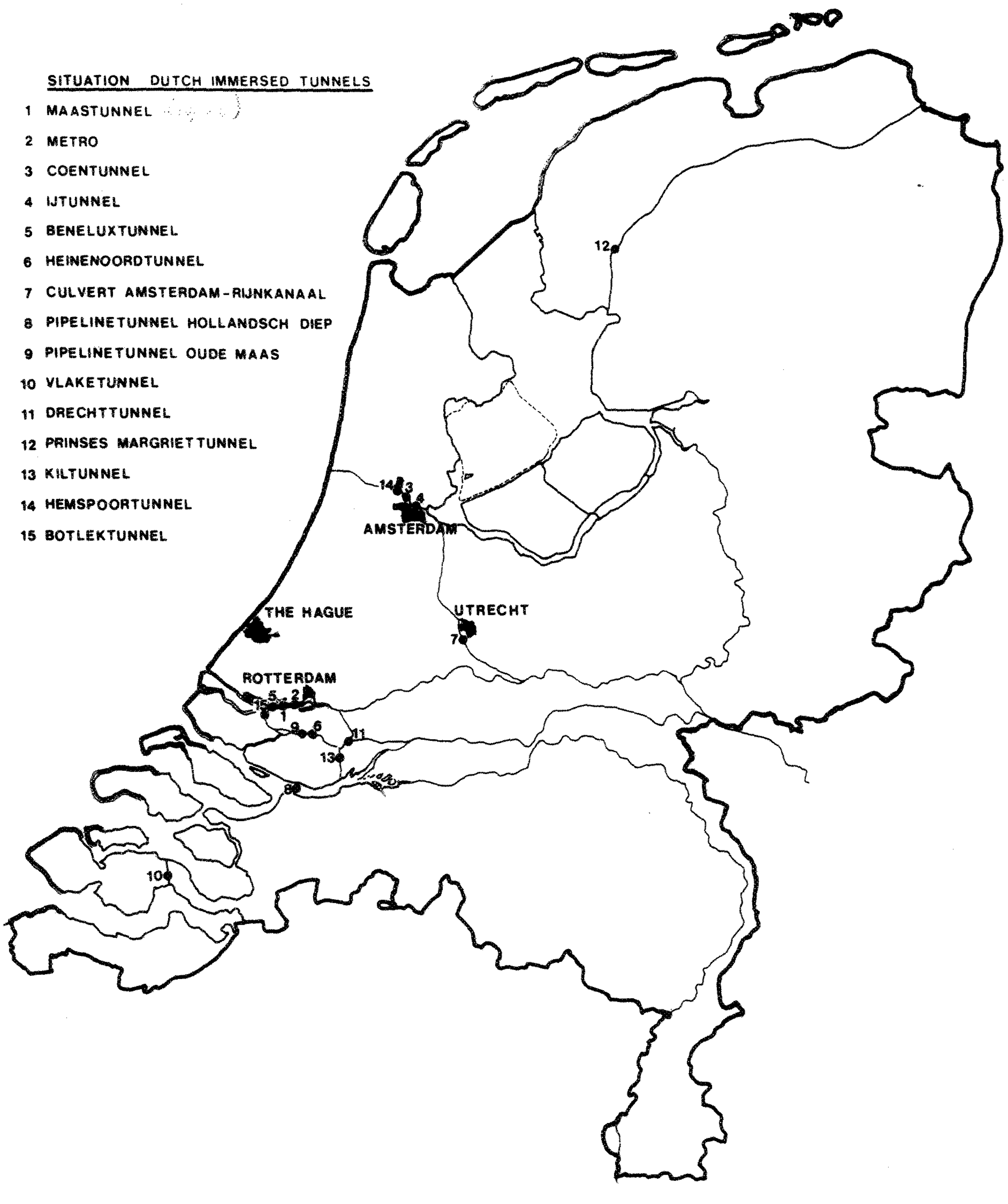
Road tunnels
The report dealing with ventilation of road tunnels has reviewed the following items extensively:
- CO-emission standards that are considerably lower than those usually applied, based on new measurements of the CO-emission of passenger cars;
- a new approach to the CO-threshold limit, taken as a function of the tunnel length;
- a modified method for determining the tunnel ventilation system in case of longitudinal ventilation by booster fans, based on fullscale experiments in a tunnel.

Since tunnel ventilation is not a specific problem of tunnels built according to the immersed tube method these reports will not be dealt with in greater detail now, although they provide a lot of important information on a subject of which too little is known.

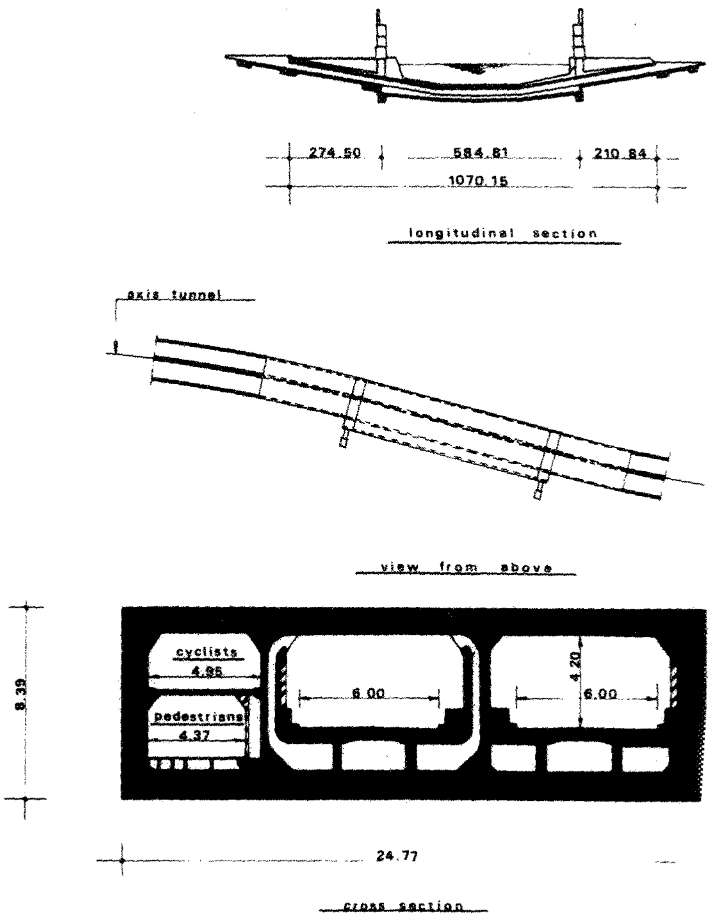
Summary
The activities of the Tunnelling Section include the studies carried out by working parties. The paper will give a short description of their main subjects and findings; costs of different construction methods; waterproofing; temperature effects on tunnel structures, ventilation; methods to prevent uplift of tunnels in ground water and recharge well pointing.

Tunnels in the Netherlands

The techniques described in the previous articles have lead to 15 tunnels so far (1978). The most important information of those tunnels follows on the next pages.
One remark must be made here: the costs are in Dutch guilders and are spent in the period of the construction!
It is self-evident that not only immersed tunnels are built in the Netherlands, so a few of other types are also mentioned. One should realise that this list of other types is not complete, there are just a few examples.
As said before the information of the tunnels (immersed or other types) is not complete. If you need more detailed information, you can contact the designers or involved contractors.

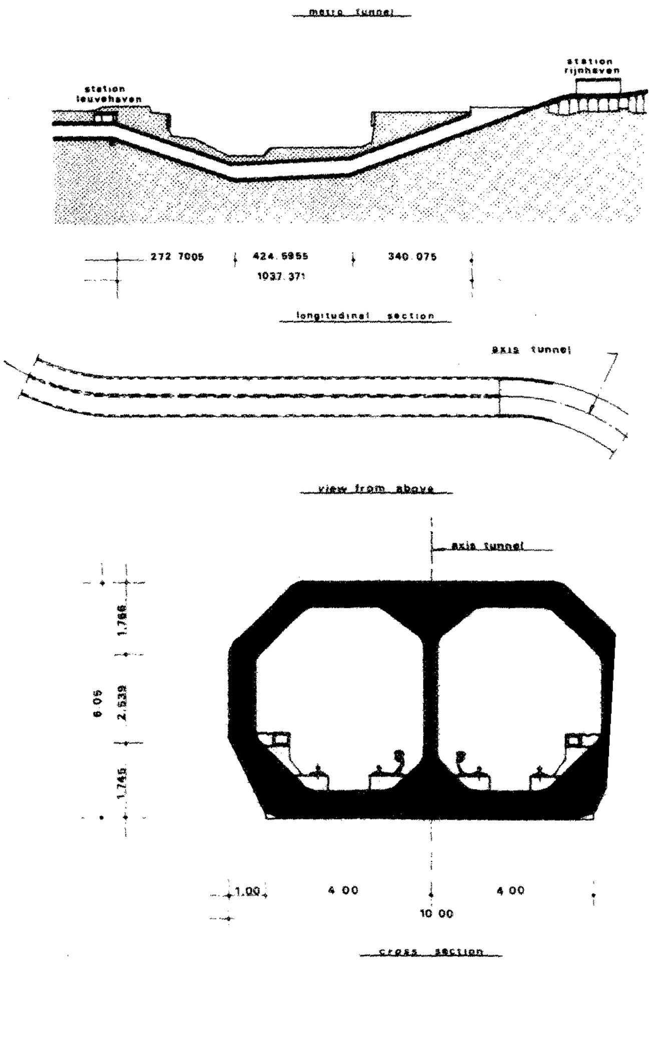


1. Maastunnel



Type:
roadtunnel (also for pedestrians and cyclists)
Location:
Rotterdam, under Nieuw Maas-river
Construction period:
1937-1942
Designer
Rotterdam Municipal Works
Advisors:
Christiani & Nielsen (sand jetting system)
Delft Soil Mechanics Laboratory
Laboratory for Aero- and Hydrodynamics at Delft (ventilation)
Construction costs:
f 18,000,000.
Method of construction:
9 immersed units of 62 m length, 2 pneumatically founded ventilation buildings, 2 open ramps on pile foundation.
Main contractor:
NV Maastunnel, joint venture of NV Nederlandsche Aanneming Mij (now member of Ballast-Nedam Group), Christiani & Nielsen, Van Hattum & Blankevoort (now member of Stevin Group), NV Internationale Gewapend Beton Bouw (now member of Royal Bos Kalis Westminster Group)
Subcontractors:
Franki (pile foundation)
Gusto – Penn & Bauduin (steel watertight lining)
NV Utrechtse Asfaltfabriek (bituminous watertight lining)
Suppliers:
Werkspoor (ventilation)
Philips (lighting)

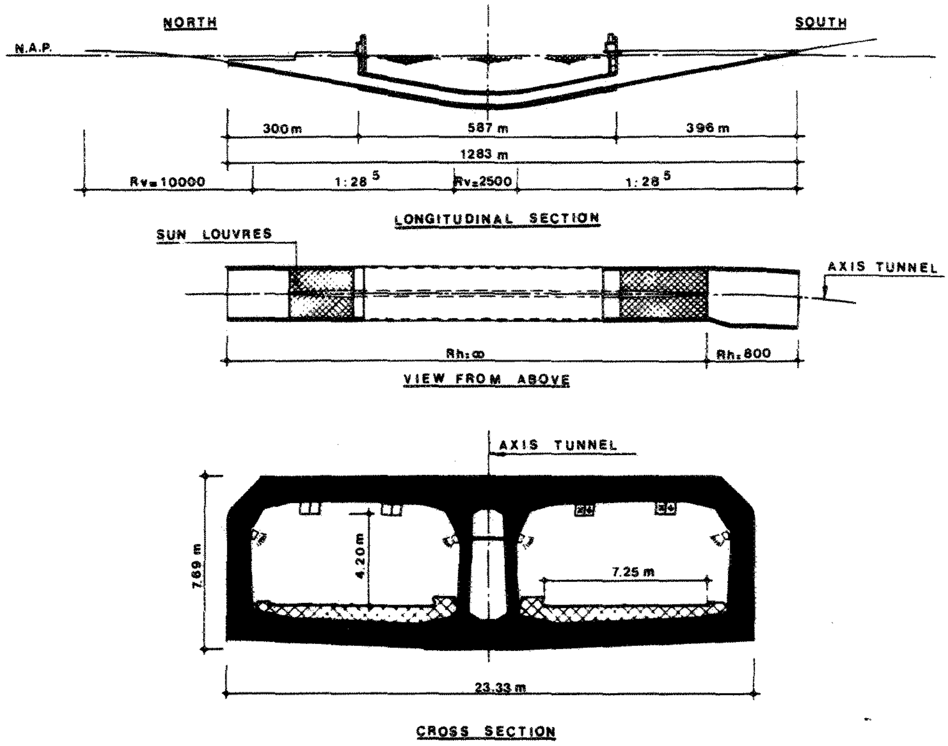
2. Metro



Type:
rapid transit railway tunnel (2 x 1 tracks)
Location:
Rotterdam, under Nieuwe Maas-river and in the city centre
Construction period:
1960-1966
Designer:
Rotterdam Municipal Works
Advisors:
Delft Hydraulics Laboratory
TNO Delft (Applied Scientific Research Organisation)
Stevin Laboratory of Delft University of Technology
Construction costs:
f 45,000,000 for river crossing
about *f* 15,000 per m^1 for land part
Method of construction:
8 units of 91 m, 4 units of 75 m length for river crossing
24 units of 65 m (including station units) for land part. The units for the land part are constructed in two artificial basins in the city-centre and immersed in waterfilled canals between sheet piling.
All units are founded on concrete piles with adjustable head to come to equal load for every pile.
Main contractor:
Combinatie Bouw Metrotunnel, joint venture of Christiani & Nielsen and Hollandse Beton Mij (member of Hollandsche Beton Group) for river crossing.
Koninklijke Nederhorst Bouw (now member of OGEM Building Division) for the land part
Subcontractors:
Royal Adriaan Volker (dredging)
Cindu Key & Kramer, now called Cekasol (watertight lining)
Troost Pernis (excavation and backfilling)
Rietveld (pile driving)
Suppliers:
Vredestein Loosduinen BV (watertight gaskets, type Gina and Omega)

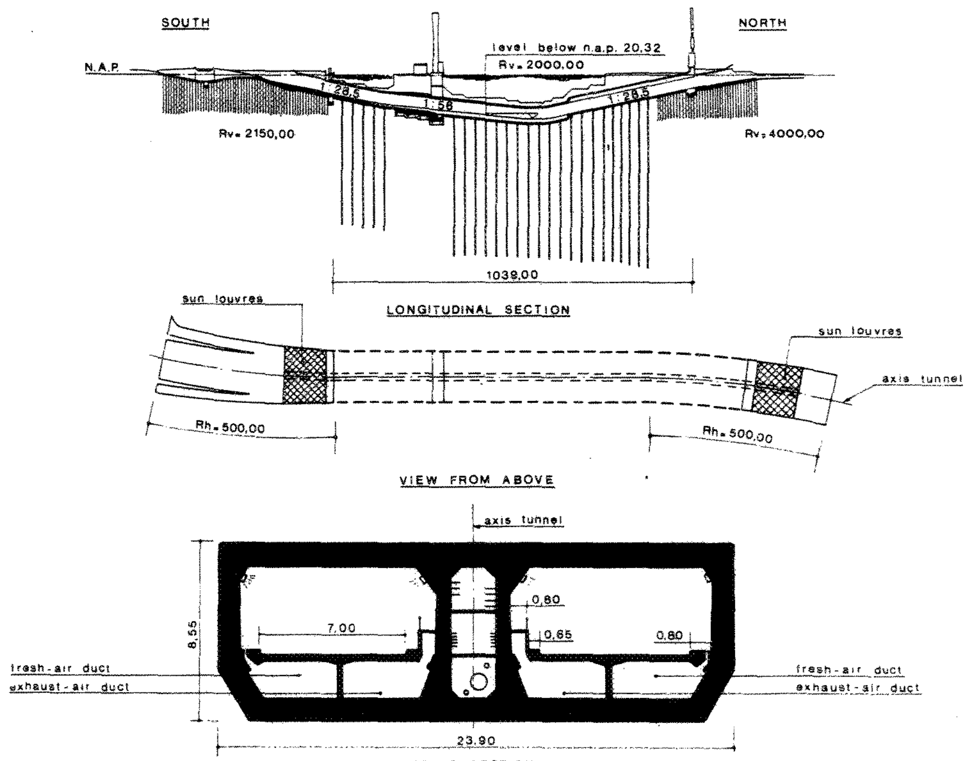
Coentunnel

pe:
ad tunnel
cation:
sterdam, under Northsea canal
struction period:
961-1966
esigner:
ional Public Works Department, Locks and
irs Division
dvisors:
elft Soil Mechanics Laboratory
ft Hydraulics Laboratory
ional Institute for Watersupply (well pumping
ystem)
rchitectenbureau De Vlaming, Salm en Fennis
chitecture)
nstruction costs:
0,000,000 civil engineering
5,000,000 equipment
ethod of construction:
mmersed units of 90 m length, open ramps
nded on piles made in open excavation
lain contractor:
nnemingsmaatschappij Hillen & Roosen BV
bcontractors:
ristiani & Nielsen (immersing of the units and
and jetting)
oyal Bos Kalis Westminster Group (dredging)
y en Kramer, now called Cekasol BV (water-
t lining)
an Swaay (lighting and control centre)
IV Apparatenbouw (sun louvres and sound absorbing ceilings)
Suppliers:
Vredestein Loosduinen BV (watertight gaskets)
Stork (ventilation)
Pannevis, now member of RSV Group (pumps)
Philips Nederland (lighting)



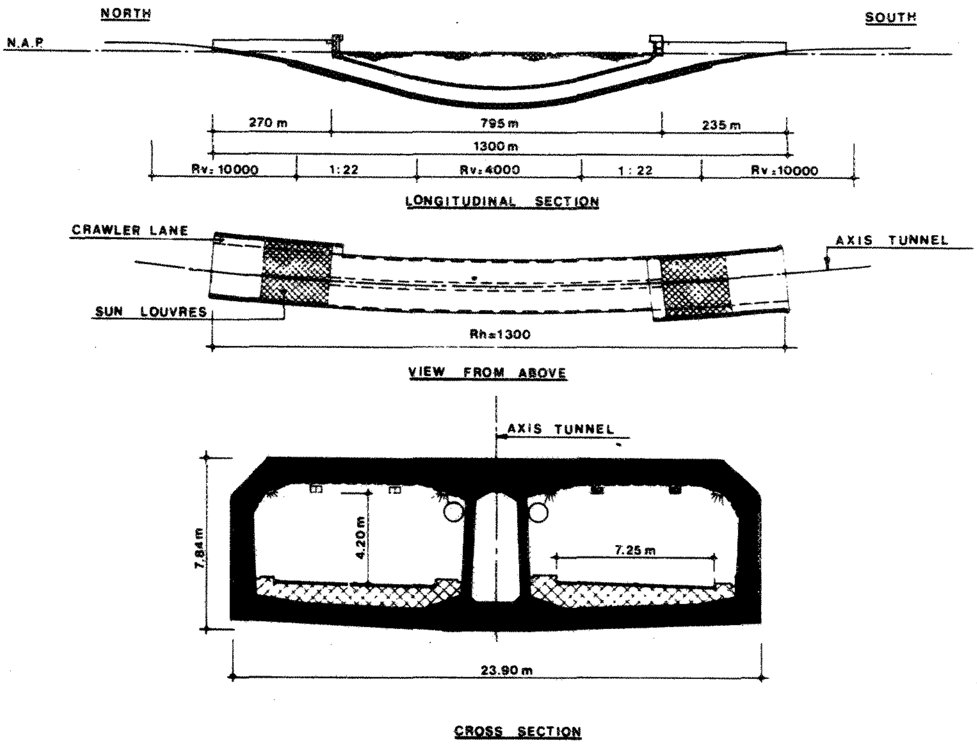
. IJtunnel

pe:
ad tunnel
Location:
nsterdam under IJ-river
nstruction period:
61-1968
Designer:
epartment of the Public Works Service of Am-
rdam
visors:
Delft Hydraulics Laboratory
National Institute for Watersupply
onstruction costs:
142,500,000 civil engineering
f 13,500,000 equipment
Method of construction:
immersed units of 90 m length, founded on
red cast in situ piles of max. 90 m length;
one section made by means of 4 pneumatic
caissons;
sections on prefabricated piles made in open
cavation between steel sheet piling walls;
e section on prefabricated piles made in open
excavation
ain contractor:
BIJT joint venture of Bataafse Aannemings
ij, Nederlandse Aannemings Mij (now member of Ballast-Nedam Group), Aannemersbedrijf
h J. P. Broekhoven, NV Tot Aanneming van
erken v/h H. J. Nederhorst (now member of
GEM Building Division), Hollandsch Aannemersbedrijf Zanen en Verstoep, Ways & Freitag,
hilip Holzman
bcontractors:
aden BV (well pumping)
Key & Kramer – now called Cekasol BV (watertight lining)
Nederlandse Staal en Aluminium Industrie (sun louvres and sound absorbing ceiling)
Groeneveld en Van der Poll – now member of GTI (electric cables)
Suppliers:
Vredestein Loosduinen BV (watertight gaskets)
Stork (ventilation)
Philips Nederland (lighting)
Schokindustrie BV (prefabricated piles)



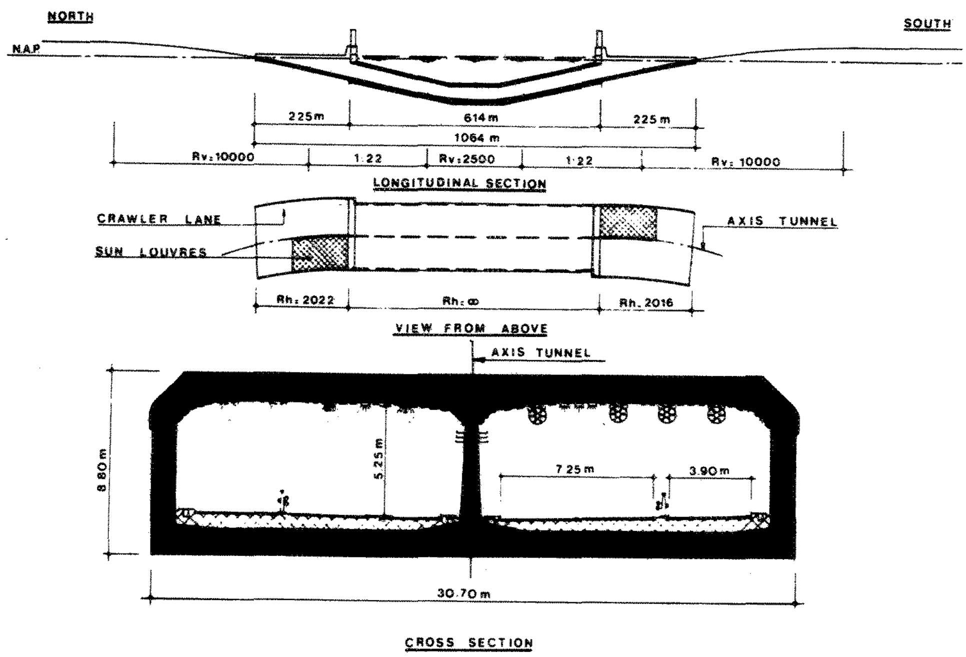
5. Beneluxtunnel

Type:
road tunnel
Location:
Rotterdam, under Nieuwe Maas-river
Construction period:
1963-1967
Designer:
National Public Works Department, Locks and Weirs Division
Advisors:
Delft Soil Mechanics Laboratory
Delft Hydraulics Laboratory
National Institute for Watersupply (well pumping system)
Architectenbureau De Vlaming, Salm en Fennis (architecture)
Construction costs:
f 72,000,000 civil engineering
f 8,000,000 equipment
Method of construction:
8 immersed units of 93 m length: steel sheet piling and under water concrete for the deepest parts of the open ramps; the other parts of the open ramps founded on prefabricated tension piles made in open excavation
Main contractor:
NV Nestum II, joint venture of NV Amsterdamsche Ballast Mij (now member of Ballast-Nedam Group), Christiani & Nielsen, Van Hattum & Blankevoort (now member of Stevin Group), Hollandsche Beton Mij NV (member of Hollandsche Beton Group), NV Nederlandsche Aanneming Mij (now member of Ballast-Nedam Group), Bato Jansen NV (now member of Royal Bos Kalis Westminster Group)
Subcontractors:
Key en Kramer, now called Cekasol BV (watertight lining)
Pre-system BBRV Spanstaal BV (prestressing of the units)
Van Swaay (control centre)
NV Apparatenbouw (sun louvres and sound absorbing ceiling)
Van der Bor (pumps)
Suppliers:
Vredestein Loosduinen BV (watertight gaskets)
Flygt (pumps)
Philips Nederland (lighting)
ACEC (control centre)
Stork (ventilation)
Derks Verf- en Lakfabriek NV (paint)



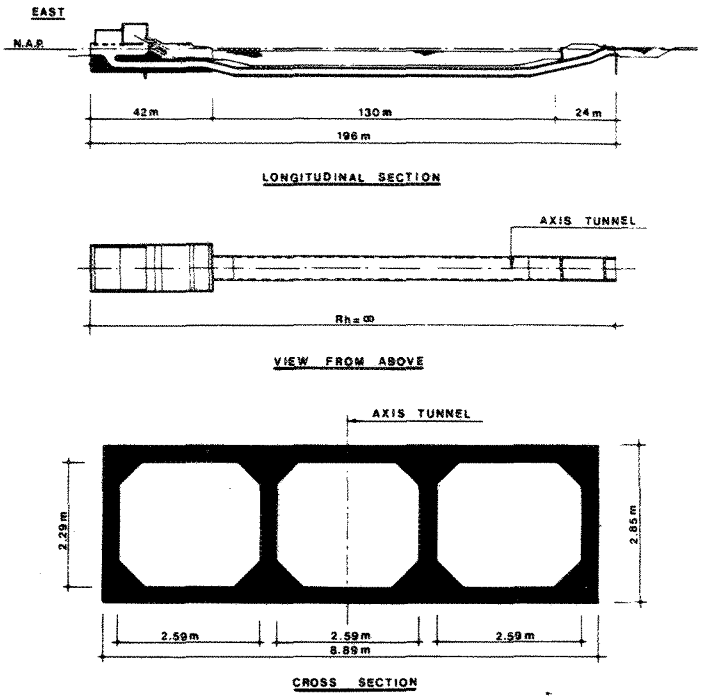
6. Heinenoordtunnel

Type:
road tunnel
Location:
near Rotterdam under Oude Maas-river
Construction period:
1965-1969
Designer:
National Public Works Department, Locks and Weirs Division
Advisors:
Delft Soil Mechanics Laboratory
Delft Hydraulics Laboratory
National Institute for Watersupply (well pumping system)
Architectenbureau De Vlaming, Salm en Fennis (architecture)
Construction costs:
f 52,000,000 civil engineering
f 6,000,000 equipment
Method of construction:
4 immersed units of 115 m length,
1 immersed unit of 111 m length,
open ramps on cast in situ concrete tension piles made in open excavation
Main contractor:
NV Nestum II see Benelux tunnel
Subcontractors:
Key en Kramer, now called Cekasol BV (watertight lining)
Pre-system BBRV Spanstaal BV (prestressing of the units)
NV Apparatenbouw (sun louvres ans sound absorbing ceiling)
Groenpol, now member of GTI (control centre and lighting)
Van der Bos (pumps)
Suppliers:
Vredestein Loosduinen BV (watertight gaskets)
Derks Verf- en Lakfabriek NV (paint)
Flygt (pumps)
Heemaf (standby equipments)
ACEC (control centre)
Stork (ventilation)
Philips Nederland (lighting)



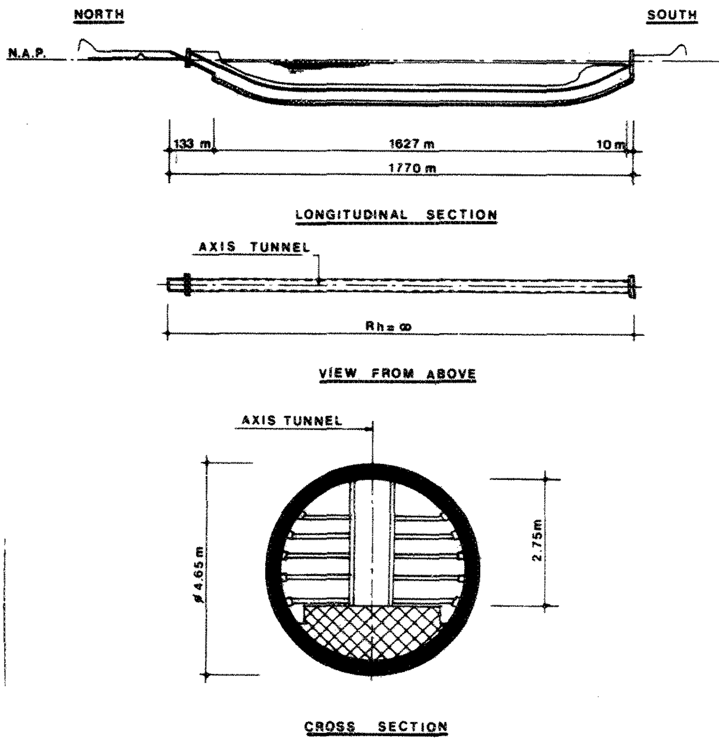
7. Culvert Amsterdam-Rhine canal

Type:
culvert for watertransport
Location:
near Utrecht under Amsterdam-Rhine canal
Construction period:
1970 – 1971
Designer:
National Public Works Department, Locks and Weirs Division
Advisors:
Delft Soil Mechanics Laboratory
TNO Delft – Applied Scientific Research Organisation (testing glue for joints)
Construction costs:
f 9,500,000
Method of construction:
1 immersed unit of 130 m length formed by 44 prefabricated parts.
The parts, of which 40 have equal dimensions and 4 near the ends are different, are cast countermoulded in a vertical position and coupled by means of prestressing and epoxy-resin in the joints.
The land parts are built in open excavation between steel sheet piling walls.
Main contractor:
NV Nestum II, see Benelux tunnel.
Subcontractor:
Pre-system BBRV Spanstaal BV (prestressing)



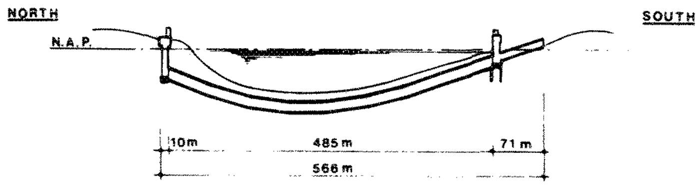
8. Pipeline tunnel Hollandsch Diep

Type:
tunnel for a great number of pipelines
Location:
under Hollandsch Diep
Construction period:
1971 – 1973
Designer: National Public Works Department, Locks and Weirs Division
Advisors:
Delft Soil Mechanics Laboratory
Delft Hydraulics Laboratory
Construction costs:
f 19,000,000 civil engineering
f 500,000 equipment
Method of construction:
27 immersed units of 60 m length. Each unit consists of 6 m long sections, cast in a vertical position and coupled by means of prestressing and injection of epoxy-resin in the 15 mm wide joints.
Land parts made by means of steel sheet piling and under water concrete.
Main contractors:
Kombinatie Tunnelbouw, joint venture of: Dirk Verstoep (member of the Royal Bos Kalis Westminster Group), Koninklijke Nederhorst Bouw (now member of OGEM Building Division)
Subcontractors:
Royal Bos Kalis Westminster Group (dredging)
Ibis (prestressing with Freyssinet system)
Boiidt BV (injecting with epoxy resin)
Suppliers:
Vredestein Loosduinen BV (watertight gaskets)

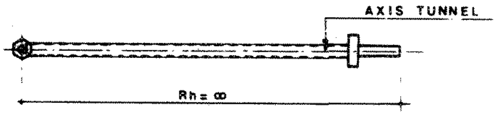


9. Pipeline tunnel Oude Maas

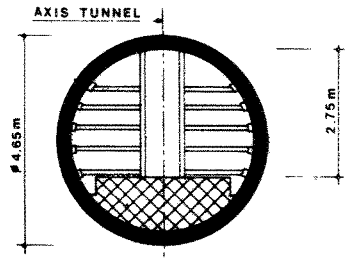
Type:
tunnel for a great number of pipelines
Location:
under Oude Maas-river
Construction period:
1972 – 1975
Designer:
National Public Works Department, Locks and Weirs Division
Advisors:
Delft Soil Mechanics Laboratory
Delft Hydraulics Laboratory
Construction costs:
ƒ 13,500,000 civil engineering
ƒ 500,000 equipment
Method of construction:
8 immersed units of 60 m length (similar as pipiline tunnel Hollandsch Diep)
One abutment by means of steel sheet piling and under water concrete, the other as an immersed caison.
Main contractor:
Kombinatie Tunnelbouw (see pipeline tunnel Hollandsch Diep) for immersed parts (tunnel and abutment), Bato Jansen BV – member of the Royal Bos Kalis Westminster Group (steel sheet piling and underwater concrete abutment)
Subcontractors:
Royal Bos Kalis Westminster Group (dredging), Ibis (prestressing with Freyssinet system), Bolidt BV (injection with epoxy resin)
Suppliers:
Vredestein Loosduinen BV (watertight gaskets)



LONGITUDINAL SECTION



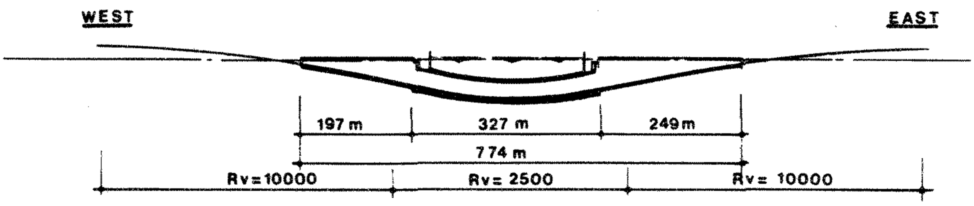
VIEW FROM ABOVE



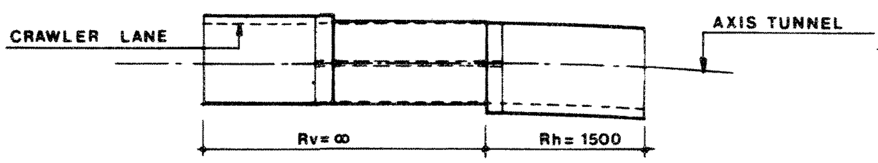
CROSS SECTION

10. Vlaketunnel

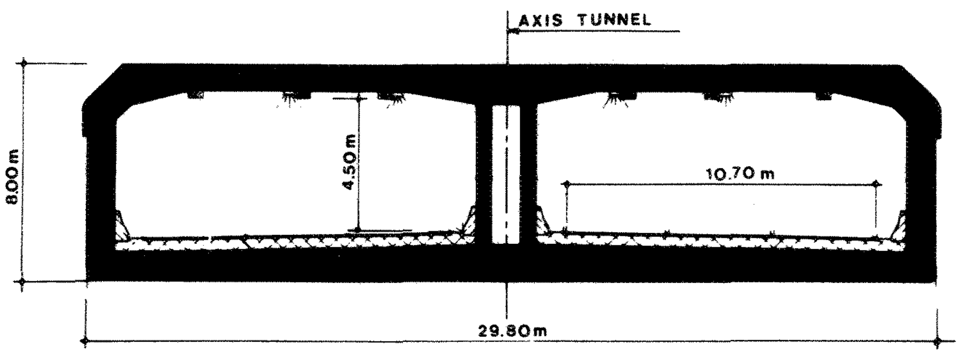
Type:
raod tunnel
Location:
under canal in south-western part in the Netherlands
Construction period:
1972 – 1975
Designer:
National Public Works Department, Locks and Weirs Division
Advisors:
Delft Soil Mechanics Laboratory
National Institute for Watersupply (well pumping system)
Architectenbureau De Vlaming, Salm en Fennis (architecture)
TNO-Delft – Applied Scientific Research Organisation (testing concrete for cooling system)
Construction costs:
ƒ 36,000,000 civil engineering
ƒ 4,000,000 equipment
Method of construction:
2 immersed units of 125 m lenght
open ramps on cast in situ concrete tension piles, made in open excavation
Main contractor:
joint venture of NV Amsterdamsche Ballast Mij. (member of Ballast-Nedam Group) and Hollandsche Beton Mij (member of Hollandsche Beton Group)
Subcontractors:
Nederhorst Grondtechniek – member of International Foundation Group (cast in situ Vibro-piles)
Pre-system BBRV Spanstaal BV (prestressing)
Emiss (injecteren)
Tjaden BV (well pumping system)
NV Apparatenbouw (sound absorbing ceiling)
Sigma Coatings bv (paint)
GTI (control centre and lighting)
Van der Bor (pumps)
Suppliers:
Vredestein Loosduinen BV (watertight gaskets)
NV Apparatenbouw (sound absorbing ceiling)
Eternit bv (Glasal plates)
Heemaf (standby equipment)
Philips Nederland (lighting)
Flygt (pumps)



LONGITUDINAL SECTION

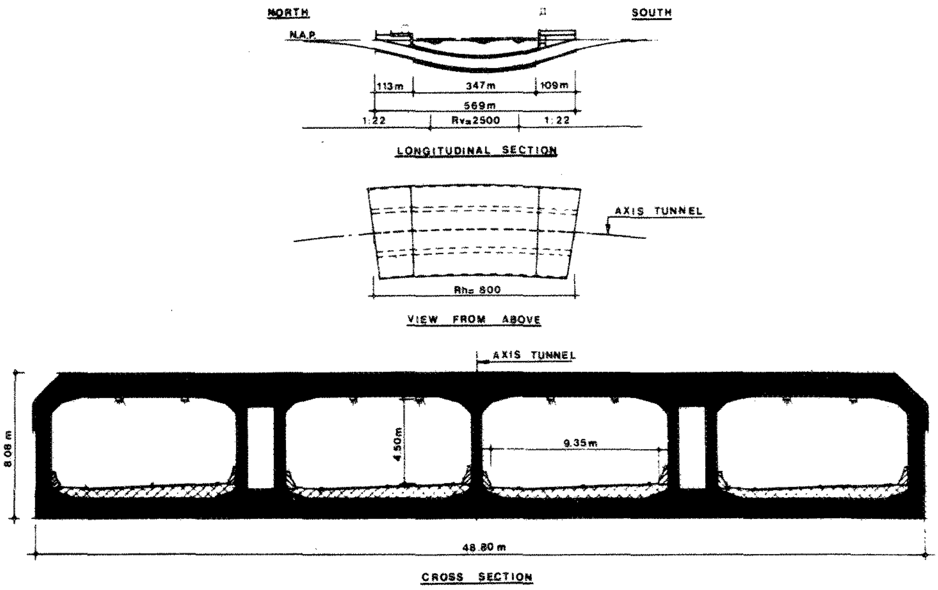


VIEW FROM ABOVE



CROSS SECTION

Drechttunnel

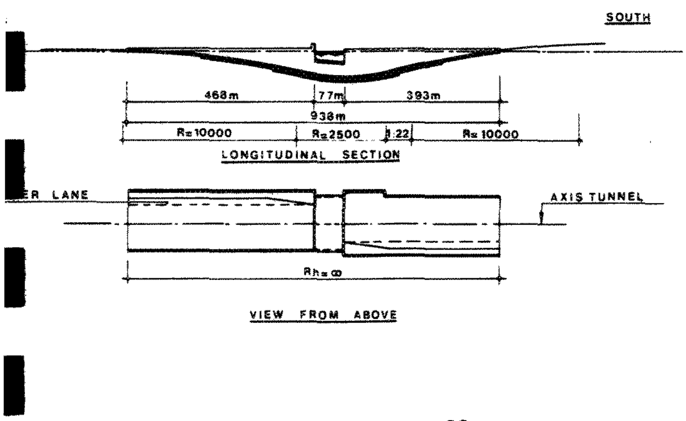


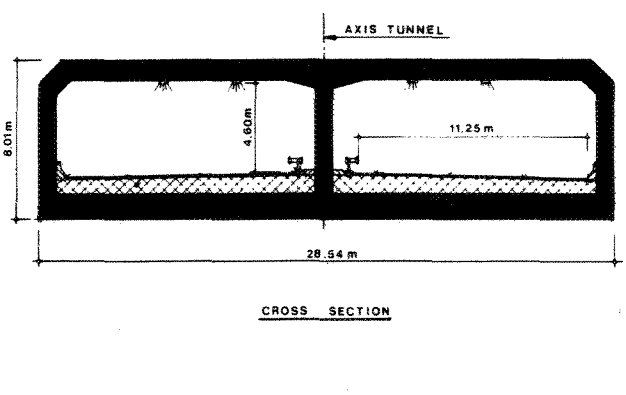
nnel
tion:
ordrecht under Oude Maas-river
uction period:
1977
gner:
al Public Works Department, Locks
eirs Division
rs:
Soil Mechanics Laboratory
Hydraulics Laboratory
al Institute for Watersupply
ctenbureau De Vlaming, Salm
ennis
truction costs:
500,000 civil engineering
500,000 equipment
od of construction:
nersed units of 115 m length land parts formed by diaphragm walls
eel sheet piling walls into a clay layer at about 25 m depth so in this
permanent lowering of the ground water level was possible.
contractors:
binatie Tunnelbouw, joint venture of Koninklijke Nederhorst Bouw BV
nember of OGEM Building Division), Dirk Verstoep BV (member of
Bos Kalis Westminster Group) and Van Hattem en Blankevoort BV
er of Stevin Group)
contracters:
Bos Kalis Westminster Group (dredging)
Kramer, now called Cekasol BV (watertight lining)
stem BBRV Spanstaal BV (prestressing)
dboorbedrijf J. Mos (well pumping system)
pparatenbouw (sound absorbing ceiling)
Sigma Coatings BV (paint)
Vogel Spuitbeton (shotcrete for wall finishing)
GTI (control centre and lighting)
Industria (lighting)
Van der Bor (pumps)
Suppliers:
Vredestein Loosduinen BV (watertight gaskets)
Eternit BV (Glasal plates)
Heemaf (stanby equipment)
ACEC (control centre)
Philips Nederland (lighting)
Stork (ventilation)
Flygt (pumps)

Prinses Margriettunnel

tunnel
on:
canal in the north of the Netherlands
uction period:
-1978
ner:
al Public Works Department, Locks and
Division
sors:
Soil Mechanics Laboratory
al Institute for Watersupply
ctenbureau De Vlaming, Salm en Fennis
itecture)
truction costs:
00,000 civil engineering
00,000 equipment

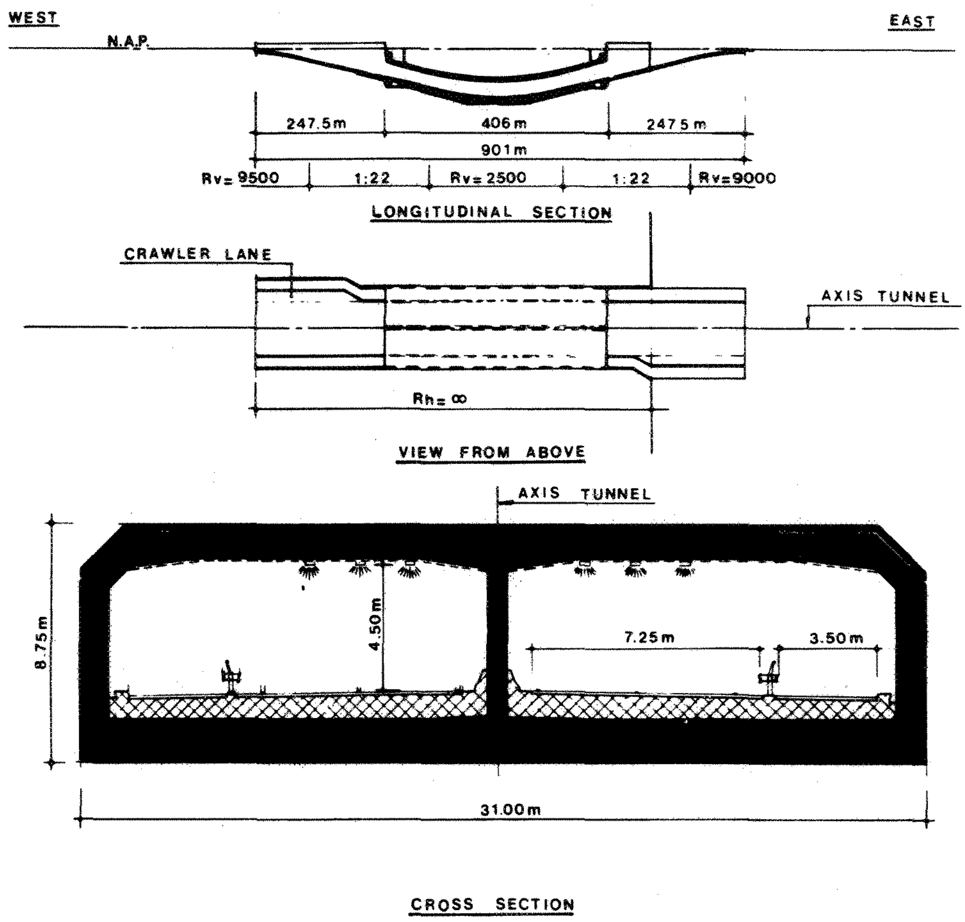
Method of construction:
1 immersed unit of 77 m length built in one of the open ramps
Immersing of the unit by means of flooding the open ramps and dredging the tempory dikes.
After immersing the joints were closed by pneumatic rubber gaskets.
The open ramps are founded on cast in situ concrete tension piles and made in open excavation.
Main contractors:
Aannemerskombinatie Prinses Margriettunnel, joint venture of BV Aaannemingsmij v/h H & P Voormolen (member of OGEM Building Division) and DURA Aannemingsmij BV
Subcontractors
J.G.Nelis & Zn IJmuiden BV (dredging and excavation)
Adriaan Volker Civil Engineering BV (immersing of the unit)
Cekasol BV (watertight lining)
Nederhorst Grondtechniek – member of International Foundation Group (cast in situ Vibro-piles)
Sigma Coatings BV (paint)
Van der Bor (pumps)
Installatie Techniek Bredero BV (control centre and lighting)
Suppliers:
Vredestein Loosduinen BV (watertight gaskets)
Flygt (pumps)





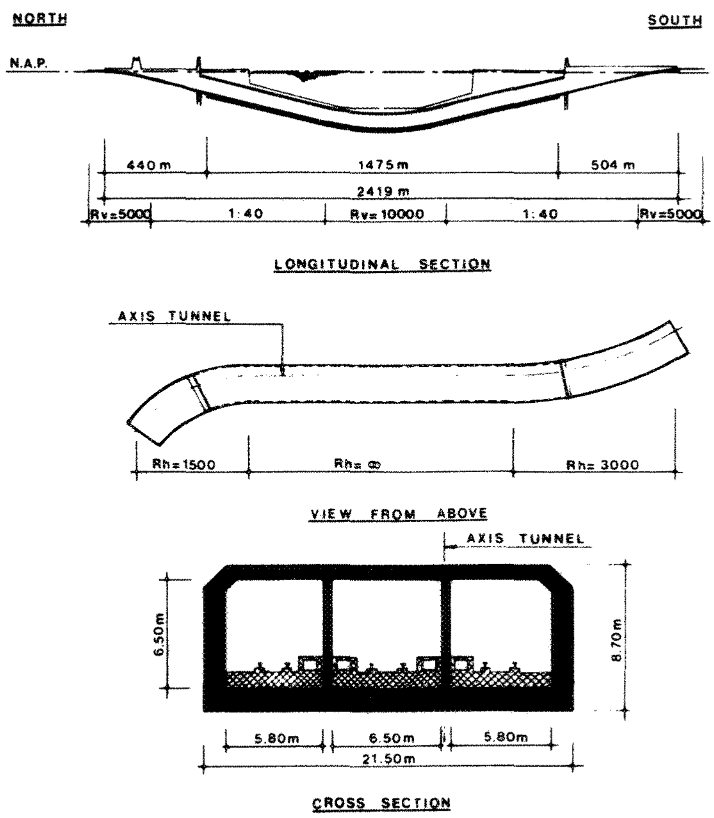
13. Kiltunnel

Type:
road tunnel
Location:
under Dordtse Kil, river near Dordrecht
Construction period:
1974-1978
Designer:
National Public Works Department, Locks and Weirs Division
Advisors:
Delft Soil Mechanics Laboratory
Delft Hydraulics Laboratory
National Institute for Water supply
Architectenbureau De Vlaming, Salm en Fennis (architecture)
Construction costs:
f 76,000,000 civil engineering
f 5,000,000 equipment
Method of construction:
3 immersed units of 110 m length. Deepest parts of open ramps formed by concrete structure founded on cast in situ concrete tension piles; higher parts formed by steel sheet piling into a deep clay layer, so a permanent lowering of the groundwater level was possible.
Main contractors:
Kombinatie Tunnelbouw (see Drechttunnel)
Subcontrators:
Royal Bos Kalis Westminster Group (dredging)
Nederhorst Grondtechniek BV – member of International Foundation Group (cast in situ Vibro-piles)
Pre-system BBRV Spanstaal BV (prestressing)
Vogel Spuitbeton BV (shotcrete for wallfinishing)
GTI (control centre and lighting)
Van der Bor (pumps)
Suppliers:
Vredestein Loosduinen BV (watertight gaskets)
Gebr. Prins Bouwstoffenhandel BV (sound absorbing ceiling)
Eternit BV (Glasal plates)
Flygt (pumps)
Heemaf (standby equipment)
Nordisk (ventilation)

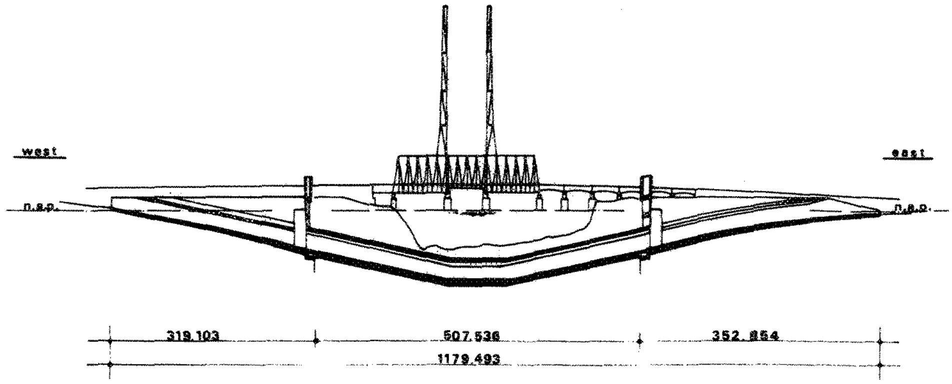


14. Hemspoortunnel

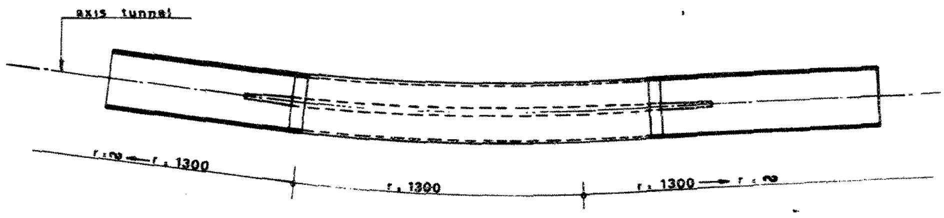
Type:
railway tunnel
Location:
Amsterdam, under Northsea canal
Construction period:
1976-1980
Designer:
National Public Works Department, Locks and Weirs Division
Advisors:
Delft Soil Mechanics Laboratory
National Institute for Watersupply
TNO Delft – Applied Scientific Research Organisation (ventilation)
Construction costs:
f 110,000,000 civil engineering
Method of construction:
4 immersed units of 268 m length, 3 of 134 m length;
open ramps founded on cast in situ concrete tension piles made in open excavation
Main contractor:
Combinatie Hemspoor, joint venture of Dirk Verstoep BV (member of Royal Bos Kalis Westminster Group) and Van Hattum en Blankevoort BV (member of Stevin Group)
Subcontractors:
Royal Bos Kalis Westminster Group (dredging and excavation)
J.G. Nelis & Zn IJmuiden BV (excavation building dock)
Mij. Grondbouw BV (cast in situ Van Parera-piles)
Suppliers:
Vredestein Loosduinen BV (watertight gaskets)



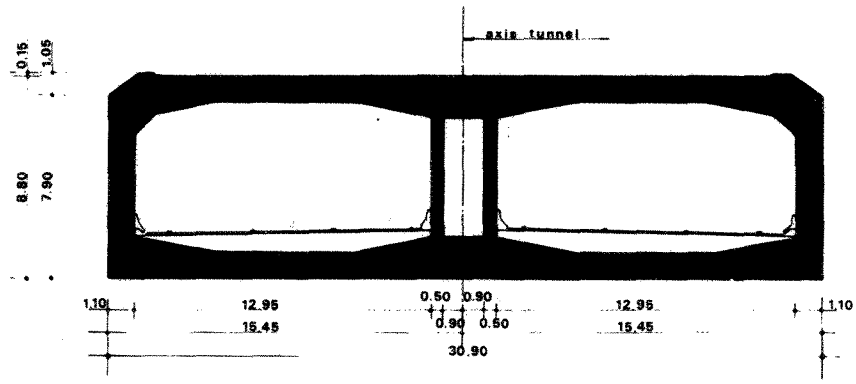
15. Botlektunnel



longitudinal section



view from above



cross section

Type:
road tunnel
Location:
near Rotterdam, under Oude Maas-river
Construction period:
1976-1980
Designer:
Rotterdam Municipal Works
Advisors:
Delft Hydraulics Laboratory
TNO – Applied Scientific Organisation (danger of explosion, fire proofness)
Construction costs:
f 126,000,000 civil engineering
f 20,000,000 equipment
Method of construction:
45 immersed units of 105 m and one of 87,5 m length;
open ramps founded on prefabricated concrete tension piles made in open excavation
Main contractor:
Aannemerscombinatie Botlektunnel, joint venture of BV Aannemingsmij. v/h H & P Voormolen (member of OGEM Building Division) and Ways & Freitag
Subcontractors:
Breejenbout NV (dredging)
BV Grondboorbedrijf J. Mos (well pumping)
Civielco (prestressing)
Troost, Oosterwijk and Romers (excavation and backfilling)
Suppliers:
Vredestein Loosduinen BV (watertight gaskets)

Other tunnels

A. Velsertunnels
Type:
road and railway tunnels 2 x 2 lanes and 2 tracks)
Location:
under Nothsea canal near coast
Construction period:
1954-1957
Designer:
National Public Works Department, Locks and Weirs Division
Main method of construction:
cut and cover
Length:
closed section road tunnel 768 m
total length road tunnel 1644 m
closed section railway tunnel 2067 m
total length railway tunnel 3324 m
Main contractor:
NV Amsterdamsche Ballast Maatschappij (now member of Ballast-Nedam Group)

B. Aquaduct Ringvaart Haarlemmermeer
Type:
road tunnel (2 x 2 lanes)
Location:
under canal near Amsterdam
Construction period:
1957-1960
Designer:
National Public Works Department, Locks and Weirs Division
Main Method of construction:
cut and cover
Length:
closed section 35 m
total length 393 m
Main contractor:
Nelis Uitgeest (now member of OGEM Building Division)

C. Schiphol road tunnel
Type:
road tunnel (2 x 3 lanes for highway and 2 x 2 lanes for local traffic)
Location:
under part of Schiphol Airport
Construction period:
1964-1966
Designer:
National Public Works Department, Locks and Weirs Division
Main method of construction:
cut and cover
Length:
closed section 530 m
total length 660 m
Main contractor:
joint venture of NV Amsterdamsche Ballast Mij (memebr of Ballast-Nedam Group), Koninklijke Wegenbouw Mij (now member of Stevin Group), P. C. Zanen

D. Pipelinetunnel Scheldt-Rhine canal
Type:
tunnel for a great number of pipelines (± 9,5 m² cross-section)
Location:
under canal in the southern part of the Netherlands
Construction period:
1973-1976
Designer:
National Public Works Department, Locks and Weirs Division
Main method of construction:
cut and cover
Length:
closed section 330 m
total length 330 m
Main contractor:
LBM Aannemingsmij

E. Pipeline tunnels Roosendaalse Vliet and Dintel
Type:
tunnels for a great number of pipelines (10 m² cross-section)
Location:
under rivers in the southern part of the Netherlands
Construction period:
1975-1978
Designer:
National Public Works Department, Locks and Weirs Division
Main method of construction:
cut and cover
Length:
197 and 187 m
Main contractor:
BV Bredase Beton- en Aannemingsmij

F. Metro Rotterdam, east-west line (first parts)
Type:
rapid transit tunnel (2 tracks)
Location:
Rotterdam
Construction period:
1975-1980
Designer:
Rotterdam Municipal Works
Main methods of construction:
underpinning under railway viaduct, school and apartments (foundation on tunnelroof) cut and cover between steel sheet piling walls; prefabricated walls + roofs together placed on cast in situ concrete floor in open excavation between steel sheet piling walls
Length:
underpinnings 4 x 24 m
prefabricated walls · roof 1050 m
Main contractor:
Van Hattum en Blankevoort BV (member of Stevin Group)

G. Metro Amsterdam
Type:
rapid transit tunnel (2 tracks)
Location:
Amsterdam
Construction period:
1970-1980
Designer:
Department of the Public Works Service of Amsterdam
Main method of construction:
pneumatic caissons; joints formed under coverage of fro
diaphragm wall panels
Length:
3200 m
Main contractors:
AMCO, joint venture of Bataafse Aannemings Maatsch
mingsmaatschappij v/h Hillen en Roosen, Philip Holzmann,
nemersbedrijf Zanen-Verstoep,
Cospoor, joint venture of Amsterdamse Ballast Beton
(member of Ballast Nedam Group), Van Hattum en Blankev
ber of Stevin Group), Hollandsche Beton Maatschappij (m
landsche Beton Group), Bouwmaatschappij Nederhorst (r
OGEM Building Division), Ways & Freitag

H. Schiphol railway tunnel
Type:
railway tunnel (2 tracks)
Location:
Airport Schiphol
Construction period:
1966-1967 and 1974-1980
Designer:
Netherlands Railways Ltd.
Main methods of construction:
excavation between steel sheet piling, bottom closed by
underwater concrete;
concrete structure founded on cast in situ concrete tensi
bricated tension piles
Length
5780 m
Main contractors:
Combinatie Spoortunnel Schiphol, joint venture of Konin
Bouw BV (now member of OGEM Building Division), H
Maatschappij (member of Hollandsche Beton Group)
Group;
Kombinatie Schiphol 4, joint venture of Dubbers Malden
group, Koninklijke Wegenbouw Stevin BV (member of S
Verstoep BV (member of Royal Bos Kalis Westminster

I. Gouwetunnel
Type:
road tunnel (2 x 6 lanes)
Location:
under river in centre of the Netherlands
Construction period:
1976-1981
Designer:
National Public Works Department, Locks and Weirs
Main method of construction:
open excavation between steel sheet piling and unde
Length:
closed section 70 m
total length 699 m
Main contractor:
Hollandsche Beton Maatschappij (member of Hollands