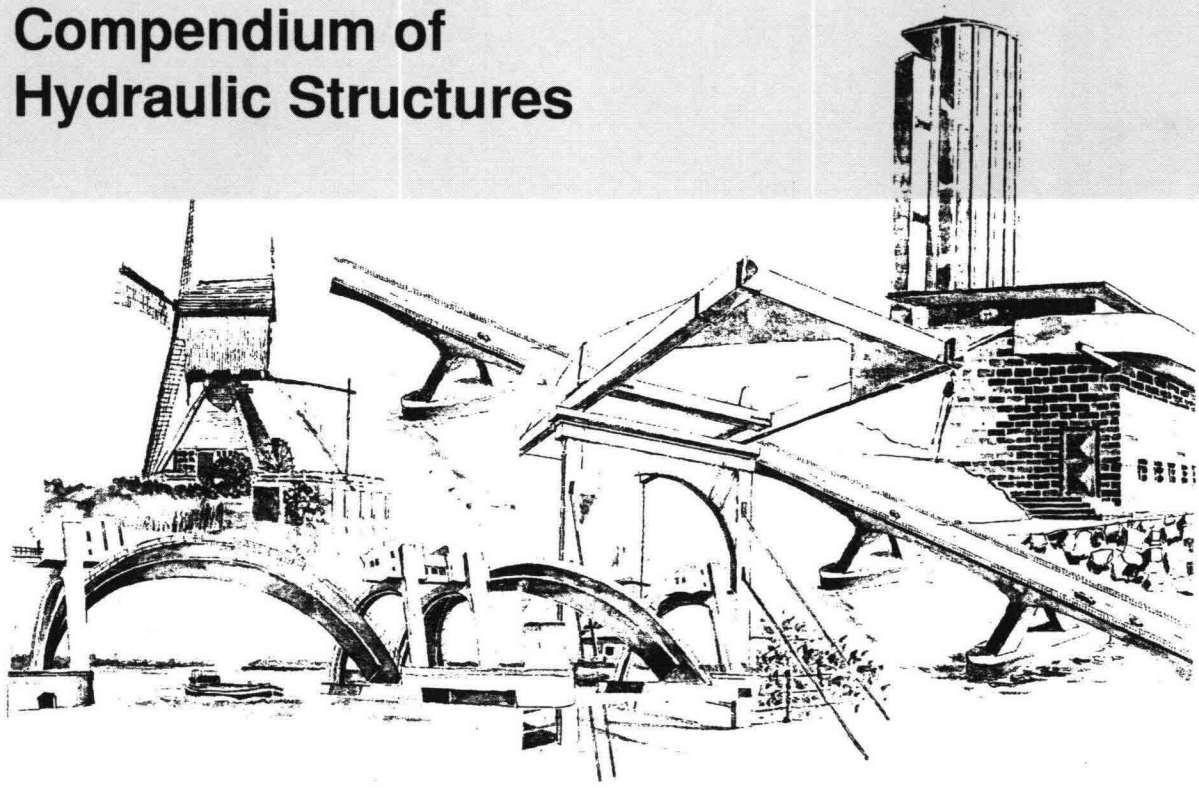
Compiled by
Ir. R. de Heer

Draft edition

Compendium of Hydraulic Structures



IHE DELFT

International Institute for
Infrastructural, Hydraulic and
Environmental Engineering

The Netherlands

Compiled by
Ir. R. de Heer

Delft, January 1993

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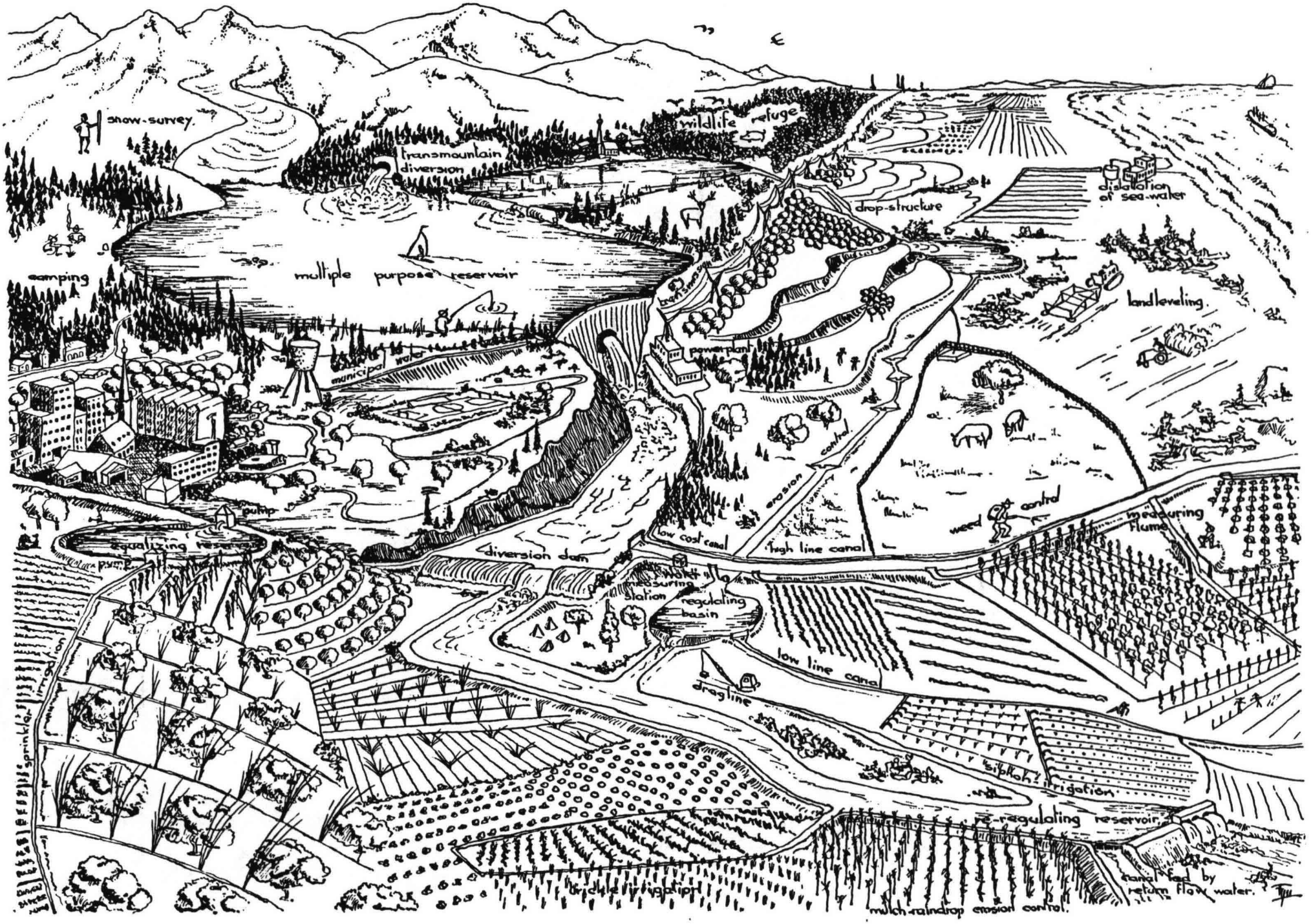
snow-survey
wildlife refuge
transmountain diversion
desalation of sea-water
drop-structure
camping
multiple purpose reservoir
land leveling.
municipal water
power plant
pump
equalizing reservoir
diversion dam
low cost canal
high line canal
erosion control
weed control
measuring flume
water measuring station
regulating basin
low line canal
dragline
sprinkler
siphon irrigation
re-regulating reservoir
canal fed by return flow water.
mulch-raindrop erosion control.

Preface

These notes are a first attempt to compile various existing material on hydraulic structures into a kind of handbook that serves two purposes:

- to acquaint the participants of the Hydraulic Engineering courses with the English terminology in the field of hydraulic structures;
- to present an overview of the field of hydraulic structures and thereby acting an an introduction to the various lecture series.

Many examples of different works, with an emphasis on the hydraulic engineering structures built in The Netherlands, are included to act as illustrations.

I realized that this compilation is far from complete and therefore welcome suggestions for improvement and/or additional material.

I hope you will find this first draft of "Compendium of Hydraulic Structures" useful.

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Subject index

1. Earth retaining structures

1.1. General

A slope 1 : 2, means that the inclination is: 1 vertical to 2 horizontal. The **natural slope** is the steepest stable inclination for an excavation or an embankment. This inclination depends on the type of ground. **Underwater**, with the influence of ground water flow and wash of waves, the equilibrium of the slope is much lower, e.g. 1 : 3 to 1 : 10. If a steeper slope is required, a protection against current and waves is necessary.

In cohesive soils, as clay and peat, sometimes it is possible to excavate perpendicular, provided that the excavated depth is restricted. On the long term, however, such vertical walls are not stable. This method is only applicable for temporary works, e.g. foundation trenches, to limit ground work.

However, often a permanent vertical separation between the higher and lower level is desired. In harbours, the ships have to come directly alongside the quay wall, in locks alongside the surrounding area. In cities, rivers and canals have embankments which are vertical to save land. In that case the ground must be supported by **earth retaining structures**.

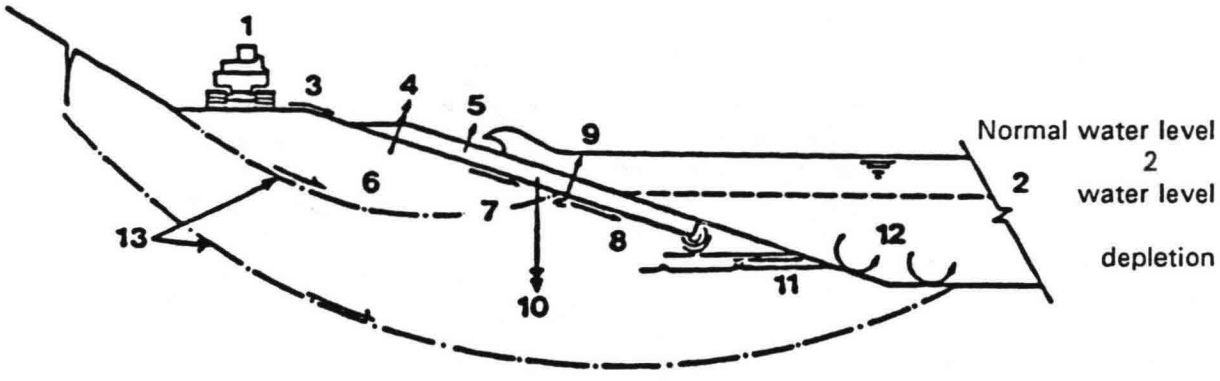


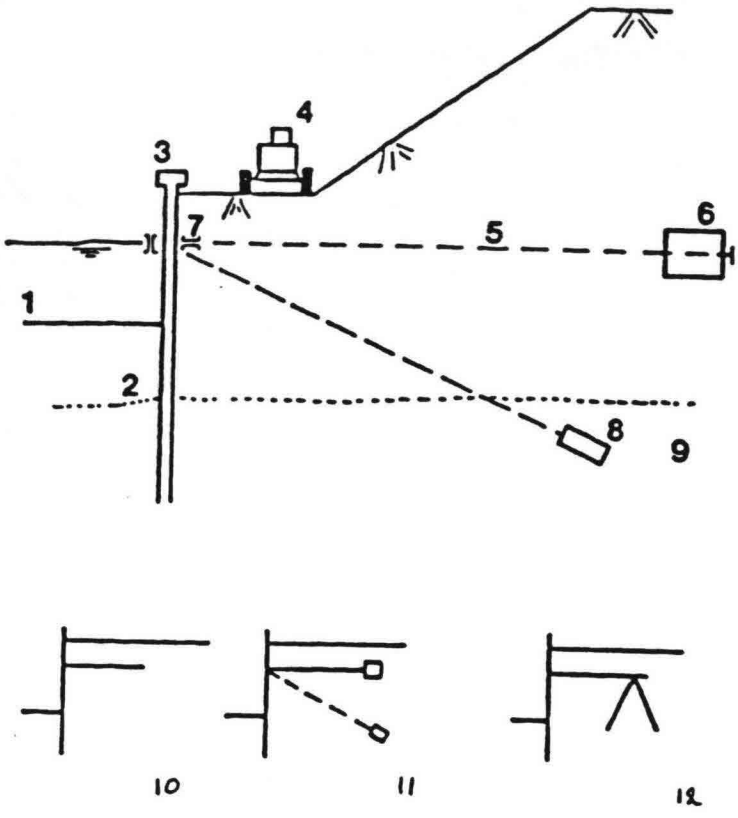
Fig. 1.1. Bank stability

——————————> Shear or slide surface
——————————> local grain transport
——————————> large scale deformation

1. surcharge loads
2. water level depressions
3. surface run-off erosion
4. heave
5. pumping
6. liquefaction
7. local sliding
8. migration
9. transport and loss
10. settlement
11. piping
12. scour
13. examples of slip surfaces

Fig. 1.2. Sheetpiling

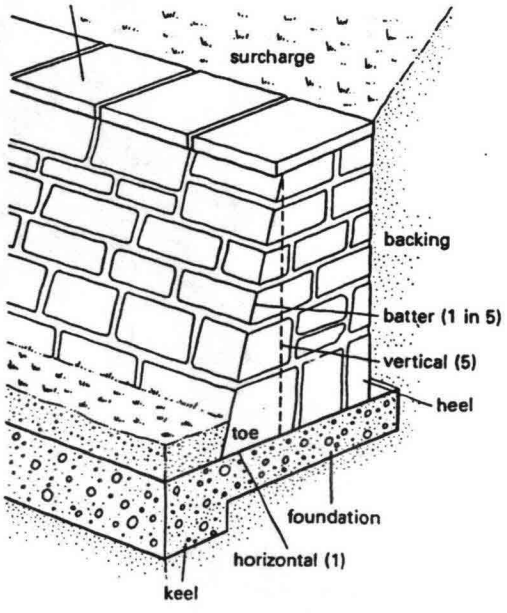
1. bed level
2. sheet piling
3. capping
4. surcharge loads
5. tie rod
6. anchor block
7. walling
8. ground anchor
9. firm strata
10. cantilever wall
11. tied wall
12. wall with relieving platform



Retaining wall. A wall built to hold back earth or other solid material (a dam holds back liquid). Important differences in design exist between **free** and **fixed retaining walls.** Fixed retaining walls are supported at top and bottom and they cannot tilt, so that the earth pressure does not fall to the reduced value of the **active earth pressure.**

Free retaining walls, however, can be **gravity retaining walls.** They need not be designed to bend, but must be able to tilt or slide enough to bring on the reduced value of the active earth pressure. They may be of **sheet-pile construction** (timber, steel or reinforced concrete). Bridge abutments are usually free retaining walls except for those which are fixed arches and portal frames. Cantilever walls are free and may be counterforted or buttressed, sometimes with a keel in the base to prevent sliding, or a **relieving platform** at the top to reduce earth pressure due to surcharge.

Fig. 1. 3. Retaining wall.



1.2. Sheetpiling flexible retaining structures)

Sheet piles. Closely set piles of timber, reinforced or prestressed concrete, or steel driven vertically into the ground to keep earth or water out of an excavation. Bored piles are often successfully incorporated with the concrete of a basement retaining wall.

Sheet-pile wall, sheet piling. A wall of sheet piles, which may be a cantilever wall or anchored back at one or two levels to a dead man as a tied retaining wall.

Steel sheet piling. Sheet piling of interlocking rolled-steel sections driven vertically into the ground along the edge of a guide waling before excavation is begun. When the sheetpiling is completed the excavation can begun in safety. It keeps out flowing ground and often also water, but requires heavy strutting against either of these, unless its penetration depth below the lowest dig level is relied upon to support it.

Tied retaining wall. A retaining wall anchored to a dead man; the contrary of a cantilever wall. A **tieback** is an anchorage or the tie rod connected to it. **Tie rod** - generally a steel rod, often threaded. A **dead man, anchorage, anchor wall/block** is a buried plate, wall or block, some distance from the sheetpile or other retaining wall, which serves to anchor back the wall through a tie between the two. The dead man is held in place by its own weight and by **passive earth pressure** from the soil, resulting in a passive anchorage. Nowadays the tie is often tensioned for added security. **Anchorage distance** : the distance between a quay wall at which the dead man must be placed so as to ensure that it will not slip with the quay wall, and that it anchors it effectively.

Cantilever sheet pile wall is stabilized by its length of penetration below groundlevel on the free side. **Cut-off** depth is the depth below excavation to which sheet piling reaches.

Bulkhead. A sheet-pile wall, usually anchored but occasionally free. It may be a dredged or a fill bulkhead. A **fill bulkhead** is an anchored bulkhead which is backfilled to make the foundation for a quay. A **dredged bulkhead** is an anchored sheetpile, from which the soil near the toe is dug or dredged.

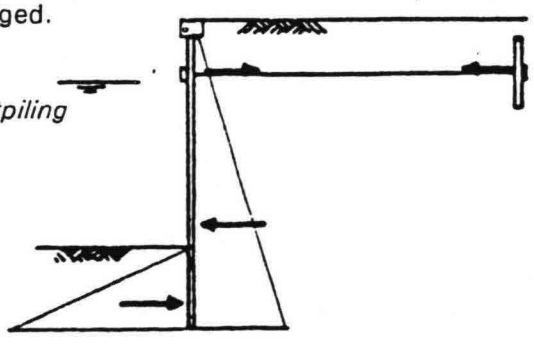


Fig. 1.4.
Loading diagram of a sheetpiling

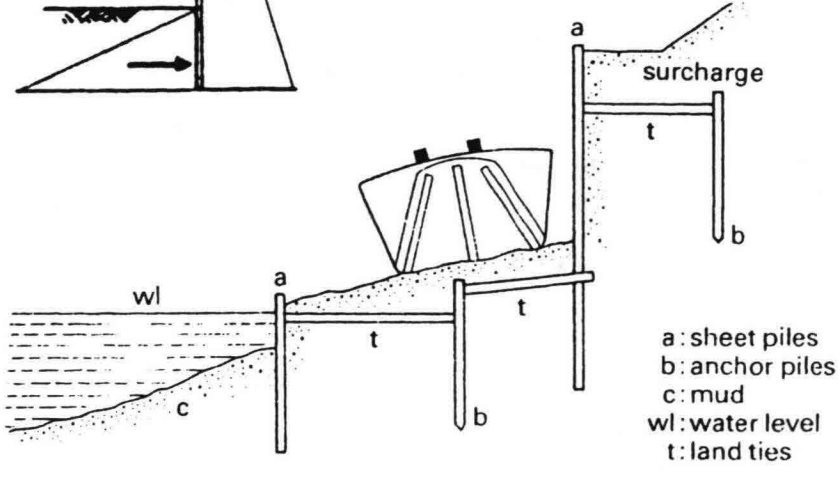
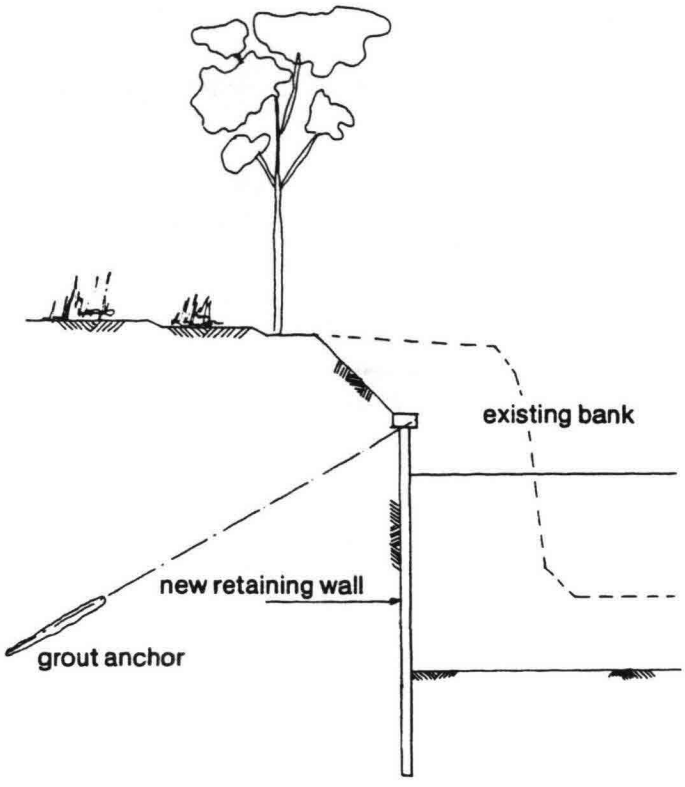


Fig. 1.5. Barge bed.
A mud bottom near the bank of a tidal river where barges can moor and sit on the mud at low tide. The bank is often protected by a double-wall cofferdam.

Grout anchors



Grout anchors are anchoring elements which are installed without excavation in an existing subsoil. An anchor consists of a high tensile bar, which at one end is embedded in a cylinder of grout, the anchoring body or 'grout socket'. The anchor pull is taken by the shear stresses developed along the outerface of this grout socket which can have a length of three to five metres.
The anchors can be of a temporary as well as of a permanent nature; in the second case a special coating is applied as protection against corrosion. After the grout has hardened, each anchor is subjected to a test loading in excess of the design road. The temporary installation is often used for anchoring retaining wall. (building pits)

technical data:
- allowable working loads:
  - up to 50 tons for temporary anchors and
  - 40 tons for permanent anchors
- length : up to approx. 45 m
- diameter of the bar : up to 36 mm.

Procedure:
After the sheet pile enclosure has been driven completely or partially, the installation of the anchors can begin immediately; for that purpose that building site is excavated to a depth of approx. 0.50 metre below the level of the proposed row of anchors. Next a hole is cut in the sheet pile and a tube is driven through it at a predetermined inclination. The tube is provided at the lower end with a conical point. When the tube, with point, has been driven to the required depth, a bar is inserted into the tube. Next, cement grout is injected under pressure into the space between tube and bar; at the same time the tube is gradually withdrawn. By a regular supply of grout under pressure, the cavity formed at the end of the tube is completely filled up for a length of several metres, thus building up a cylinder of grout around the bar. Upon completion of the actual anchoring body, the tube is fully withdrawn; the cavity, created during this extraction, is filled with grout under slight excess-pressure. After removal of the driving tube, the end of the bar extends through the sheet piling. When the grout has hardened sufficiently, the bar is post-tensioned against a bearing plate or waling by means of a jack.

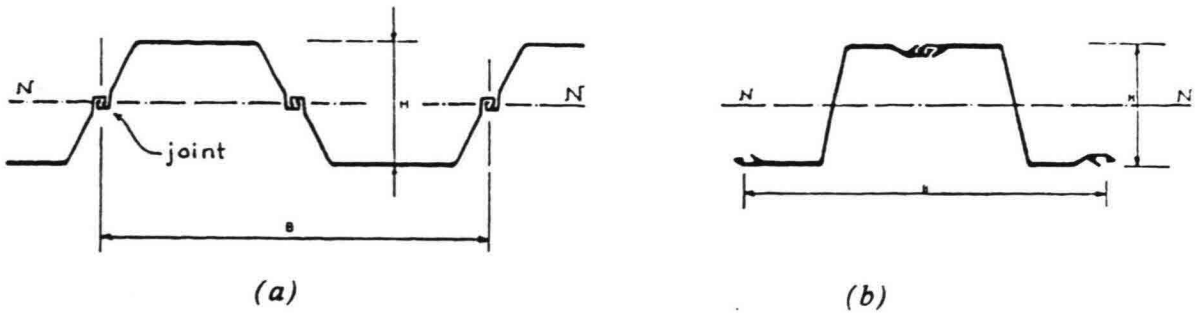


Fig. 1. 6. Steel sheetpiles
(a). Type 'Larssen' - joint in neutral axis: maximum shear in joints
(b). Type 'Hoesch' - joint in flanges : no shear in joint

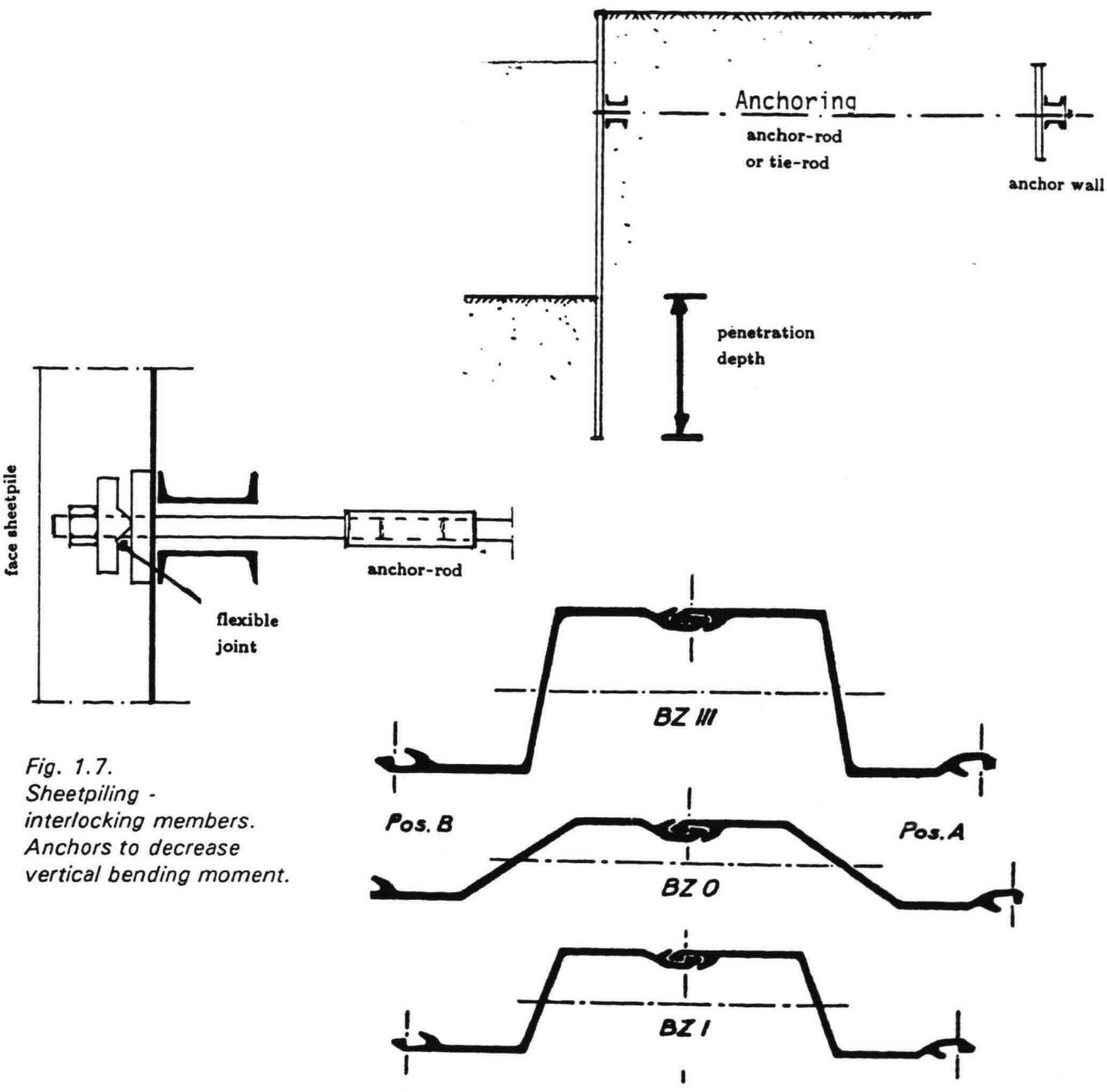


Fig. 1.7.
Sheetpiling - interlocking members. Anchors to decrease vertical bending moment.

1.3. Retaining walls on shallow foundation

Retaining walls structures with a **large weight** and/or a **wide footing**, which means stability.

These walls are subjected to 2-design-criteria:

1. **safe against overturning**: the resultant force **R** of all forces acting on the structures must **intersect the footing-base** (in the middle 1/3 part)
2. **safe against sliding**: the angle between the resultant force and the vertical may not be too large

Fig. 1.6 shows a massive wall, in former time always in **masonry**, in later time also in **mass concrete**. Fig. 1.7 shows a retaining wall in reinforced concrete; the reinforcement allows the designer more freedom in the design by constructing an **L-shaped wall**. The **necessary vertical weight** is partly given by the backfill which rests on the foundation base. This way of execution leads to saving in material.

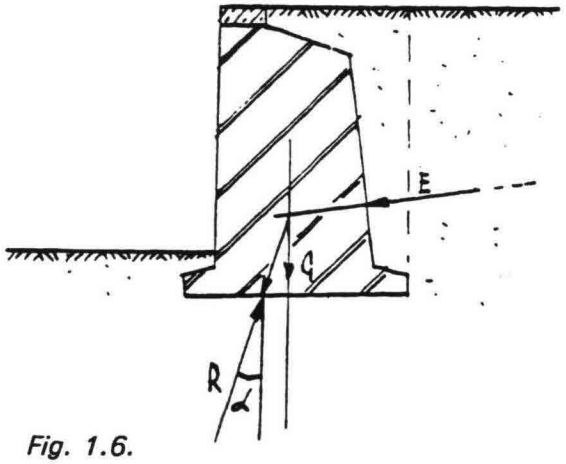


Fig. 1.6.

Massive wall
- masonry (formerly)
- mass concrete

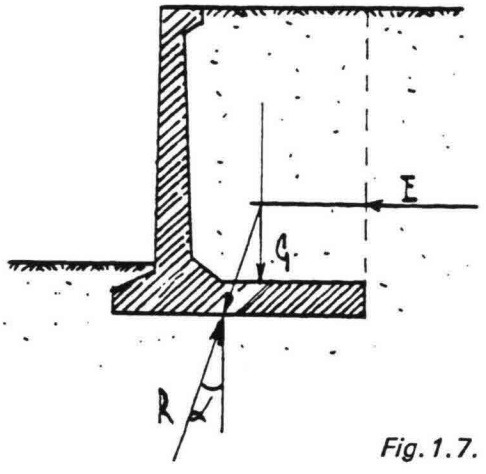


Fig. 1.7.

L-shaped wall
- re-inforced concrete

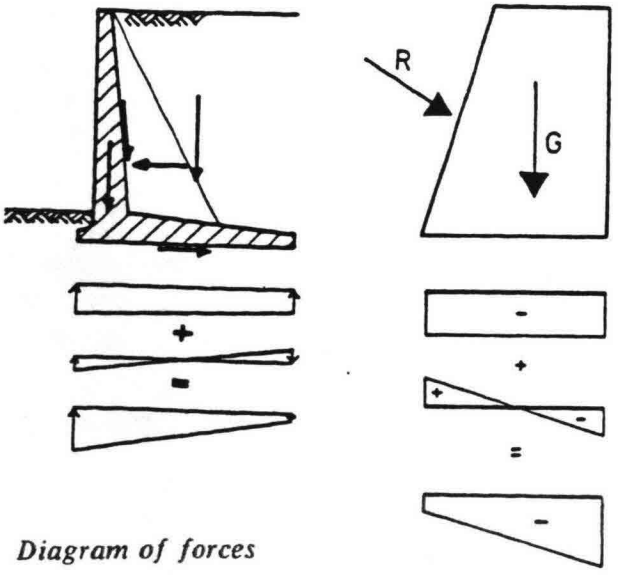
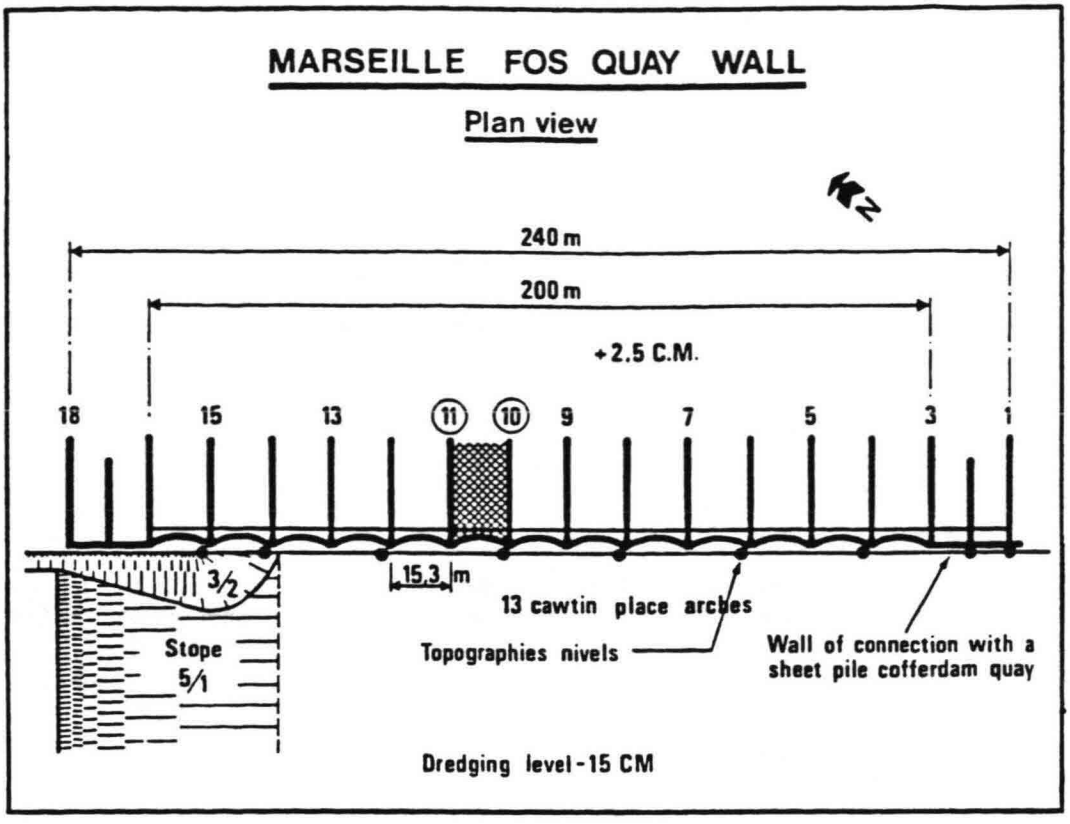


Fig. 1.8. *Diagram of forces*



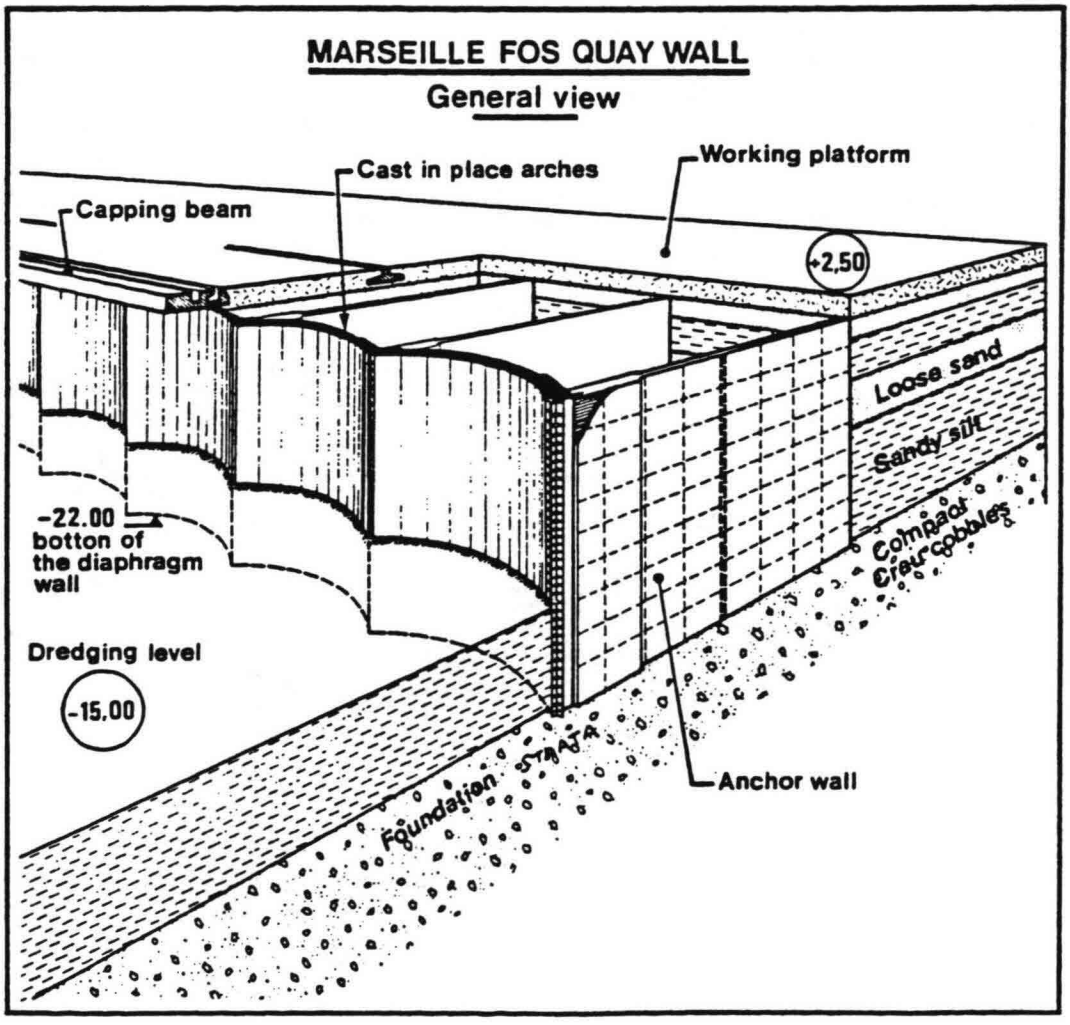


Fig. 1.11. Marseille Fos Quay Wall.

Blockwall Mina Jebel Ali - Dubai - United Arab Emirates

One of the largest man-made ports in the world is Mina Jebel Ali, built between 1976 and 1981. Two huge wet basins have been excavated within the coastline into the desert. Civil works include excavating for and constructing some 13 km. of concrete block wall to form the quay face (quay 1 to 9). Huge solid unreinforced concrete blocks are simply laid one on top of the other.

The block wall does away with steelfixing and formwork setting operations on the actual site as the blocks are pre-fabricated in a block-yard.

This blockyard of 2 km. long comprising a production line and a storage yard - produced some 65,000 - 45-tonne unreinforced blocks and another 7,000 reinforced capping blocks. One drawback with the unreinforced blocks is that they use more cement than with a more sophisticated form of construction. This is offset by a large extent by the use seawater made possible by having no reinforcing steel or lifting lugs. Precious drinking water was conserved. Handling in the blockyard was done by hydraulic clamps which exert a purely friction grip on the sides of the block.

The block wall of the outer basin were built in drained trenches of 100 m. wide at the top and more than 15 m. deep. The blocks of 6 m. long, 2 m. wide and 2 to 2.2 m. deep weighing 42-45 tonnes are placed 'in the dry' by two Manitowoc 4600s cranes. The blocks are bonded with staggered vertical joints instead of standing in individual columns. The reinforced blocks are laid with their long dimensions across the wall, in courses up to nine high depending on the dredging depth of the basin (respectively -14 m. and -11.5 m.). A row of reinforced precast capping blocks is laid on top and the final quay edge is formed by an insitu reinforced concrete recapping beam incorporating a service duct.

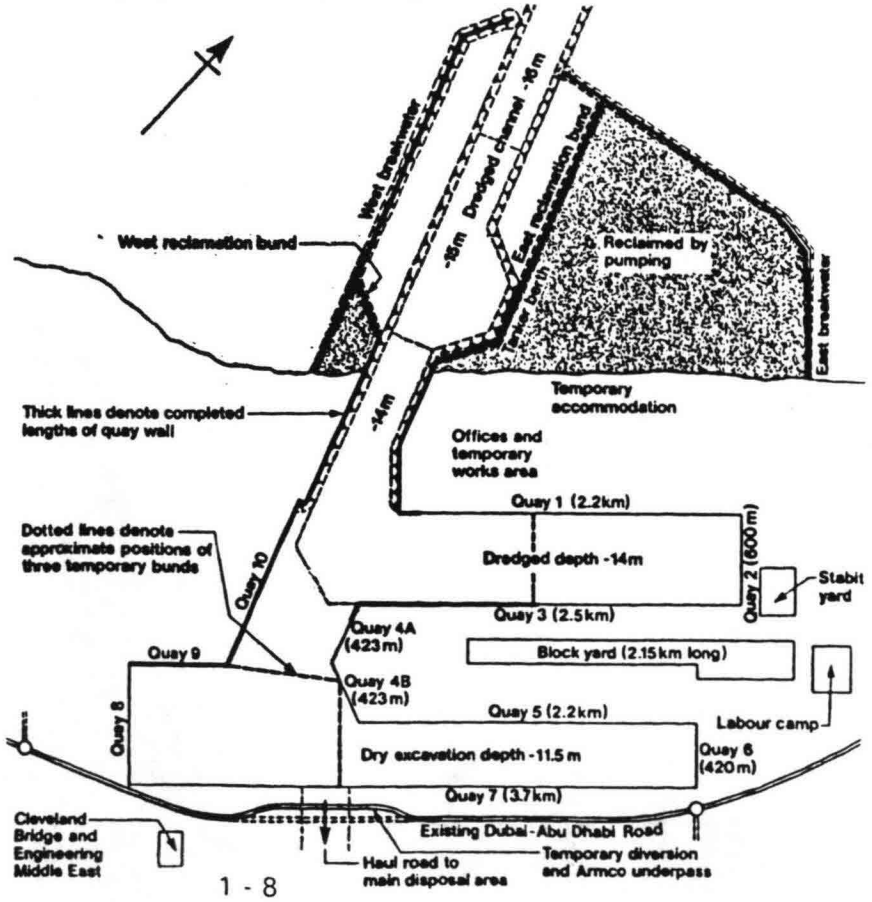


Fig. 1.12.
Plan of Mina Jebel Ali.

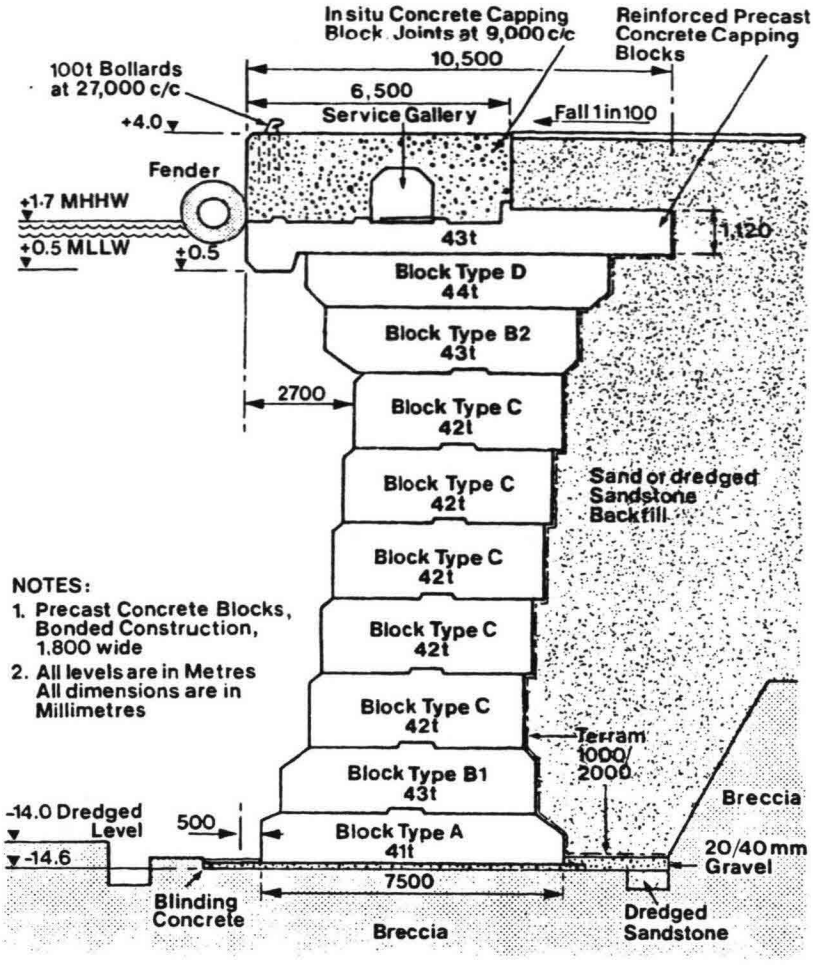


Fig. 1.10.
Mina Jebel Ali - Dubai. Typical section through Quay Wall.
Construction of 13 km. of deep water quays, including casting and placing 63,000 No. 42 tonne concrete blocks and an insitu RC capping beam which required a total of 1,200,000 cu. m. of concrete. The peak daily production of the blockyard was about 168 blocks and some 3,125 cu. m.

1.4. Gabions

A box gabion is a rectangular basket fabricated from a mesh usually of galvanized wire. It is filled with rounded river or quarried stone of suitable size. The mesh is usually prefabricated and assembled at the site, where they are filled. The gabions must be securely wired together at all corners and edges. It is far easier to lace adjacent gabions together when they are empty.

The best filling material is one which allows flexibility of the structure, but at the same time fills the gabion compartments with the minimum of voids and to the maximum weight. Ideally it should be small, just slightly larger than the size of the mesh. The packing of the stone inside the compartment must always be as tight as practicable. To avoid bulging on the outside of the structure, tie wires are fitted inside the gabion compartment (horizontally or vertically).

Standard sizes : 2 x 1 x 0.50 m, 3 x 1 x 0.50, 2 x 1 x 1 m, etc.

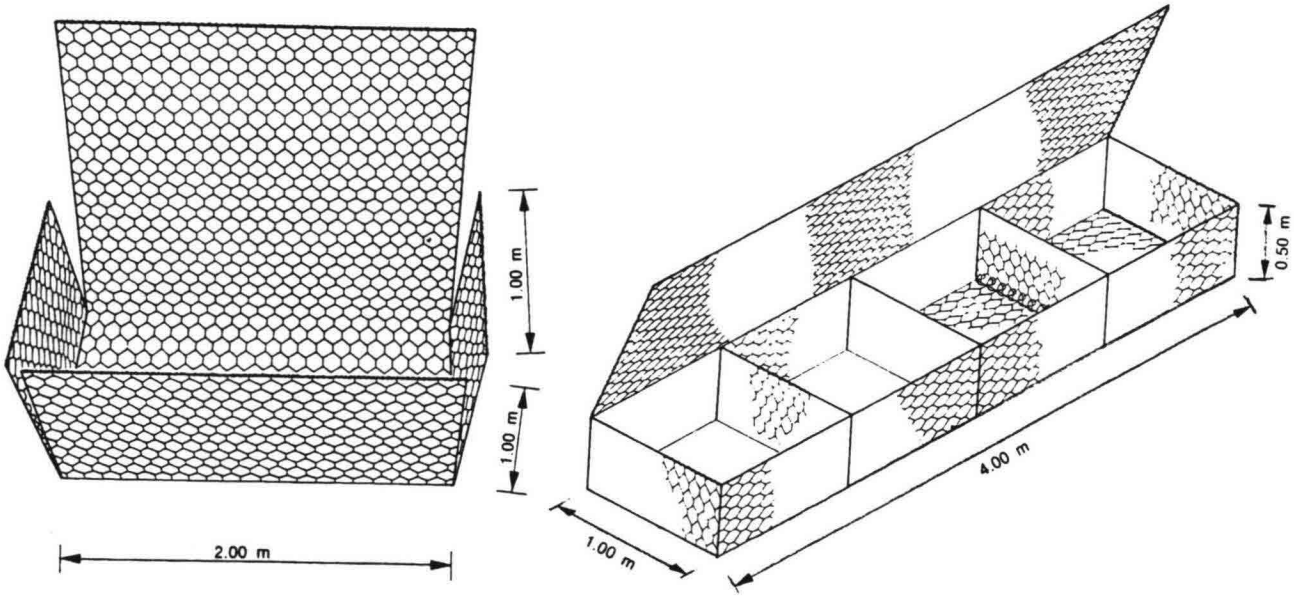


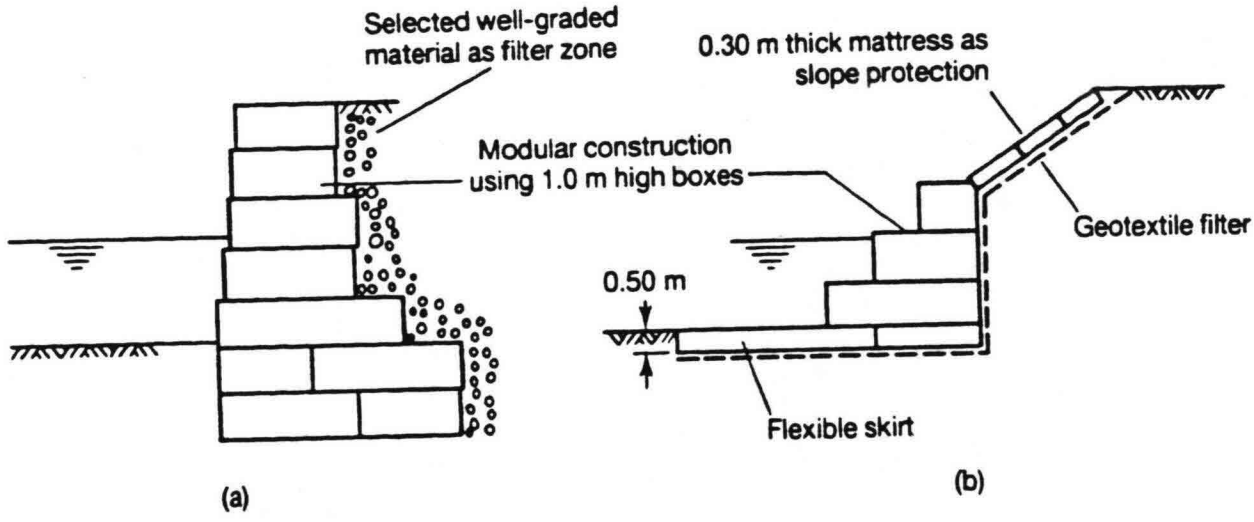
Fig. 1.14. Gabion box of hexagonal, double twist galvanized wire.

Use of gabions and mattresses

Wire gabions and mattresses filled with stone may be used in a conventional sloping revetment.

Gabion boxes are also commonly used to form gravity-type walls, which may be tied back into the soil mass as a composite structure. This form of structure, like other stone revetment systems, can accommodate settlement or consolidation of the foundation after construction. Gabion walls should be built with the front face at a slight angle (10:1) to the vertical, to allow for future forward horizontal tilting as settlement takes place. This battering of the wall may be achieved either by sloping the entire foundation or by stepping back individual units. The gabion wall concept is illustrated in Figure

Careful attention should be placed on provision of an effective filter behind the gabion wall to prevent leaching out of the fine material it retains. Detailed design advice and experience of similar applications should be sought from a gabion manufacturer.



(a) Gabion box wall structure; (b) combined box wall/mattress structure

Fig. 1.15.

1.5. Terre armee (reinforced earth)

The basic principle of Reinforced Earth is simple: the frictional association of non-cohesive granular soil and linear reinforcements creates a new cohesive material, highly resistive to lateral forces and loads (see Fig. 1.11). Reinforced earth structures consist of linear reinforcement strips placed in low cohesion soils. The strips are made from either galvanized steel or glass fibre reinforced plastic. The most common granular materials are sand and gravel; also low cohesion soils can be treated in this way.

An earth retaining wall is made from interlocking facing panels, which can be easily handled and positioned without the need of scaffolding. Reinforcing strips are bolted to these facing panels. Reinforcing strips and compacted backfill are placed in successive layers. This technology has by now been used to build a wide range of seawalls, bulkheads, quays and dams. As a coherent yet flexible gravity mass, Reinforced Earth, is well suited for construction in seismically active regions.

The orientation of the strips has to be in the same direction as the maximum loading as the reinforcement works in one direction only. Another application of **'Reinforced Earth'** is the so-called **'nailing'** whereby only use is made of reinforcement strips.
One may distinguished between treatment of the subsoil (reinforcing earth below footing) and the treatment of embankments (steeper slopes). Finally there are various methods to enwrap soil with firm materials to create a larger unit that has its own cohesion. The simplest example is the sandbag.

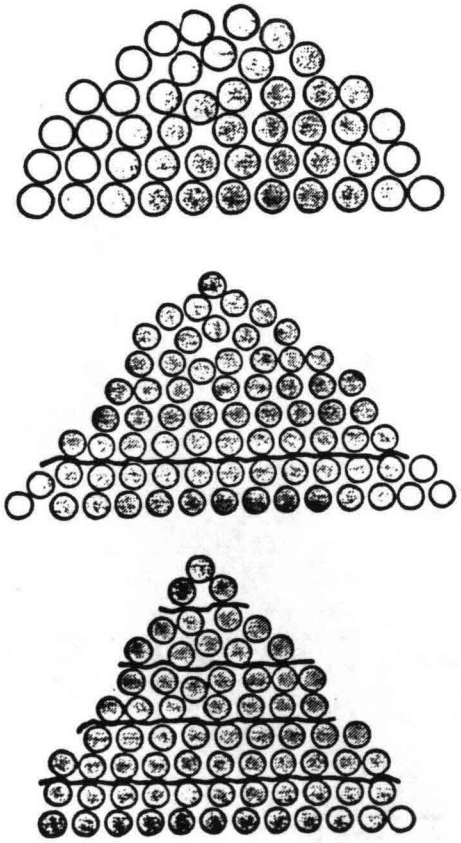


Fig. 1.16. Principle of reinforcement.

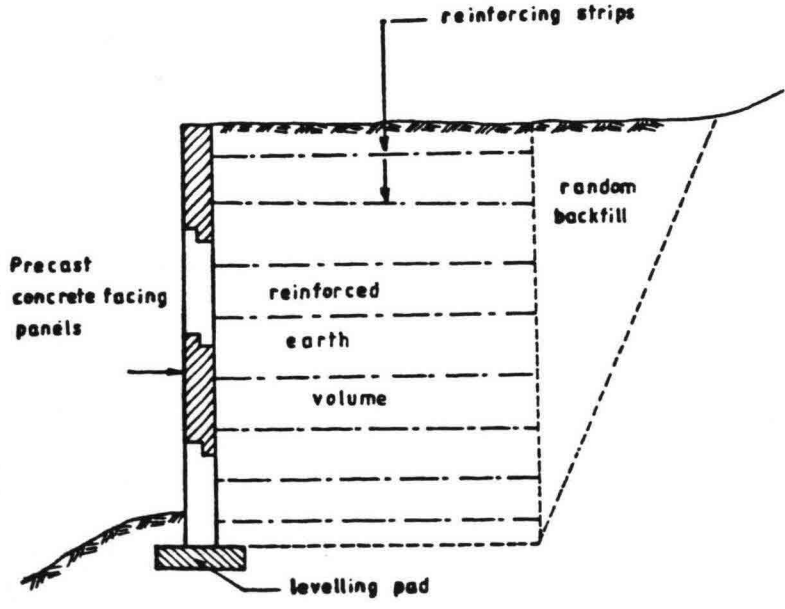


Fig. 1.17. Typical section of a Reinforced Earth retaining wall.

Reinforced Earth. In the USA road or railway cuttings often have sides of reinforced earth. The cut may be faced with precast slabs with tiebacks. The earth is reinforced by the wall and holds it at the same time. Bars used near the sea should be non-rusting.

The Romans built **reed-reinforced** earthen levees along the river Tiber, and 1000 years earlier the people of Iraq built earthen ziggurats. We now use **geosynthetics** of the most varied types, laid horizontally, vertically or sloping. They may allow drainage without piping or merely strengthen the soil mass.

Probably the originator of reinforced earth was Henri Vidal, in France, who in 1963 with his children saw the effect of building sand castles reinforced with pine needles and then in 1966 built a reinforced-earth structure and patented his method.

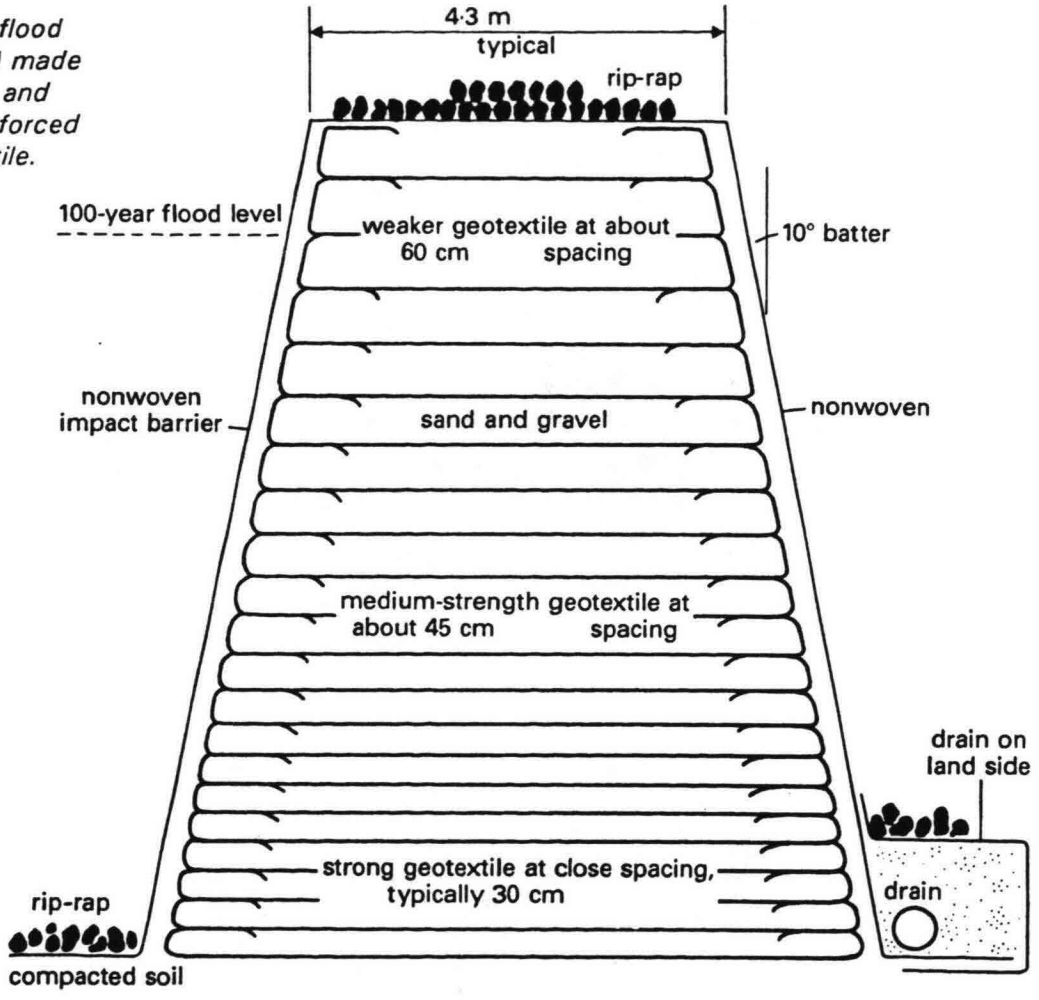


Fig. 1.18.
Ohio river flood wall (USA) made from sand and gravel reinforced by geotextile.

1.6. Retaining walls on deep foundation/quay walls

If the bearing capacity of the subsoil is low and a deep foundation is necessary a combination of a L-shaped retaining wall founded on piles and a sheetpiling is usually applied. The horizontal and vertical forces are taken up by the piles. There are pressure piles and tension piles (see Fig. 1.13). The function of the sheetpile is to retain the soil and possibly water. In some cases the sheetpiling might also have a bearing function.

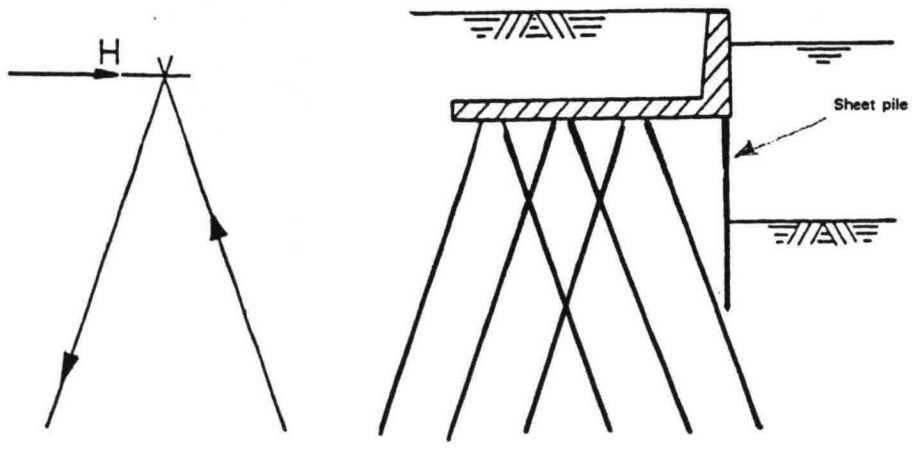
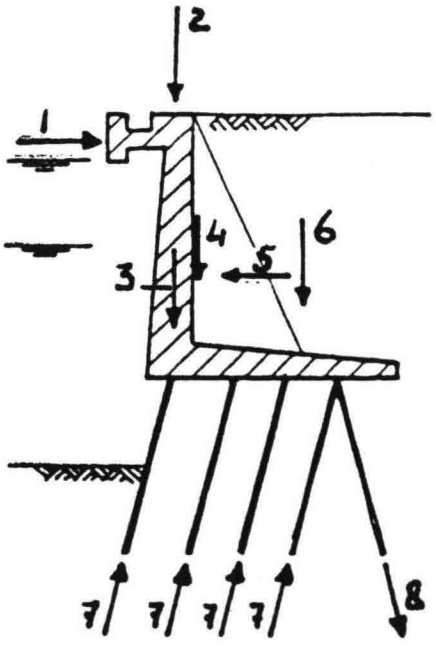


Fig. 1.19. Combination of sheetpile and L-shaped retaining

A quaywall is an example of such a combination. A quaywall serves as a berthing place for ships, often to be loaded of unloaded. The quaywall will be horizontally loaded by the ship and vertically by a crane (see Fig. 1.14).

Fig. 1.20.
Forces acting on a quaywall.
1. Horizontal load
2. Vertical load
3. Weight of structure
4. Soil friction
5. Horizontal earth pressure
6. Weight of soil body
7. Reaction forces (pressure)
8. Reaction force (tension)



A quaywall is often a combination of a sheetpile and a retaining wall.

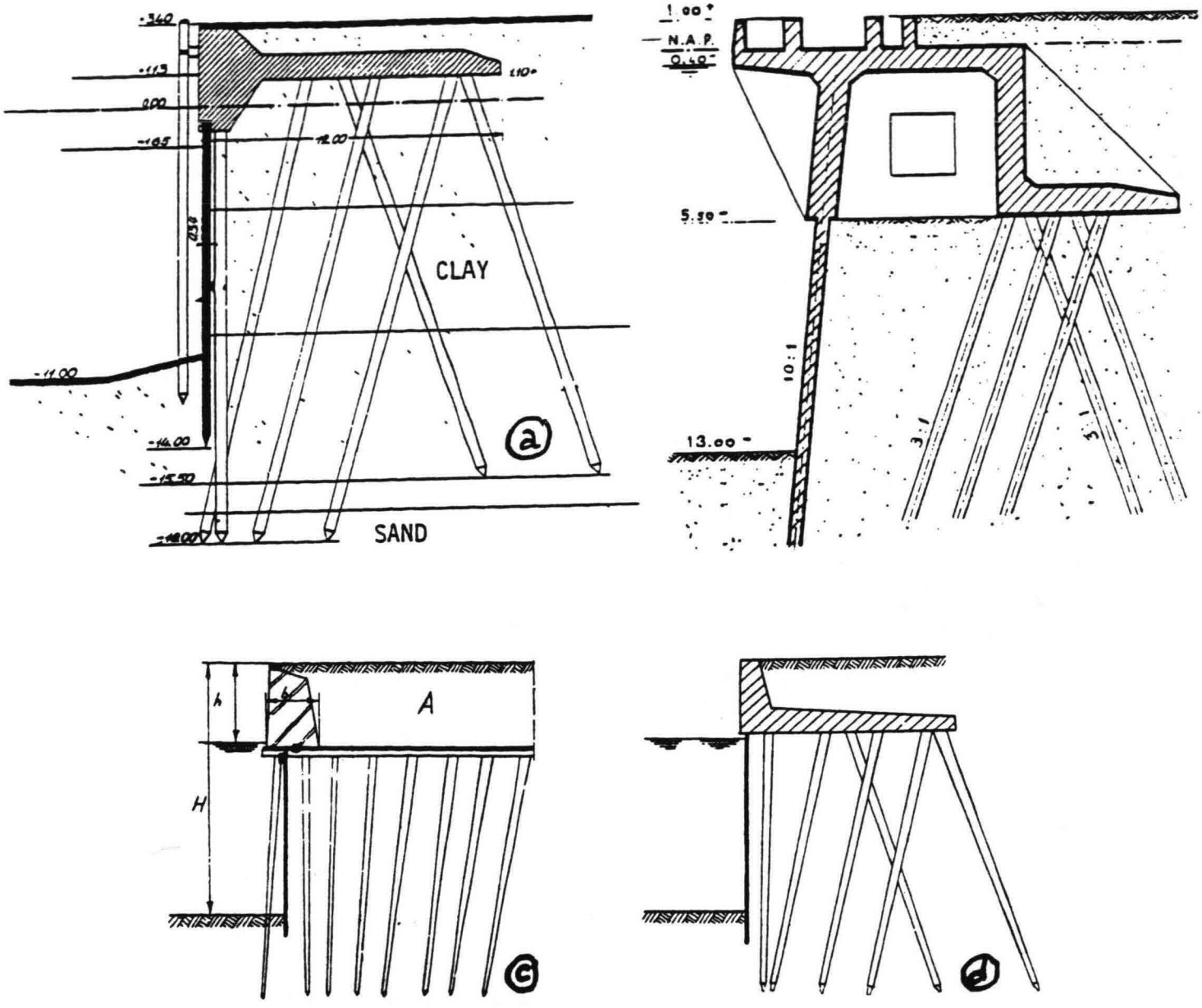
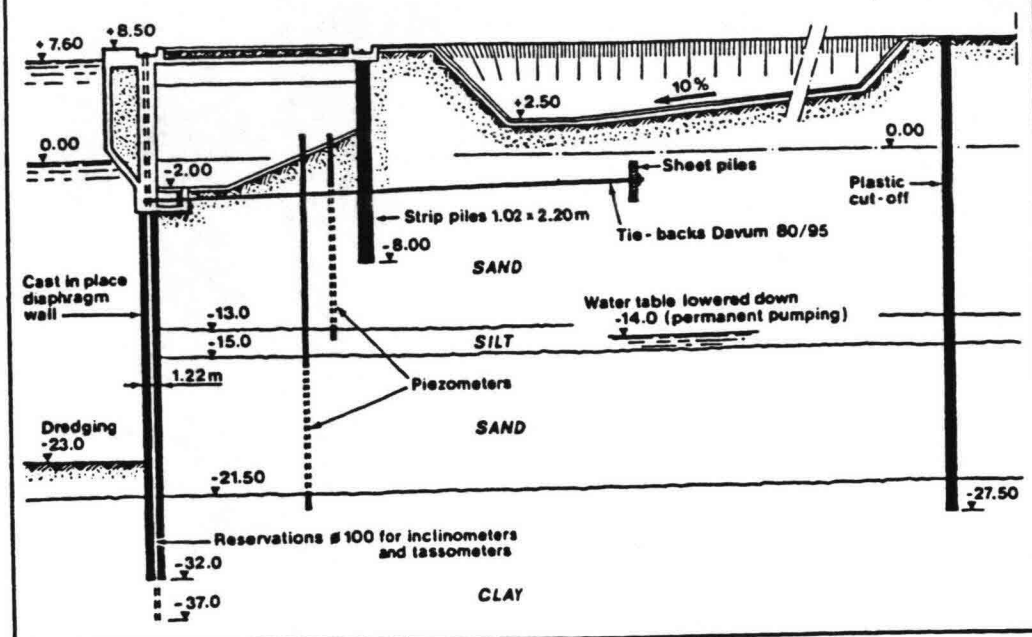


Fig. 1.21. Various quaywall cross-sections.

a. normal quaywall at Delfzijl (The Netherlands).
b. quaywall of a different shape; the quay had to be provided with a double crane rail for a very heavy bridge crane.
c. the earth retaining function is fulfilled partly by the retaining wall and partly by a wooden sheeting.
d. today the timber works and masonry have all been replaced by a (reinforced) concrete superstructure and reinforced concrete piles with sheetpiling either in steel or reinforced concrete. Now the superstructure can be placed entirely above waterlevel. To give the construction more rigidity in horizontal direction, piles are often given an inclination.

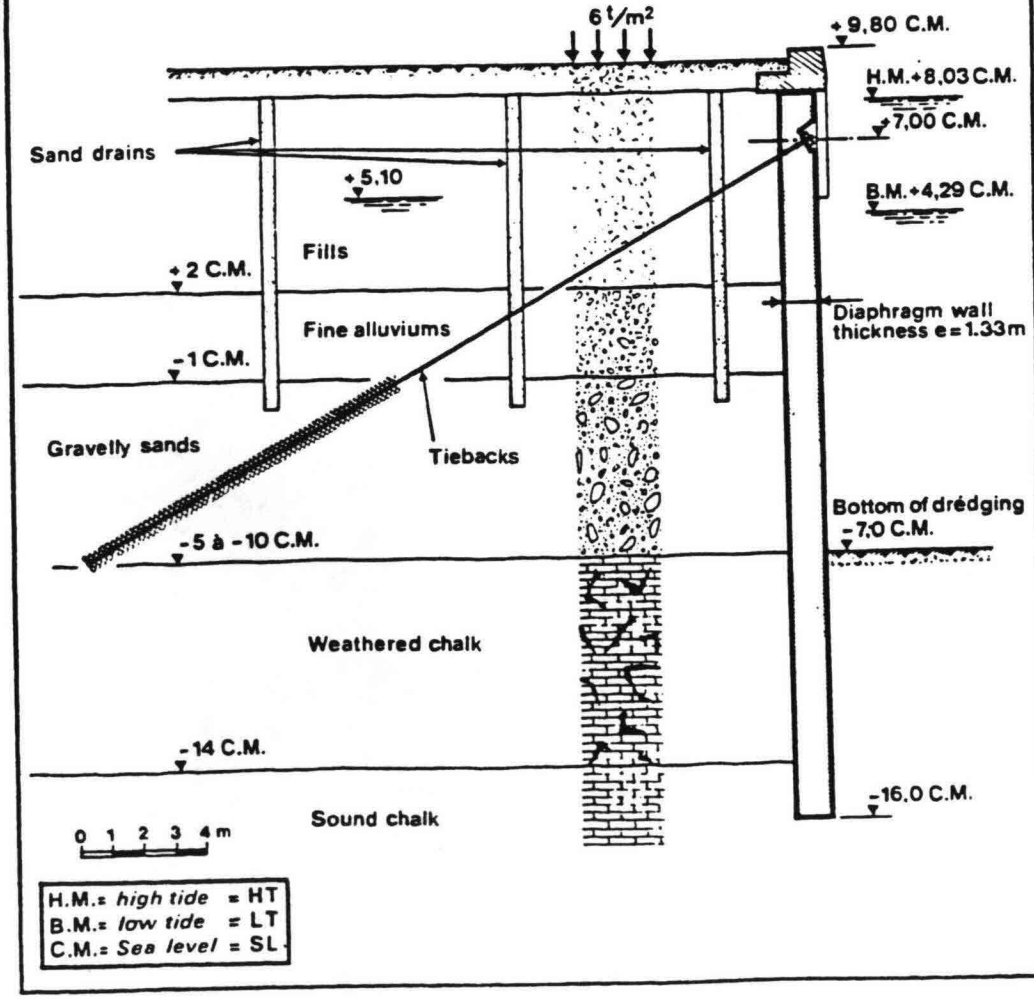
DUNKERQUE QUAY WALL

Typical cross section



ROUEN LES MOULINEAUX QUAY WALL

Typical cross section



Diaphragm wall

Diaphragm wall, ICOS wall, slurry wall. A concrete retaining wall underground , which may be as much as 24 m deep, built in a mechanically excavated trench that has been filled with bentonite-loaded or ordinary mud to support it during excavation. Reinforcement is dropped into the mud and the concrete is lowered into the bottom of the trench by **tremie**. The method is relatively silent and vibrationless compared with driving sheet piles.

In 1973 a French method of converting bentonite slurry to concrete by adding cement (Sepicos) was said to be economical, because, there was no need to buy concrete nor to pay for the removal of waste mud. About 150 kg of cement were added per m3 of mud, with some lignosulphite retarder. The 28 day strength was very low, varying from 1 to 3 N/mm2, but the material was impermeable and crack-free and sometimes this is all that is needed.

To prevent dilution of the slurry it must always be at least 1 m above the standing-water level in the surrounding ground. The grab may be supplemented by an air-lift pump when cleaning solids from the bottom of the trench before concreting. The slurry density should be less than 1.1. At more than 1.1 the slurry should be desanded to prevent it mixing with the concrete. The concrete should be sloppy with a slump of between 150 to 200 mm, but this should be achieved by a **superplasticizer** to ensure a low W/C ratio. The wall is usually cast in lengths (barrettes) of 4-6 m. Precast, post-tensioned, prestressed wall units have also been used.

Masthead gear. Any gear carried on a mast or jib; this has come to mean two drums carried on a jib to control two subsidiary ropes over pulleys near the head of the jib. These two ropes can exactly orient a chisel grab digging a trench for a **diaphragm wall** and can thus control the direction of the wall. Any trench during excavation is liable to be overloaded by the crane standing near its edge. This device enables the crane to stand well away, so that it does not surcharge the edge of the trench.

Slurry wall, slurry trench. A type of diaphragm wall built for watertightness rather than strength, e.g. to create a **groundwater dam**. The slurry may include soil, bentonite, fly-ash, cement, other materials or a mixture.

Bentonite. A clay composed, like fuller's earth, mainly of the same clay **montmorillonite**. Found in Wyoming (USA), where it is known for its remarkable, accordion-like expansion when its water content increases. **Bentonite mud** is a thioxotropic suspension of bentonite in water, used for holding up the sides of deep trenches excavated by machine. Bentonite slurry is often used as a lubricant to reduce skin friction in pipe-jacking or in pile-driving or in the sinking of a caisson.

1.7. Quay wall design in Rotterdam : historical review

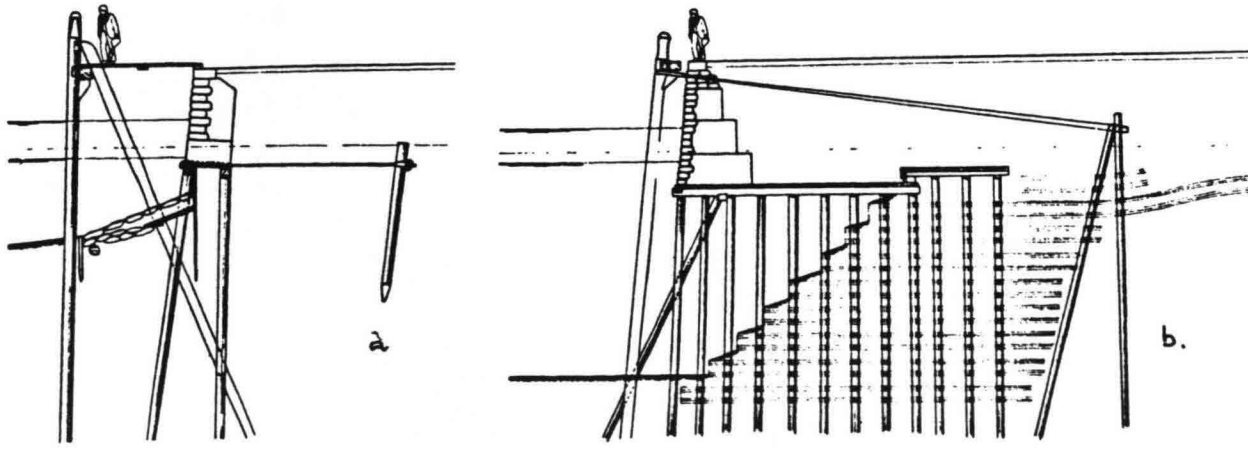
Before 1850 firm upright harbour walls were not yet considered necessary in Rotterdam although many banks had been protected. In those days (18500 Rotterdam was a fairly important port already : ships usually moored to a long ow of poles set-up in the river parallel to the norther bank just outside the town. Here part of the cargo was transhipped into smaller vessels.

The soil in Rotterdam consists of soft layers of watery peat and clay varying in thickness from 18 to 20 metres or in some places even 22 metres, before a firm layer of sand is reached. The very first quay wall built was a 750-metre-long quay along Boompjes. This quay wall partly collapsed shortly after it had been completed. The project was started by dumping fairly heavy stones along the sloping border of the river until a firm dam had been created which just emerged from the water at low tide. This dam was the base on which a wall of basal blocks was built; the space behind it was filled up with sand. This wall however could not withstand the combined effects of the settling process in the vertical plane and the horizontal earth pressure of the sandfill.

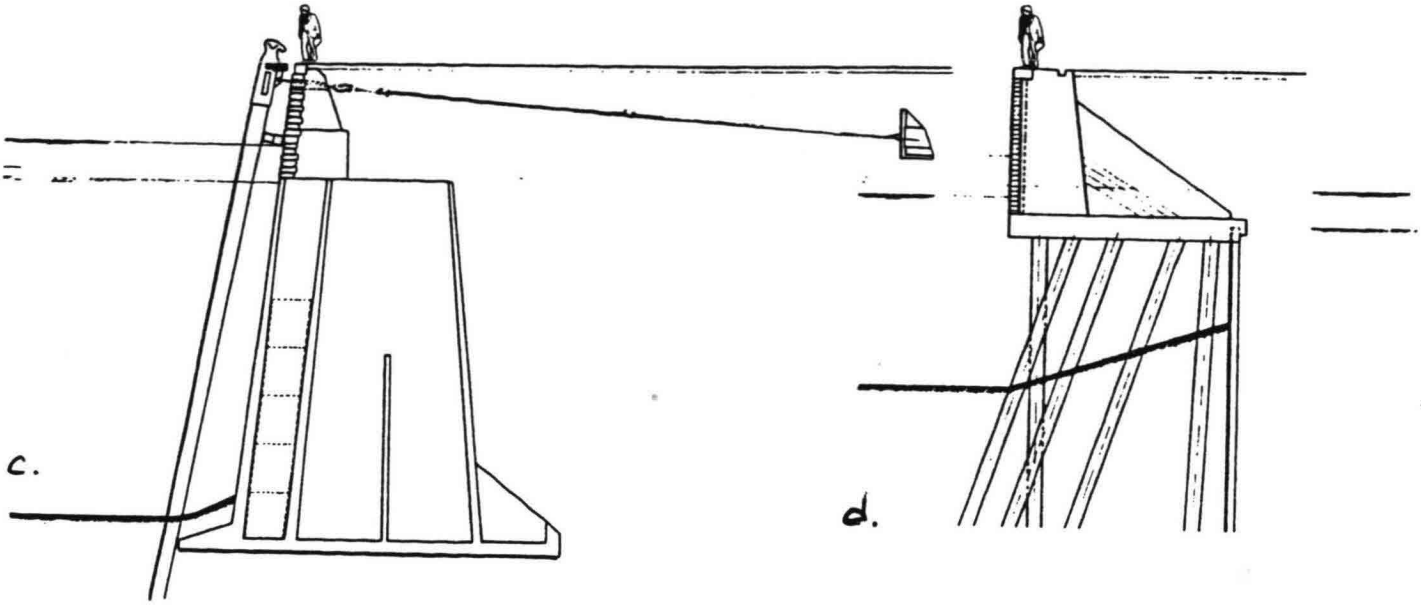
Over a hundred years ago the first quays were founded on tall fir poles which were driven through the layers of soft peat and clay into the firm layer of sand (Fig.a). The tops of these poles were provided with grating that was strong enough to bear a wall. The constructors saw to it that the wooden parts of the construction always remained below the water level to prevent them from decay.
Masoning under water is impossible; therefore the base of the quay wall was never much below the low-water level. This meant that deepening the harbour was sharply limited: going too deep would mean that the sand behind the quay wall would spill underneath the wall into the harbour.

In the first few years of this century a major breakthrough was made when a method was developed to get rid of the soft subsoil. First a deep trench was dug at the place where the quay wall was to be erected. Initially this trench was not as deep as the thick layer of sand, but later it was made even deeper. Then large numbers of fascine mattresses weighted with stones were lowered into the trench, after which the surfaces of the mattresses were levelled with a layer of sharp sand.

By piling up one layer of mattresses after another along the whole length of the trench, dyke was made, which was strong enough to keep the masses of soil behind it in place (Fig.b).



The next step was to drive a great many wooden poles (this time pitch-pine was used instead of fir) into the bottom on which a strong wooden floor was laid. The quay was built on this floor and the sand which had to be dumped behind the quay wall also rested on it. The advantage of this construction was that only this layer of sand exerted pressure on the quay.



The wooden construction underwater were made with the aid of a large diving bell.
Around 1910 the first caisson walls were constructed in Rotterdam. Caissons are heavy concrete boxes which can float. This allows for building them elsewhere and towing them by water to their destination. Here the soft subsoil has meanwhile been dredged and the resulting trench has been filled up with sand on which, after smoothening, the caisson can be sunk. The water inside is later replaced by sand. Thus the earth-retaining-function and the mooring function have been combined in a very heavy and expensive construction element. (Fig.c)

The introduction of quay walls with ground-stemming steel-sheet piling offers a new possibilities. It has proved to be extremely suitable to check masses of ground provided it is anchored solidly to keep it in place.

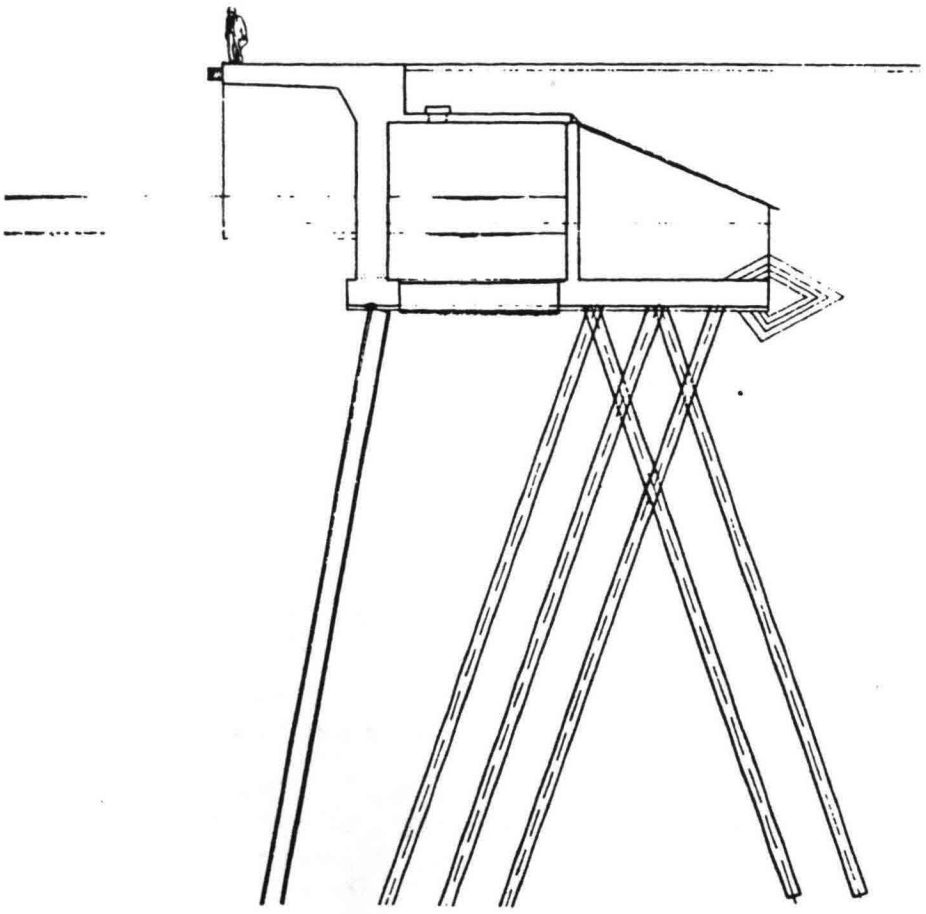
Quay walls with earth retaining steel-sheet piling have been built in large numbers along small Rotterdam inner harbours since 1945. Concrete piles, usually driven into the ground with a slight tilt, absorb the forward ground pressure. They support a heavy concrete groundplate, which reaches as far as the piling and on which the quay wall is erected.

The heaviest quay now in use in Rotterdam is in Maasvlakte.
Situated along Mississippihaven, it had dealt with huge shipments of ore and coal over the years (Fig.e). At low tide it is accessible to ships drawing 21 metres of water. The difference in height between the top of the quay and the bottom of the harbour is nearly 27 metres about as high as an eight floor apartment building.

Here the quay consists of a concrete superstructure resting at the front on an extremely heavy quay wall with earth retaining steel-sheet piling, which tilts slightly forward. The back

of this concrete construction is fastened securely to a few row of concrete piles, which have all been given a slight tilt: some forward and some backward.
The pressure exerted on the ground-stemming steel-sheet piling from the back and top are transferred via the superstructure to the piles in the back field. Thus forward pressure on the quay wall with ground-stemmimg steel sheet-piling will result in a pulling force: the piles with a forward tilt would go deeper into the hard soil, whereas those tilting backward would be drawn out of the ground. Everything will remain in balance as long as the piles are strong enough.

To prevent very heavy-laden and very deep-drawing ships from hitting the quay walls, the superstructure (for it can hardly be called a wall here) has been equipped with a protruding platform which will keep the ship's shell at a distance.



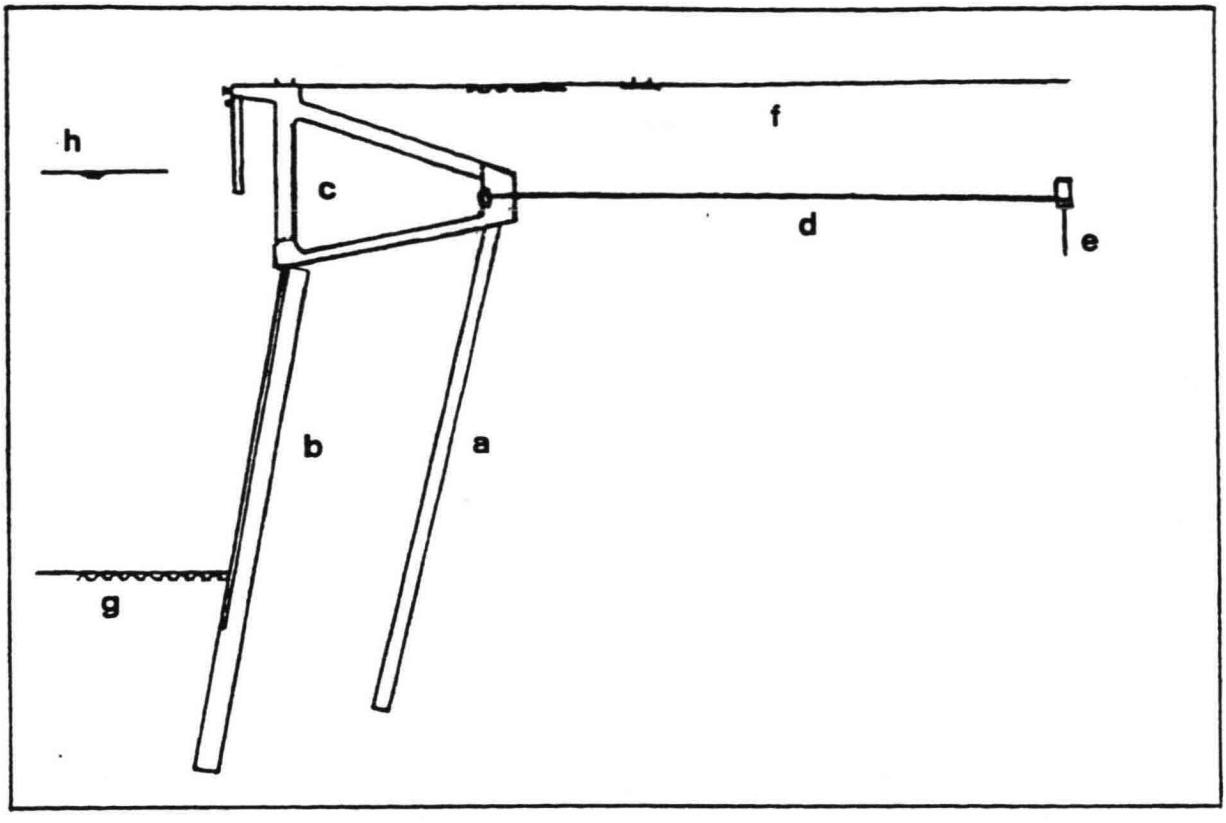


Fig. 1.24. Quay wall - Rotterdam - EECV Terminal.
A much simplified drawing of the new EECV quay, probably the biggest of its kind in the world.

a. indicates the row of 79 concrete piles, manufactured in the ground. They are all 1070 mm thich, but their length varies between 31 and 36.5 metres, depending on subsoil conditions.

b. the actual quay wall, consisting of 244 hollow steel box-piles, ranging in length between 32.6 and 34.1 metres and placed with the greatest precision. They are linked by profiled steel sheets. All elements fit together by means of U-shaped extensions. The steel sheets needed not be so long as the piles, as they reach only a few metres below the dock bottom.

c. the concrete delta-shaped top construcyion is the binding element of the quay. It is hollow and in open connection with the water. 23,000 m^3 of concrete were needed to build this construction which is hundreds of metres' long, 11.5 m high and 14 m wide.

d./e. notwithstanding the oblique position of the bearing elements, horizontal pressures remained to be absorbed, so that a five-metre-high anchor wall had to be built at the rear of the quay, to which the top construction was attached with a large number of anchor cables.

f. the surface of the EECV site is 5.5 m above sea level.

g. the dock bottom; the so-called construction depth marked here is 25.5 m below sea level.

h. sea level is measured at the so-called Normal Amsterdam Level, the benchmark of which is a bronze nut on top of a pile driven into Amsterdam's main square. The nut is about one metre below street level.

1.7. Quay wall design in Rotterdam : historical prospective

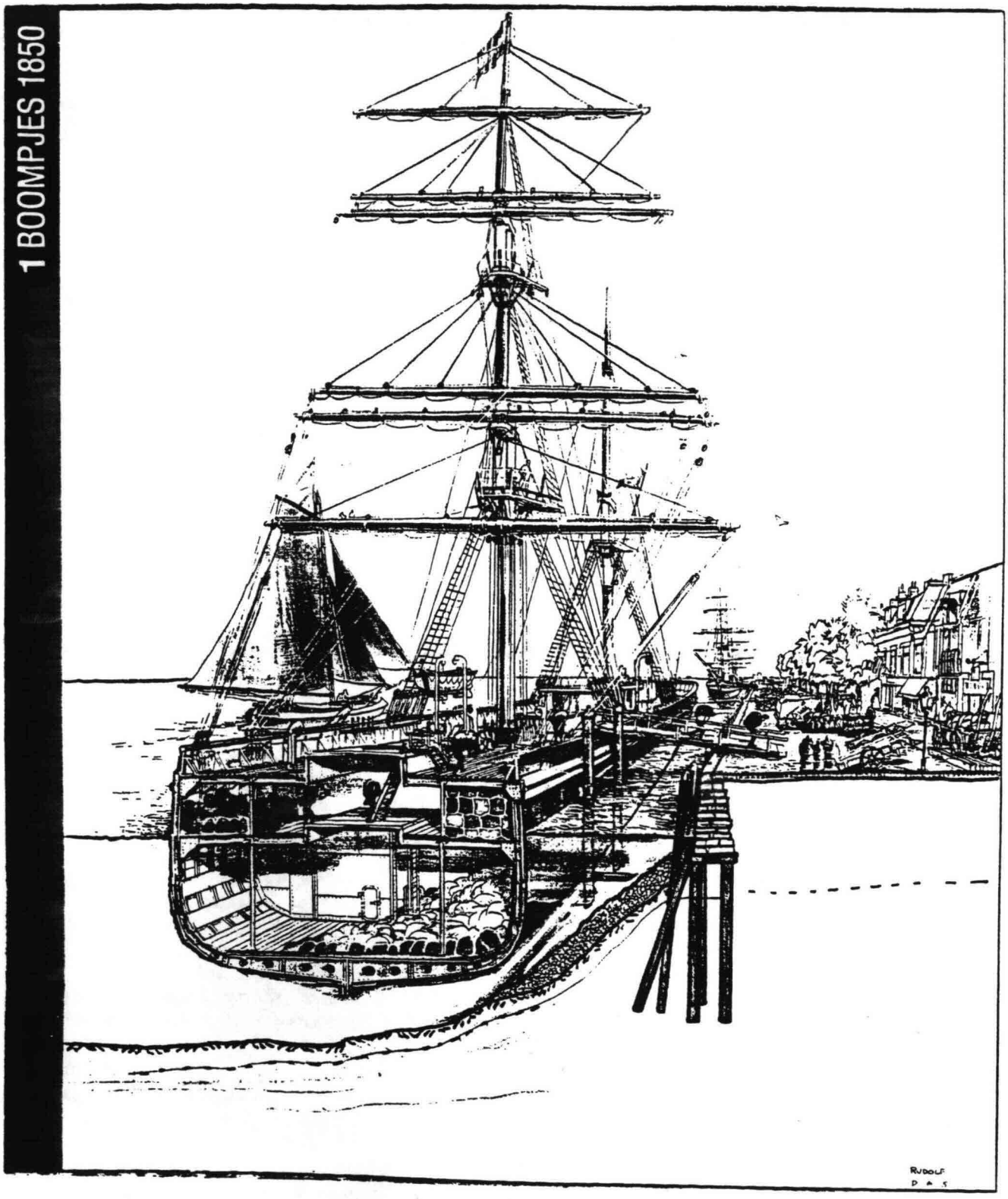


Fig. 1.25. QUAY WALL - ROTTERDAM - Boompjes (The Trees) - 1850
Quay : a brickwall on wooden piles. The brick work is about 3 metres high (-0.30 m NAP to + 2.65 m); the underwater slope is protected by stones. Ships of up 4000 tonnes are moored against wooden poles at a depth of approximately 6.5 m.

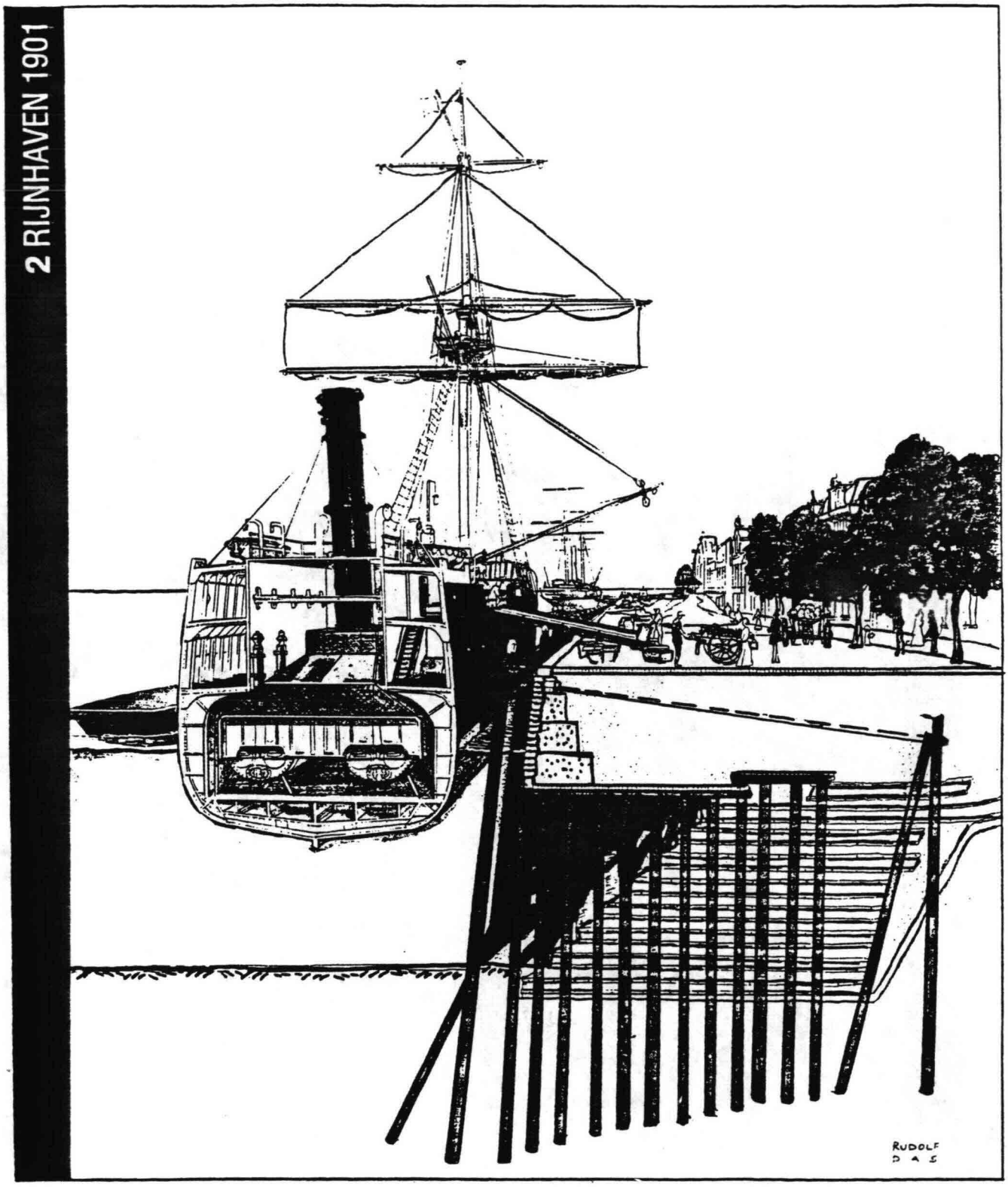


Fig. 1.26. QUAY WALL - ROTTERDAM - Rijnhaven - 1901
Quay : Concrete blocks behind basalt stones on top of a wooden bearing construction. A pile of fascine mattresses prevents sliding of the soil. The fender piles are anchored in order to cope with the forces in the ship's ropes. Height of quay wall: 3.25 m + NAP.
Depth of harbour basin: 7.65 - NAP, accessible for ships of upto 5,000 tonnes.

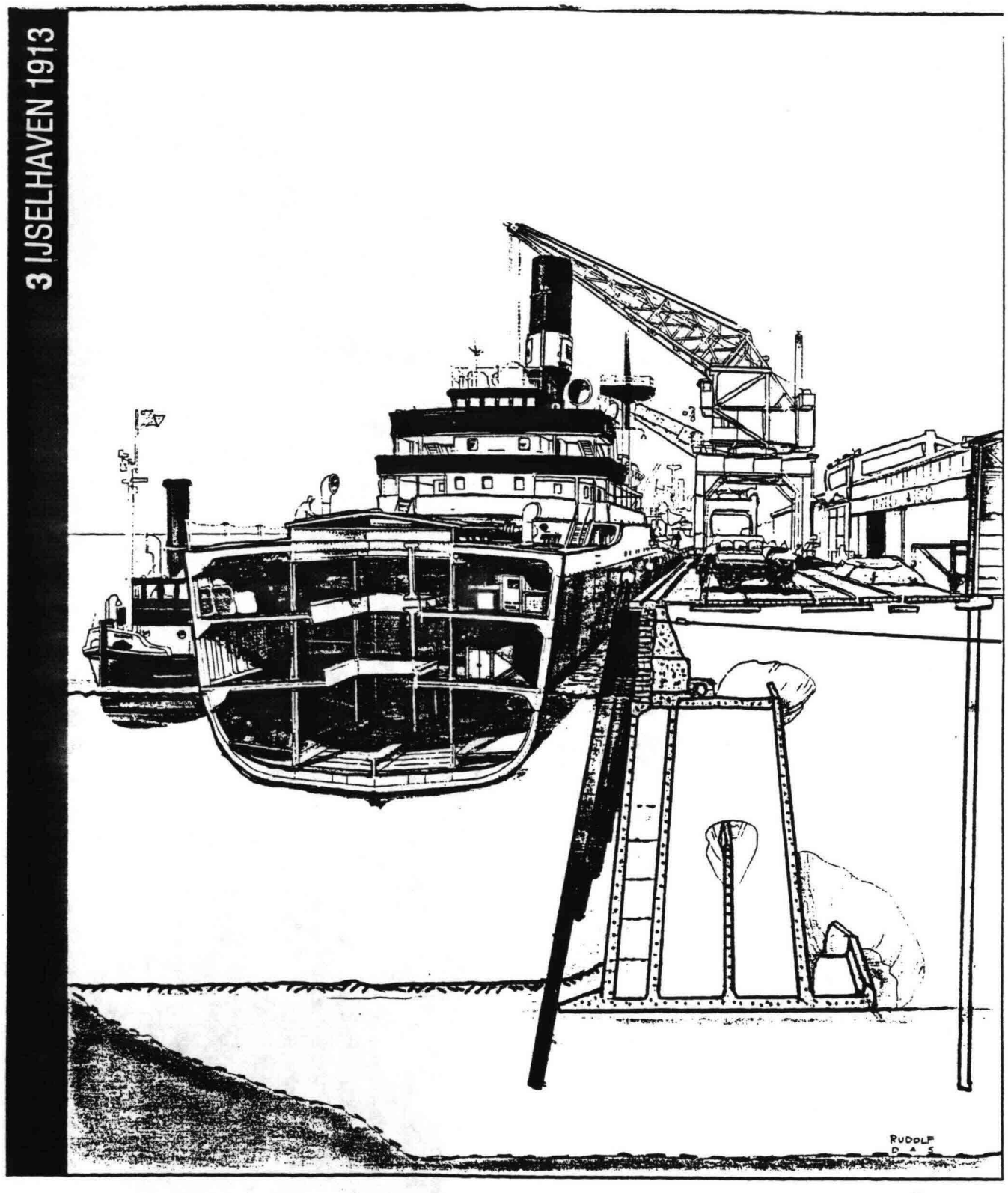


Fig. 1.27. QUAY WALL - ROTTERDAM - IJsselhaven - 1913
Quay: Concrete caisson with concrete blocks and basalt stone on tope. The caissons were made somewhere else in a dock and placed in predredged trench.
Height of quay wall: 3.35 m + NAP. Depth of harbour basin: 9.65 - NAP, accessible for ships of upto 20,000 tonnes. Portal cranes and trucks appeared on the scene; large sheds are available for cargo which has to be stored.

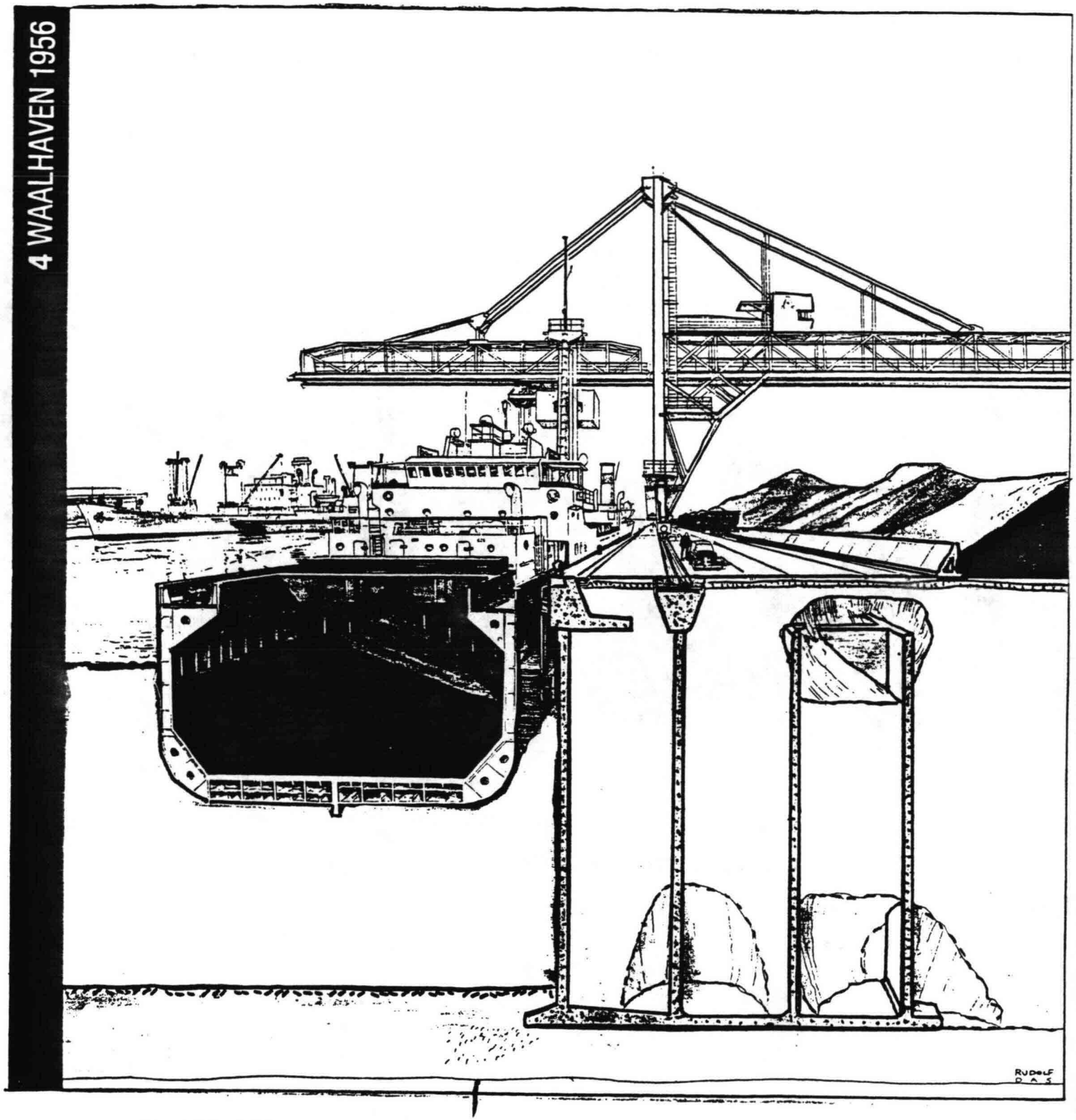


Fig. 1.28. QUAY WALL - ROTTERDAM - Waalhaven - 1956
Quay: Caissons on top of a soil improvement (the soil under the caisson has been replaced by sand.) Concrete superstructure. Height of quay wall: 3.25 m + NAP. Depth of harbour basin: 12.00 m - NAP, accessible for ships of upto 30,000 DWT. On the quay are located tracks, loading bridge with grab and heavy coal - and ore mounds.

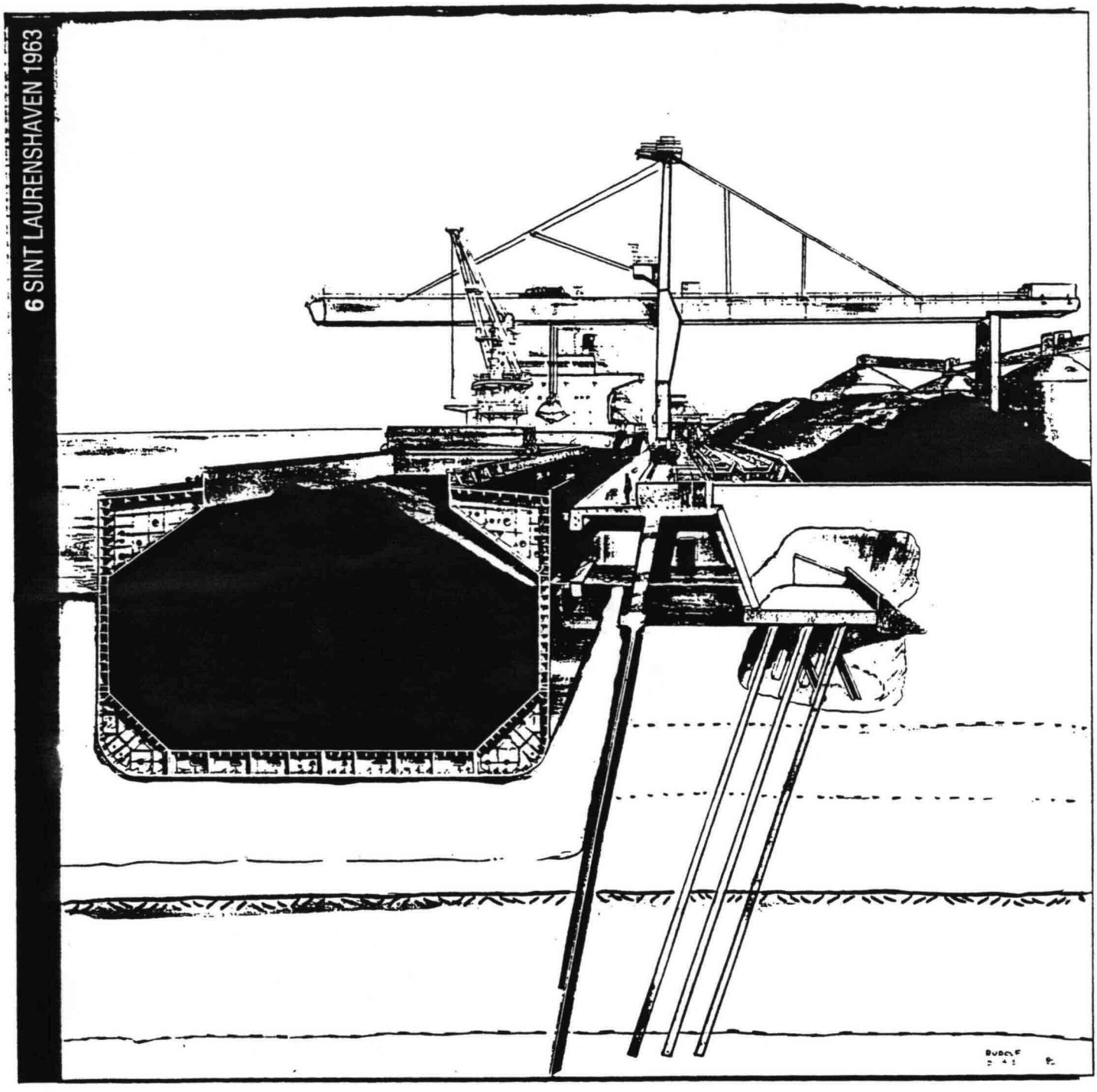


Fig 1.29. QUAY WALL - ROTTERDAM - Sint Laurenshaven - 1963 (Botlek)
Quay: Concrete superstructure on concrete piles and steel sheetpiles. This sheetpile is earth-retaining and bearing. The structure is heavily loaded by the crane and the ore storage. Height of quay wall: 3.35 m + NAP. Depth of harbour basin: 13.65 m - NAP, accessible for ships of upto about 80,000 DWT. At high water ships of upto 120,000 DWT can be received.

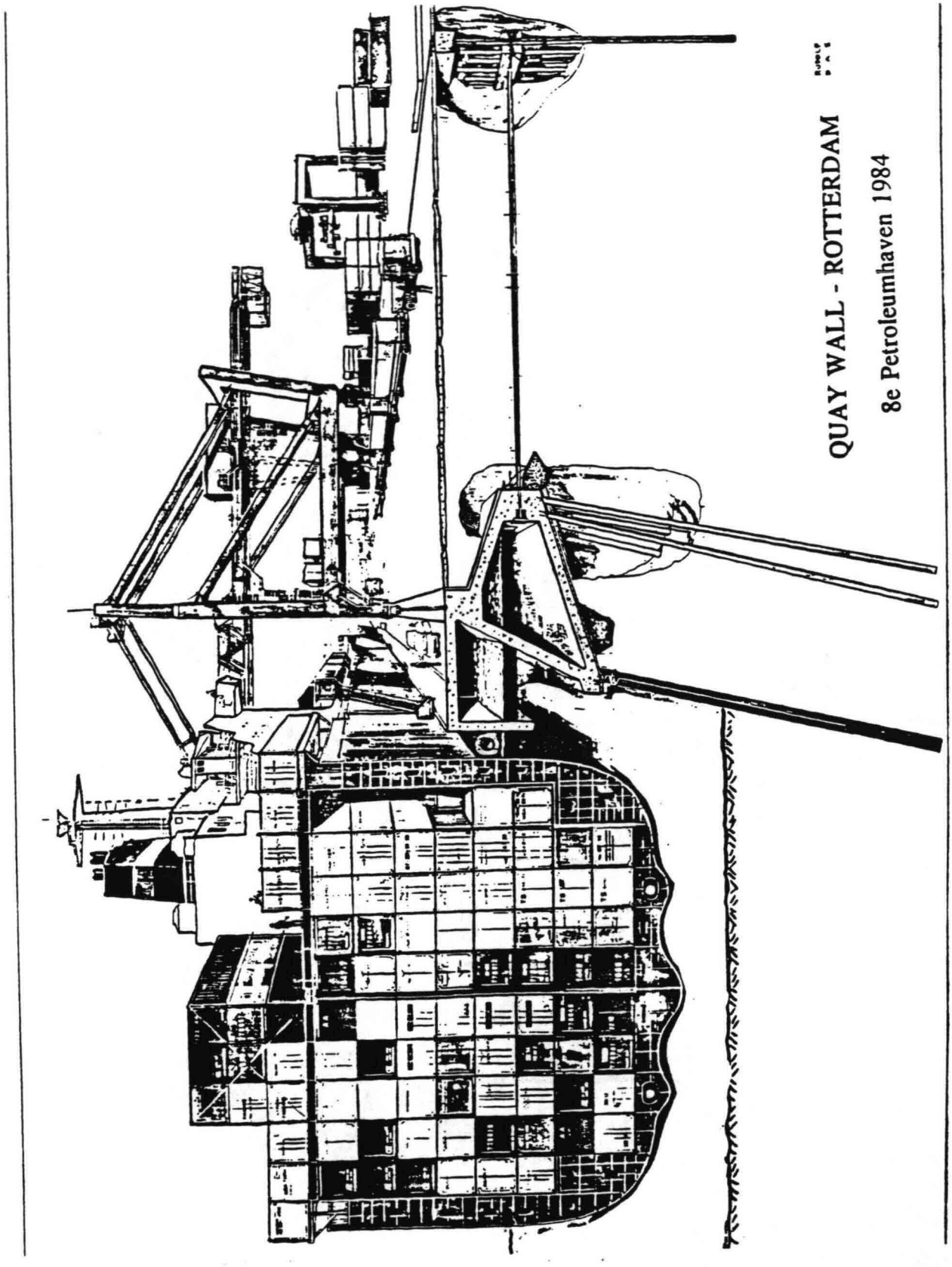


Fig. 1.30. MAASVLAKTE : Euro Container Terminus (ECT) - 1984
Quay: Concrete delta-shaped top construction on top of concrete pile and hollow steel-box-piles. Anchoring to anchor wall that also functions as the foundation for the backleg of the container crane. Height of quay wall: 3.5 m + NAP. Depth of harbour basin : 13.65 m - NAP, (with further deepening to 15.65 m - NAP possible). Accessible for container ships of up to 50,000 DWT.

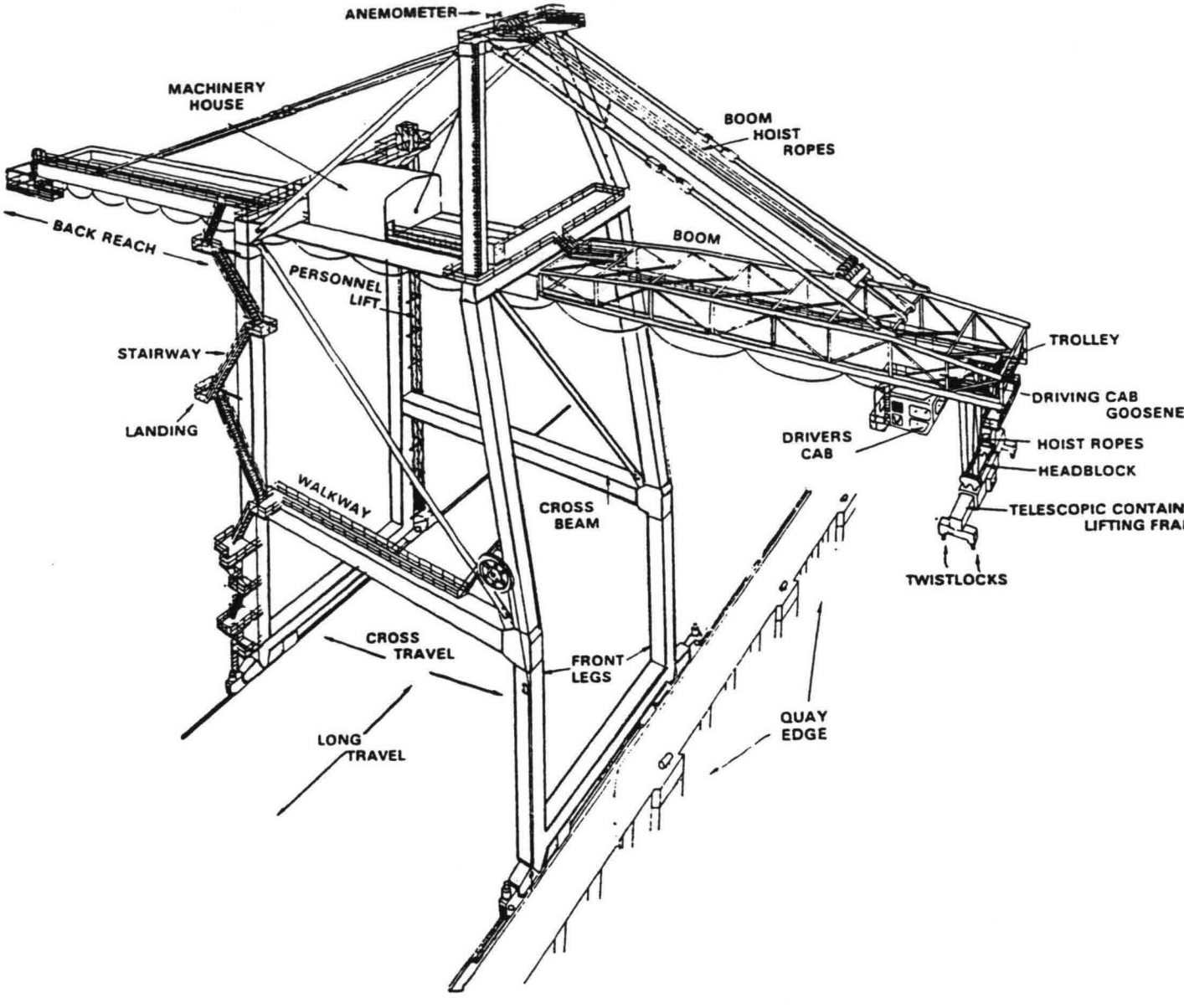


Fig. 1.31. General arrangements of a container crane.

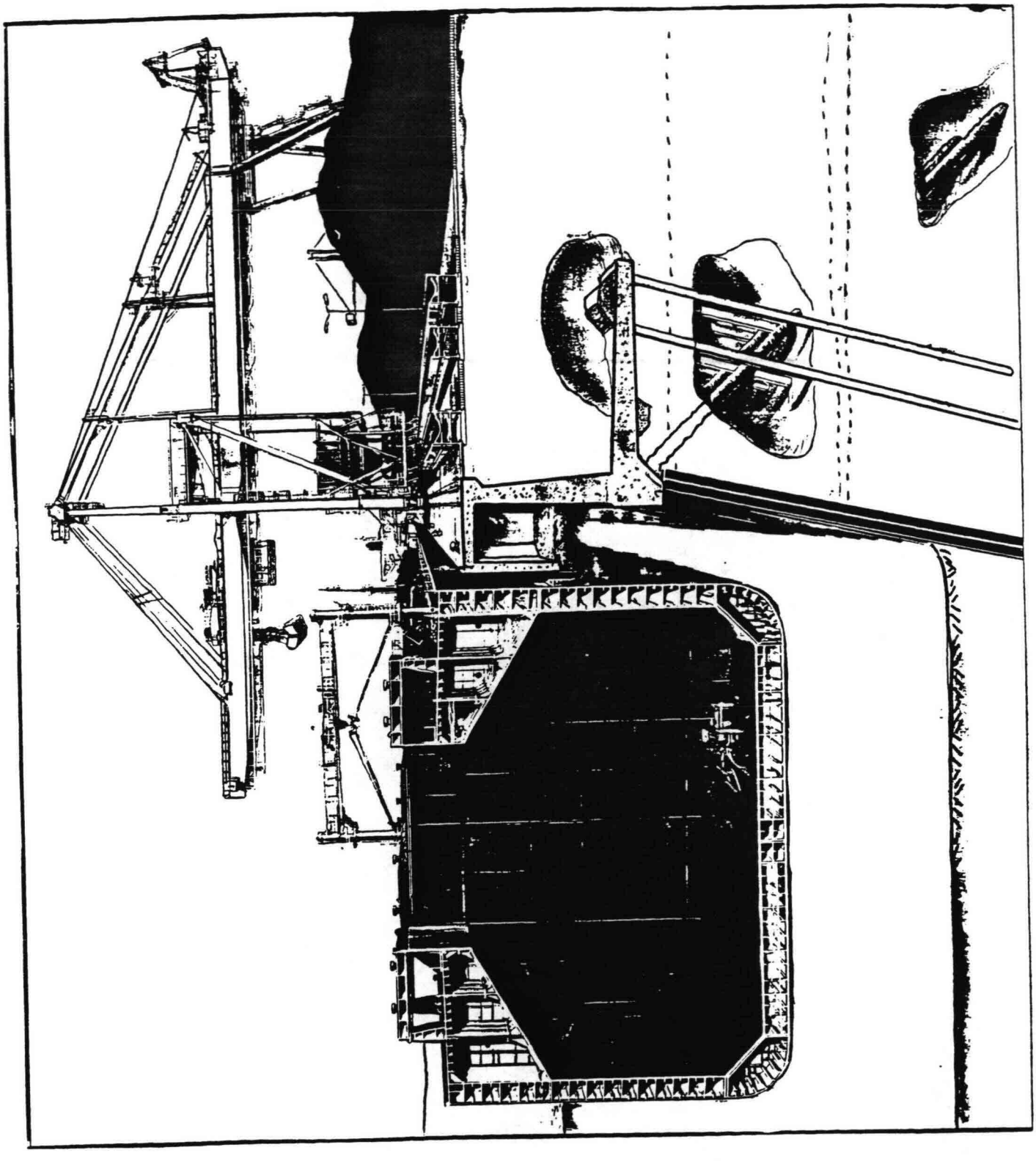


Fig. 1.32. QUAY WALL - ROTTERDAM - Mississippihaven - 1984
Maasvlakte: EMO (European bulkhandling company). Quay wall consists of a heavy concrete L-shaped wall, that at the watersite is founded on very heavy hollow steel box-piles and at the backside on concrete piles. Additional tension piles are provided for sufficient anchorage to withstand the heavy earth pressure. Height of quay wall: 5.00 m + NAP. Depth of harbour basin: 23.00 m - NAP, accessible for ships of upto about 300,000 DWT.

2. Piers, jetties

Another method to overcome the difference between high and low level is to bridge over the slope by a **pier or jetty**. Formerly built in wood, nowadays more and more in reinforced concrete and steel. In fact, a pier or jetty is **simple a slab on piles**. They are useful as **landing-stages** for passenger ships, oil-tankers, etc. When the harbour is very deep and the slope is flat, the length of the pier will become considerable. In that case it is possible to design the pier., like Fig. 2.1. or protect the slope against waves.

Especially the front piles will have a great length and are not sideways supported. That means a **low resistance against horizontal forces**! For that reason the landing-stage will be protected by **dolphins**, (see Chapter 20), which are detached from the jetty. The dolphins must be somewhat flexible in order to resist ship-collisions. It is also possible to resist ship-collisions without unacceptable deformations by means of rubber **fenders**.

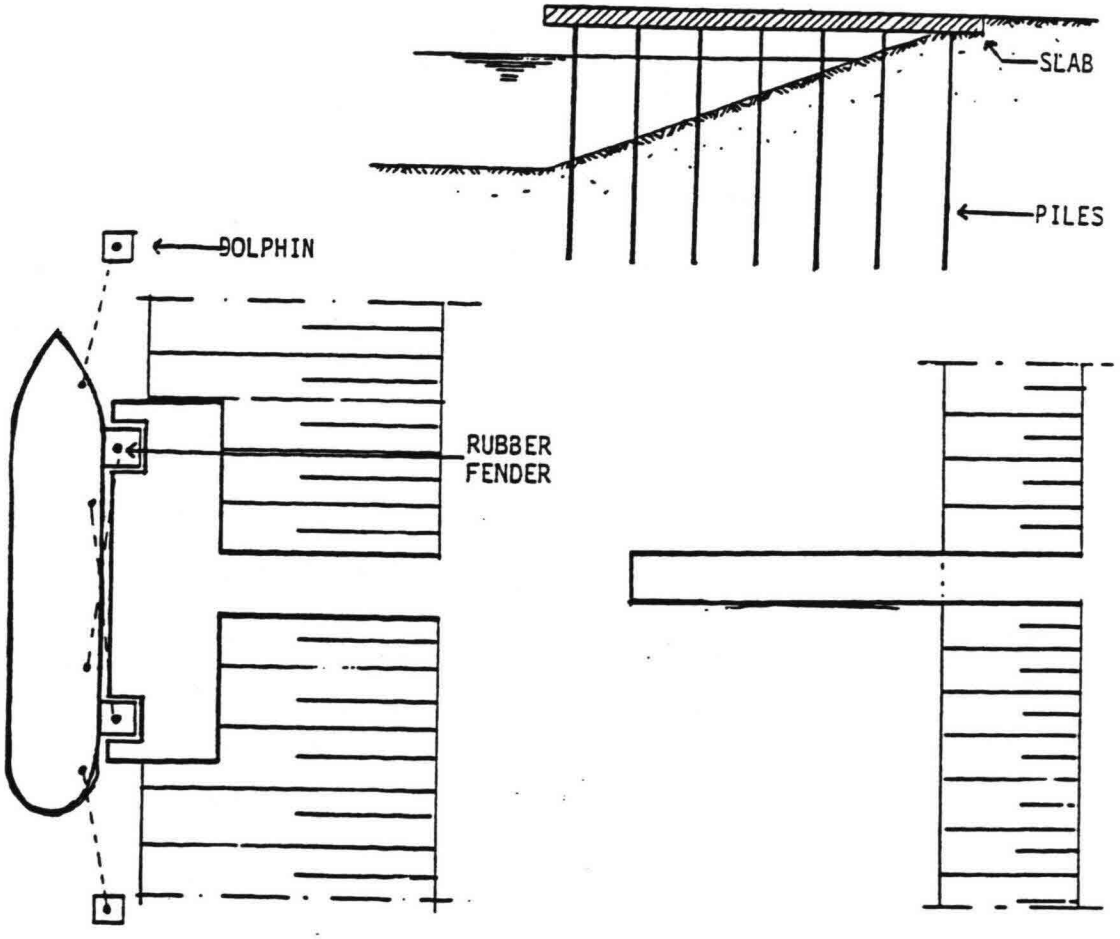


Fig. 2.1. Landing stages (piers, jetties).

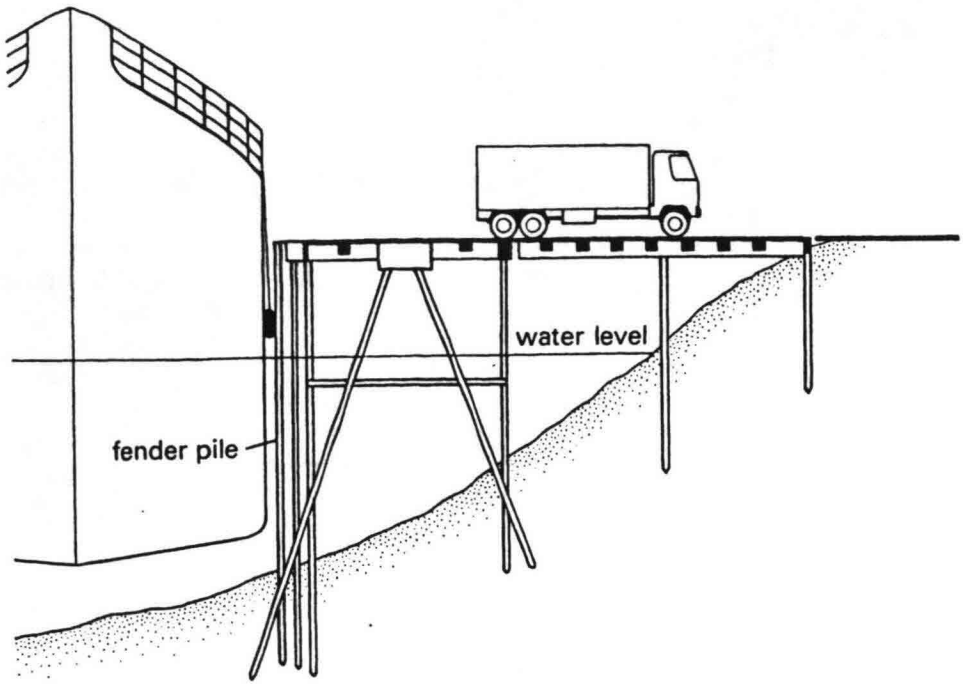


Fig. 2.2. Open jetty, built on driven piles.

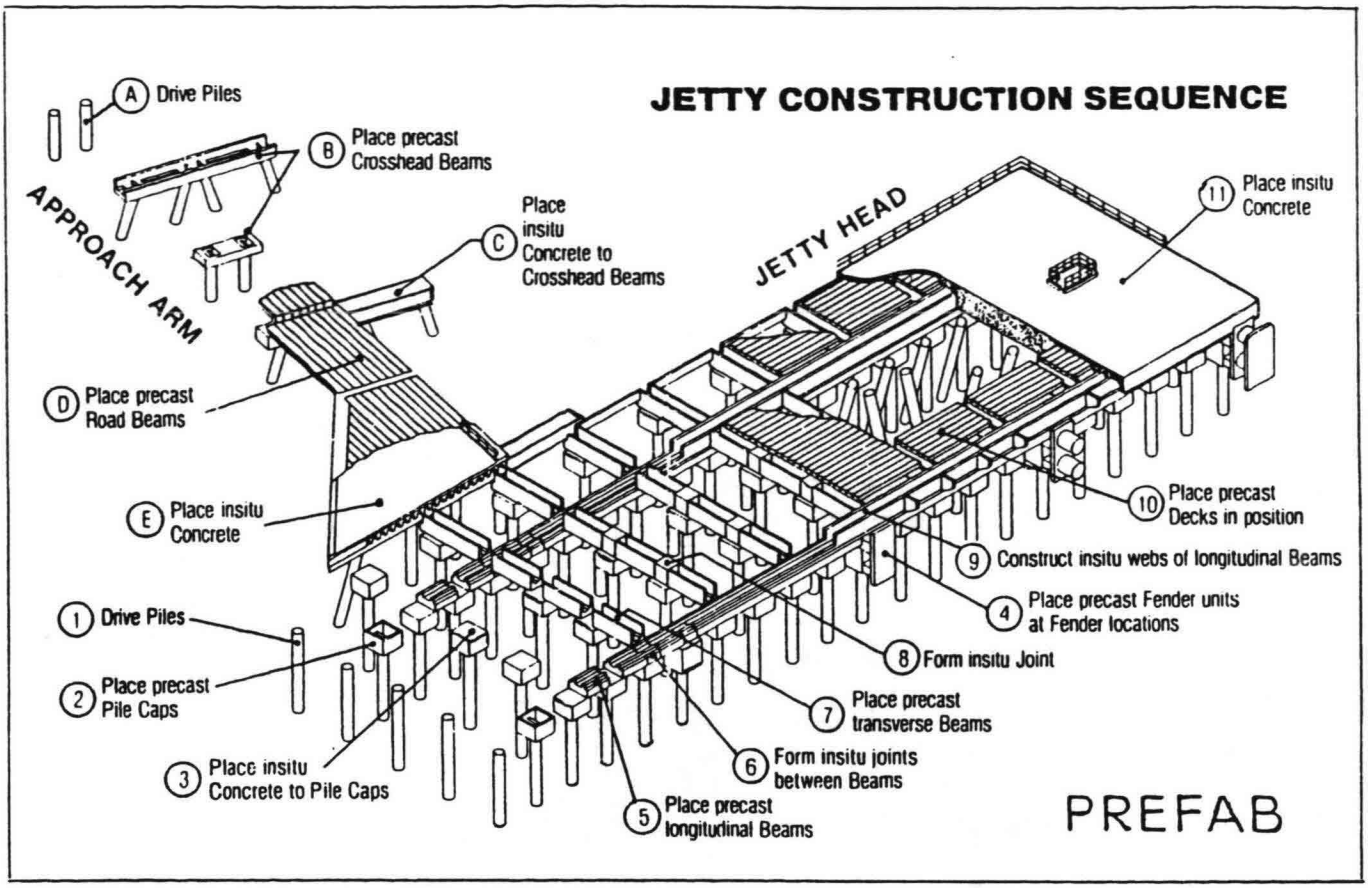


Fig. 2.3. After the Second World War the use of prefabricated elements, in particular for bridges, jetties and factories, became common, as it usually resulted in a saving of shuttering and a shortened construction period.

The **longest deep water jetty** in the world is the Quai Hermann du Pasquier at Le Havre, France, with a length of 1523 m. Part of an enclosed basin, it has a constant depth of water of 9,8 m on both sides.

Jetty, landing stage. A berth, usually one projecting out from the shore line.

Wharf. A berth parallel to the waterfront. It may be of solid or open construction. In solid construction the wharf holds back all the earth behind it like a retaining wall. In open construction no earth is retained; the wharf is carried on piles of timber, steel or precast concrete driven into the bed. Open wharfs can be used only for vessels of shallow draught.

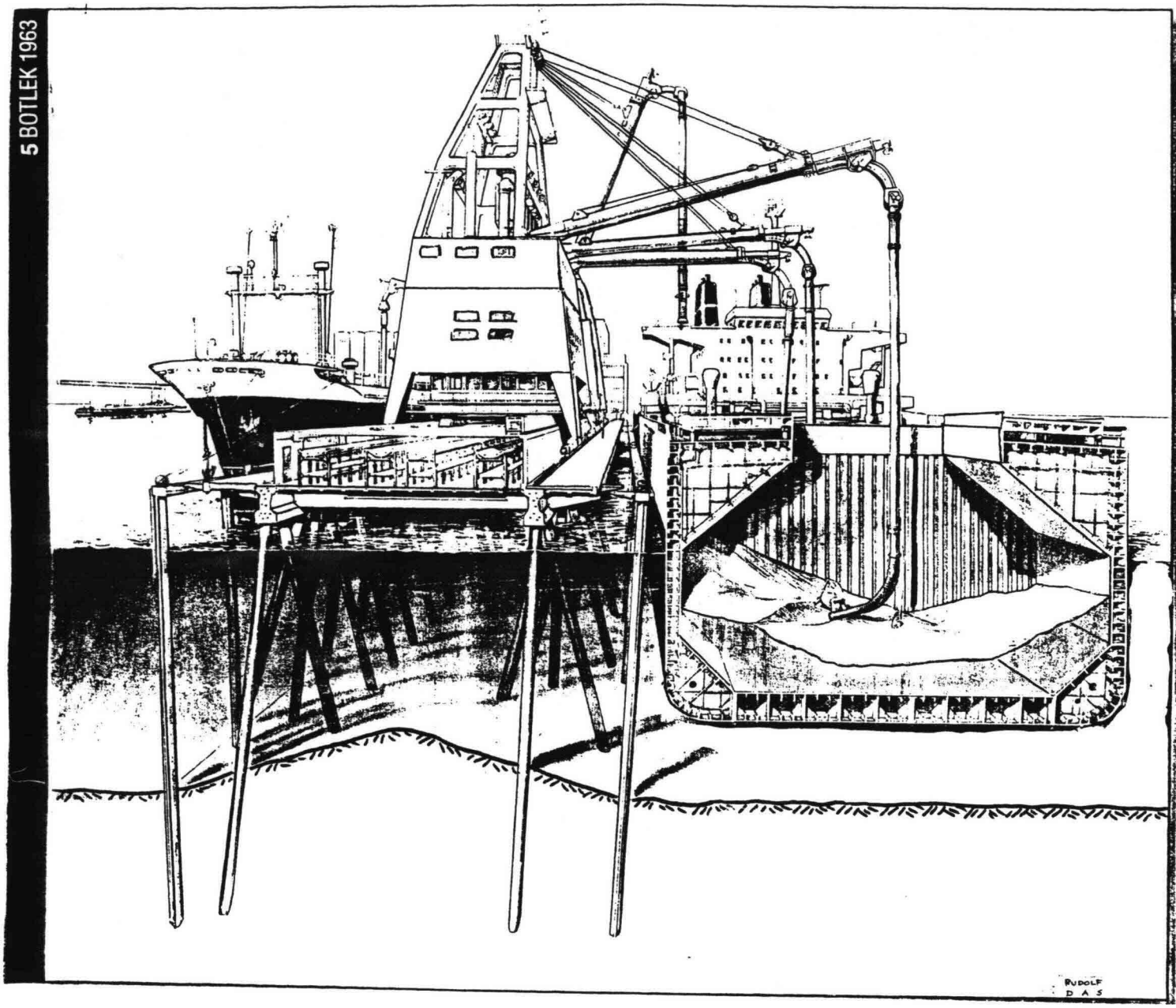


Fig. 2.4. JETTY - ROTTERDAM - Botlek - 1963
Unloading pier for grain carriers. The jetty, with a length of about 425 metres, is made from concrete. The construction is 3.45 m above ordnance datum. The bottom of the port is 13.65 below ordnance datum (suitable for ships of up to about 90,000 DWT). The grain is removed pneumatically from the ship by high-capacity installations on the pier and transported by belt conveyors to the silos on land.

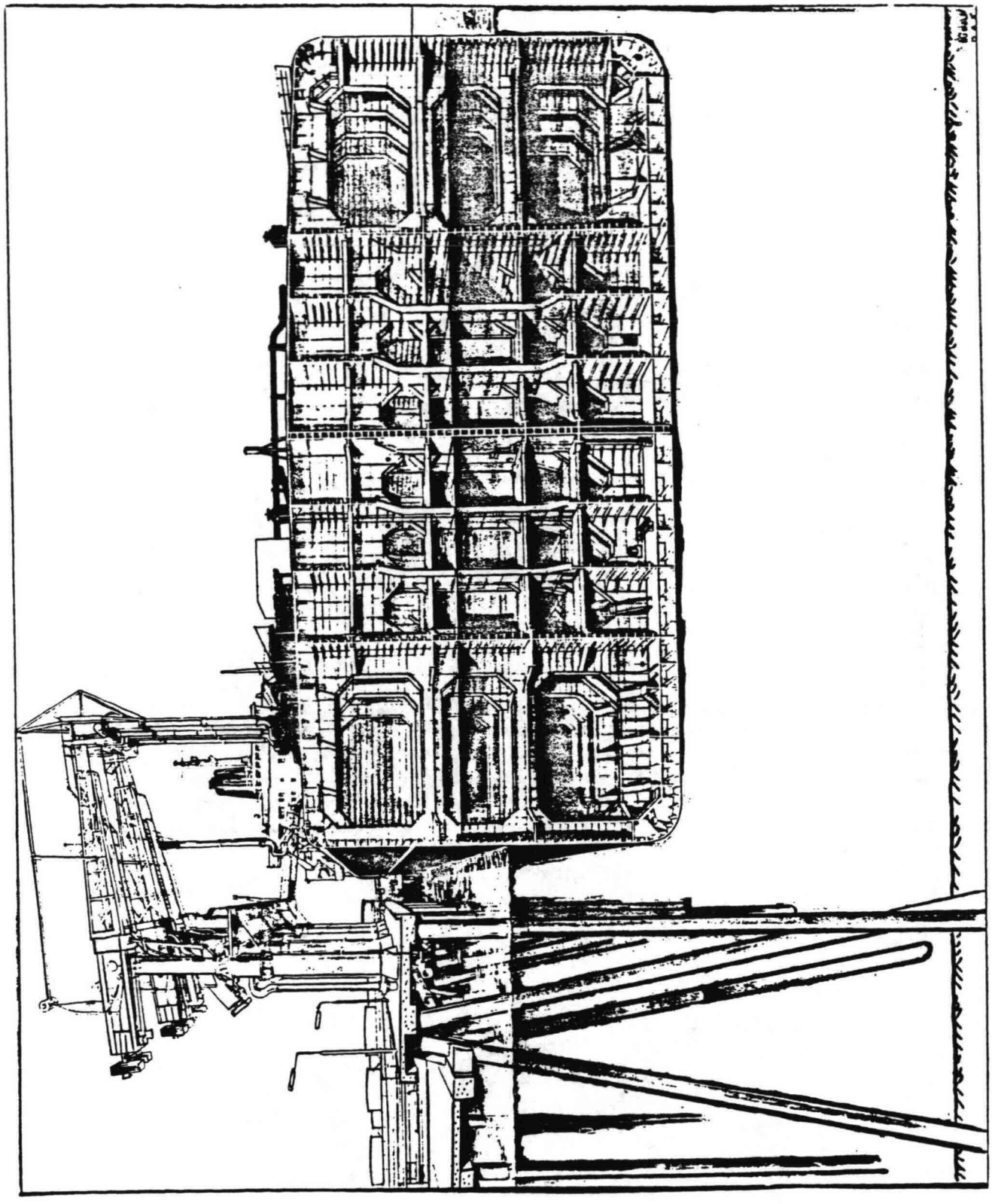


Fig. 2.5. JETTY - ROTTERDAM - 8e Petroleumhaven - 1974
At the oil terminal in 8th Petroleumhaven this unloader, accessible to very large vessels, was built. The pier is 7 m. above ordnance datum, the bottom of the harbour is 23.50 m below ordnance datum. The ship in this picture, which measures about 250,000 DWT, is equipped for bulk transport of inflammable liquids. The pier is equipped with several pumping installations which can be connected to the ship's loading and unloading systems.

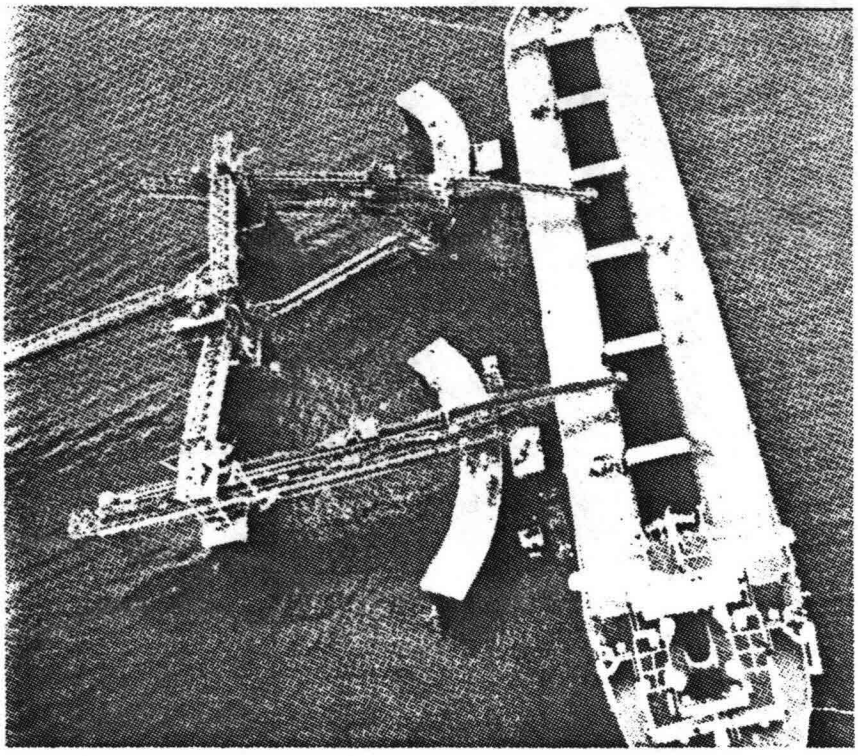
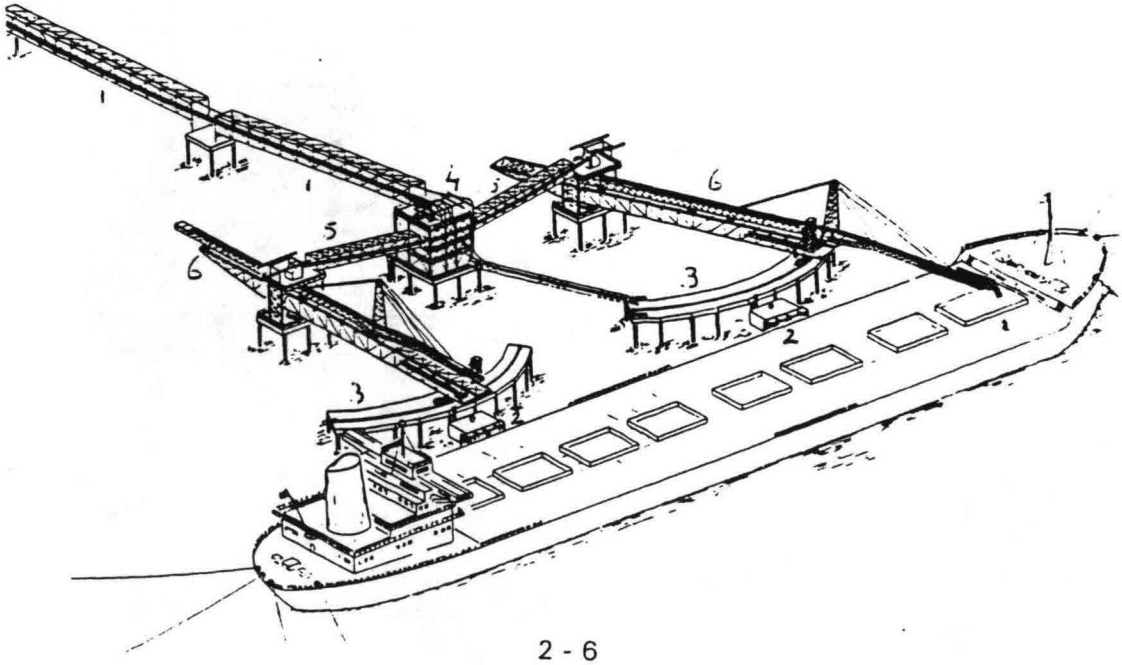


Fig. 2.6. Offshore ore loading facility, Port Lattia, Tasmania, Australia.
In concept the shiploading facility creates a buoy-type offshore anchorage that vessels can approach and depart with or without tug assistance. The loading is done by two 81 m long slewing bridge type shiploaders. Each loader incorporates a 30 m horizontal shuttle movement. Steel box trusses, 60 m long and supported on bents at 60 m spacing were selected as a gallery for the 1.6 km long approach conveyor belt. Completion 1969.

1. Conveyor bridge
2. Dolphins (2 no.)
3. Slewing platforms (2 no.)
4. Drive house (1 no.)
5. Cross-conveyor bridges (2 no.)
6. Slewing-bridge type shiploader (2 no.)



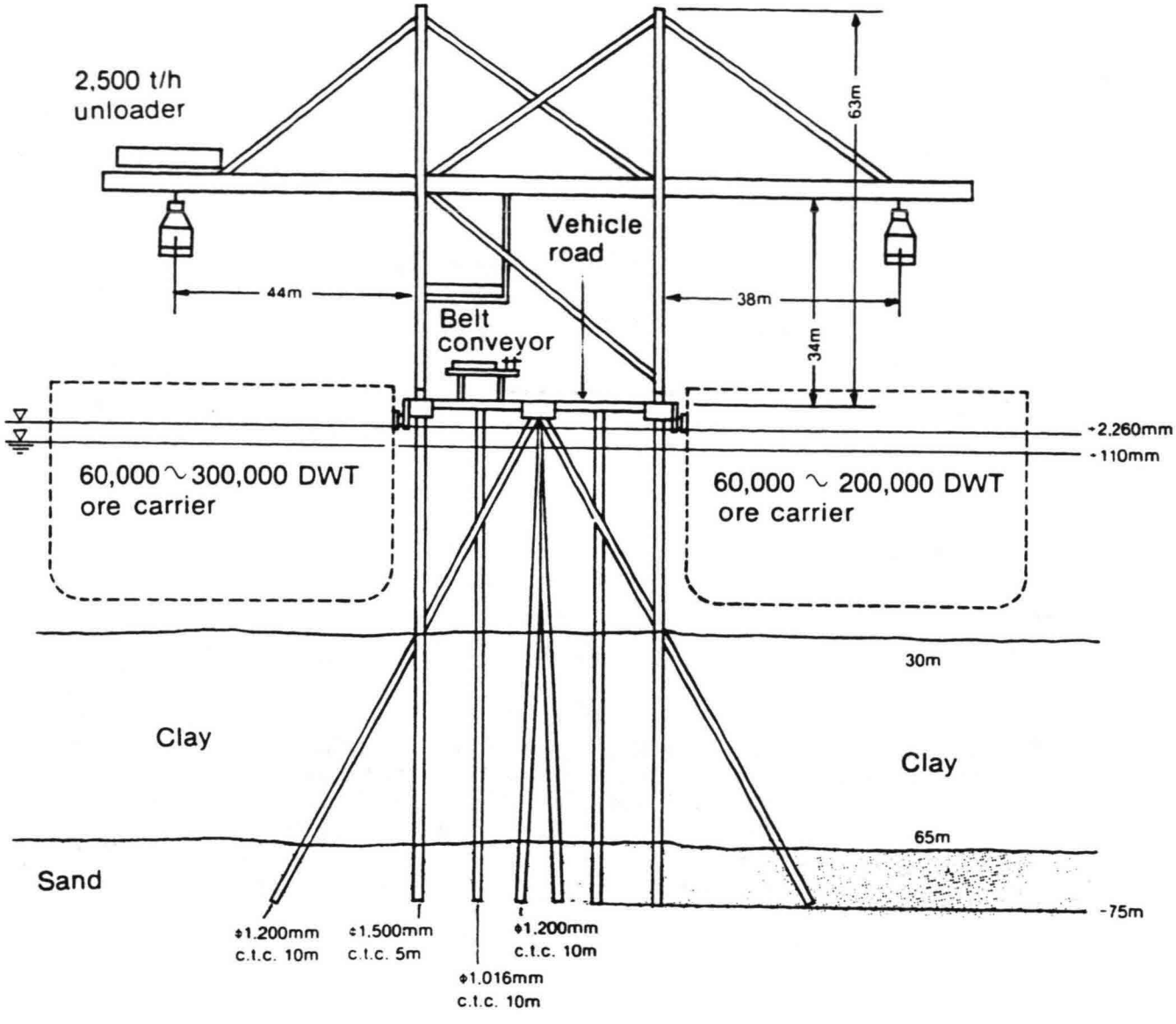


Fig. 2.7. Offshore raw materials berth utilising steel pipe piles (Nippon Steel Corporation's Oita Works - Japan). Commercial availability of large diameter, heavy-walled and long pipe piles, along with advances in execution practice, have made it possible to build deep-water offshore berths.

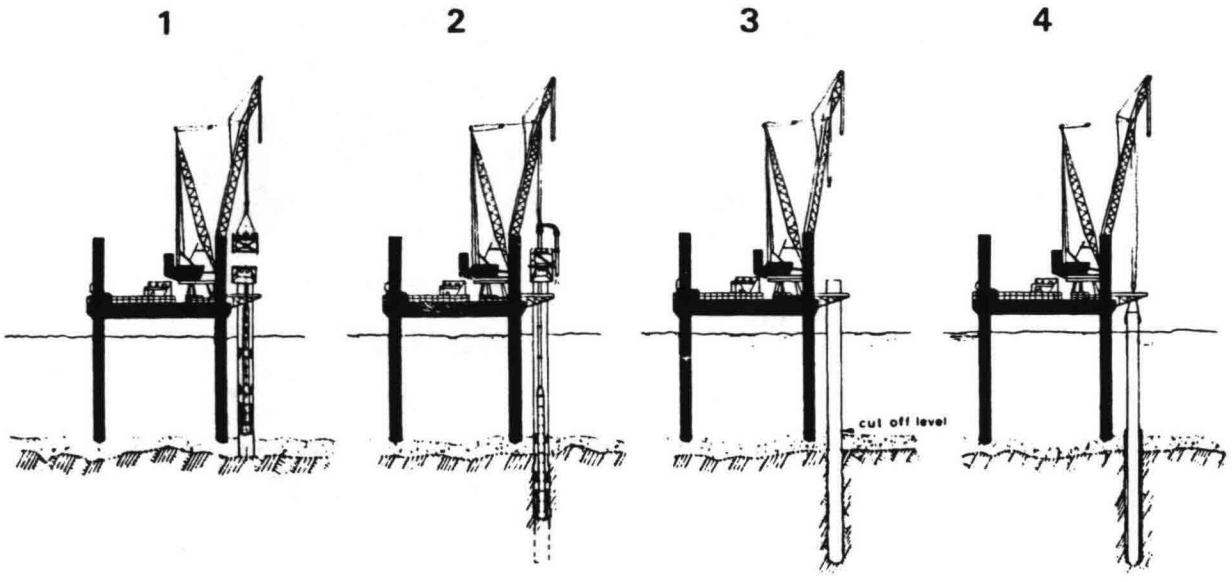


Fig. 2.8. Jetty construction - Crude unloading facility Pahi Bay, Greece.

1. Guide casing through the overburden in contact with bedrock
2. Drilling blind holes of 2,630 and 2,950 mm to target depth
3. Guide casing cut by divers 0.5 m above the bottom of the sea
4. Inserting of pre-fabricated steelpile in open hole and grouting of annulus according to displacement method.

In 1980 a crude oil unloading facility capable of berthing ships up to 500,000 DWT was constructed in the Bay of Pahi approximately 40 kilometres west of Athens. The works included a.o. the installation of eight trestle - and platform-piles four breasting dolphins, erection of pre-fabricated steel top structure sections etc. The marine works, in water depth varying from 17 to 32 metres, were carried out using a self-elevating platform. The berthing line was only 100 metres from the shore (very steep bottom profile).

A preliminary soil investigation revealed that driving of piles was not feasible due to the compressive strength of the bedrock. An alternative scheme was developed resulting in the limited number of eight foundation piles and four breasting dolphins with very large diameters of respectively 2,400 and 2,800 mm. The self-elevating platform with a hydraulically operated side-frame was placed within one metre of the theoretical pile position. A guide casing, outside diameter 3,100 mm, was placed in this frame and positioned by means of hydraulic jacks.

The guide casing was subsequently driven through the overburden using a diesel piling hammer. A 'Wirth' drilling rig was used for drilling holes of 2,630 and 2,950 mm. Upon completion of the drilling operation the guide casing was cut by divers 0.5 m above the bottom of the sea. The pre-fabricated permanent steelpile was inserted in the open hole and the annulus grouted (maximum quantity 28 m^3).

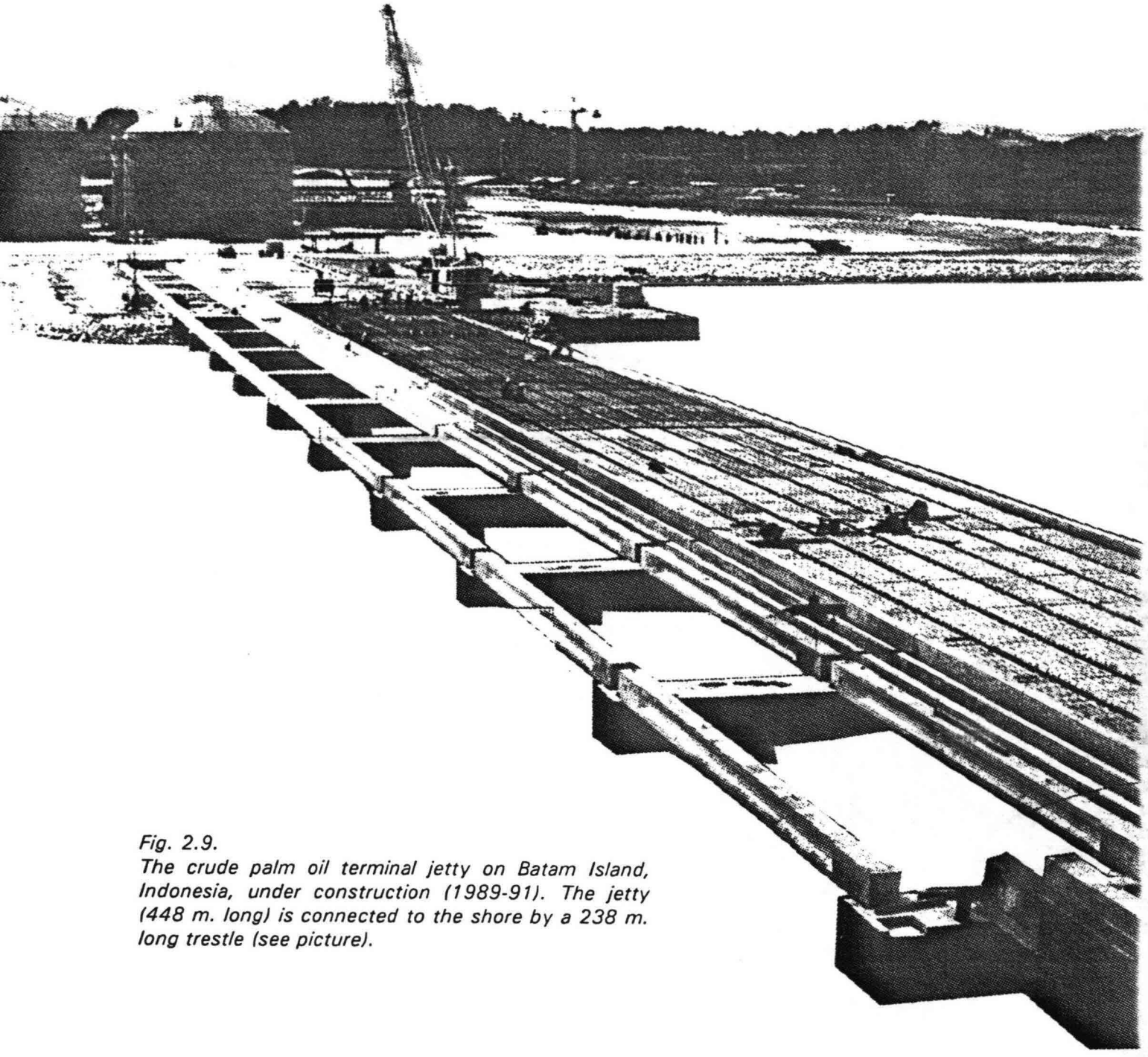


Fig. 2.9.
The crude palm oil terminal jetty on Batam Island, Indonesia, under construction (1989-91). The jetty (448 m. long) is connected to the shore by a 238 m. long trestle (see picture).

3. Bridges, viaduct, aqueducts

Bridge. A structure which covers a gap. Generally bridges carry a road or railroad across a river, canal or another railway/road. Bridges can be **fixed** or **movable.**

Viaduct. A bridge of many spans. A **viaduct** conveys a road (traffic or railway) across a **road** (traffic or railway).

Aqueduct. A conduit (which may include tunnel and bridge) for carrying water over long distances. The term is usually confined to water carrying bridges.
A **aqueduct** conveys a **water-course** across a **valley, a road** or a **railway.**

These crossings can be perpendicular and inclined.

Bridge bearings. The supports on a bridge pier, which carry the weight of the bridge and control the movements at the bridge supports, including the temperature expansion and contraction. They may be metal rollers, knuckles or slides, or merely rubber or laminated rubber (rubber with steel plates glued into it).

Bridge cap, bridge pier. The highest part of a bridge pier, on which the bridge bearings or rollers are seated. It may be of stone, brick, or plain or reinforced concrete, usually the last for heavy loads.

Bridge pier. A support for a bridge. It may be of masonry, timber, concrete or steel, but in any case is founded on firm ground below the river mud.

Bridge deck. The load-bearing floor of a bridge, that which carries and spreads the loads to the main beams. It is either of reinforced concrete, prestressed concrete, welded steel or (rarely) light alloy.

3.1. Fixed bridges

Long span, limiting span. A span is long for a particular slab, beam, girder or bridge type if it is near the greatest economical length for that type. Some long spans are: solid concrete slab, 10 m; hollow-tile concrete slab, 11 m; arches, 300 m; cantilever bridges, 600 m; suspension bridges, the Golden Gate Bridge, 1280 m between centres of support.

The spans of long-span marine bridges are decided on the basis of site conditions on the one hand and construction costs (from the engineering viewpoint) on the other. Nevertheless, most bridges are required to have long spans to permit the safe passage of large ships. The following types of structures have proved economically and structurally satisfactory for spans of over 400 m:

- suspension bridge, maximum span - 1,470 m, Humber Bridge (U.K.)
- cable stayed bridge, maximum span - 458 m, Hoogly Bridge (India)
- Gerber truss bridge, maximum span - 549 m, Quebec Bridge (Canada)
- arch bridge, maximum span - 504 m, Bayonne Bridge (India)

The recent tendency is to use the suspension type for spans exceeding 500 m, and to use the diagonal cable bridge for spans of less than 500 m.

850 BC	Bridge built over **River Meles** (Izmir, Turkey), still standing
600 BC	First stone bridge in **Rome**
512 BC	Mandrokles of Samos builds a pontoon bridge over the **Bosporus** for Darius I of Persia's army
265 BC	Suspension bridges in **China** hang from spun-cane-fibre ropes
206 BC	Iron-chain suspension bridges erected in **China**
99 AD	**Trajan's** bridge over **Danube** near Orsova (the Iron Gate at Turnu Severin), with 20 stone piers joined by wooden arches
494 AD	Submersible suspension bridge built in **China** over the Huai River
1176-1209	**London Bridge** built; not replaced until 1831
1377	**Duke of Milan** bridges over River **Adda** with a 72 m span arch
1750	**Westminster Bridge** piers built using cofferdams of dovetailed squared timbers and wooden foundations 9 x 24 m
1774	Perronet's **Pont de Neuilly** (not demolished until 1956)
1779	Abraham Darby's cast iron bridge spans the **Severn** at **Ironbridge**
1801	James Finley's iron-link suspension bridge built over **Jacob's Creek**, Pennsylvania, with two spans of 21 m, patented in 1808, followed by similar but longer bridges
1826	Telford's **Menai suspension bridge** (part of London - Holyhead road)
1834	**Grand Pont Suspendu** over the Sarine at Fribourg, Switzerland, built by Joseph Chaley, spanning 273 m, a world record until 1849. This was the first European wire-rope suspension bridge
1849	In France, Joseph Monier's **reinforced-concrete** tubs for orange trees; he patents RC beams in 1877, later RC bridges **Brittannia Bridge**, tubular wrought-iron railway bridge over the Menai Strait, built by Robert Stephenson, helped by Fairbairn and Hodgkinson (see 1826) **Ohio Bridge** at Wheeling, West Virginia, built by Charles Ellet, a French American, to 308 m span, beating Charley's suspension bridge record of 1834
1850	**Basse Chain** Suspension Bridge over the Maine at Angers, France, collapsed when 478 soldiers marched over it; 226 died. This bridge was on two steel wire ropes and spanned 102 m. Virtually no French rope suspension bridges were built then until after 1870
1851	**Pneumatic caissons** used for building the piers of bridges at Rochester and Chepstow
1857	**Cezanne's bridge** over the River Tisza, Hungary, uses compressed-air caissons for building the bridge piers and an early steamdriven concrete mixer
1859	Bridge over the River Rhine at **Kehl**, near Strasbourg, built by F. Saint-Denis, using pneumatic caissons for the piers
1877	Monier patents **reinforced-concrete beams**
1879	**Tay Bridge** (UK), the world's longest bridge, collapses, 73 drowned
1883	**Brooklyn Bridge** completed; suspension bridge by Roebling, 486 m span
1883-90	**Forth Railway Bridge** built to the design of Fowley and Baker
1905	**Victoria Falls Bridge** over the Zambezi River; 150 m span, designed by Ralph Freeman
1930	**Plougastal Bridge** near Brest built by Freyssinet with three arches of 187 m span; destroyed in 1945 and then rebuilt Robert Maillart's **tied-arch** bridge for the Rhaetion Railway over the River Landquart at **Klosters**
1940	**Tacoma Narrows suspension bridge** fails because of flutters in wind.

Pontoon Bridges

A temporary or permanent bridge which floats on pontoons moored to the river bed. Permanent bridges are built in this way when the foundation material is very poor. In this case the pontoons may be of reinforced concrete (Lake Washington Bridge, near Seattle).

Transporter Bridges

A bridge consisting of a lattice girder spanning between two towers at each side of a gap. It carries vehicles across the gap in a container slung at road level by ropes under a crane crab on the girder. A lattice is a description of an open girder, beam or column,etc., built up from members joined by intersecting diagonal bars of wood, steel or light alloy.

Bailey Bridges

A military bridge developed in Britain about 1942, the first British welded lattice bridge. It is built in panels, which are connected at the four corners by steel pins to the next panels. To build bridges of higher strength, panels can be connected together in two or three storeys with one, two or three panels in each storey. No lifting tackle is needed; each part can be lifted by one or at most two people, so it is a most useful temporary bridge.

Cantilever Bridges

Generally a symmetrical three-span bridge of which each of the outer spans is anchored down at the shore and overhangs into the central span about one third of the span. The suspended span, resting on the cantilever arms, occupies the remaining one third of the central span.

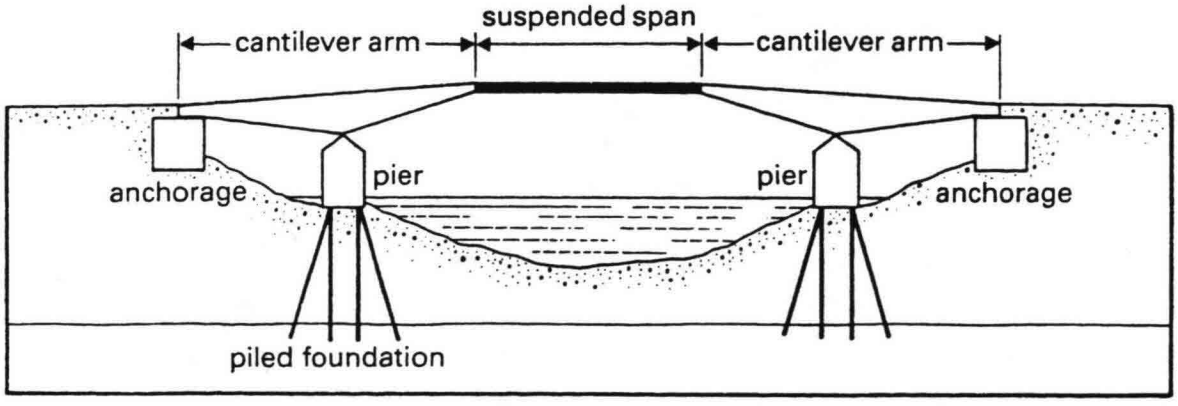


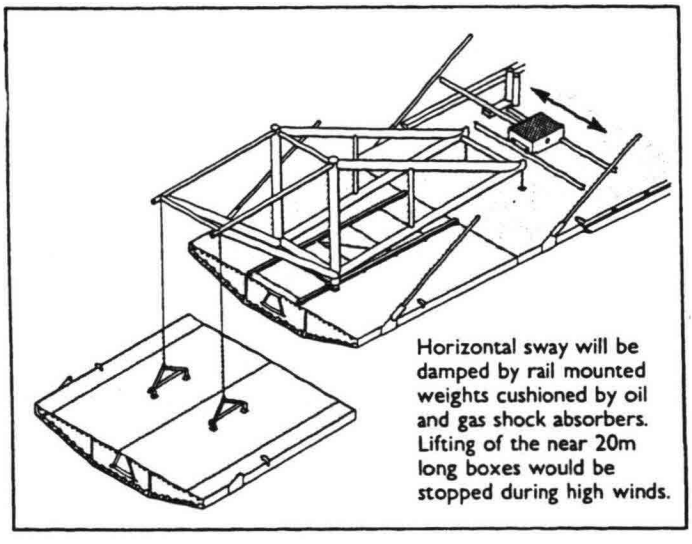
Fig. 3.1. Cantilever bridge.

Forth Bridge (Scotland) - a cantilever bridge over the Firth of Forth near Edinburgh, built in 1890 of plates riveted into shapes like boiler shells. It has the unusual number of two main spans of 520 m clear (slightly less than the Quebec Bridge), flanked by two side spans of 210 m each. The centre pier therefore carries a bridge element which overhangs on both sides. It has, however, a width at the base of 76 m. The railway bridge is 1630 m long overall.

The **Quebec Bridge** (Canada, 1917) - one of the largest bridges in the world - is a normal cantilever bridge with central span of 550 m, with 157 m side spans.

Pont de Normandie, Le Havre to Honfleur crossing over the river Seine, France. In November 1992 work begun on the concrete deck elements of the two longest cantilever structures ever attempted. Still to be resolved is the method of controlling wind induced oscillations on cantilevers which will be just over 426 m long (the ends of the mainspan are 116 m long) before closure. A wind flutter restraint for the cantilevers while Danish contractor Monberg & Thornsen lifts the 624 m of central section steel box deck is still undecided. A choice has to be made by March 1993 between an expensive array of cables anchored 250 m upstream and downstream in the bed of the river Seine and a tuned mass damper of 40 t mounted on each cantilever. With a 856 m span it will be the world's longest **cable stayed span**. The two pylons will have a height of 214 m. Flint boulders met in the piling subcontract costed the job nine month delay. Total estimated cost: around Dfl. 600 million.

Fig. 3.2.



Arch bridges

Arch. A beam curved usually in a vertical plane, for carrying heavy loads such as bridges. Arches are usually made of any solid material but usually of light alloy, pre-stressed concrete, steel or reinforced concrete, stone, mass concrete, timber or brick.

Voussoir Arch. An arrangement of wedge-shaped blocks set to form an arched bridge. After about 1910 this method was completely displaced by concrete or steel bridges, which where both cheaper and lighter in weight, until 1964, when the **Parramatta Bridge** was completed.

Sydney Harbour Bridge (Australia, 1932). A steel, two-hinged trussed arch bridge of 509 m clear span, erected by temporarily cantilevering the ends from each shore until they met in the middle.

Kill Van Kull Bridge (USA) - now called **Bayonne Bridge**. A steel, two hinged trussed-arch bridge near New York of 510 m clear span built in 1931, the longest of its type in the world. Unlike Sydney Harbour Bridge it was built with the help of intermediate supports.

Plougastel Bridge (France, 1930). A bridge first built in 1930 over the Elorn River in North-West France, having three arch spans of 187 m centres. It is of reinforced concrete with a very small amount of reinforcement (less than 0.3 % total) and is also known as one of the earliest sites where Freyssinet used flat jacks. The formwork for one arch was built on the shore and floated out on barges into position under the first arch. When this arch was built, the formwork was lowered and floated away to the next arch. The bridge was destroyed in 1944 but rebuilt to the original design.

Träneberg Bridge (Sweden). A reinforced concrete-arched bridge near Stockholm with a clear span of 181 m. The arches are rectangular, hollow box-girders with about 0.75 % steel.

Sändo Bridge (Sweden). Until its completion of the Parramatta Bridge, this was the largest reinforced-concrete arch bridge built, having a clear span of 264 m. The concrete in the 200 mm test cubes had an average crushing strength of 38 N/mm2 at only seven days. It spans the Angerman river, north Sweden.

Parramatta Bridge, Gladesville B. (1964). An elegant arch, and the longest concrete span in the world, over the Parramatta River at Gladesville, Sydney, a short distance from the Sydney Harbour Bridge over the same river. This arch bridge, built of 50-tons precast concrete blocks, is a revival of the ancient technique of the **voussoir arch**, which has been moribund for 50 years. The 6-lane carriageway is 22 m wide and has 1.8 m footways. Maunsell and Partners of London designed it.

Segmental Bridges

A bridge built usually of precast concrete segments held together by prestressing tendons, placed by incremental launching or by a launch gantry, or cast on a form-carrying gantry. This method is popular because it avoids the expense and trouble of supports in the spans, and can speed construction. Precasting of spans can begin while foundations are being dug. The **Pont de Ré** is a fine curved bridge of this type. The St-Jean Bridge at Bordeaux is 475 m long but has no expansion joints except at each end. The intermediate supports have neoprene bearings, which allow temperature movements. A bridge at Cardiff has segments joined by glue.

Tendon. A prestressing bar, cable, rope, strand or wire.

Pont de Ré (France, 1988). A bridge more than 3 km long (from La Rochelle on the west coast of France to a holiday resort, the Ile de Re), built in only 19 month by the contractor Bouygues in 1987-88. It has 29 concrete box-girder spans of 110 m each. Its extraordinarily fast construction speed was probably in large measure due to Bouygues's launching gantry, designed for the job and weighing only 500 tons, carried on 48 wheels, each one motorized. Only six hours were needed to advance the gantry 110 m from one span to the next.

A launch gantry is a long movable beam above a bridge being built, which moves forward as the bridge is built, placing sections of the spans.

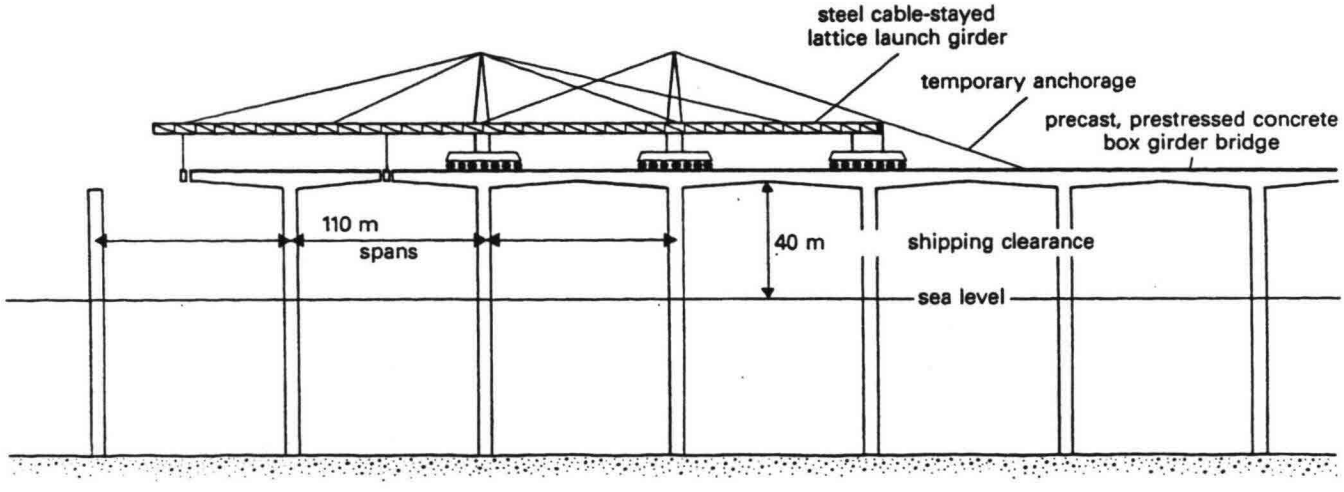


Fig. 3.3. Travelling launch gantry building the Pont de Ré from La Rochelle to the Île de Ré (France).

Incremental launching. Launching from one bank may be the only way of bridging a gap in battle conditions. In peace time, especially for a segmental bridge of many equal spans, it can be economical. The complete bridge is built one span or half-span at a time on the bank and launched on the prepared piers as each unit is completed. To reduce the launching stresses from overhanging weight, a lightweight metal nose (span or half-span) may be attached purely for launching and discarded when the last gap is closed.

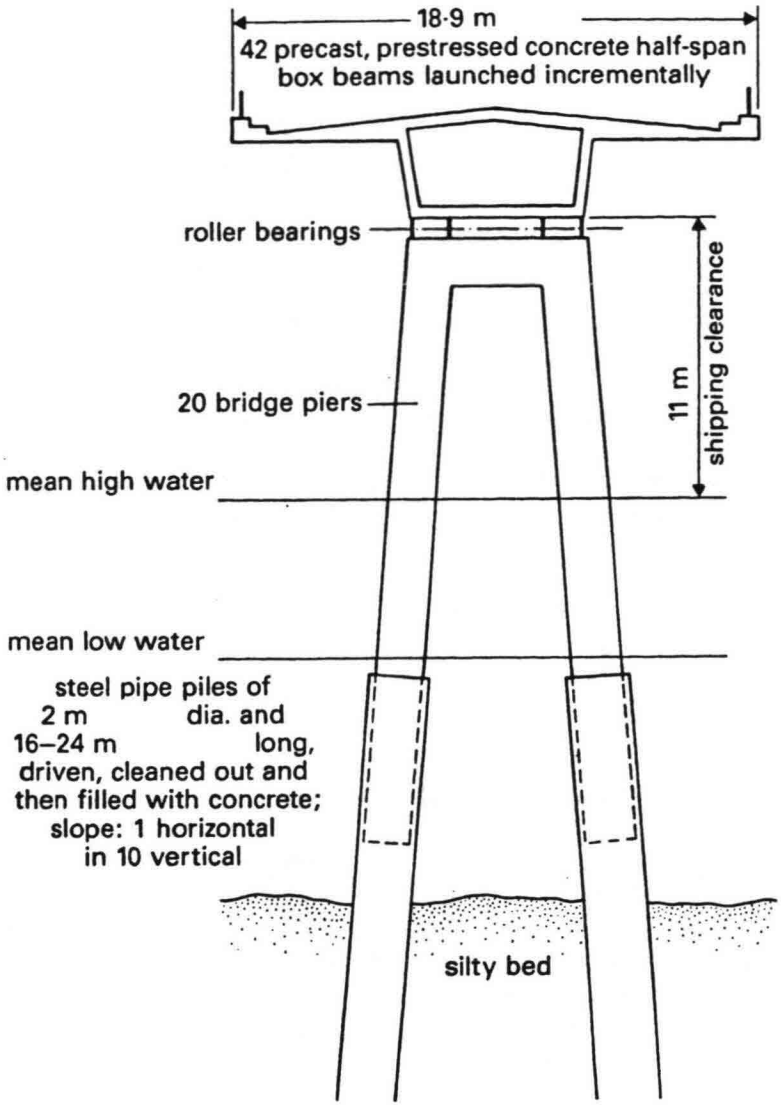


Fig. 3.4. Incrementally launched roadbridge, 890 m long, over the Dornoch Firth, completed in 1991 (North of Scotland). 19 spans of 43.5 m and two-end spans of 31.7 m. The 20 piers are carried on pairs of 2.1 m diam steel raking piles 16- 24 m long, excavated inside and then filled with concrete and reinforcement. The box-girder units of the bridge, 18.9 m wide, 2.6 m deep, were precast in halfspans of 21.75 m and connected by prestressing. Contractor: joint venture between Christiani and Nielsen and Morrison Civil Engineering.

Stressed-ribbon Bridge (Spannbandbrücke in German)

A bridge that may be of prestressed concrete, but has an uncommon, **catenary**-shaped deck. Several examples exist in Europe, over the Rhone at Lignon near Geneva, another at Freiburg and a conveyor-belt bridge in Switzerland that passes through a tunnel. Heavy abutments or anchorages are needed to carry the tension at each end but big differences in level are possible. (catenary = the curve into which an uniformly loaded rope falls when hung between two points).

Cable-stayed Bridges

Any bridge with straight cables from masts connected directly to the deck girders without suspenders. The longest span (475 m) is over the Hoogly near Calcutta (**Hoogly Bridge**). The **Dartford Bridge** is nearly as long. All these bridges are of striking appearance, especially those with the cables in a single, central plane, such as the **Sunshine Skyway Bridge** in Florida (1988). But a survey published in early 1988 of 200 built since 1978 showed that most of the cables suffered severe corrosion, even those with PVC tape wrapping, including the **St-Nazaire Bridge** over the Loire.

Dartford Bridge (1990). A cable-stayed bridge over the Thames at Dartford, spanning 450 m at a height of 55 m above the water. This first large British cable-stayed bridge is also the first recent British bridge built privately, as a franchise. Trafalgar House, its promoter, will own it until 2010, levying tolls from motorists and eventually giving it to the Ministry of Transport. The Essex and Kent County councils in 1988 were demanding 3 m high walls each side as windshields to enable vehicles to cross at 80 kph in all but the severest wind. Since this would probably triple the wind load and sideways bending on the bridge, resulting in increased cost and delay, the need for it was disputed. Construction began early in 1988. The four cable towers are of steel above the deck, concrete below it.

Skarnsundet Bridge, Norway (1992). The current cable stay record holder at 530 m over the Trondheim Fjord. The planned **Tatara Bridge** in Japan with a 890 m span is not scheduled for completion until 1999.

The bridge under construction at **Honfleur** over the River Seine(see under cantilever bridges), not far from Le Havre, is expected to use locked-coil ropes and to have a main span of 856 m, the world's longest cable-stayed span. A **locked-coil rope** is a steelwire rope which is not stranded but built of concentric rings of specially shaped, closely packed wires in opposite lays. The outer ring is of S-shaped wires locked together so that if one breaks it cannot work loose. The surface of the new rope is very smooth but the ropes are stiff.

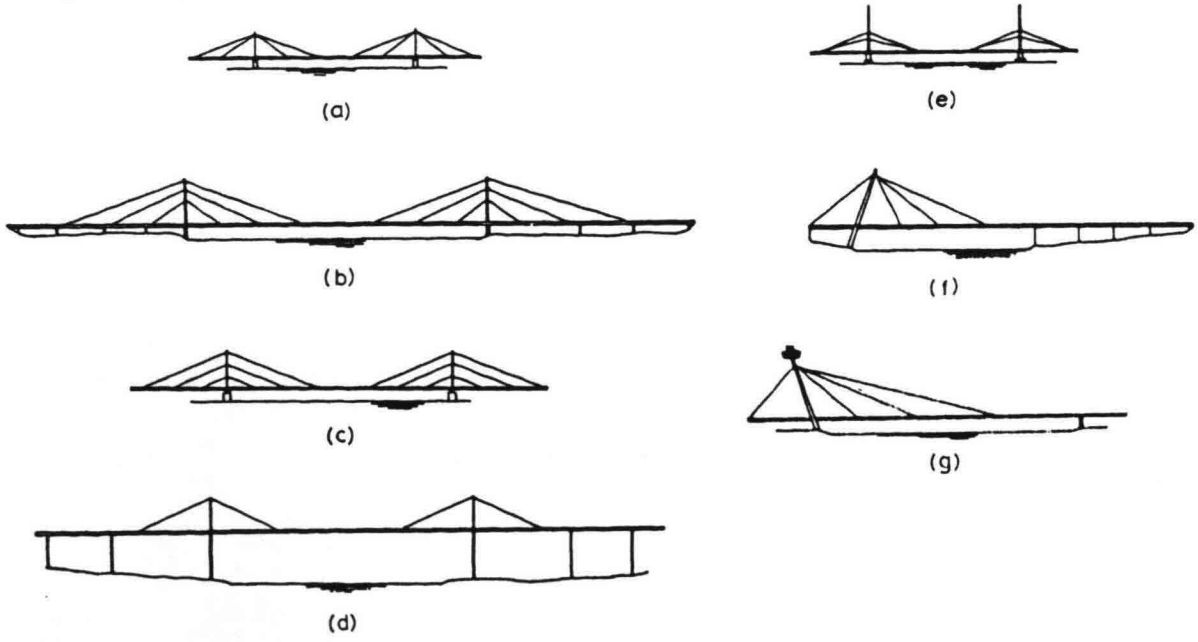


Fig. 3.5. Examples of different cable systems.
(a) Fan (Stromsünd); (b) modified fan (Duisberg - Neuenkamp); (c) harp (Theodor Heuss); (d) single cable (Erskine); (e) star (Norderelbe); (f) asymmetric systems (Batman); (g) Bratislava.

Suspension Bridges

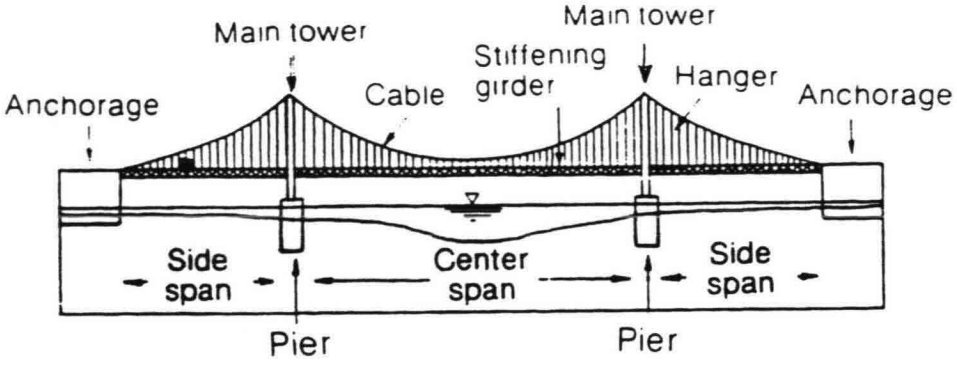


Fig. 3.6. Structure of suspension bridge.

A bridge hung from ropes, chains, cables, or pinned steel or iron bars passing over towers at each bank. The cables or ropes are held by anchor blocks or solid rock behind the towers. The track or road is hung from the cables above by suspenders - rods or ropes each side carrying the weight. Suspenders are usually vertical but on the **Severn Bridge** for the first time they were made sloping as to improve the stability of the bridge in wind.

Early **Chinese** decked suspension bridges had eight or more chains mostly below the deck, carrying it directly, perhaps with two more above the deck acting as railings. Consequently decks were curved, not flat. An ingenious bridge built in AD 494 on the Hoei River in eastern China had ten ropes which could be slackened by windlasses to submerge it. The bridge then blocked the river against large boats. It could not be burned and no enemy could cross the river. Another Chinese suspension bridge, built in AD 65, was the Lan Chin Bridge over the Lantshang river in Yunnan, spanning 76 m. Its iron chains carried a timber deck. Footbridges of rope with no deck have long been known and still exist, some of them made of ropes made of spun bamboo fibres or even of growing vegetation (creepers).

Suspender. A vertical hanger in a suspension bridge, by which the road is carried on the cables.

Suspension cable. A steel-wire rope carrying a suspension bridge. Two are needed for each bridge. They are often spun on the site.

Suspension-cable anchor. A mass of masonry in soft ground, or a fixing deep into rock, on the land side of a suspension bridge tower, to hold the ends of the suspension cables.

Stiffening girder. A girder built into a suspension bridge to distribute the loads uniformly among the suspenders and thus to reduce the local deflections under concentrated loads. A suspension bridge does not need stiffening if the maximum deflection is less than one three-hundredth of the span.

Self-anchored suspension bridge. A suspension bridge with no anchorages because the cables are attached to the ends of the stiffening girders beyond the towers.

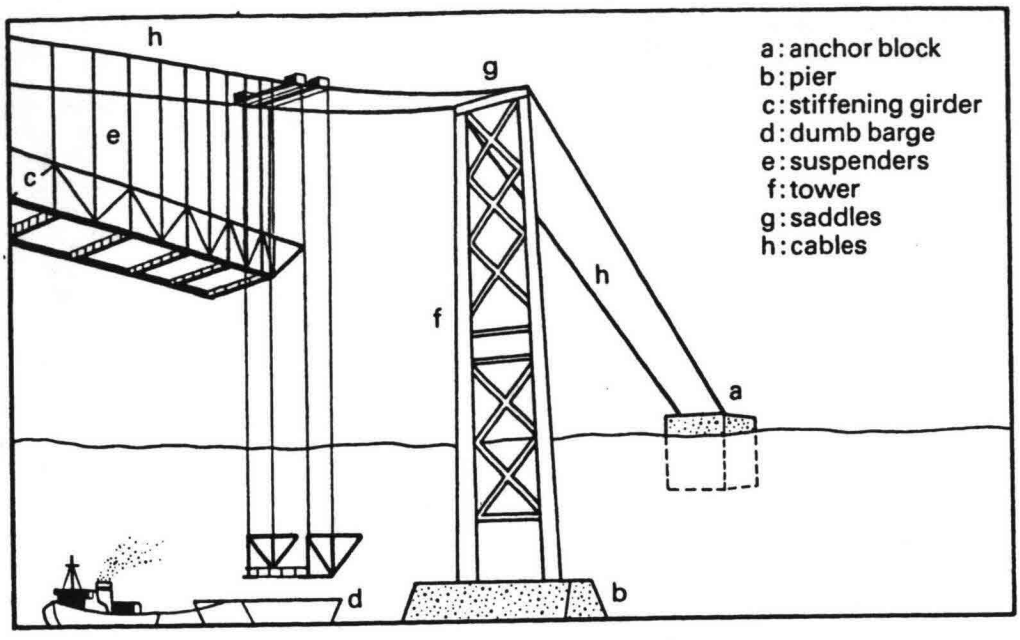


Fig. 3.7. Suspension bridge.

A suspension bridge could be regarded as an inverted arch. Except for the road deck, which is subjected to small and local bending moments and is therefore stiffened accordingly, all the stresses are tensile. Materials such as steel can take a bigger load in tension than they can in compression (failing by buckling). Another point which makes the suspension bridge the most economical type for long spans, is its flexibility. It gets extra strength from this by moving in order to distribute the stresses placed upon it. But this has also been the reason of a number of failures.

The fundamentals of the wire suspension bridge were developed in France (1825-30) by **Marc Seguin**, who constructed several with spans up to 91 m and by **M. Vicat**, who devised a method of spinning the cables in situ. In 1834 such a bridge was finished over the **Sarine Valley** (Switzerland); its 265 m span was supported by four 140 mm diameter wire cables.

Charles Ellet (born in the USA, educated at the Ecole Polytechnique, Paris) introduced the wire suspension bridge in the USA in 1832. He designed and built several, the longest being over the Ohio River at Wheeling (308 m span, 30 m above the river), but it was wrecked in 1854 by a storm.

Clifton Bridge (1864). A suspension bridge carried by wrought-iron links over the Avon gorge, built to the design of I.K. Brunel, but after his death, using links from his Hungerford Bridge over the Thames in London. In 1977 weighbeams were installed in both approaches to bar any vehicle with an axle load above 2.5 tons. A weighbeam is a steelplate measuring about 3 x 1 m which weighs any axle passing over it and may transmit the figures to a transducer. It is smaller than a weighbridge.

Brooklyn Bridge (New York, 1883). Construction time: 14 years. With a centre span of 486 m the longest span in the world until the Forth Railway Bridge. Designed and built by John and Washington Roebling. A **pneumatic caisson** was used to get through the mud on the river bed to solid layers. In a pneumatic caisson, air pressure is used to keep the water and mud out of the working chamber at the bottom. There men work, excavating material and the caisson sinks to a solid foundation.

George Washington Bridge (Hudson River, 1931). Its towers were designed on the same principle as a skyscraper, that is: a steel frame clad with non-load bearing stone. Thus they could be higher and stronger than traditional masonry towers as used at the Brooklyn Bridge, and they enabled the 1067 m span to be more than twice that of the Brooklyn Bridge. The foundations of the 181 m tower at the New Jersey side were built inside two coffer dams (each 33 by 30 m). Construction time: 4 years.

Golden Gate Bridge (1937). Formerly the largest suspension bridge of main span 1280 m and side spans each 343 m. During an 80 kph gale which lasted for four hours this bridge caused some anxiety by vibrating with a double amplitude of 3.4 m, 8 times per minute. Its 27 m wide deck is suspended from two 910 mm diameter cables on towers 227 m tall. The most difficult problem was the foundation of the south tower, 343 m from the shore and exposed to the open sea with a tidal current of 4 m/s.

Tacoma Narrow Bridge (1940). A suspension bridge of 853 m central span and 335 m end spans which collapsed in November 1940 a few month after its completion. It failed during a stiff breeze which caused a fluttering movement of oscillation and twisting (aerodynamic instability), but was rebuilt in 1950 to withstand winds of 200 kph.

Verrazano Narrows Bridge (1964). The world's longest suspension bridge when it was built in New York, and possibly still the heaviest. Its main span is 1298 m, with six lanes of each of its two decks.

Forth Road Bridge (1964). A motorway suspension bridge close to the Forth Bridge. The main span is 1006 m and the overall gap 1660 m. The towers are 156 m high. Six years construction time.

Severn Bridge (1966). A road bridge over the river Severn in the west of England, a suspension bridge of 988 m span, designed by Freeman, Fox & Partners. Wind tunnel tests led to a reduction of the side trusses. Experiments on box section road decks with different shaped triangular sidepieces were very successful and a revolutionary design was thus produced, saving twenty per cent on the steel.It had been general practice to make the towers of a cellular construction, but the towers on the Severn bridge are each made of four stiffened steel plates (economical). Fatigue damage to the deck was caused by 38-ton lorries for which the bridge was not designed. In 1987-89 this was repaired. In 1988 a second Severn bridge was under consideration.

Bosporus Bridge (1973). When built it was the longest (1074 m) suspension bridge in Europe. It cost $ 36 million, but there was so much traffic that this cost was recovered from tolls in three years time. Built like the Severn Bridge to Freeman Fox's design, it aroused interest with its economical aerofoil deck design and the off-vertical, continuous suspender ropes each side of the deck, carrying its weight up to the main cables above them. It was built in 3.5 years instead of the normal 5.5 years.

Humber Bridge (1981). The longest UK suspension bridge, resembling the Severn Bridge but of 1410 m main span.

Second Bosporus Bridge (1988). Main span of 1090 m, two four-lane carriageways (two lanes wider than the first bridge). Construction started in May 1985; completed in June 1988, 6 month ahead of schedule, but could not be opened then because the 28 km of approach roads were not ready. Designed by Freeman Fox; constructed by a three-country joint venture, including Impregilo of Italy; three Japanese contractors - IHI, Mitsubishi and Nippon Kokan - and Turkes-Feyzi Akkaya Insaat from Turkey. One difference from the first bridge is that its suspenders are vertical. Each main cable has 16,648 galvanized 5 mm diam. hard-drawn steel wires in 37 strands. The cables are coated with red-lead paste, wrapped with soft-steel galvanized wire and then painted.

Akashi-Kaikyo Bridge (Japan). A suspension bridge with many changes of plan and decades of research behind it for the earth-quake area of the Akashi Strait. The bridge towers are in waters 35 m deep with heavy scour. The 1986 proposal, being built in 1988, was for the world's longest span of 1990 m, 960 m side-spans, and towers originally designed for a height of 333 m. They may be reduced to about 300 m. The railway originally proposed will not now be carried.

Tsing Ma (Hong Kong, 1996). The new Chek Lap Kok airport will be connected by the Lantau Fixed Crossing to Hong Kong city. This crossing will consist of two major suspension bridges and viaduct. The link would consist of an upper deck six-lane expressway and a lower deck carrying a high-speed rail-link and two lanes for vehicles. When completed the Tsing Ma bridge (1377 m span) would be among the longest suspension bridges in the world.

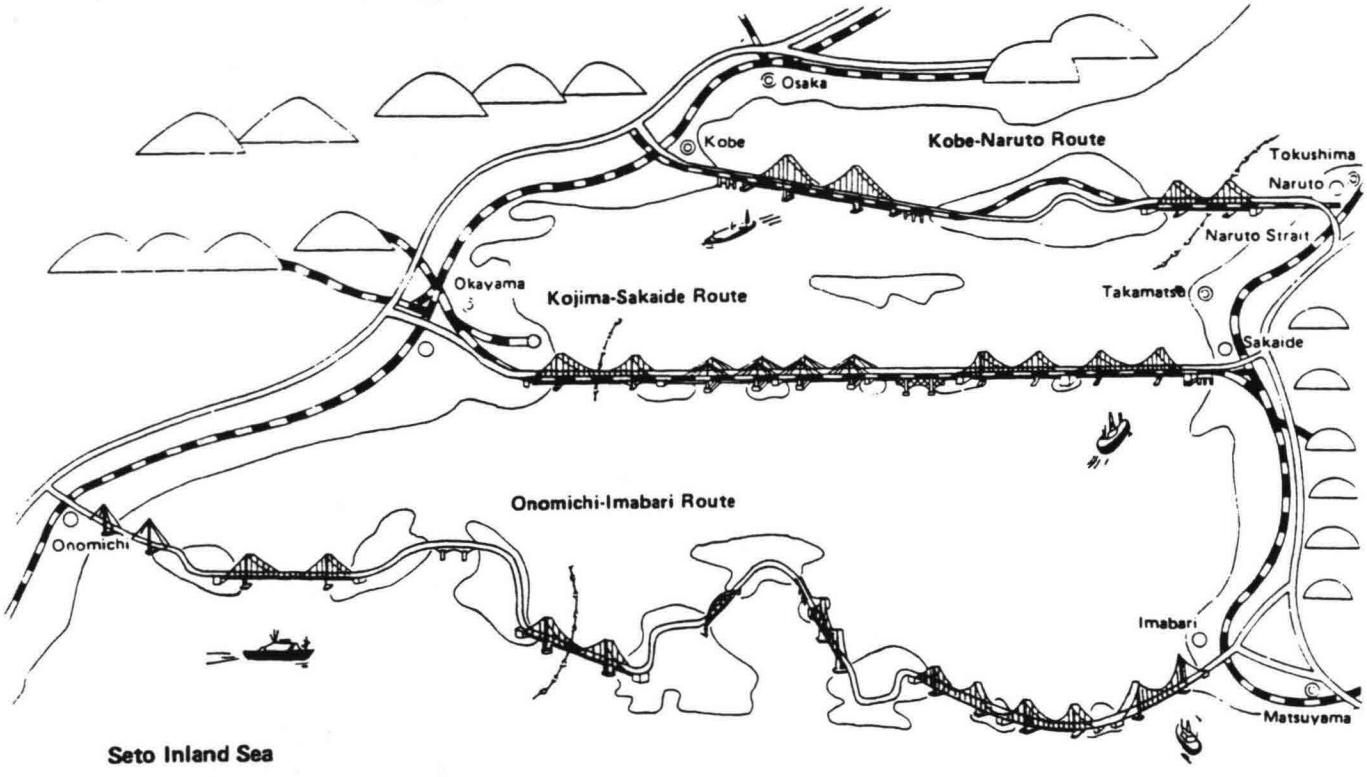


Fig. 3.8. Bird's eye view of Honshu-Shikoku bridges, Japan.

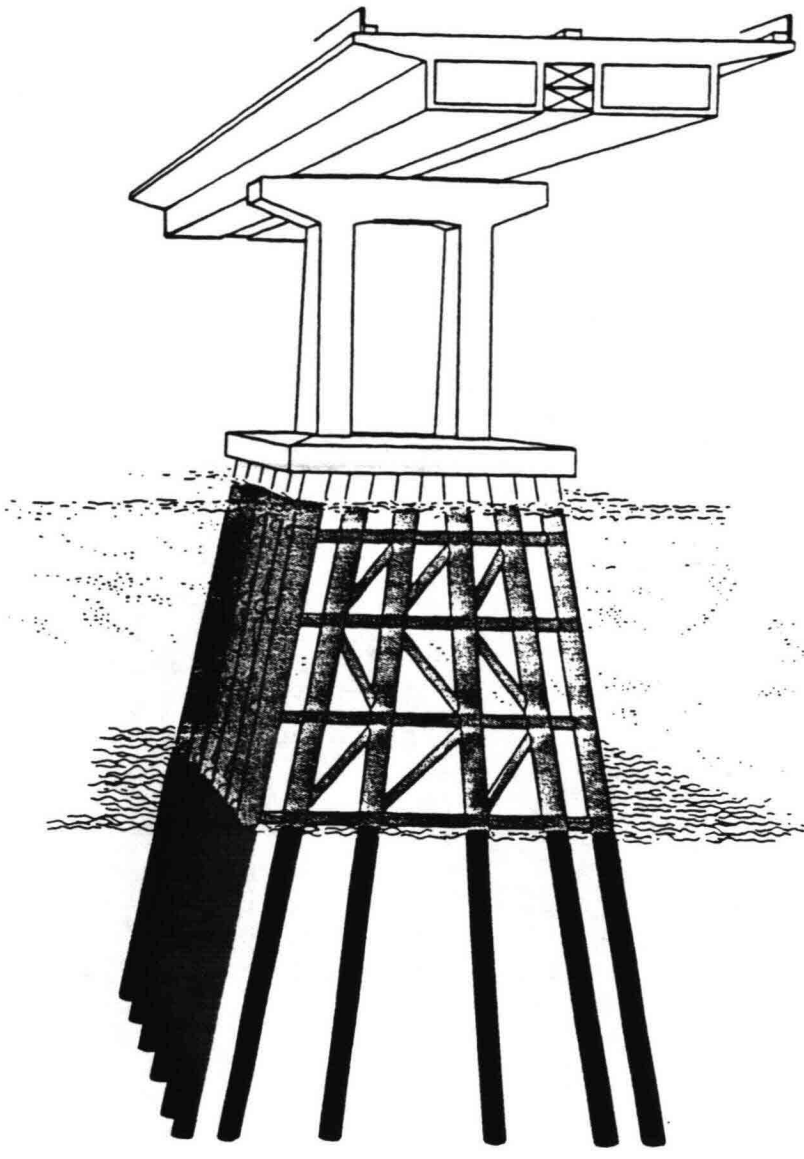


Fig. 3.9.
Jacket foundation in sea.
This jacket type has been considered for the Tokyo Cross Bay Highway. The jacket and pile weight 6300 metric tons.

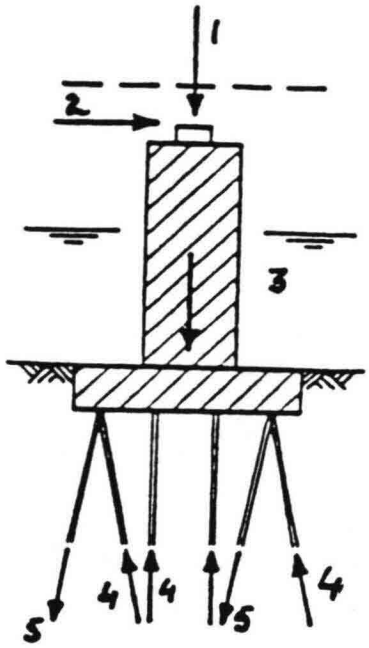


Fig. 3.10.
Forces acting on a bridge pier.
1. vertical load
2. horizontal load
3. weight of pier itself
4. reaction forces (pressure)
5. reaction forces (tension)

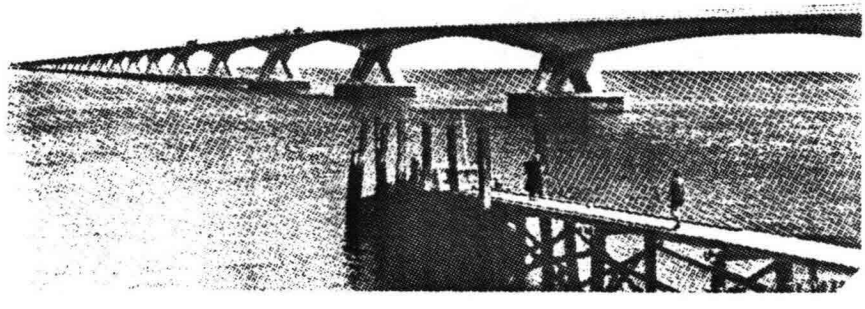
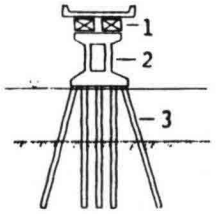
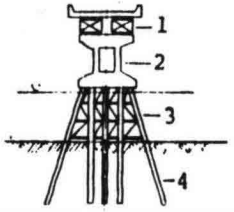
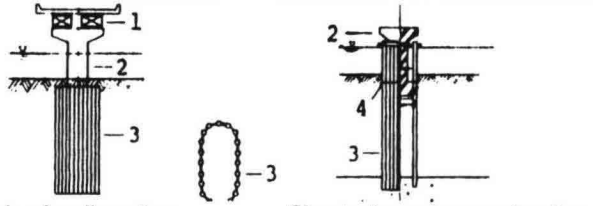
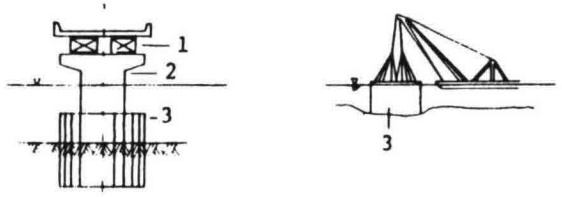


Fig. 3.11. The 5 kilometre long Zeeland Bridge over the Eastern Scheldt is the longest bridge in The Netherlands. Completed in 1965.

Different types of bridge foundation

System	Outline		Feature
Multi-pile foundation		1. Superstructure 2. Bridge pier 3. Steel pipe pile	1. Applicable to soft ground 2. Applicable to deep waters 3. Quick construction effected with large machines 4. Absence of underwater work
Jacket foundation		1. Superstructure 2. Bridge pier 3. Jacket 4. Steel pipe pile	1. Suitable for offshore structures such as oil drilling platforms and bridge substructures 2. • Increased binding among steel pipe piles • Improvement in steel pipe pile installation accuracy
Bell-type foundation		1. Superstructure 2. Bridge pier 3. Steel pipe pile	1. Fabrication of bridge pier as integral unit· short construction time 2. Small space requirement for substructure. Favourable for estuary and canal areas.
Steel pipe pile well foundation	 Example of well section Sheet pile under construction	1. Superstructure 2. Bridge pier 3. Steel pipe pile 4. Subwater cross-section of steel pipe pile	1. Characteristics approach rigid structure, caisson structure 2. Bridge pier built in the air, not sea 3. Small space requirement for substructure. Favourable for estuary and canal areas
Stationary caisson foundation		1. Superstructure 2. Bridge pier 3. Steel caisson (filled with concrete)	1. Applicable to hard ground 2. Applicable to deep waters 3. Quick construction effected with large machines

Storebaelt West Bridge - Denmark (under construction)

A low-level bridge, 6.6 kilometres long and built in concrete designed to carry railway- and motor traffic across the Western Channel between Knudshoven and Sprogø in 1994 and 1997 respectively is under construction since 1989. The principal concept of the project is manufacturing of all 250 elements on land, and subsequently transporting them to their final destination with a self-propelled floating crane, "Svanen". The construction of the West Bridge requires 62 caissons, 124 piershafts, 63 road girders and 63 railway girders plus 2 abutments (cast insitu).

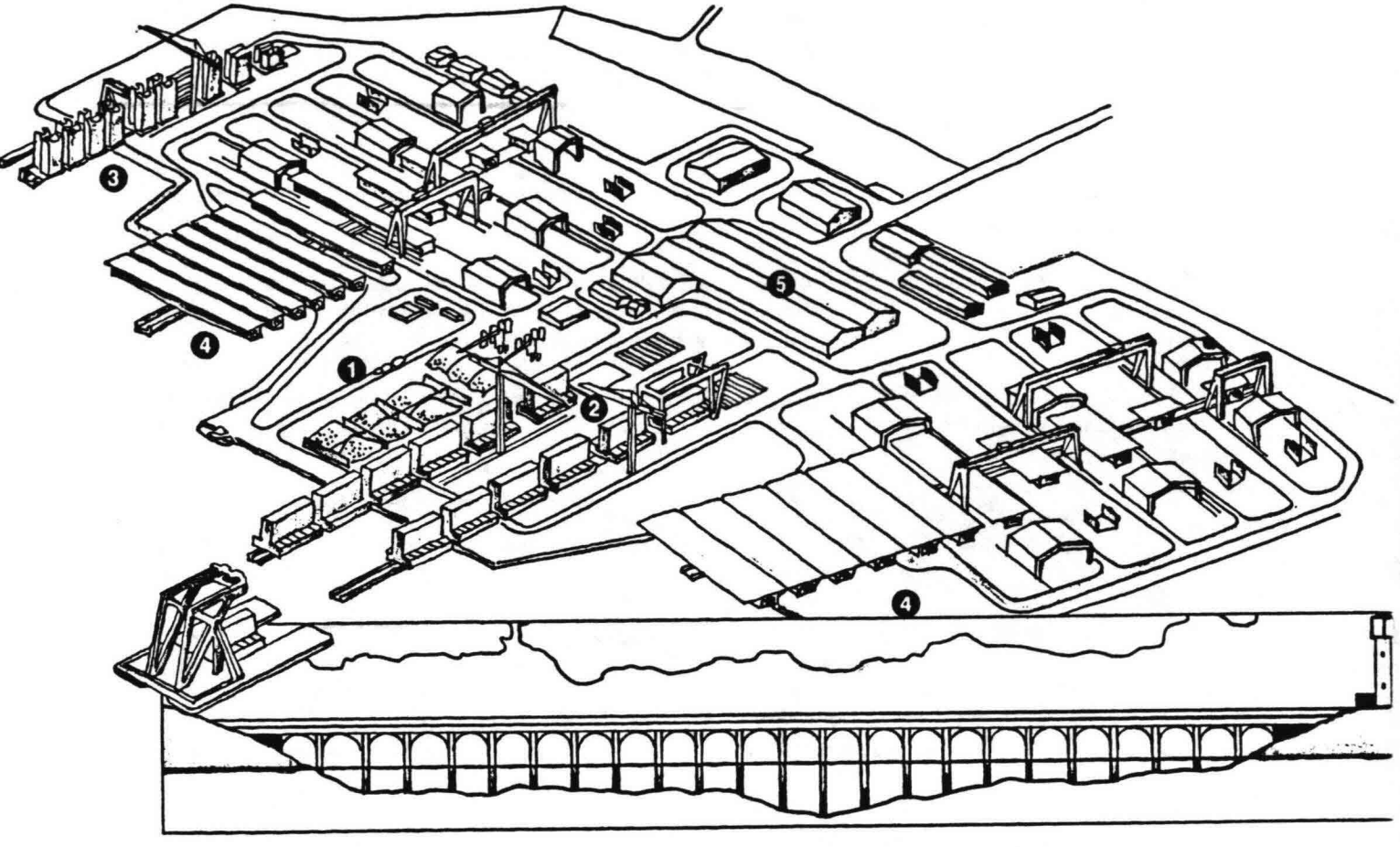


Fig. 3.12. Construction yard (Lindholm).
400,000 m³ of sand was pumped in from Storebaelt to form a site of 30 hectares with production lines for the concrete sections, batching plants and workshops.

1 = **Concrete** - two batching plants, each with a max. capacity of 120 m³ of concrete per hour, aggregates and sand stockpiles, laboratory for quality control. Ready-mix trucks transport the concrete to the production lines.

2 = **Caissons** - 2 production lines.
The 62 caissons vary in size and weight according to their position in the alignment. The caissons are the "feet of the West Bridge" and the shared foundation for the road- and railway part of the bridge. They consists of 3 sections - a base slab, walls and a plinth. Casting of the walls is done by slipforming. A special built gantry crane, 40 m high, carries the slipform by use of a network of heavy cables. By means of hydraulic jacks, these cables pull the form upwards and concurrently concrete is poured into the form. Weight caisson: 4,000 - 7,000 tons, height : 7 - 28 metres.

3 = **Piershafts** - one production line.
Ten platforms are in circulation; on each platform between six and seven pairs of piershafts of varying sizes are casted. The casting takes place in three operations: the actual piershaft in two stages and the head section which will carry the bridge decks. Average weight one pair of piershafts : 2,600 tons; height 4.5 - 21.5 metres.

4 = **Bridge girders** - two production lines
The girders for the road- and railway sections are cast on separate production lines. Out of the 2 x 63 girders 51 (x2) are 110 metres long, the other 12 (x2) 82 metres. a girder consists of 5 sections : a hammerhead (A) and sections B,C,D and E. Each girder has to pass through three casting stations on its way down the production line before it is completed.
Road girder : weight 6,000 tons, 24 m wide
Railway girder : weight 4,500 tons, 12 m wide

5 = **Reinforcement** - 27,000 kilometres or 75,000 tons
The reinforcement is processed in the 10,000 m^2 rebar workshop. Here the reinforcement steel is cut and bent and the assembling to form the rebar mats on U-cages takes place.

Skidding units transport the concrete elements down the production lines. These machines are capable of pushing/lifting the more than 6,000 tons bridge sections down towards the launching pier, from where the floating crane "Svanen" takes over.

Prior to the placing of the 62 caissons (with a distance of a little over 100 m apart) in the alignment of the West Bridge, the sea bed has to be prepared. This is done in several ways:

1. **Excavation** of each pit down to firm boulder clay by a bucket ladder dredger (total 800,000 m^3). The depth of the Western Channel varies between 7 and 32 metres.

2. **Preparation** of sea floor by self-elevating platform the "Buzzard". Its task is to level the area where the caissons will be positioned, and afterwards to place a layer of crushed stones 1.5 m thick (a foundation bed) upon which the caissons will be placed. Operations are controlled by sophisticated computer technology and monitored on video screens.

Placing of concrete elements by the self-propelled floating crane "Svanen" (94 x 65 m, 6,900 tons lifting capacity). The "Svanen" takes one working day to place one concrete element. After lifting the element from the load out pier and the transport to its position the 'Svanen's' eight anchors are laid out. The elements must be placed with an accuracy of just a few centimetres and the final positioning can take up to 5 hours.

Once in place on the foundation bed, the caisson is sand filled. The top of the caisson is 3 metres below the water surface and a temporary steel cofferdam is needed to permit "dry" casting of the joints between caisson and a set of piershafts.

Between each bridge girder is a gap of 2 metres. Concrete for this gap is placed insitu.

3.2. Movable bridges

Drawbridge. A movable bridge lifted at one end by chains or ropes either to let vessels pass under or to stop traffic passing over it. All parts are above ground.

Strauss bridge. As all parts are above ground there is no need to provide a tail chamber, but care must be taken to avoid obtruviness. This type is rather inelegant and requires greater depth behind the quayside than the fixed trunnion bascule.

Bascule bridge. A bridge which is hinged at the bank to allow ships to pass under it by raising the part over the river and lowering the part over the bank behind the hinge. Modern bascule bridges are of light alloy. Other names : **Balance bridge, Counterpoise bridge.** The rolling lift **(Scherzer)** bascule with its overhead counterweight and rolling track has the problem of rolling tracks deteriorating with age. The very high bearing pressures at the points of contact may lead to local crushing. This can be overcome by using wider tracks on heavier support girders.

The **trunnion bascule** is the type used in many modern examples and may be driven in several ways. The rolling lift bascule gives a wider clearance with the bridge in the open position than a fixed trunnion bridge of the same span, although it therefor requires a greater depth behind the quay.

Designing for wind loading on the opened bridge sets an economic limit to the single-leaf bascule span. With the double-leaf type, considerable care must be taken in the nose-locking arrangement between the two leaves; if the bridge is to carry a railway track it may be difficult to obtain a satisfactory joint.

Modern bascule bridges may lift from the middle, with hydraulic jacks.

Lift bridge. A bridge in which both ends of the deck are lifted at the same time to let ships pass. The vertical lift-bridge sets a headroom limit and is expensive for narrow crossings. However, it can be used for very long spans without the nose-locking problems of the double-leaf bascule. The lifting machinery may be either at the top of the lifting towers or in the piers; a mechanical or electrical linkage connects the separate motors to ensure synchronized parallel motion of the corners of the lifting section.

Swing bridge, pivot bridge, turn bridge. A movable bridge that swings on a vertical pivot at its centre, to allow vessels to pass. The swing-bridge often provides the cheapest solution for a given span. Its main disadvantage is the need to protect the bridge in the open position.

Traversing bridge, retractable bridge. A movable bridge that retreats from the waterway to allow a ship to pass. This type is not often used as it requires a suitable approach to accommodate the span in the open position and heavy rolling or sliding ways.

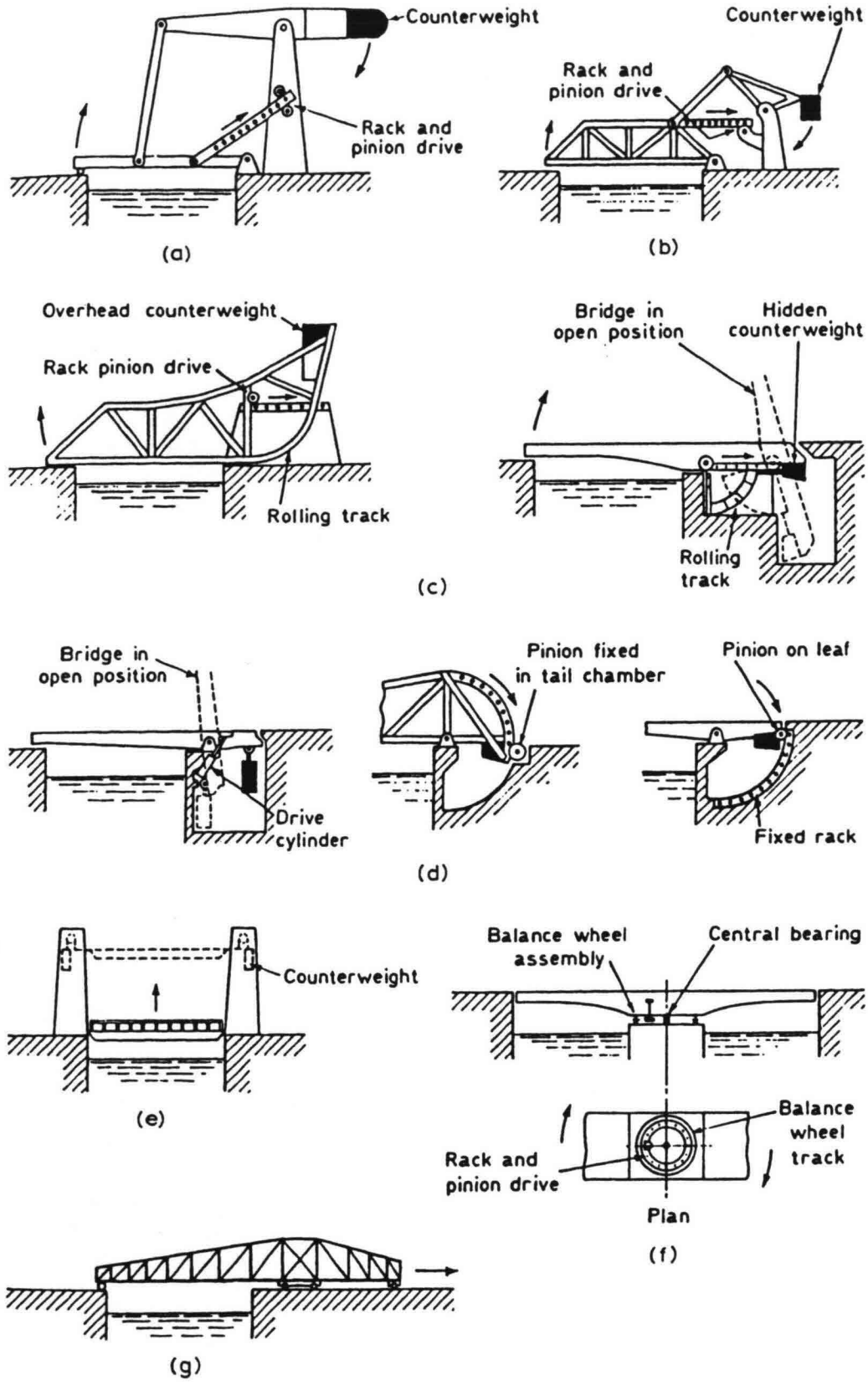
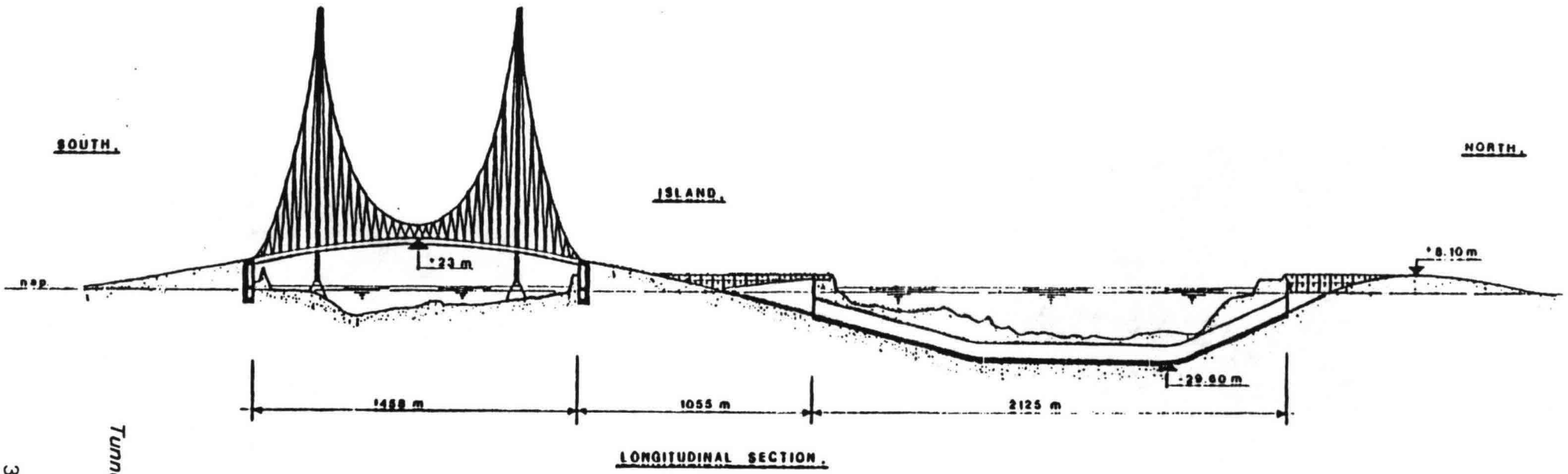


Fig. 3.13. Types of movable bridges. (a) Drawbridge; (b) Strauss; (c) Rolling lift (Scherzer) bascules; (d) Trunnion bascules; (e) Vertical lift; (f) Swing; (g) Retractable.

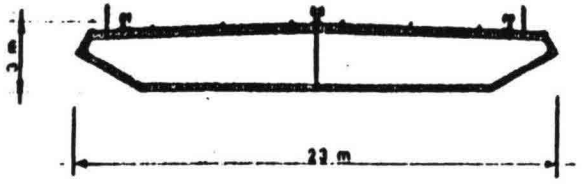
Bridge-tunnel combination

Chesapeake Bay bridge-tunnel. Two high road bridges, 20 km of low precast concrete bridges, four artificial islands, and two tunnels over 1.6 km long each, completed in 1964. The tunnels provide a clear seaway for shipping, and the high bridges help also. The combined road bridges and tunnels are 28 km long.

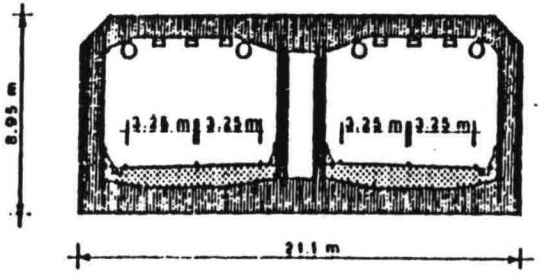
Tokyo Bay crossing. Two bored tunnels 4.9 km long connected to two artificial islands and to a 4.4 km bridge, due for completion in the year 2000, to cost rather more than the Channel Tunnel. The tunnels, of 13.9 m outside diam. are to be driven in poor silt and sandy clay with their crowns 15 m below the seabed. The Kawasaki artificial island of 200 m diam. will carry the tunnel ventilation machinery, and is at the tunnel midpoint. The other artificial island, Kisarazu, provides the way out from the tunnels to the bridge.



LONGITUDINAL SECTION.



CROSS SECTION SUSPENSION BRIDGE.



CROSS SECTION TUNNEL.

Tunnel-bridge combination with an island in the middle

3.2. Aqueducts

Historical review

700-600	BC	Assyria	A stone-lined aqueduct 20 m wide and 80 km long was built by Nebuchadnezzar in 15 month to bring water to **Nineveh.**
312	BC	Italy	**Aqua Appia**, the first aqueduct, brings clean water to Rome
145	BC	Italy	**Aqua Marcia**, the first high-level Roman aqueduct.
18	BC	Franc	**Pont du Gard** built by Agrippa to supply water to Nimes.
226	AD	Italy	The last of 11 aqueducts was built to bring water to **Rome**, but by AD 1000 the only Roman water supply was the Tiber.
1845	AD	USA	**Allegheny Aqueduct** at Pittsburg completed by John Roebling, the first US suspension bridge to be built by spinning in place.

The oldest known aqueduct was built near the Nineveh, the capital of Assyria in 703 B.C. A 48 km long feeder canal had to pass at **Jerwan** over a valley on an aqueduct (263 m long, 21 m wide, 8.5 m at its highest).
The tallest of the 14 arches of **Aguas Livres Aqueduct**, built in Lisbon, in 1784 is 65 m.
The structure consisted of a stone block dam with five corbelled arches. Many aqueducts were built by the Romans. Eleven aqueducts supplied **Rome** with water; the first, the **Aqua Appia** opened in 312 B.C. and the last, the **Aqua Alexandrina** in AD 226.

The biggest and best, and supplying the purest water, was **Aqua Marcia** (145 BC). Its source was 37 km from Rome in a direct line but the aqueduct was 92 km long, because, as in all Roman aqueducts, the water fell by gravity alone, and this meant that the channel had to meander along the contours to keep its steady gradient. For 80 km it was underground in a covered trench, on it last 10 km. into Rome, it was carried on an arcade. It was the first of the high-level aqueducts and supply water to all of Rome.

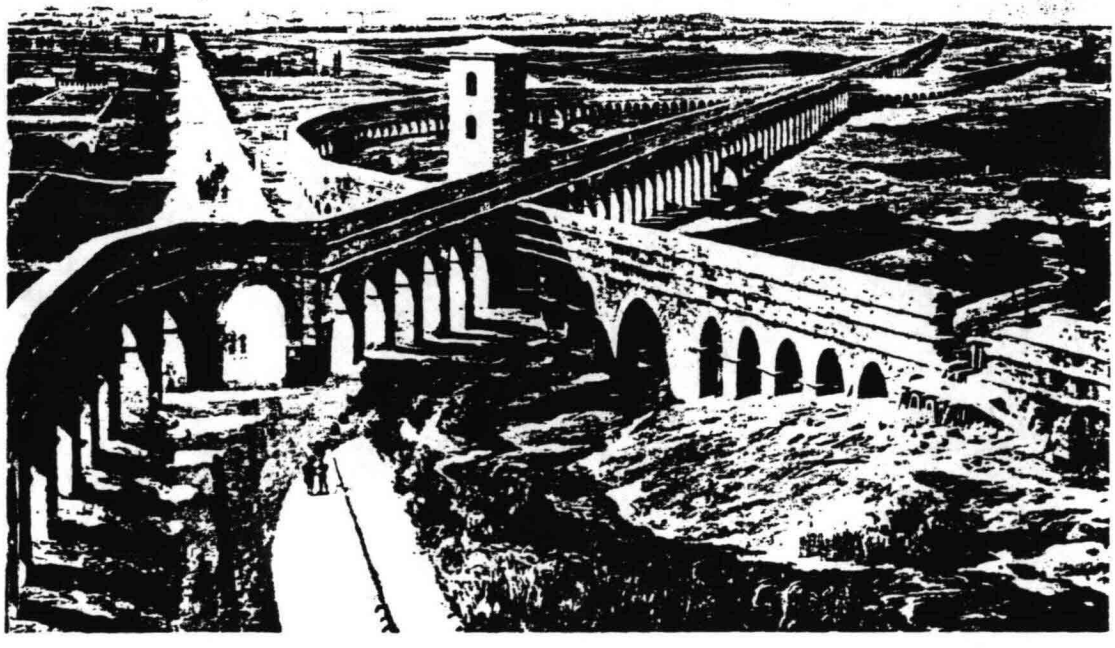


Fig. 3.15. Roman aqueducts supplying water to Rome.

The greatest of ancient aqueducts was the **Aqueduct of Carthage** in Tunisia, which ran 141 km from the springs of Zaghouan to Djebel Djougar. It was built by the Romans during the reign of Hadrianus (AD 117-38). By 1895, 244 arches still survived. Its original capacity has been calculated at 31.8 million litres per day.

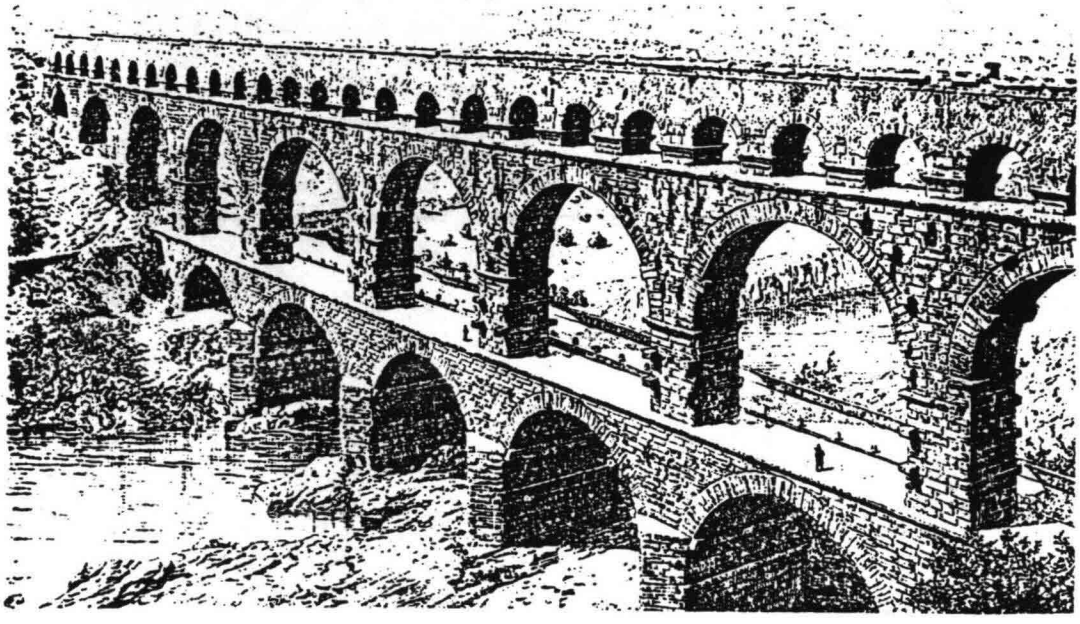
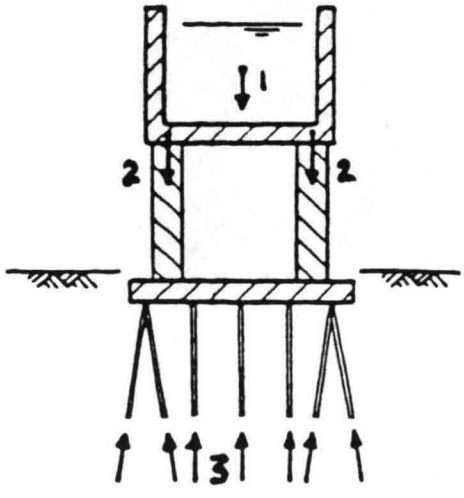


Fig. 3.16. Roman aqueduct built in AD 19. The triple-tiered Pont du Gard carried water to the town of Nimes in southwest France. It is 49 m high.

The **world's longest aqueduct**, in the modern sense of a water conduit, as opposed to an irrigation canal, is the California State Water Project aqueduct, completed in 1974, to a length of 1329 km of which 619 km is canalised.

Fig. 3.17.
Forces acting on an aqueduct.
1. weight of water
2. weight of structure + water
3. reaction forces (pressure).



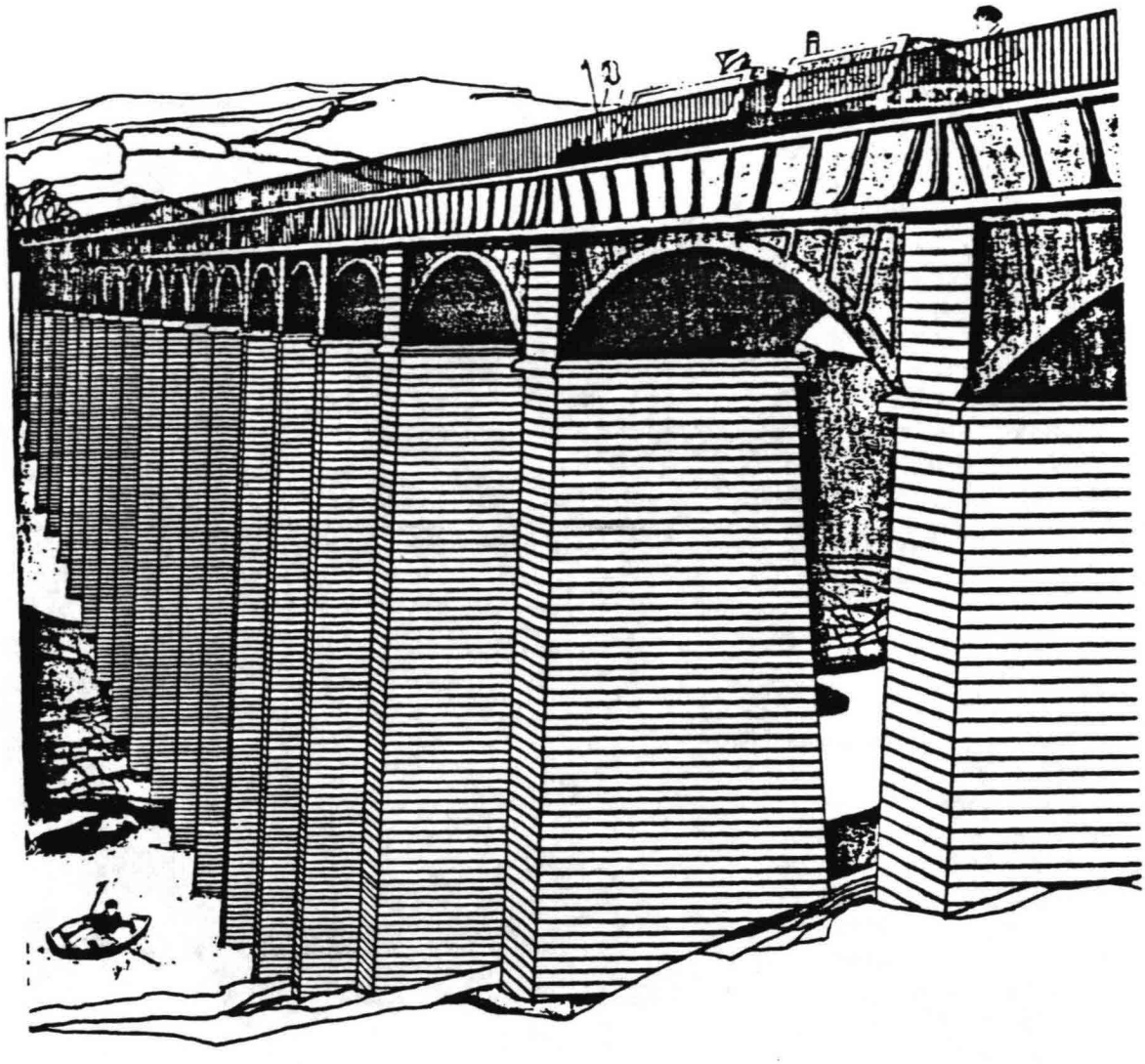


Fig. 3.18.
The Pontcysyllte Aqueduct carries the Shropshire Union canal across the river Dee about 5.5 kms east of Llangollen in Wales. Designed by Thomas Telford and completed in 1805, it is 307 m long and has 19 spans with a maximum height of 38.7 m. It is one of the earliest structures in which a trough was made from cast iron units bolted together and even today the many dovetailed joints hardly leak at all (longest bridged aqueduct in Britain).

Fig. 3.19. Bhima Aqueduct, Maharashtra, India. The aqueduct across the Bhima River carries water (discharge of 42.5 m^3/s) for irrigation. The aqueduct comprises a continuous precast tube prestressed longitudinally and transversally. The structure is 947 m long with spans of 41.5 m; it has a truncated circular cross section of 4.8 m diam with 3.75 m roadway on the top. The thickness of the tube is 200 mm. The aqueduct is constructed by a free cantilever method. Service date: 1985.

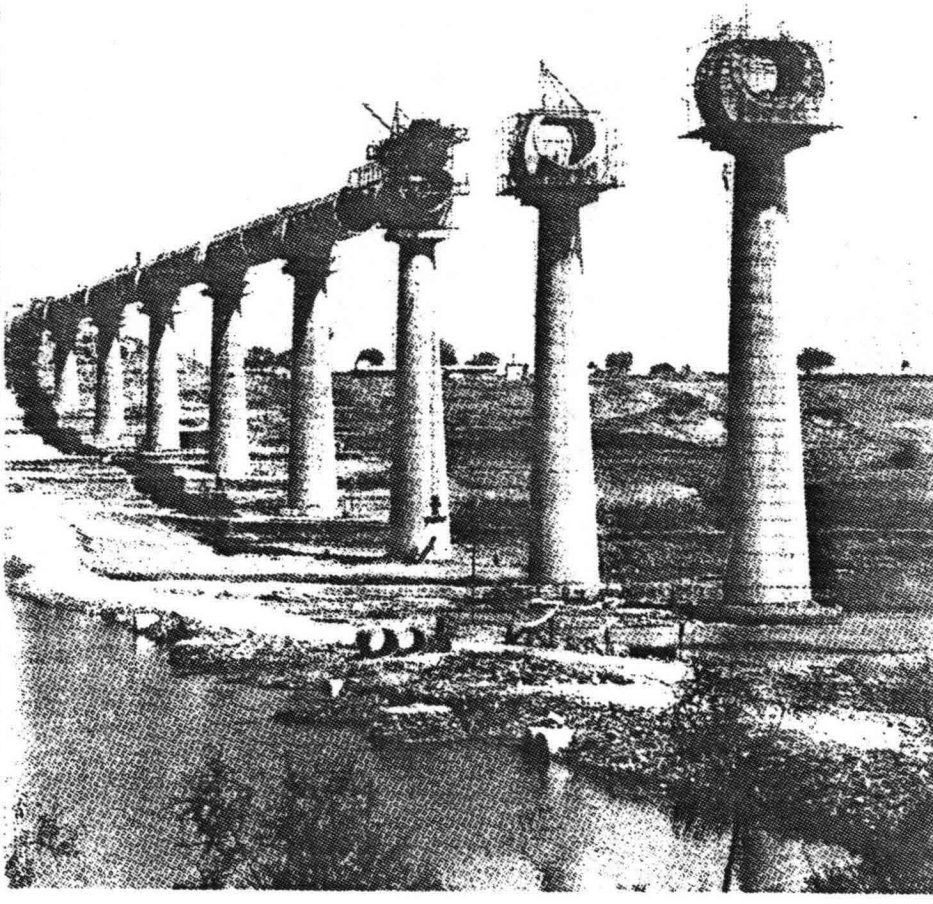
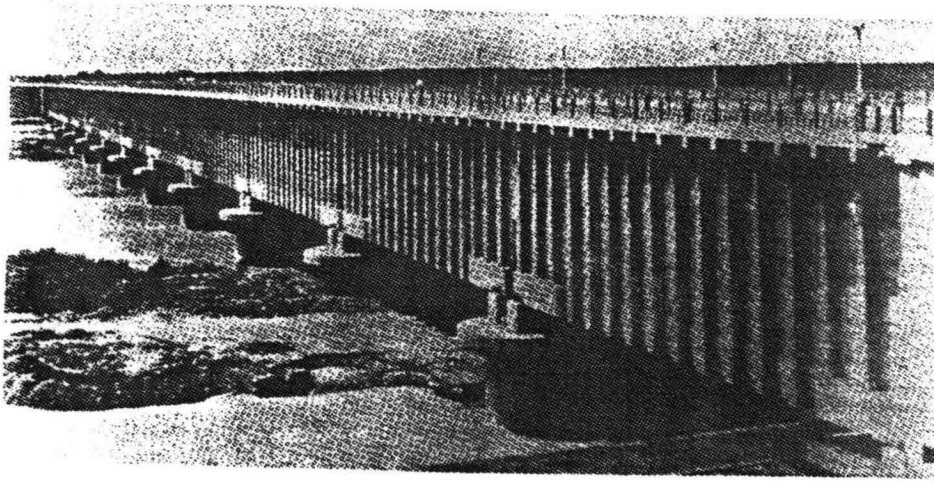


Fig. 3.20. Gomti Aqueduct, Uttar Pradesh, India. This is one of the biggest aqueducts constructed in India. It was executed 1978 for the Irrigation Department. The aqueduct is 382 m long and consist of 12 equal spans of 31.8 m each. The aqueduct box section of 12.8 x 6.8 m carries a discharge of 357 m^3/s.The depth of prestressed concrete girders is 9.9 m and weighs as much as 500 tons per girder per span. The foundations for the aqueduct are double D wells of size 12 x 27 m. The depth of well is 32 m below low water level.



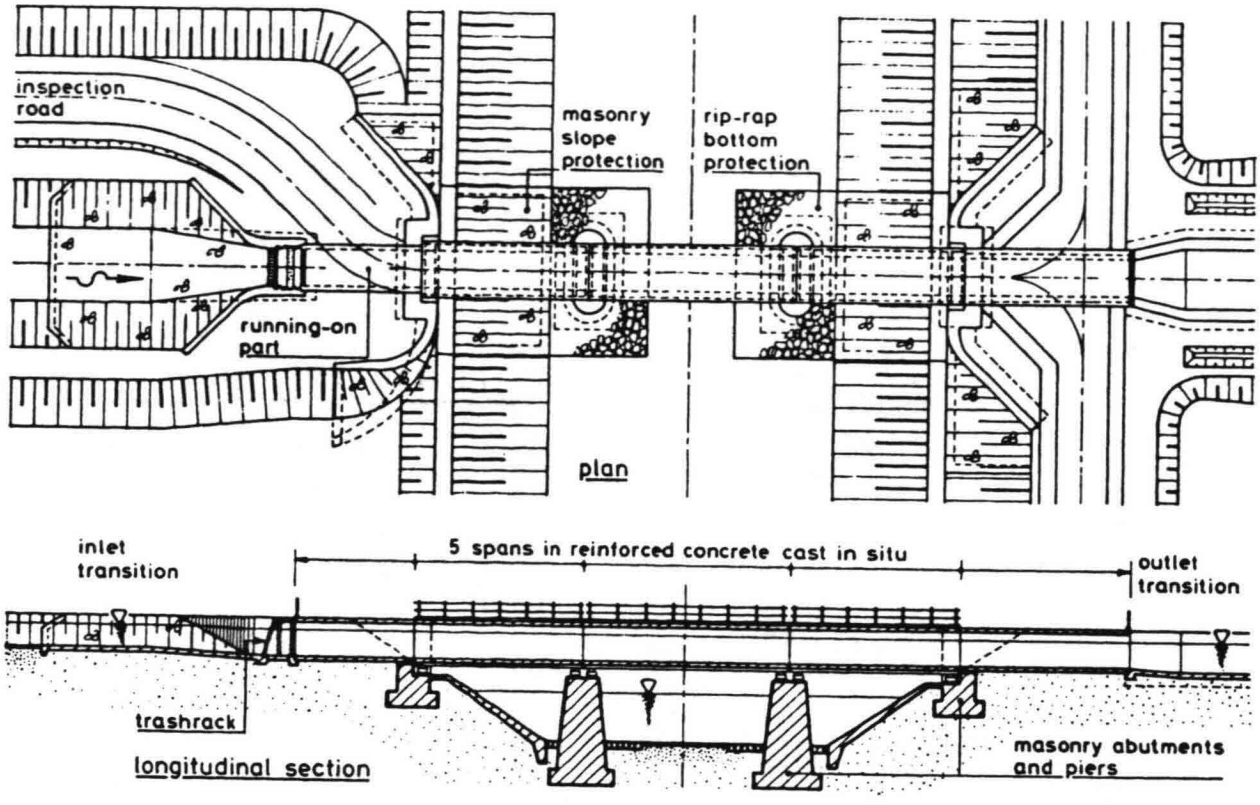
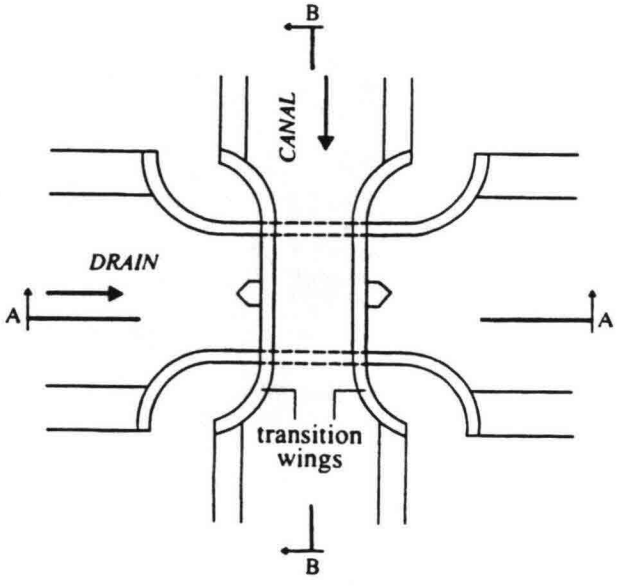
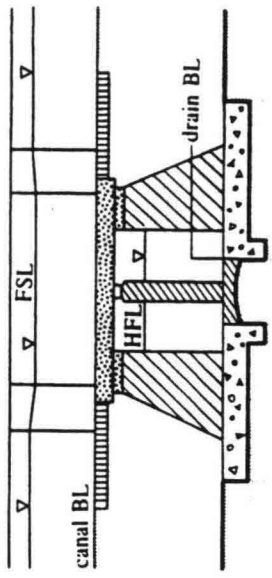


Fig. 3.21. Example of aqueduct.

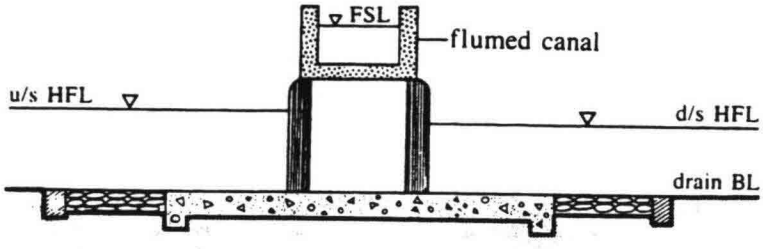
(a) Plan of aqueduct



(b) Section B – B (aqueduct



(c) Section A – A (aqueduct)



(d) Section A – A (inverted siphon aqueduct)

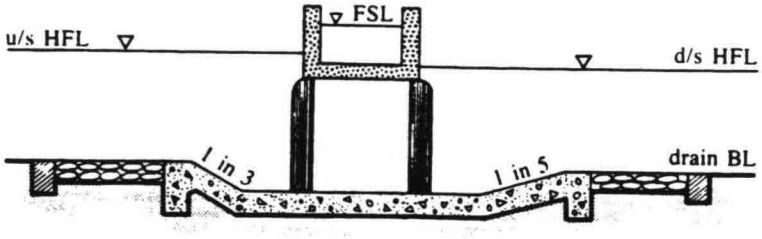


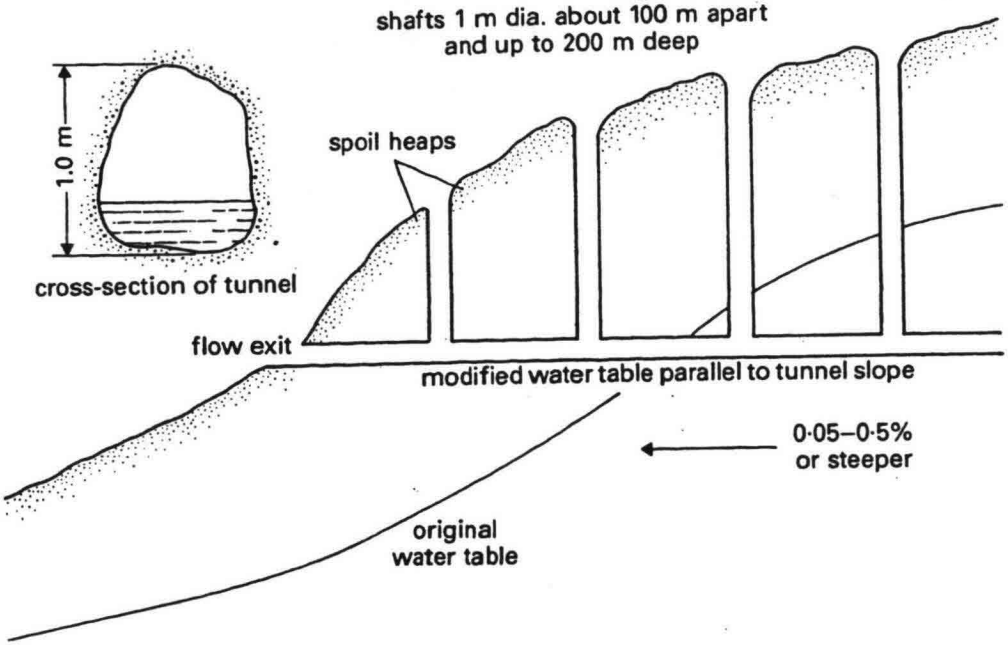
Fig. 3.22. Layout of an aqueduct.

4. Tunnels

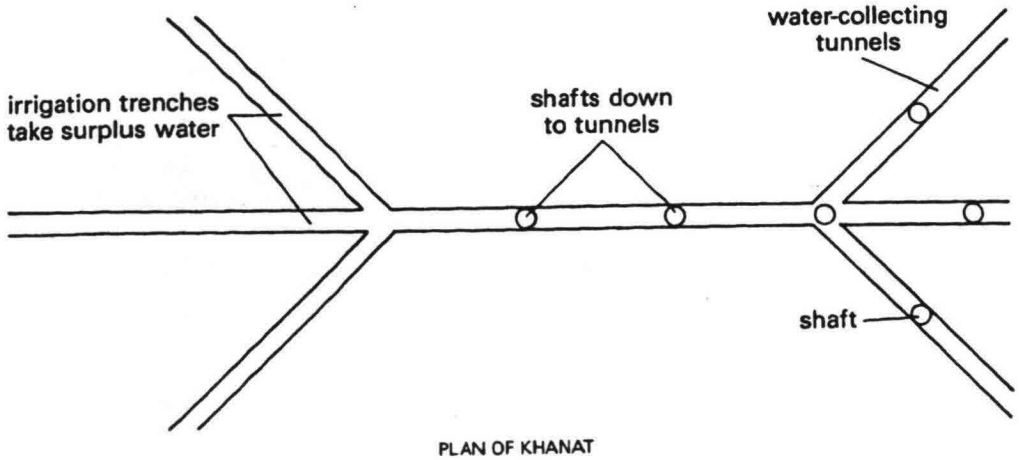
Historical Review

2800 BC	Middle East	**Khanats**
530 BC	Greece	Eupalinos digs the first water tunnel on the island of **Samos**, 1 km long, starting at both ends simultaneously.
1681 AD	France	First tunnel driven by blasting
1828	U.K.	**Marc Brunel** uses concrete made by hydraulic cement for infilling the bed of the Thames over his tunnel
1830		Earth-drilling derrick developed from ship's derricks
1831	Germany	Furnace gases from blast furnaces used for heating the blast first at Wurtemberg
1842	U.K.	First **shield-driven** tunnel under the **Thames** completed by Marc Isambard Brunel and his son Isambard Kingdom Brunel. It took approximately twenty years to build and it is rectangular and completely brick built. The exterior is 11.5 m wide and 7 m high, with passages approx. 450 m long. When the tunnel was being built there were all kinds of technical difficulties and financial problems. The experiences initially acquired in using this method were so depressing that a second attempt to use this system for making a tunnel was not made until 1869.
1863	U.K.	First **London underground railway** links Paddington with Farringdon St; 4 km long, powered by steam locomotives
1869	U.K.	Peter Barlow drives a 2.13 m dia. tunnel under the **Thames** from the Tower to Bermondsey, lined with iron rings, later to become the **first tube railway**, with one car containing 12 people hauled by steel wire rope
1857- 71		First Alpine tunnel, Mont Cenis, 12 km long, begun with hand-drilling, finished with compressed-air drills
1872- 82		St Gotthard tunnel driven with the loss of 300 lives
1886	U.K.	Severn railway tunnel opened
1890	U.K.	First **electric tube railway** opened, using 120 hp electric motors (City and South London Railway from the City to Stockwell)
1893	U.S.A.	**New Croton aqueduct tunnel** completed, New York;
	U.S.A.	**Immersed** tube sewer tunnels sunk in **Boston**
1874- 1905	U.S.A.	First **Hudson Tunnel**; this impressive tunnel was built using the shield method under compressed air. Work was started on both banks working towards the centre, the double tunnel was 2,600 m long, of which 1,50 m lay below the river.
1905	U.K.	**Inner Circle underground** railway in London electrified
1906-10	USA/Canada	Railway tunnels under the **Detroit River** sunk as immersed tubes (Michican Central Railroad - MCR tunnel)
1907-11	Germany	First **Elbe tunnel** built using the shield method; the only access to this tunnel is by means of a lift shaft and it is therefore unsuitable for modern traffic
1925-34	U.K.	**Queensway Tunnel** under the **Mersey** between Liverpool and Birkenhead, the shield method was used
1933-40	USA	**Lincoln tunnel** between New York City and Weehawken by the shield method.
1936-40	USA	**Queen Midtown Tunnel** (new York)- shield method

1942	Netherlands	First immersed tunnel, the **Maastunnel** in Rotterdam, in The Netherlands opened: enclosed part: 1070 m
1964	USA	**Chesapeake Bay bridge-tunnel**, 28 km long, with four artificial islands, completed.
1987	UK/France	Start construction **Channel Tunnel**.
1988	Japan	**Seikan tunnel**, the longest public service tunnel in the world, nearly 54 km long, 23 km of it under the sea, opened. Terrible geological difficulties, including flooding about 50 times, caused the tunnel driving to last 24 years. By the air transport had become cheap and the 'bullet train' company no longer wished to use the tunnel. Now it is used by other rail services.



CROSS-SECTION OF KHANAT



PLAN OF KHANAT

Fig. 4.1. The Khanat is one of the most useful and ancient of man's creations. It cannot be called a structure because it is entirely dug and not built. In Iran they reach to 70 km in length.

Tunnel. An underground passage, open to daylight at both ends. If open at only one end it is a **drift** or **adit**. The London underground railways are probably the world's largest underground network and include tunnels 30 km long; but one of the world's longest tunnels in 1973, completed in that year, was the Orange-Fish tunnel (South Africa). Of 5.35 m finished dia., with a concrete lining 230 mm thick, it is 82 km long. It irrigates the semi-arid Great Fish River Valley, in the eastern Cape Province, with water from the Orange River.

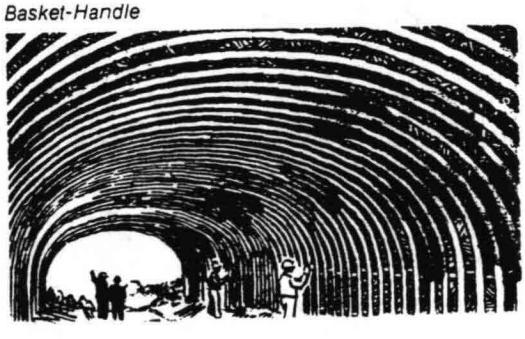
Khanat. Other names: kanat, ganat, ghanat, infiltration gallery, galeria (Mexico,hyd.). A long, ancient tunnel or network of tunnels dug to collect groundwater, usually from alluvial deposits, certainly used 2500 years ago in Persia, possibly 1000 years earlier in Armenia. They are also in use in Afghanistan, Egypt, Mexico and Chile. About 25,000 are said to be in use in Iran. A khanat is an adit or several adits branching off each other, dug by sinking vertical shafts at a spacing of around 100 m to remove the rock from the tunnel and provide ventilation for the tunnellers. The adits are about 1 m high and 0.8 m wide and slope at 0.5 - 5 m/km. The shafts may serve as wells when the khanat is completed. If there is enough water it is distributed, when the adit reaches the surface, through irrigation canals to the farmers near by.

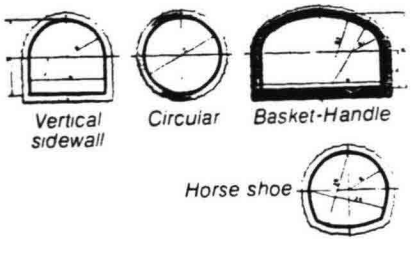
Longest tunnel - Water supply
The world's longest tunnel of any kind is the New York City West Delaware water supply tunnel, begun in 1937 and completed in 1944. It has a diameter of 4.1 m and runs for 168.9 km form the Roundout Reservoir into the Hillview Reservoir, on the border of Yonkers and New York City, NY, USA.

Longest tunnel - Sub-aqueous
The 53.9 km long Seikan Rail Tunnel will be 240 m beneath sea level and 100 m below the seabed of the Tsugaru Strait between Tappi Saki, Honshu, and Fukushima, Hokkaido, Japan. Once due to be completed by March 1979 at a cost of Yen 200,000 million, major flooding on 6 May 1976 has put back completion beyond 1982. Tests started on the sub-aqueous section (14.5 miles, 23.3 km) in 1963 and construction in June 1972. Currently the world's longest sub-aqueous rail tunnel is the Shin Kanmon Tunnel, completed in May 1974 which runs 18.7 from Honshu to Kyushu, Japan.

Longest tunnel - Hydroelectric, irrigation or sewerage
The Majes project in Peru involves 98 km of tunnels for hydroelectric and water supply purposes. The dam is at 4200 m altitude. The Chicago TARP (Tunnel and Reservoir Plan) in Illinois, USA involves 193 km of sewerage tunnelling.

Fig. 4.2. Kinds of tunnels.
Tunnels are built in different shapes. Many times tunnels are round. They are called **circular tunnels**. Circular tunnels are built for going through soft ground or under a river or lake. Some tunnels are round at the top but square at the bottom. These are called **vertical side-wall tunnels**. The flat floor in these tunnels is used for roads. Sometimes tunnels are shaped like a horseshoe. **Horseshoe tunnels** are built when the ground is soft but a flat floor is needed. Another kind of tunnel is the **basket-handle tunnel**. It is a wide tunnel. It is built when two or more roads are needed.

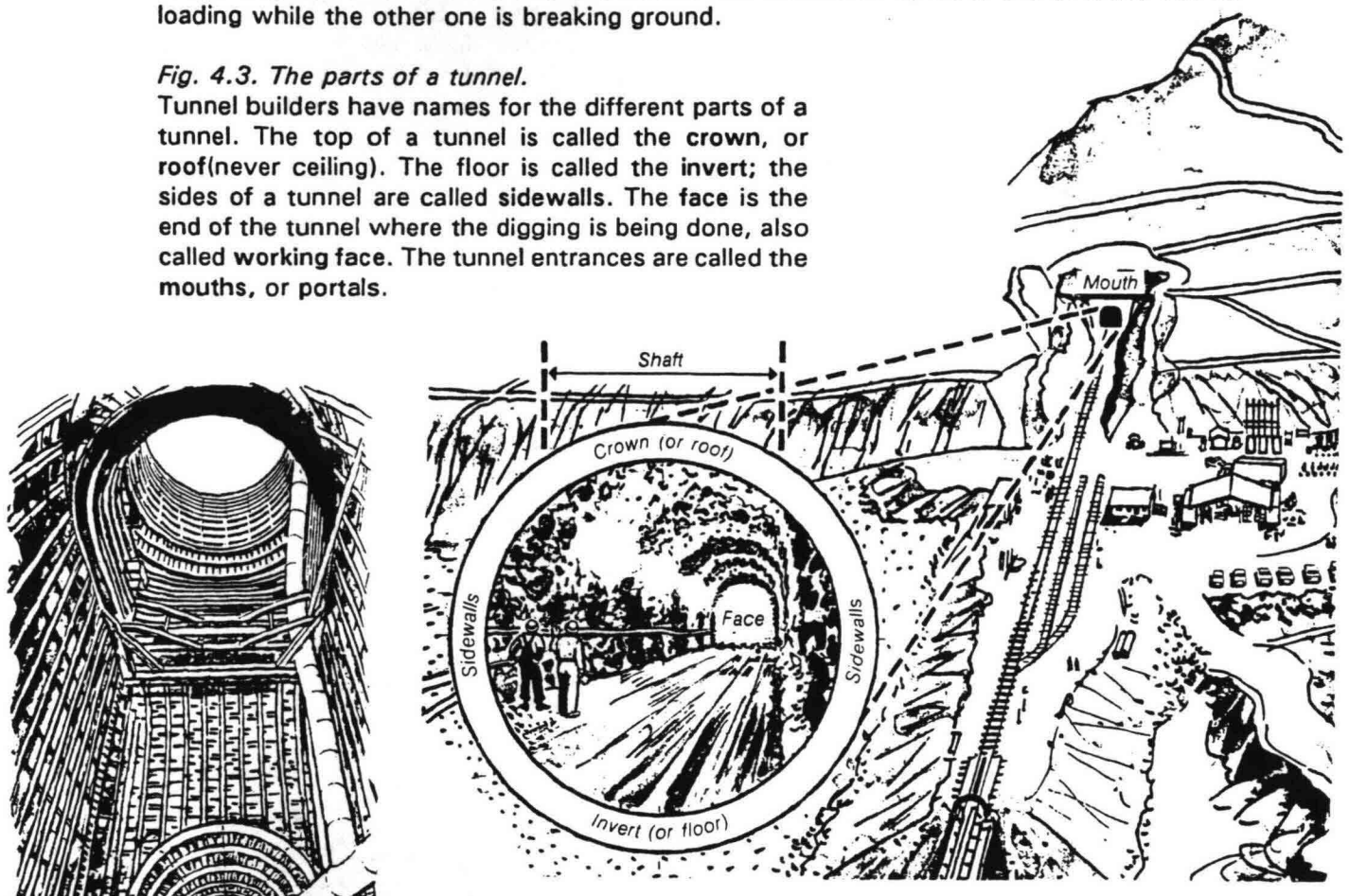




4.1. Bored tunnels

Tunnelling. Tunnelling methods vary with the hardness of the ground, the size of the tunnel and whether the ground is waterlogged. Most tunnels in soft ground are driven by **shields,** mechanically with **tunnelling machines (TBM).** In medium-hard ground and even in the hardest rock, tunnels are now driven by **fullface TBMs.** Drilling and blasting, formerly the only method in hard rock, is giving way either to a TBM or to a heavy **hydraulic hammer** of up to 3500 kg in weight, powered by the hydraulics of the excavator that carries it. But it is not possible to get every hydraulic excavator into a tunnel cross-section below 30 m2. In the largest tunnels where two excavators can work side by side, one of them will be loading while the other one is breaking ground.

Fig. 4.3. The parts of a tunnel.
Tunnel builders have names for the different parts of a tunnel. The top of a tunnel is called the **crown,** or **roof**(never ceiling). The floor is called the **invert;** the sides of a tunnel are called **sidewalls.** The **face** is the end of the tunnel where the digging is being done, also called **working face.** The tunnel entrances are called the **mouths,** or **portals.**



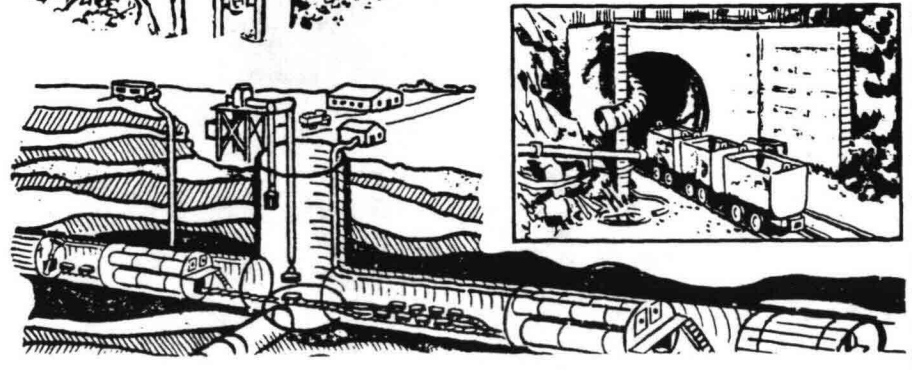


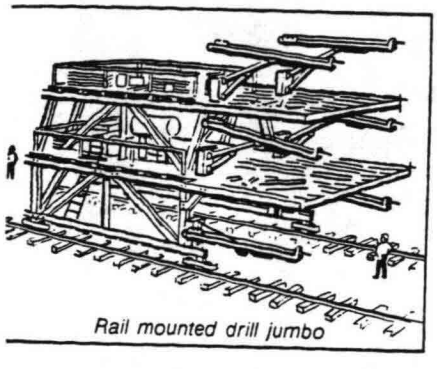
Fig.4.4. Starting a tunnel.
The building equipment is hauled in and out through the open end of the tunnel. Some tunnels are deep underground. Then, men and equipment enter the tunnel through a **shaft.** The shaft is a hole made straight down from the top of the ground to the tunnel. Sometimes the shaft is dug near the middle and the tunnel is dug both ways.

Fig. 4.5. Tunnelling through rock.
The most common way of building a tunnel through rock is called **blasting** and **mucking**. **Muck** is the broken rock and dirt that must be hauled away as the tunnel is being built. **Mucking** means to clear away all the muck. Holes are drilled into the rock face, filled with explosives and blasted. After the smoke has cleared, the muck is removed.

Conveyor belt moves muck out of tunnel.

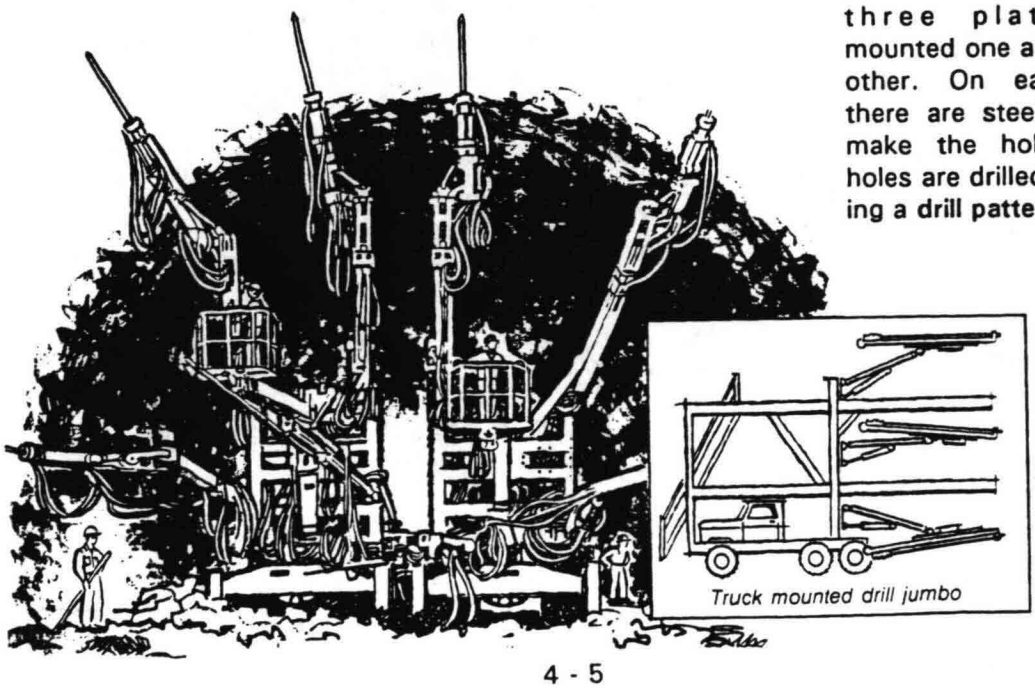
Holes are drilled in the rock face and filled with explosives.

Steel drill bits are used to make the holes.



Rail mounted drill jumbo

Fig. 4.6.
The drill jumbo.
The drill jumbo is a big platform on wheels with drills fastened onto it. A jumbo may have two or three platforms mounted one above the other. On each drill there are steel bits to make the holes. The holes are drilled according a drill pattern.



Truck mounted drill jumbo

Fig. 4.7. The muck train.
The muck is often removed by a muck train. A powerful little engine pulls the muck train to the mouth of the tunnel. When the cars are outside, a special machine turns them upside down to unload the muck.

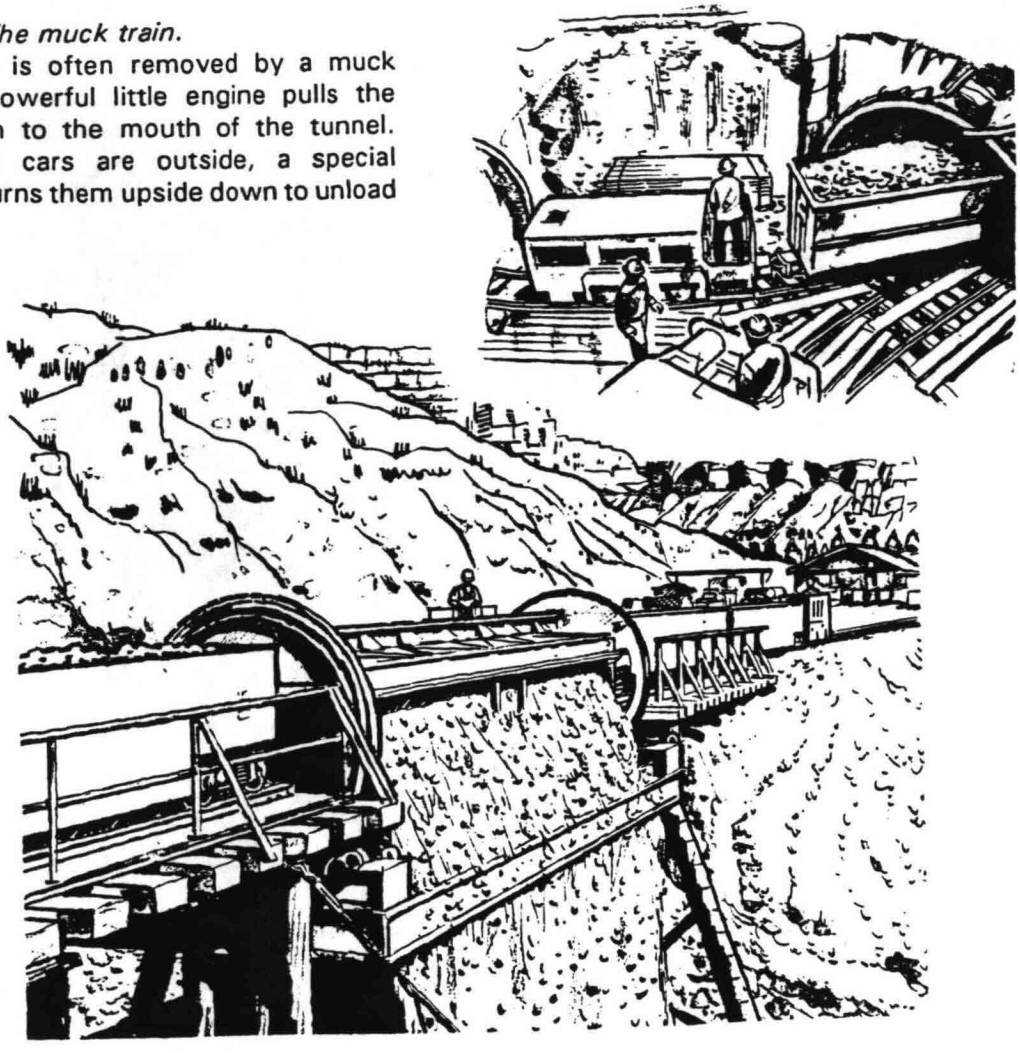
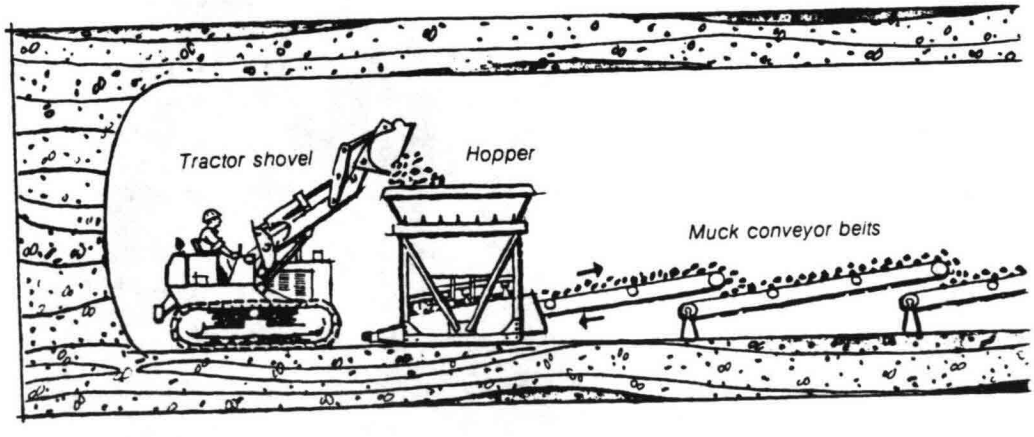


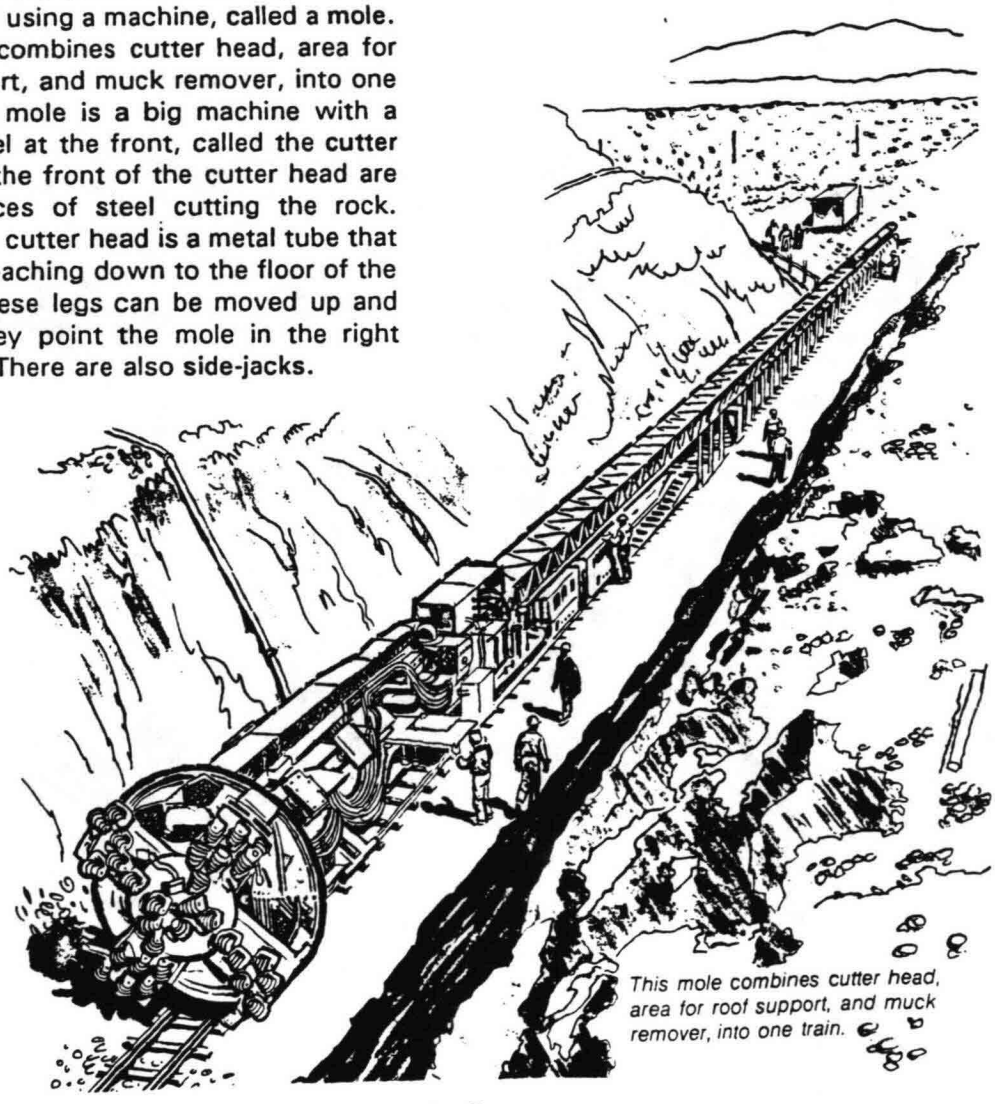
Fig. 4.8. Conveyors.
Sometimes the mucking machine loads the muck onto a conveyor belt. The muck is first dumped into a metal box called a hopper. Doors in the bottom of the hopper drop the muck onto the moving conveyor belt. Several conveyors are put together in a row when the muck needs to be moved a long way.



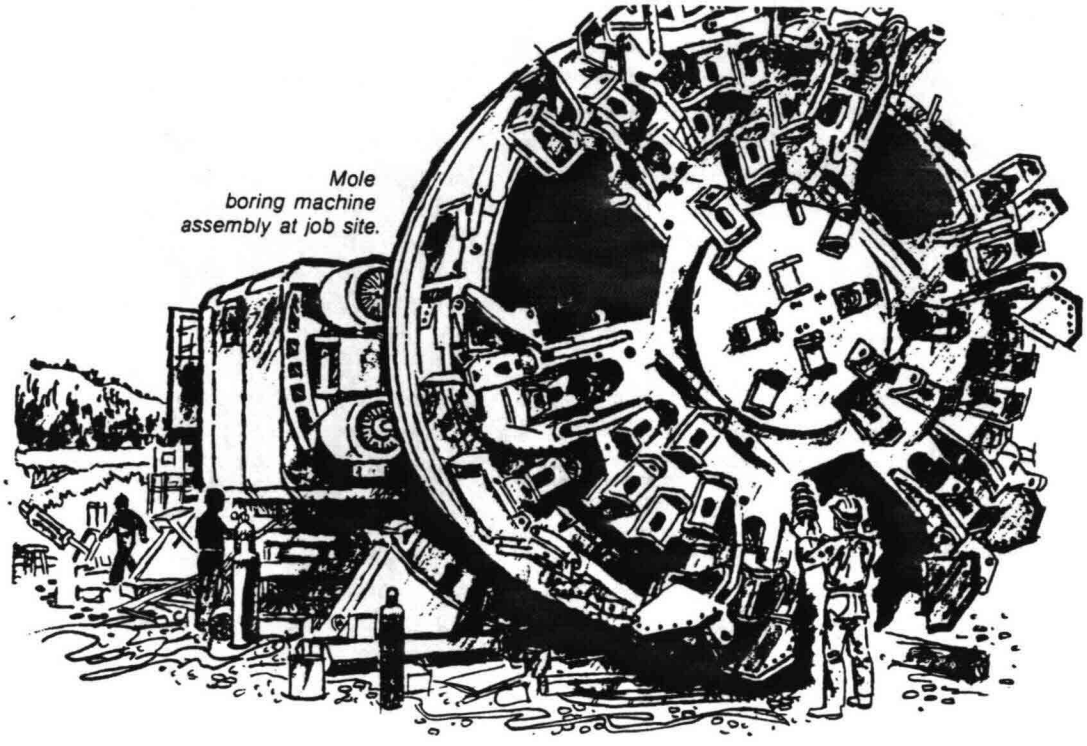
Tunnelling machine, tunnel boring machine (TBM). A machine for excavating a circular tunnel. The first ones were designed for penetrating relatively soft rocks but they are now able to drive fast through the hardest rock. A rotating cutting wheel breaks the ground, which drops through slots in the cutting wheel for removal. The cutting wheel diameter is larger than the bore of the lined tunnel by twice the thickness of the tunnel lining. In soft ground, instead of the TBM a backhoe may be used. TBMs have been successful in soft ground but they are inflexible and generally suitable only for the type of rock they were designed for.In variable ground the TBM may have to be changed, a cause of long delay. In soft ground a shield must be used, normally a **close-face shield**, with a **tail-skin** at the rear end long enough for building at least two segmental rings.

Behind the cutting wheel in soft-ground TBMs is a watertight steel wall, the bulkhead, through which the muck is removed, either by a screw conveyor or by pipeline transport. The bulkhead provides protection against an inrush of mud or water and enables the face to be pressurized if need be, either by compressed air or by injected slurry. TBMs do not weaken the surrounding ground as explosives do but they are a heavy investment. One TBM for the Channel Tunnel has a train of associated equipment 215 m long behind it. At large diameters the cutting wheel is often supported and driven from its outer drum next to the shield. In early TBMs it was driven by a central shaft which obstructed the removal of muck. Other tunnelling methods involve **boomheaders** or **hammer tunnelling**.

Fig. 4.9. Tunneling machine.
Sometimes the tunnel can be dug without blasting by using a machine, called a mole. The mole combines cutter head, area for roof support, and muck remover, into one train. The mole is a big machine with a giant wheel at the front, called the **cutter head**. On the front of the cutter head are **teeth**, pieces of steel cutting the rock. Behind the cutter head is a metal tube that has legs reaching down to the floor of the tunnel. These legs can be moved up and down. They point the mole in the right direction. There are also **side-jacks**.

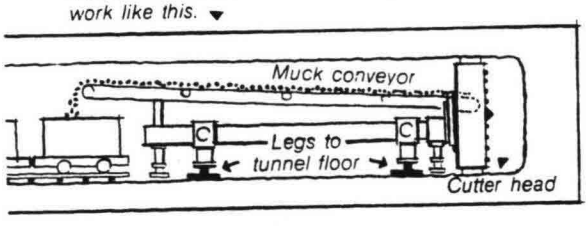


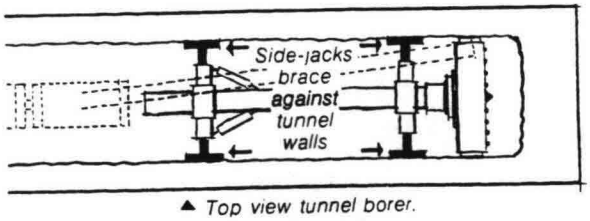
This mole combines cutter head, area for roof support, and muck remover, into one train.



Mole boring machine assembly at job site.

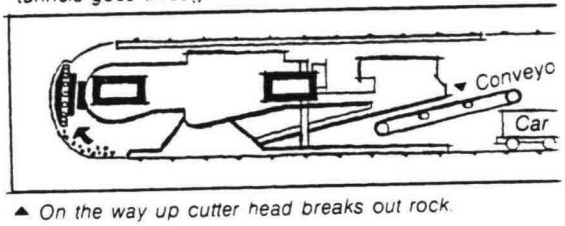
Modern conventional tunnel boring machines work like this. ▼



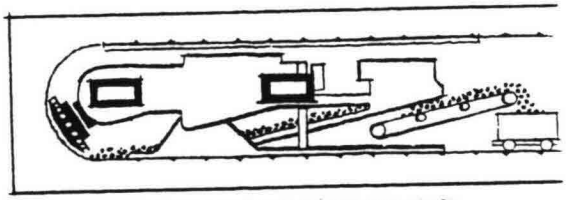


▲ Top view tunnel borer.

New minimole spinning head tunneler for small tunnels goes through hard rock like this. ▼



▲ On the way up cutter head breaks out rock.



▲ On the way down cutter head sweeps muck onto conveyor.

Shield. A steel protective tube used in soft ground, inside which miners drive a tunnel with pick and shovel. Instead of miners now there is often a TBM or, in a small tunnel, a **microtunnelling machine.** The shield has jacks around the edge which push on the tunnel lining of segmental rings and advance the shield with the TBM. The protection of the shield eliminates timbering but other dangers, such as inrushes of water or mud, cannot be completely forgotten though they are greatly reduced by earth-pressure-balance or slurry shields. The original open-face shields, with miners loading the muck into wagons, had protective overhead half moons and are still common with hand-mining. A **close-face shield** is a tunnelling shield which, like the slurry shield or the earth-pressure-balance shield, has a steel partition (bulkhead) protecting the tunnel from inrushes of mud or water.

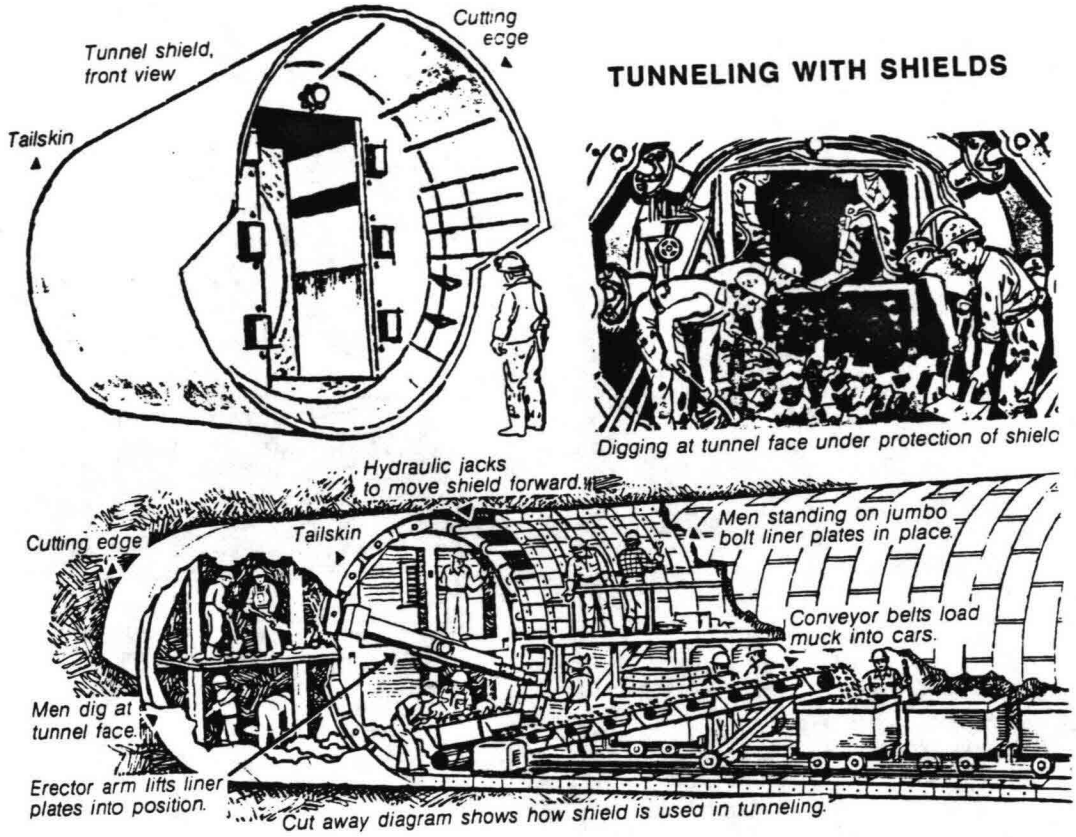
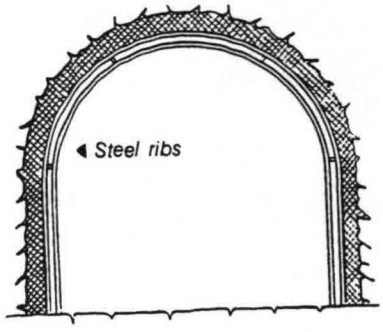
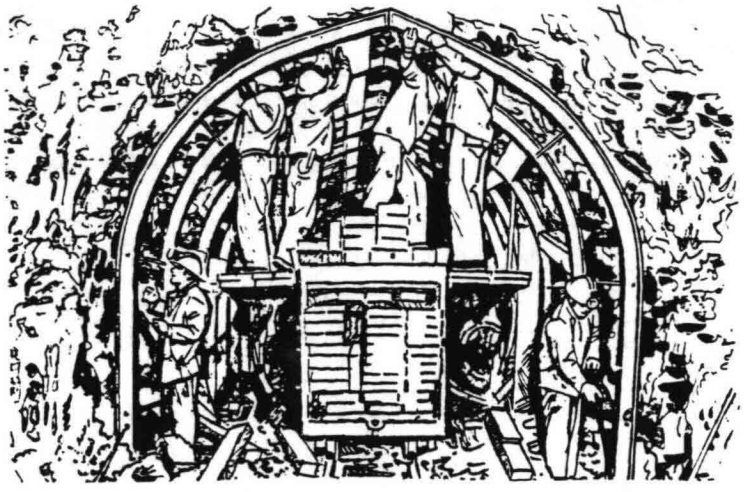
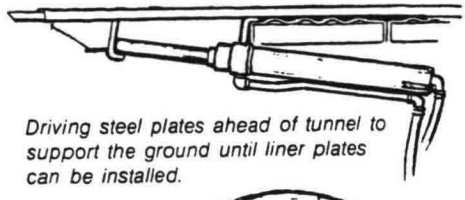


Fig. 4.11. Tunneling with shields.
Some tunnels are shield driven; the shield is like a very large tin can with both ends open. The shield is pushed through soft ground br **thrust jacks.** The front edge of the shield is called the **cutting edge.**The back part is called the **tailskin.** As the shield moves forward, the tunnel lining is built behind the tailskin. The lining can be steel liner plates or concrete or stone.

Fig. 4.12. Supporting the tunnel.
Digging the hole is just part of building a tunnel. Support is needed to hold the sides and the top of the tunnel in place, Even rock tunnels have to be supported to make them safe.







Driving steel plates ahead of tunnel to support the ground until liner plates can be installed.

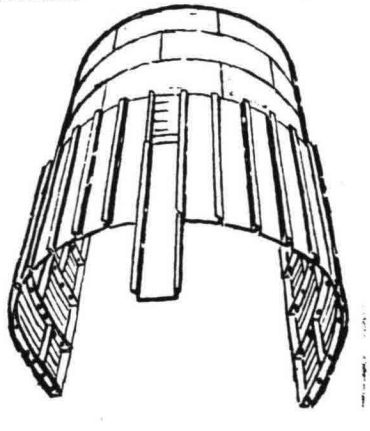


Fig. 4.13. Tunnelling through soft ground.
If a tunnel goes through soft ground special care is to be taken to keep the ground from caving in. Sometimes pointed boards are driven around the roof and sidewalls of the tunnel. Then the dirt is dug away. Another way to dig soft ground is to drive steel plates ahead of the tunnel. These plates will hold up the roof while the men dig away the dirt. If the dirt is very soft, it is held back by **breast boards**. The boards are braced again the working face.

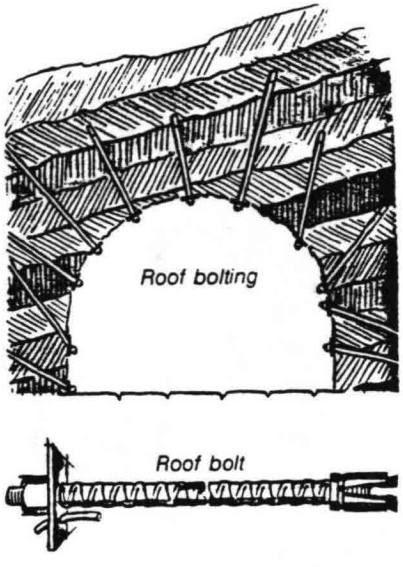


Fig. 4.14. Roof bolts.

Sometimes roof bolts are used to hold up the roof of tunnels built through rock. The roof bolt is a long steel rod. A hole is first drilled into the rock above the tunnel. Next the roof bolt is pushed up into the hole. The end that is pushed into the rock gets bigger when the bolt is twisted. This holds it tight inside the hole. Then a piece of steel with a hole in it is placed on the outside end of the bolt. This flat piece is held firmly against the tunnel roof by twisting a nut on the outside end of the roof bolt.

Installing rock bolts using jumbo.

Tensiling the bolt.

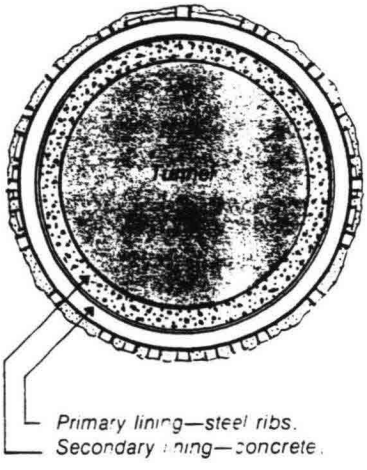
Another way to support a tunnel through rock is by **spraying** the inside of the tunnel with concrete. A pump forces the concrete through a big hose and is sprayed in an about 100 mm thick layer onto the walls and roof.

After finishing digging of the tunnel **another lining** must be put on the inside of the tunnel. The lining installed to prevent the tunnel from caving in during the excavation is called the **primary lining**. The new lining is called the **secondary lining** and consist of reinforced concrete.

Tunnel lining. In soft ground, tunnel linings are normally segmental rings of wedged concrete in good ground or bolted cast iron in bad ground. They may be made waterproof sometimes by grouting behind after erection. In rock, protection against water is achieved by continuously welded plastic sheets next to the shotcreting over the rock, all round the upper part of the tunnel. The plastic sheets are designed for drainage by dimpling, channels or felt bonded to them. Concrete lining are usually not less than 300 mm thick, and are cast over the plastic waterproofing.

Segmental ring. A ring for lining a tunnel or shaft, built up from segments of precast concrete, pressed steel or (originally) cast iron bolted to each other. Some concrete segments are not bolted but wedged. A 1 m dia. tunnel usually needs only three segments, but for a 3.65 m internal dia. tunnel, about 15 segments could be used and either one or two short key segments at the crown. Concrete segmental rings have been adopted for much of the Channel Tunnel because of their fast erection speed (5 m/h), to keep up with the TBMs. With segment-erection machines forming part of the TBM, they are almost as reliable as cast iron, and much cheaper.

Tunnel with primary lining in place. Secondary concrete lining to be installed.



Primary lining—steel ribs.
Secondary lining—concrete.

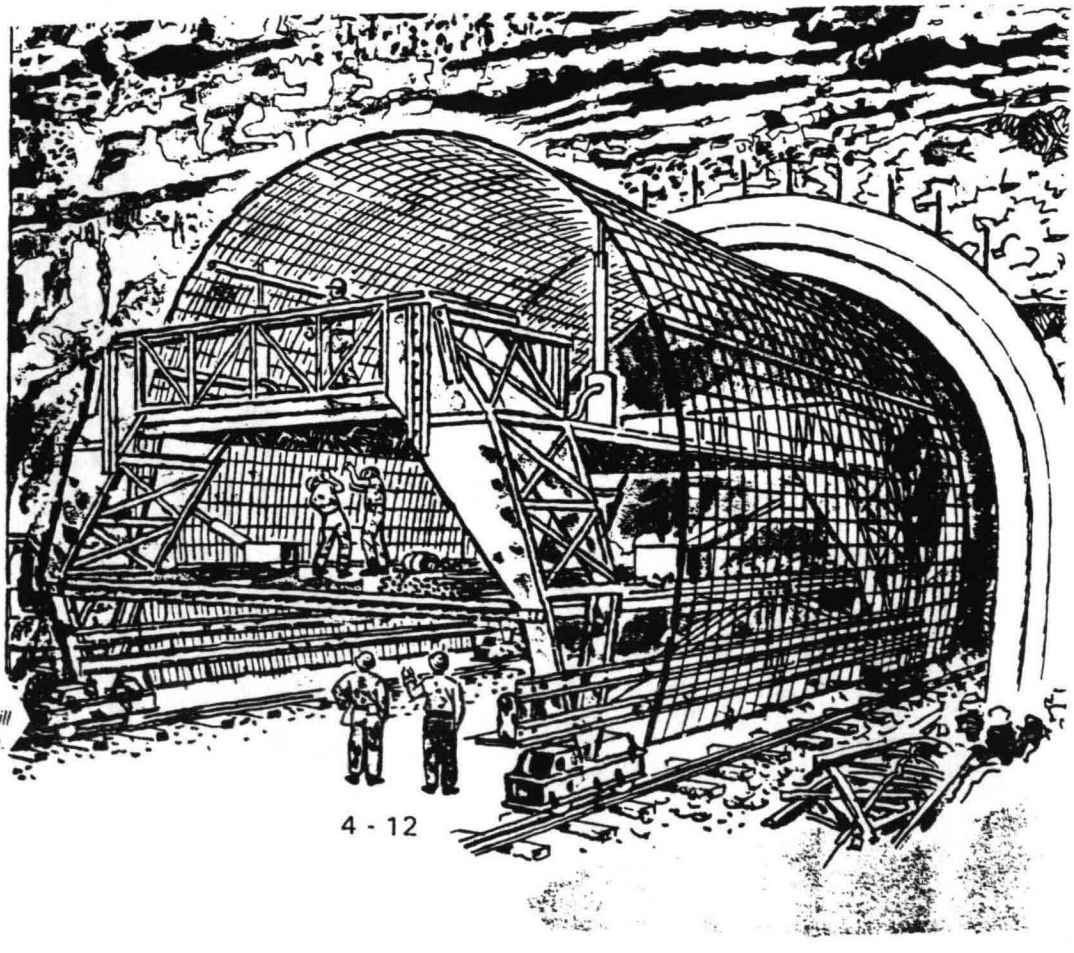
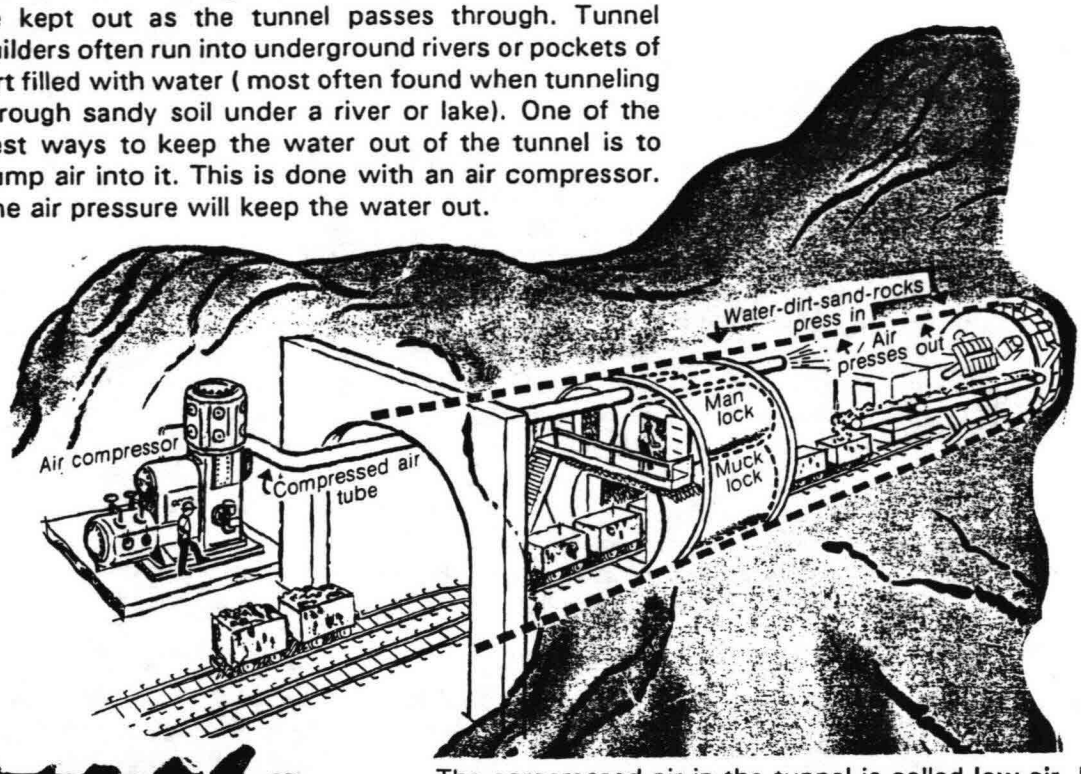


Fig. 4.15. Lining the tunnel with concrete.

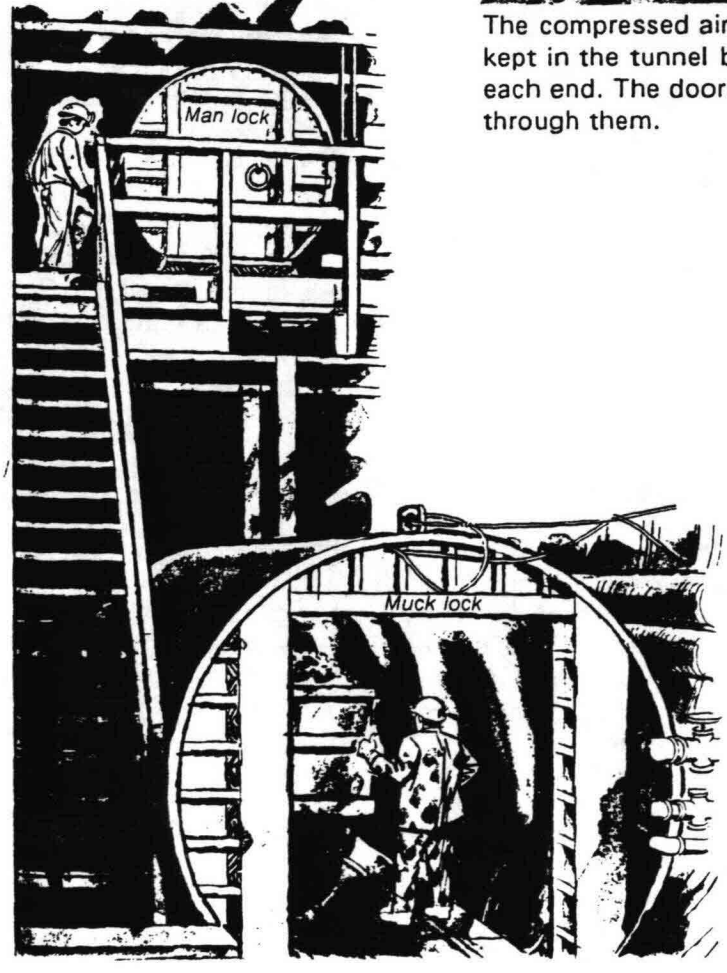
Construction jumbo ready to transport wired together reinforcing bars into tunnel. There they will be covered with concrete.

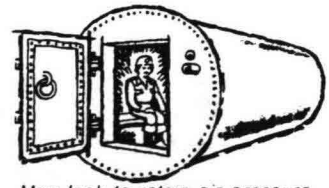
Tunneling through wet soil

Sometimes there is ground water present, which must be kept out as the tunnel passes through. Tunnel builders often run into underground rivers or pockets of dirt filled with water (most often found when tunneling through sandy soil under a river or lake). One of the best ways to keep the water out of the tunnel is to pump air into it. This is done with an air compressor. The air pressure will keep the water out.



The compressed air in the tunnel is called **low air.** It is kept in the tunnel by an **air lock,** which has a door at each end. The doors close so tightly that air cannot go through them.





Man lock to retain air pressure and allow passage of men.

Muck lock

Slurry shield, bentonite shield, hydroshield. A closed-face shield for a slurry tunneller excavating a face into which water or a clay or bentonite slurry is pumped under controlled pressure to restrain inrushes of water or mud. The slurry lubricates the cutters and its return pipe remove the cuttings from the face. An air-lock may have to be provided for workers to go into the face and remove break boulders. The slurry shield is used in Japan for driving through any weak soil from soft clay to water-bearing gravel, above or below water, but the stones must be smaller than 50 mm. One of the world's largest tunnels, driven in this way, of 11.2 m dia. is an aqueduct. The tunnel is kept on line by a laser beam aimed at a target on the shield. Using water instead off bentonite reduces the difficulties of settling solids from a bentonite slurry, which may be considerable.

Slurry tunneller. A tunnelling machine or microtunneller (MT) for soft waterlogged ground which makes compressed air unnecessary. The machine works in a slurry shield. Slurry may also drive the cutting wheel if it is not electrical.

Compressed air. A compressed-air atmosphere in a tunnel or a shaft excludes water from it even under 30 m of water, but this depth requires 3 at of air pressure to exclude the water. Someone working in this pressure needs 2.5 hours of decompression and a short working shift. Fine silts or soft clays are best for compressed-air work. In gravels, air losses apart from being expensive may damage the surface. To prevent such troubles, any shield should have ample soil cover - at least its own diameter. To avoid these as well as the medical hazards of compressed air (caisson disease, bone necrosis), slurry shields and earth-pressure-balance shields have been developed.

Earth-pressure-balance shield (EPBS). A shield that eliminates the need for a compressed-air atmosphere, and is used with a tunnelling machine. The volume excavated is mechanically controlled. A screw conveyor delivers it through a pipe passing through the bulkhead on to the conveyor or wagon. The 'sand plug' thus formed in the pipe by the conveyor can withstand the pressure of a collapsing face. At the delivery point on to the conveyor, an additional control may be provided in the form of a rotary discharger. One EPBS type controls the pressure of the cutting wheel on the face rather than the volume of excavation. Removal of muck may be by belt, pipeline, etc. EPBSs have become a Japanese speciality.

Fullface tunnelling machine. A tunnelling machine which, unlike a boomheader, breaks out a face of circular cross-section with its cutting wheel. Probably the first fullface machines were those that in 1882 bored circular 2.1 m dia. tunnels 1800 m long from Dover and from Calais in the first attempt at a Channel Tunnel. They were known as Beaumont-English machines after the inventor and developer.

Multi-face tunnelling machine. Two or more fullface tunnelling machines can dig overlapping tunnels, with one machine ahead of the other. This is claimed to save 13 % of the excavation needed for one large circular tunnel.

Hammer tunnelling. The use of a hydraulic hammer (heavy concrete breaker), powered by a hydraulic excavator or loader for breaking ground instead of the usual drilling and blasting. In 1988 five large Italian tunnels were being driven by this means, one of them nearly 100 m2 in cross-section. Advantages claimed are that the rock is less shattered, the tunnel is safer, and the method is less noisily than blasting. The same excavator can be used for loading broken rock and breaking ground. The hammer can be changed for a digging bucket in a few minutes, even though a heavy hammer weighs 3.5 tonnes.

Boomheaders, roadheader, cutter boom tunneller. Machines of many different types for breaking ground in rock that does not have to be blasted. They have existed since the 1930s for driving headings in mines and until 1970 they were relatively small, with up to 25 Kw power. (the largest, of 500 Kw, now weigh more than 100 tons). They move on caterpillar tracks or wheels, or on a gantry or monorail in a shield. The 'pineapple' rotating head at the end of the massive boom projecting ahead can be directed upwards, sideways or downwards. In some machines the boom is telescopic. Like fullface machines they make no overbreak as explosives do, nor do they shatter the rock, and they can choose their tunnel profile, unlike fullface machines. The many makers in Europe and the USA have machines that cut 6 m wide and 6 m high from one position. The cutting-head motors use half the power.

Water-jet assisted boomheader (WJAB). In 1987 more than 60 of these boomheaders were in use, developed jointly by manufactures, the US Bureau of Mines and the British Coal Corporation. The water jet, applied at the point of cut, greatly reduces the dust and eases rock cutting. Pressures vary from 14 to 42 MPa, with a maximum water consumption of about 50 litres/min. Possibly in order to reduce water nuisance, German researchers are investigating much higher pressures of 140 Mpa.

Pilot tunnel, pilot shaft. A tunnel, microtunnel or shaft driven sometimes at a fraction of the size of the final excavation. It enables the final tunnel to be driven from either end at will, with full knowledge of groundwater and rock, with good ventilation and the simplest method of soil removal. The pilot tunnel also ensures that the final tunnel method is in the right place. Some microtunnels are reamed several times before the final diameter is achieved. But in the Channel Tunnel the pilot is driven at its final diameter. It is nevertheless a pilot because it precedes the running tunnels.

Pipe jacking, pipe pushing, thrust boring.

In soft ground, building an underground pipeline by assembling rigid pipes at the foot of the assembling pit, and jacking them through the ground to a reception pit, instead of digging a trench to lay them. It was the first method of microtunnelling. The jacks react against a strong concrete wall at the back of the launch pit. Usually the soil inside the pipe is excavated by a microtunnelling machine, but thin steel rods or pipes can sometimes be pushed without prior excavation.

Where the thrust becomes too much for the jacks available an interjack station must be inserted to recommence pushing. Polythene pipe, the most popular replacement pipe, cannot be jacked, being too flexible, though it can be pulled. Jacked pipelines are usually straight or even slightly curved.

Launch pit, jacking pit, thrust pit. A pit for starting pipe jacking or a microtunnel.

Reception pit, receiving pit. A pit dug at the opposite end from the launch pit of a microtunnel or pipe-jacking job.

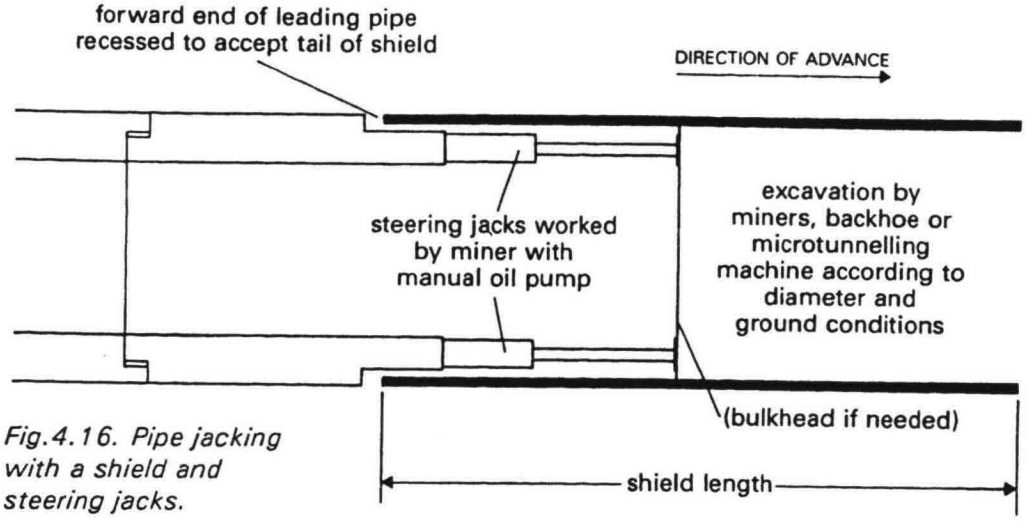


Fig.4.16. Pipe jacking with a shield and steering jacks.

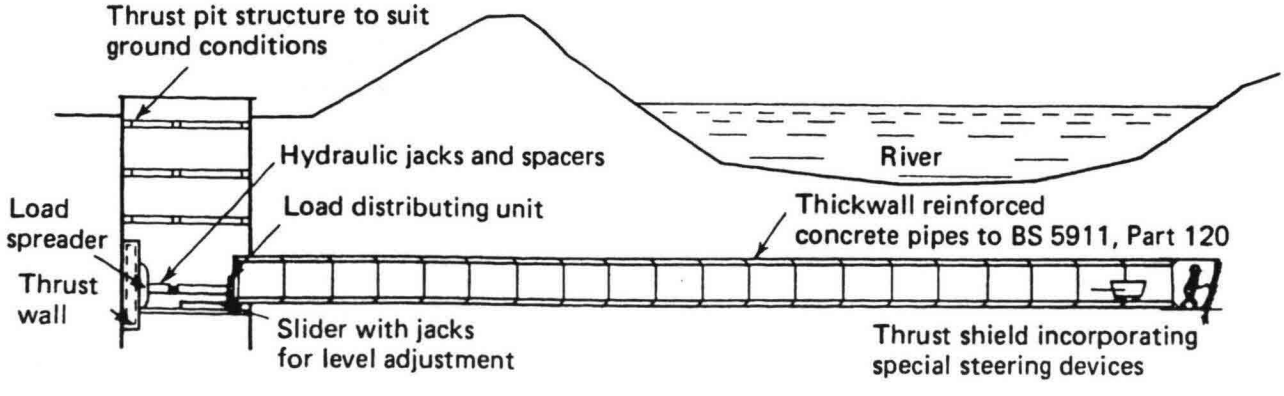


Fig. 4.17. Section through a typical pipe jacking operation.

Orange River Project, Orange-Fish Tunnel

The multipurpose Hendrik Verwoerd Dam provides for electric power generation in two ways: a power house is located at the toe of the dam and a small plant is envisaged at the downstream end of the Orange-Fish Tunnel. The dam, with its maximum height of 90 m above foundation level, provides for a storage capacity of round 6 milliard m^3. About 25% of the water of the Orange River is drawn from the reservoir through the intake structure and delivered by the 82.8 km long 5.35 m dia low-pressure tunnel into the Fish River Valley, where it is used for irrigation, further for urban, industrial and recreational purposes.

It is the longest irrigation tunnel in the world. Construction begun in 1967 and the boring was completed in April 1973. The lining has a minimum thickness of 230 mm.

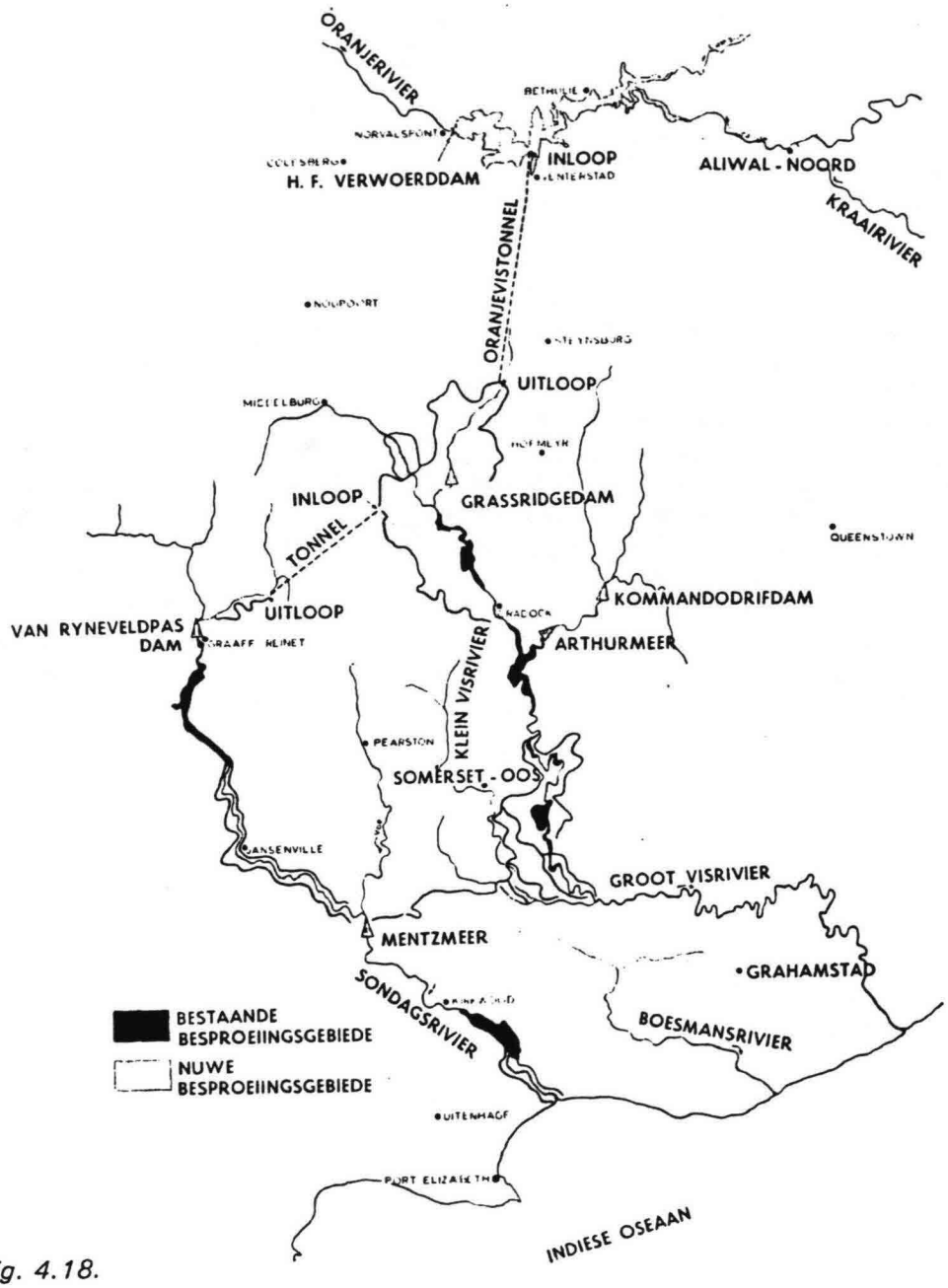


Fig. 4.18.

Channel or Eurotunnel

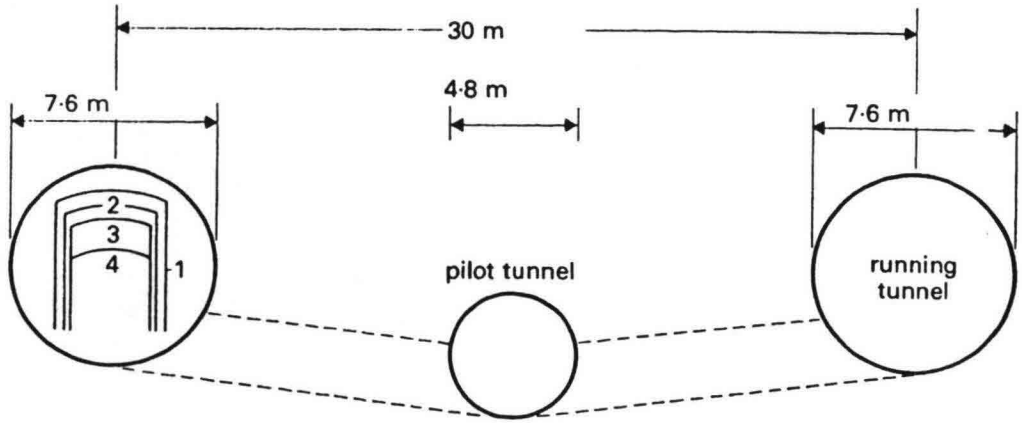
A project begun and stopped on both sides of the Channel in 1882 and 1975, but began again in 1987 with the achievement of the break-through of the pilot tunnel at the end of 1990 at the planned speed of 1 km/hour from both ends. There are to be three tunnels 49 km long from Frethun near Calais to Cheriton near Folkestone. The smallest tunnel, of 4.8 m dia., the pilot tunnel for service, access and drainage, was driven first. On each side of it but slightly higher are the running tunnels of 7.6 m dia. All are driven by **tunnel boring machines.**

In France the serious geological difficulties of 1974 caused French engineers in 1987 to sink a vast 55 dia. shaft 70 m deep at Sangatte to provide access for the driving of six tunnels at 40 m depth, three of them towards Paris and three northwards. Chalk excavated from the tunnels is pumped as 20 % slurry to a settling pit 1.8 km away. The pipeline transport uses fresh water to make sure that the aquifer below the settling pit is not polluted.

Transmanche Link is the main contractor, working for the client, **Eurotunnel.** representing the governments and the shareholders who have invested 6000 million pounds. Financing of the tunnel is based on the tolls received over the 55 years of franchise from its opening (originally planned in mid-1993).

Transmanche Link (TML) is a joint venture of five British and five French contractors; on the British side Balfour Beatty, Costain, Tarmac, Taylor Woodrow, Wimpey; on the French side Bouygues, Dumez, SAE (Societe Auxiliaire d'Entreprises), SGE (Societe Generale d'Entreprises) and Spie Batignolles.

Eurotunnel is the promoter of the Channel Tunnel, a Franco-British partnership between France Manche and the Channel Tunnel Group. Responsible to the governments and shareholders, it employs the contractor TML. By 1989 it had ceased to be a promoter and was wholly project manager. After completion it should again change its role and become the tunnel operator. The client is also, in part, the contractor. Ten of the 15 main founder shareholders are the contractors building the tunnel; the other five founders are banks. To improve the independence of the client the two governments stipulated an Anglo-French maitre d'oeuvre to be staffed by independent consulting engineers. Engineering managers from Bechtel Corporation are employed by the maitre d'oeuvre to manage the project.



Train sizes
1. Channel shuttle (double deck)
2. SNCF
3. British Rail
4. London tube

Fig. 4.19. Channel Tunnel (typical cross-section).

4.2. Microtunnels

Microtunnel, non-man-entry tunnel. Any machine made tunnel too small for a person to work in. The absolute minimum, even for someone small, is 900 mm. A microtunnel usually start from a launch pit and finish in a reception pit. **Pipe jacking** methods have been used to drive very small pipe without excavating.

Microtunnelling machine, microtunneller. Many types exist, and they may include augering, jetting, pipe jacking, fullface tunnelling machines, etc. In 1988 a small microtunneller of 135 mm dia. could drive length up to 150 m. The microtunneller with its rotating electrical cutting head has to drag an electric cable. a high-pressure water pipe and a pipe containing bentonite slurry to lubricate the walls of the tunnel. It cannot drill rock or loose gravel.

At this small diameter the cost was estimated to be twice the cost of trenching. But in Berlin in 1988 microtunnels at 150 mm dia. were cheaper than trenching. Microtunnels can be enlarged indefinitely by reamers.

Slurry tunneller (MT). A microtunneller for soft waterlogged ground. The machine works in a slurry shield. Slurry may also drive the cutting wheel if it is not electrical. The electric power cable, apart from supplying the driving power, may send back information about the direction and position of the cutting head. One Japanese MT of 400 mm dia. uses a temporary series of steel linings with integral slurry feed and return pipes. When the hole has been completed, these 'cans' are pushed into the receiving pit and the new permanent pipe is pushed in with much less force than normal pipe jacking demands.

Since **deep trenching** had always been more expensive than shallow trenching there has hitherto been no incentive to lay pipes or cables deeper than the minimum 1 m. This situation has now changed because a deep microtunnel is little more expensive than a shallow one. When a microtunneller meets an obstacle and cannot advance, it is usually withdrawn, and an attempt is made to drive more deeply. The important implication is that records of the location of pipes and cables must now be more accurate than formerly to avoid conflicts underground.

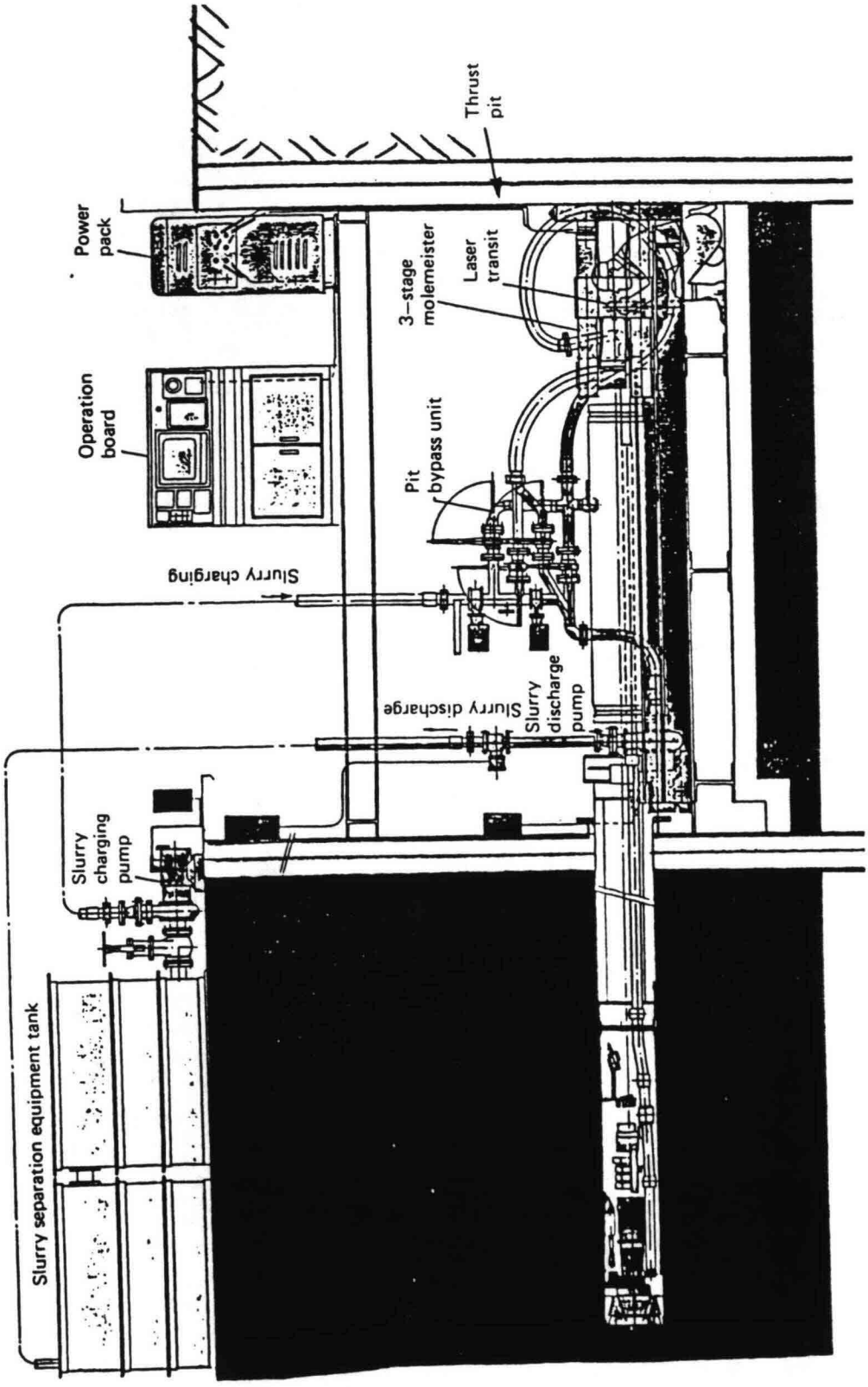
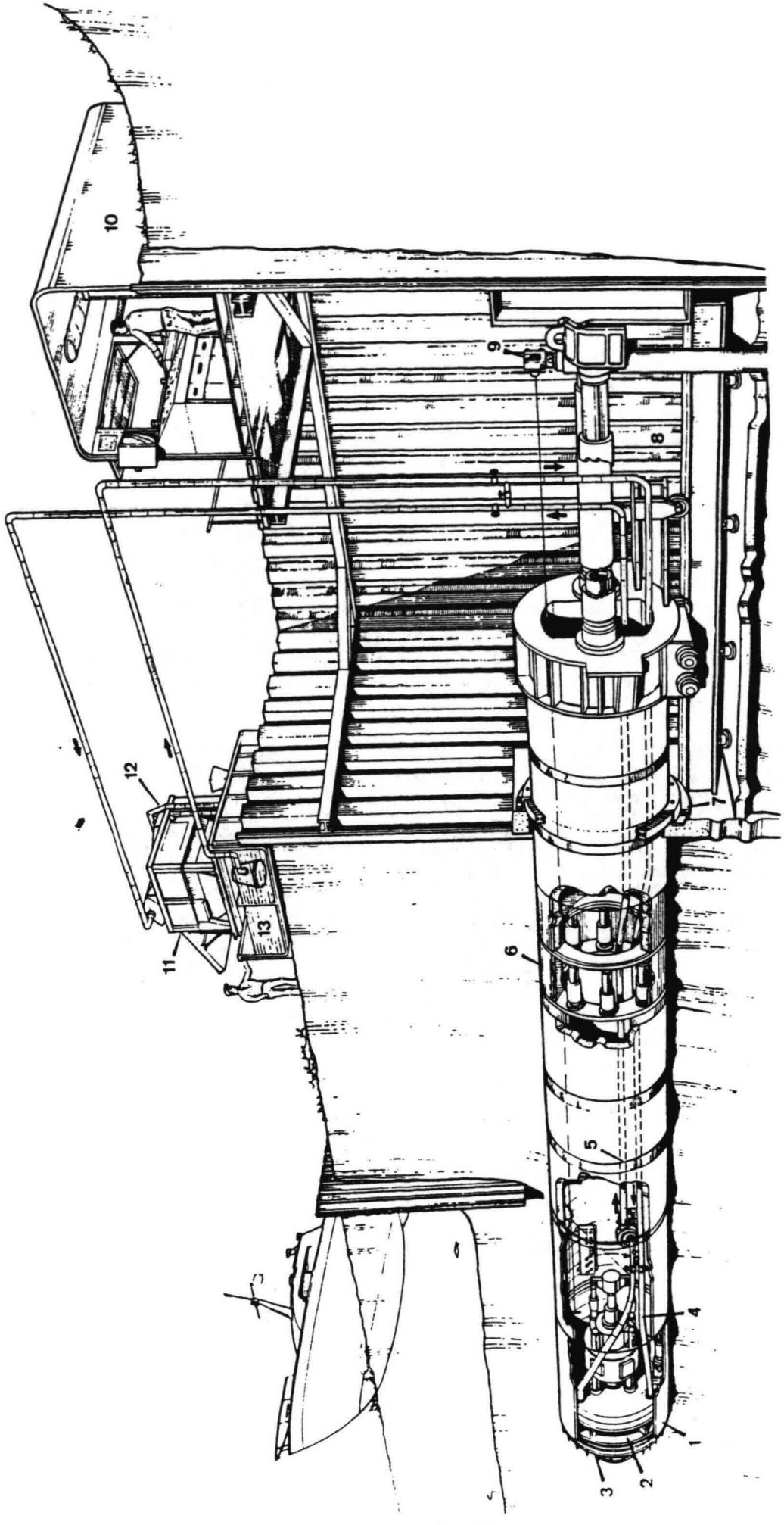


Fig. 4.20. Microtunnelling with a bentonite slurry shield.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4.3. Immersed tubes

An underwater tunnel made by sinking precast concrete boxes (tube elements) into a channel dredged for them and joining them up under water. Concrete tubes are often provided with a steel watertight 'skin', while 'steel' ones have to be loaded with a concrete base to overcome their buoyancy. The precasting dock, if specially excavated for the job, is a large part of the expense. Each element is closed by diaphragms at the ends, floated out, accurately sunk and rammed to fit its neighbour. Immersed tubes can be less deep than conventional tunnels, which need much more than the few metres of cover that protect an immersed tube from ship's anchors.

The first immersed tube for vehicles was 800 m long, built in 1910, and contained two railway tunnels between Michigan, USA, and Ontario, Canada, under the Detroit River.

The first British immersed tube, begun early in 1987 under the **Conwy** River in Conwy, North Wales, was expected to cost 100 million pounds. The 700 m long immersed tube on the North Wales coast road has a rectangular cross-section, 24 m wide. At both ends are conventional tunnels, making a total tunnel length of 1090 m with dual carriageways. The six precast tubes each weighing 35,000 tons are set in a suction-dredged trench from which 6 million m3 of mud must be taken. The bottom and sides of the tube are lined with 6 mm thick steel plate as permanent shuttering, with shear connectors welded on. The steel is cathodically protected. The carriageways are 10.45 m wide by 5.1 m high. Design by Travers Morgan & Partners; contractor: Costain-Tarmac.

In 1988 the concept of a toll-paying motorway in an immersed tube along the bed of the Thames from Chiswick to the City 25 km away was mentioned as a possible relief to east-west traffic congestion in London, but costing nearly as much as the Channel Tunnel.

Another concept, but in 1850, was an immersed steel tube across the English Channel proposed by a Frenchman. It would have had ventilation towers above the sea at intervals.

Possibly the world's largest immersed tube, across **Hong Kong** harbour, was completed in 1988. It has two rail tunnels, two road tunnels an a service tunnel divided in two to provide two ventilation ducks. To reduce the heat of hydration of the heavy concrete units, 20 % of the cement was replaced by fly-ash, improving the sulphate resistance of the concrete. Visible cracks in the concrete were grouted with epoxy resin, and the sides and roof of the units were sprayed all over with an epoxy rubber membrane. In the placing of the 15 tube units, a cylindrical, rubber, water-filled bag was placed under each end of the unit, enabling it to be set to precise level while its sand foundation was pumped in below it. The water in the **jack bag** is pumped in or out to adjust the level of the unit.

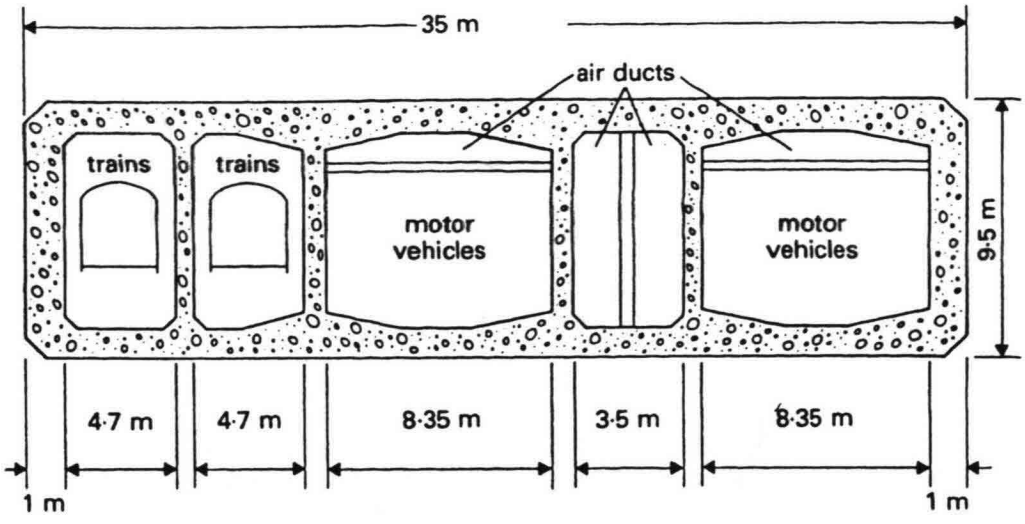


Fig. 4.22. Immersed tube, Hong Kong. Typical cross-section of the 15 concrete immersed tube units, from 122 to 128 m long, laid to cross Hong Kong harbour in 1987-88, and weighing from 40,000 to 42,000 tons each. Contractor: Kumagai Gumi; consulting engineers: Freeman Fox and also Maunsell and Partners.

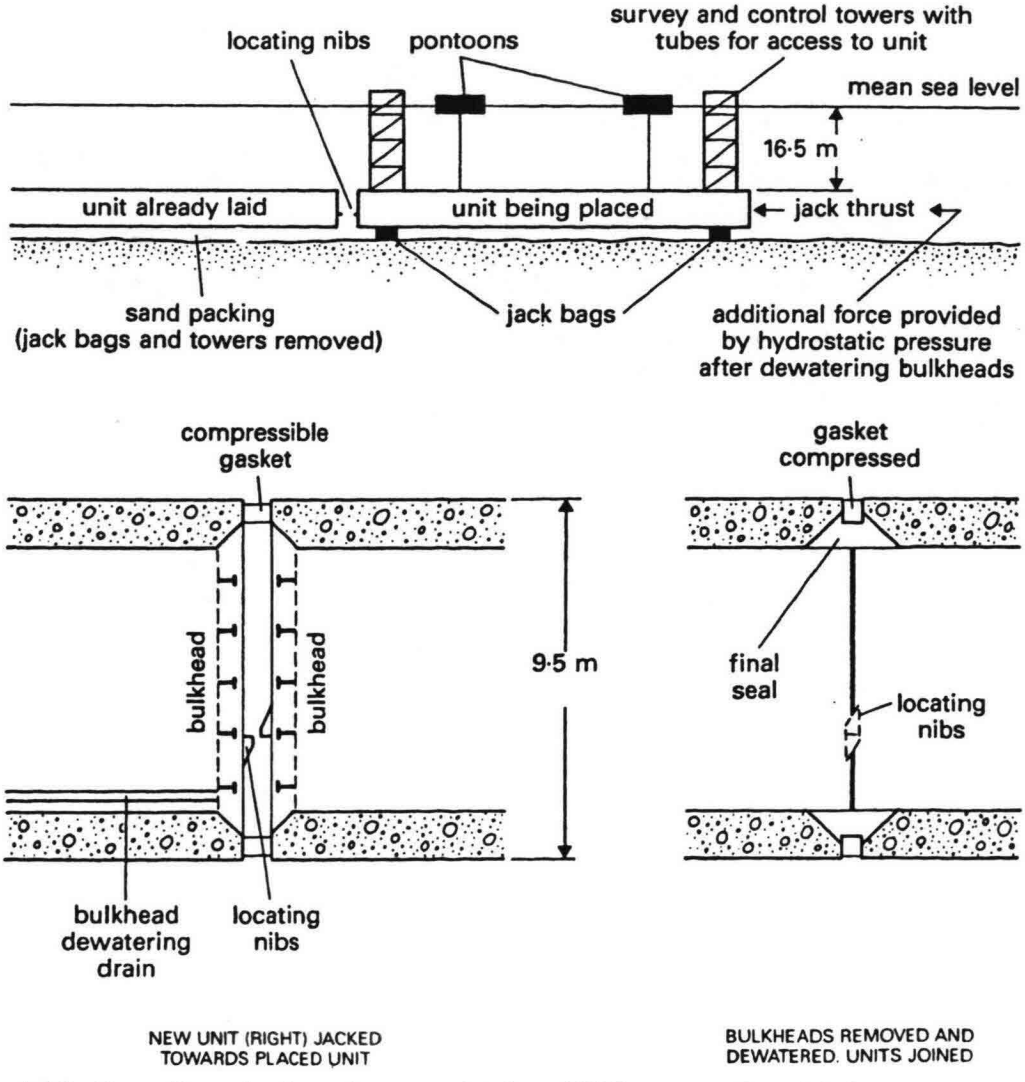


Fig. 4.23. Hong Kong harbour immersed tube, 1988: connection details.

Tunnels in The Netherlands

On the contrary to bridges, a tunnel leads the traffic under the water-course to the other side. Because of the increasing traffic and hold-up problems, fixed bridges have preference nowadays. The bridge-level must be in such a way, that navigation can pass. In Holland for inland-navigation applies : 9 m (Rhine-navigation), 7 m (2000 DWT), 5.5 m (1350 DWT), etc.

Sea-ships need a clearance up to 50 m, which means very high and expensive fixed-bridges. In that case a tunnel solution is preferable. The investment of a tunnel, as a rule, is slightly higher than a bridge, but maintenance is lower. Long tunnels must be strongly ventilated to remove engine-gases.

The construction of tunnels in The Netherlands for the purpose of the traffic started more than fifty years ago. Tunnels for road and rail transport and tunnels passing under water and shipping channels are built using various methods, of which the shield method, the immersion method and the open construction pit method are the best known.

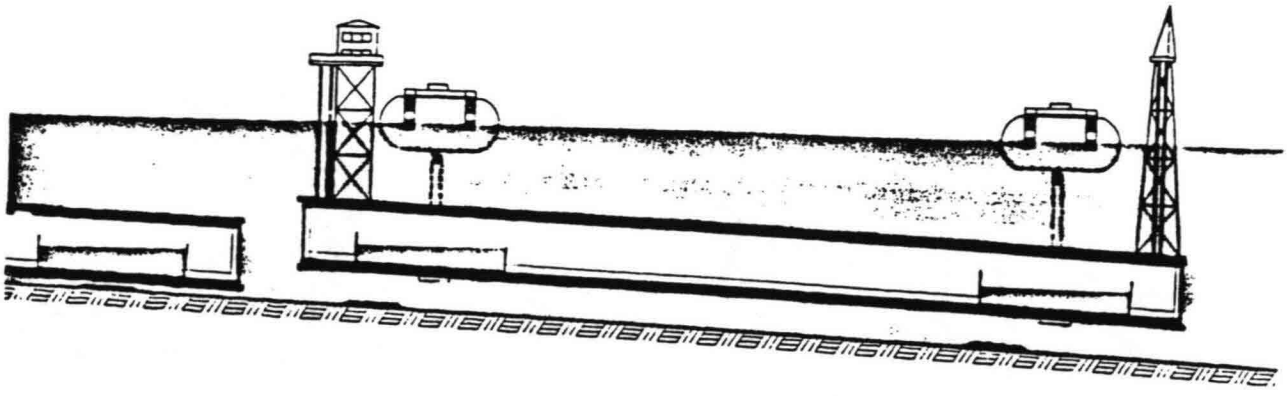
The principle of the immersion method involved the construction of individual tunnel sections (in steel and/or concrete) in different locations, which sections are then transported and lowered into the water in the trench of a river or a canal. The first tunnel to be built according to this method was the Michigan Central Railroad (MCR) Tunnel (1906 - 1910).

When the town-government of Rotterdam decided in the late 1920's to build a tunnel, three engineers from the Department of Public Works were sent to America to study this method. This resulted in the construction of the Maas Tunnel, using the immersion method, which was completed before the second World War. Because of its enormous technical and financial advantages, this method has been very popular since the 1960's for large tunnel projects under channels and waterways.

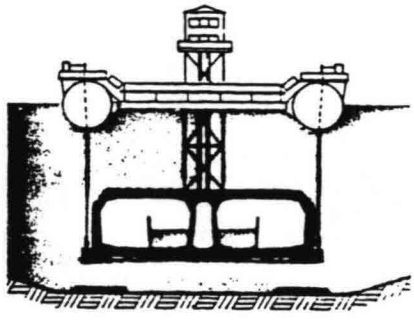
The open pit method has the advantage that the tunnel is constructed directly in its final position together with the foundation. This method is fairly popular for the construction of so-called "land-tunnels" and for tunnels under relatively small shipping ways and waterways. The method of construction using an open pit is mainly determined by the need to lower the water table temporarily. (In the west of The Netherlands the water table is, on an average, only just below the surface (0.5 - 1 m).

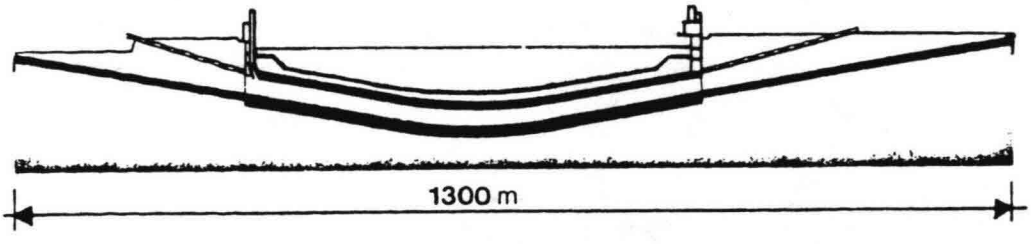
A combination of the immersion and construction pit methods is frequently used for the construction of tunnels in The Netherlands, the immersion method in particular for the part of the tunnel below the canal or river, the open pit method for the adjacent parts underground. The first to be constructed was for a road in the city of Rotterdam.

Immersion method

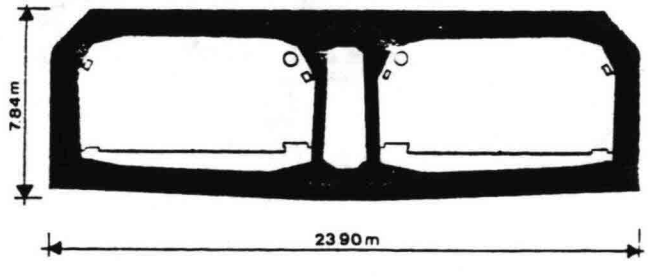


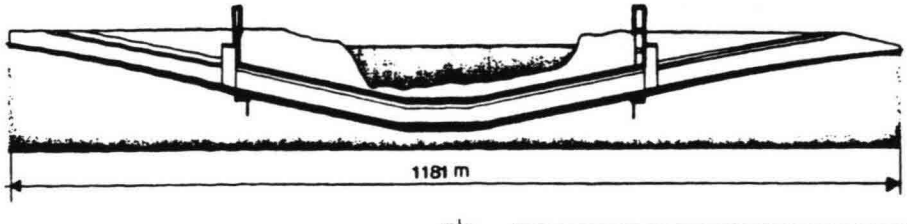
The tunnel section is immersed with the aid of two positioning towers and a twin-set of immersible pontoons: a cross-section and a longitudinal section are shown here.



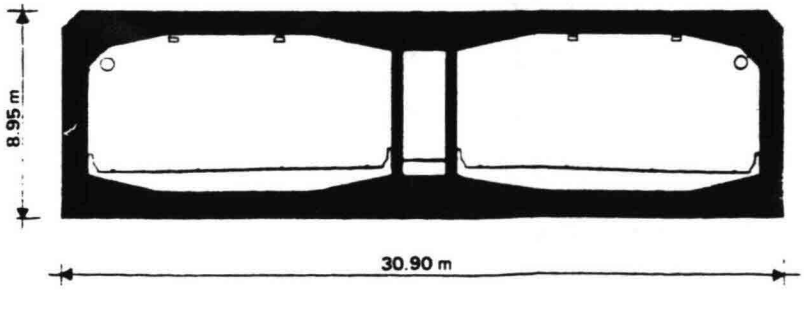


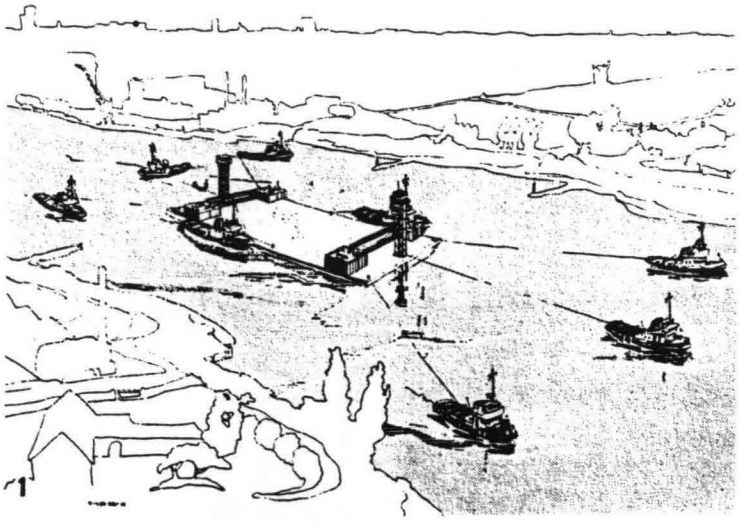
Benelux tunnel

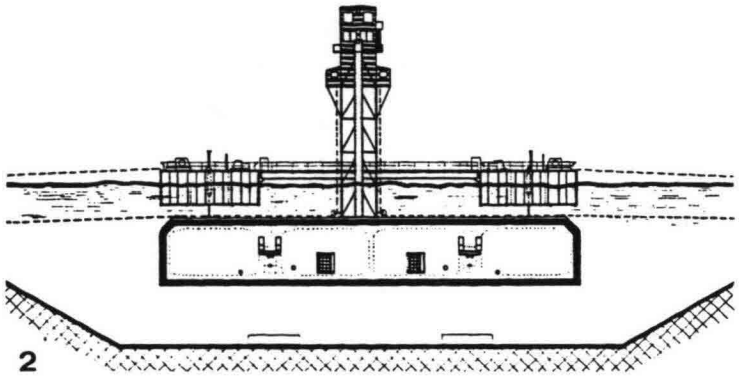


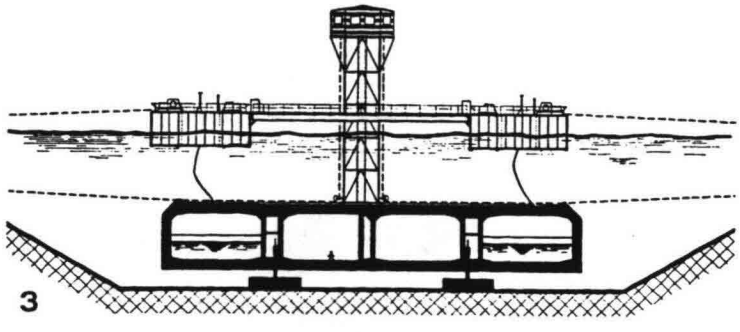


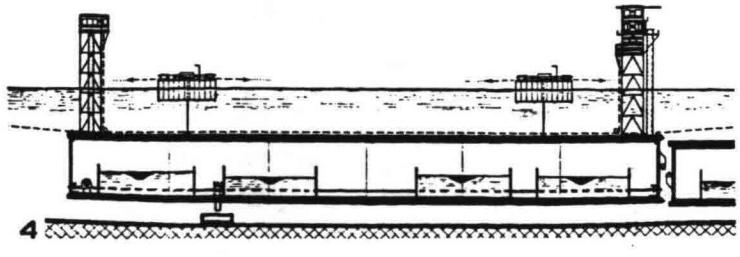
The Botlek tunnel











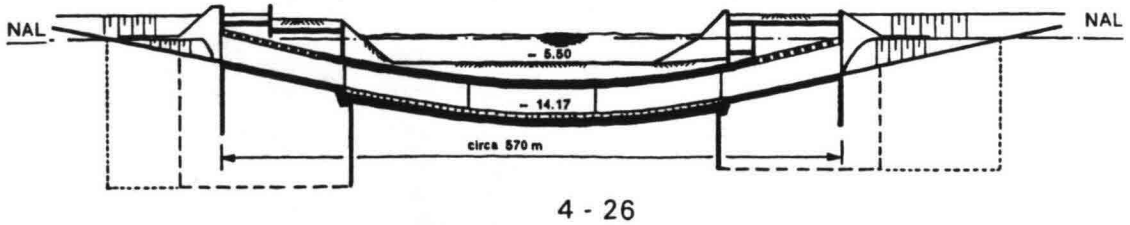


Fig. 4.25. Construction of the Drecht traffic tunnel near Dordrecht - completed in 1977 (The Netherlands).

1. An artist's impression of the transportation of the tunnel elements from the construction dock to Dordrecht.

2. The commencement of the sinking operation (cross section).

3. The completion of the sinking operation.

4. Longitudinal section of a sunken element.

Longitudinal section showing the sunken river tunnel and the covered and open access roads on both sides.

Benelux tunnel

Year of opening	:	1967 (construction 1962 - 1967)
Method of construction	:	immersed tunnel
Enclosed part	:	795 m (width 23.90 m, height 7.84 m)
Including entrance and exit ramps	:	1,300 m

The finances were largely raised through loans made available by a group of insurance companies and pension funds. Upon its completion the tunnel was to be used as a toll tunnel until such time as it was taken over by the state. In this way it was possible to start on the construction a considerable time before this would have been possible if it had been solely a state venture.

The Benelux tunnel forms part of the ring-road around Rotterdam. It is the western link between the banks of the river "Nieuwe Maas".

The enclosed part of the tunnel was built by immersion of eight section of 93 m long each. These sections were built in a construction dock on the southern bank of the river "Nieuwe Maas". At the tunnel location a curving trench was dredged with 800 litre bucket dredgers down to 27 m - N.A.P. When the eight sections of the tunnel have been completed, the construction dock was filled up with river water and then the construction dock was connected to the river by dredging away part of the dike.

With the help of tugboats, each section of 93 m - with temporary bulk-heads sealed - was towed into position above the trench that had been dredged and was then connected with the winch cables of the lowering system. Then the section was weighted down by filling the ballast tanks with water and placed on the temporary foundation plates on the layer of gravel laid there before. The section was pulled against the previous section in such a way that the rubber gasket around the whole circumference of the bulkheads was pressed together for a few centimetres creating a watertight joint. After being placed into position accurately in the trench, a sandbed was jetted into the opening between tunnel and riverbed on top of the above-mentioned foundation plates.

Traffic tunnels in The Netherlands

Name	Location	Year of opening	Building method	Enclosed part	Including approaches
Maas tunnel	Rotterdam	1942	immersed tunnel	1070 m	1373 m
Velser tunnel	Under Northsea Canal near Velsen	1957	open construction pit	768 m	1644 m
Coen tunnel	Amsterdam	1966	immersed tunnel	587 m	1283 m
Schiphol tunnel	Schiphol airport	1966	open construction pit	530 m	660 m
Benelux tunnel	Vlaardingen-Pernis	1967	immersed tunnel	795 m	1300 m
Y-tunnel	Amsterdam	1968	immersed tunnel	1039 m	1685 m
Heinenoord tunnel	Barendrecht (South of Rotterdam)	1969	immersed tunnel	614 m	1064 m
Vlake tunnel	Province of Zeeland (South Beveland Canal)	1975	immersed tunnel	327 m	773 m
Drecht tunnel	Dordrecht Zwijndrecht	1977	immersed tunnel	555 m	823 m
Kil tunnel	Dordrecht	1977	immersed tunnel	406 m	901 m
Prinses Margriet tunnel	Province of Friesland	1977	immersed tunnel	77 m	938 m
Botlek tunnel	Rotterdam	1980	immersed tunnel	539 m	1181 m
Gouwe aquaduct	Gouda	1981	construction pit	70 m	699 m
Zeeburger tunnel	Amsterdam	1990	immersed tunnel	546 m	946 m
Alblasserdam		under construction	immersed tunnel		

Rail tunnels

Name	Location	Year of opening	Building method	Enclosed part	Including approaches
Velser tunnel	Northsea Canal (line Haarlem - Uitgeest)	1957	construction pit	2076 m	3324 m
Rotterdam metro	North-South line -extension -- East-West line	1968 1974 1986			total 40 km
Schiphol tunnel	Schiphol	1981	Construction pit	5140 m	5780 m
Amsterdam metro		1981	Caisson	3500 m	
Hemspoor tunnel	Northsea Canal	1983	immersed tunnel	1520 m	2418 m
Spijkenisse metrotunnel	Oude Maas	1985	open pit	530 m	1611 m
Rotterdam rail-road	Nieuwe Maas	under con-struction	immersed tunnel	2608 m	3164 m

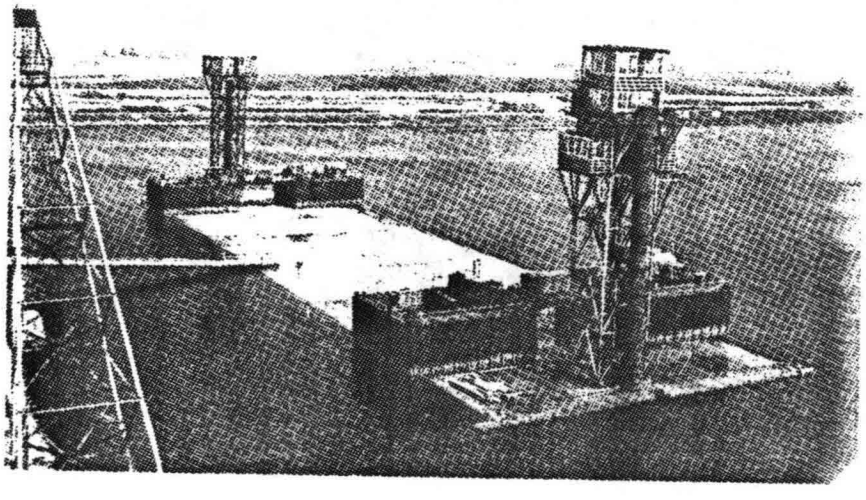


Fig.
Section of the Heinenoord road tunnel with equipment for sinking into position. In a construction dock sited to the west of the tunnel on the north bank of the river four ections of each about 115 m long were built. Year of opening : 1969

4.4. Directional Drilling

The directional drilling technique was first developed in the United States, where it was applied for the development of petroleum and gas, onshore and offshore. Today, this techniques is used worldwide and extensively for pipelines, utility conducts, cables and dredging pipes, to be installed under rivers, channels, highways, railways, etc... without disrupting the environment and the flow of traffic, as well as keeping the disturbance at the points of entrance and exit to a minimum. The technique of directional drilling can offer a solution for many previously insurmountable problems, where traditional techniques are more cumbersome and sometimes deficient. Often these techniques is more economical than the traditional open cut methods.

At the beginning of the project, a working area is prepared, where the drilling rig can be installed. When the drilling rig is positioned, and all drilling components are tested the actual drilling of the pilot hole can start. The pilot rods are pushed into the ground along the prescribed profile. When friction along the pilot rods becomes excessive, and progress is difficult to make, it is necessary to relieve this friction by washing over the pilot rods with a larger drill pipe. This pattern of drilling pilot rods and wash-over rods continues until the exit point is reached. The positioning of the pilot rods is executed continuously by using a measuring device, related to a computer which will calculate and plot the real traject of the drilled hole, in relation to the prescribed profile. The real traject can be controlled and rectified to the theoretical traject.

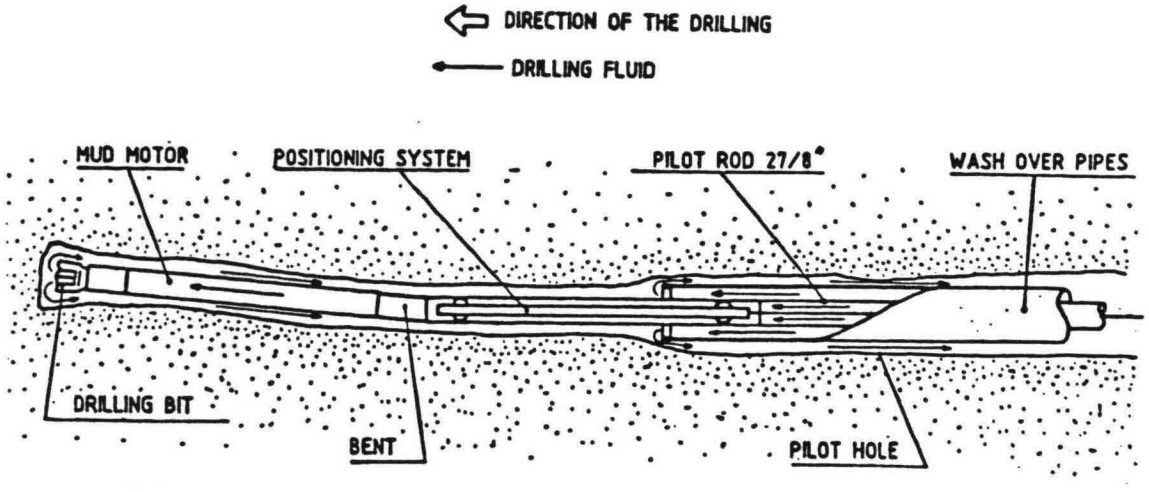


Fig. 4.26. Positioning and steering system

For the drilling mud, a mixture of water and bentonite is used. The viscosity and density of the bentonite drilling mud are determined by the soil conditioning. Once the drill pipe and the wash-over pipe exit at the other side of the river, the cutting tool is removed. The inner string (drill pipe) is pulled back and removed. Consequently, only the wash-over pipe remains in the soil and the reaming operation can start.

The wash-over pipe is attached to a reamer with diameter of 550 mm and is pulled back rotating with the drilling rig. This means that the tunnel is reamed from 125 mm up to 550 mm, this is the first phase of the pre-reaming operation. Once the reamer exits the starting point, the reamer will be removed and the wash-over pipe is connected to a second phase reamer with a 750 mm diameter so that the second phase of the pre-ream operation can start. During the second phase pre-ream process the tunnel is reamed from 550 mm up to approximately 1.5 the external diameter of the pipeline.

In all these operations (pilot-hole and pre-ream processes), a bentonite drilling fluid is used for the powering of the cutting tool, to reduce the friction and to support the bored tunnel. The pipeline string is positioned and laid down on the landing rollers in the drilling axis by means of cranes. Again the final reamer is coupled on the wash-over pipe. On this reamer the pipeline string is coupled to its pulling head. At this stage the final reaming and pipeline string installation process can start. The pipeline string is pulled back into the tunnel, which is called "the pull-back operation".

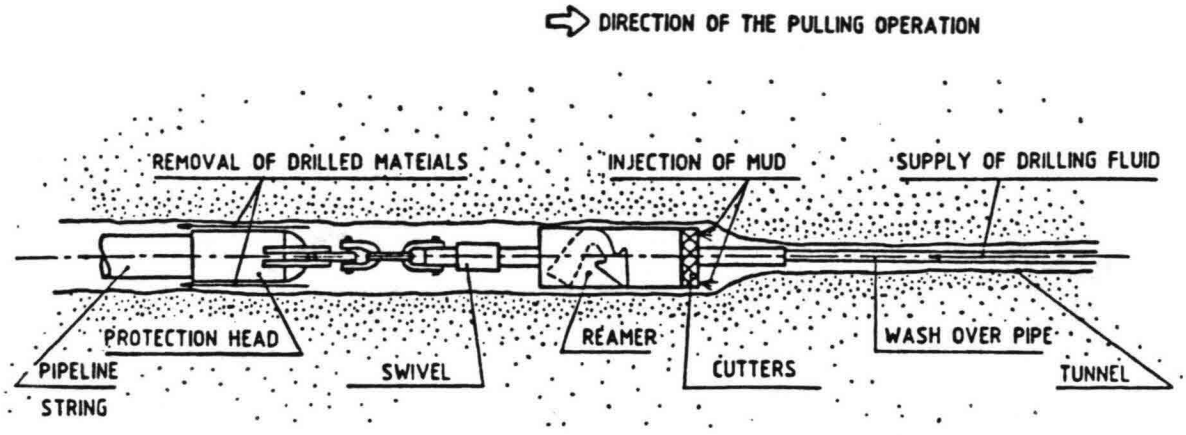
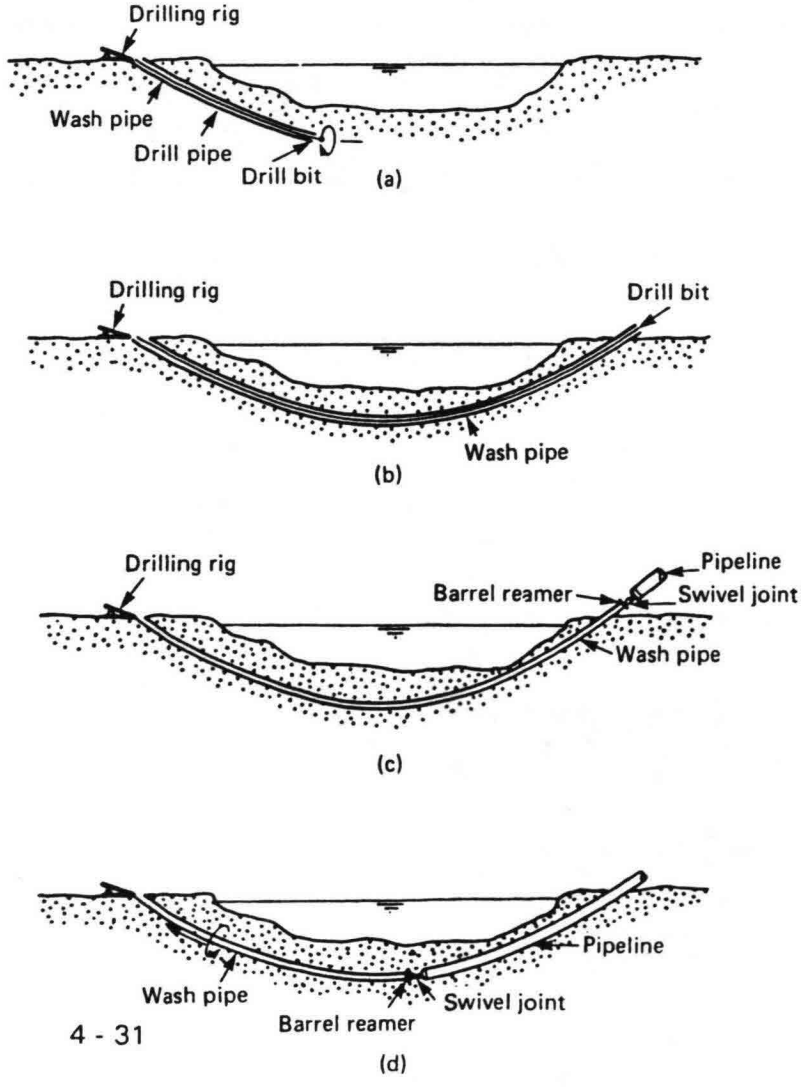
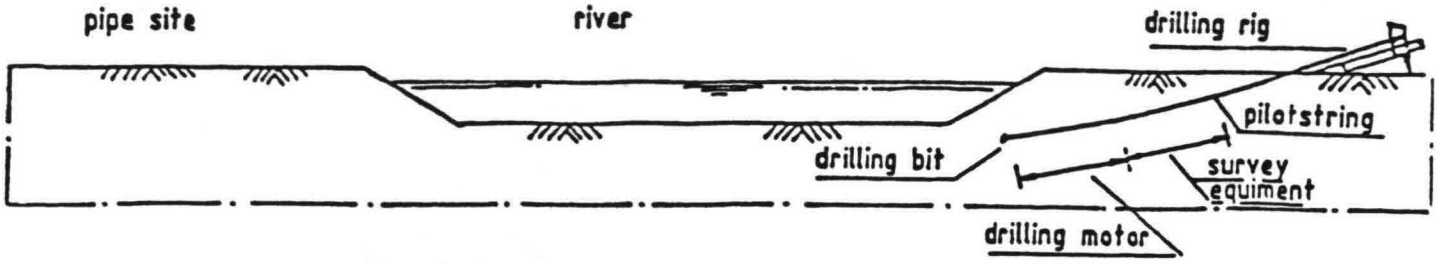


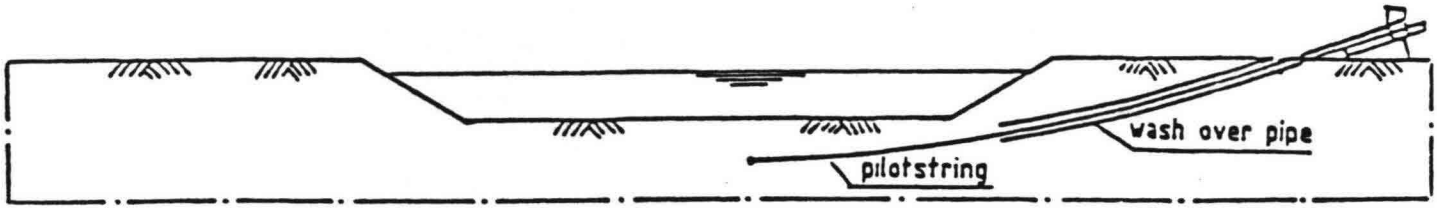
Fig. 4.7. Reaming and pull-back operation

Fig. 4.8. Directional drilling.
(a) Stage 1: pilot hole drilled by advancing the drill sting and over-drilling with the washover pipe in stages of approximately 80 m;
(b) Stage 2: pilot hole completed when both drill sting and washover pipe exit on opposite bank;
(c) Stage 3: drill pipe is removed, barrel reamer connected to wash-over pipe which is in turn connected to the pipeline pulling head by a swivel joint;
(d) Stage 4: the barrel reamer is pulled back and rotated by the drill rig positioning the non-rotating pipeline into the formed hole.

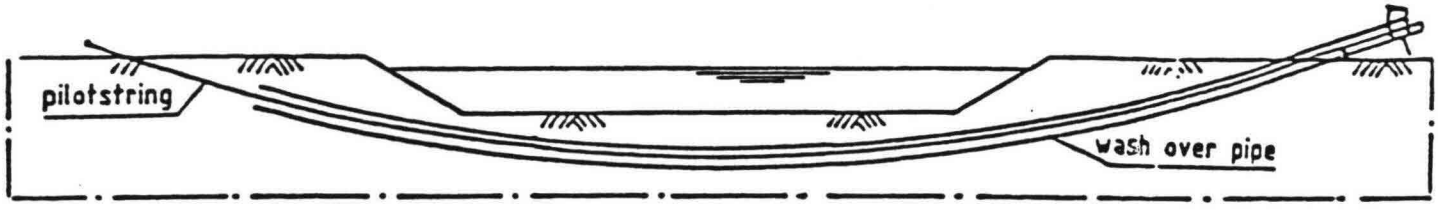




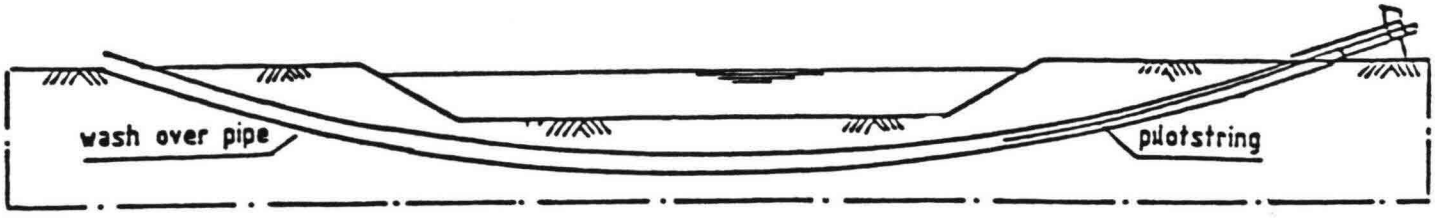
1. *Drilling of the pilot string*



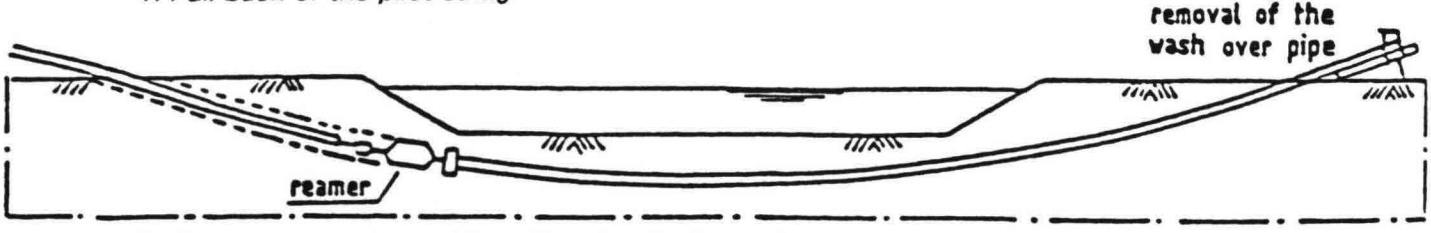
2. *Drilling of the pilot string and the wash-over pipe*



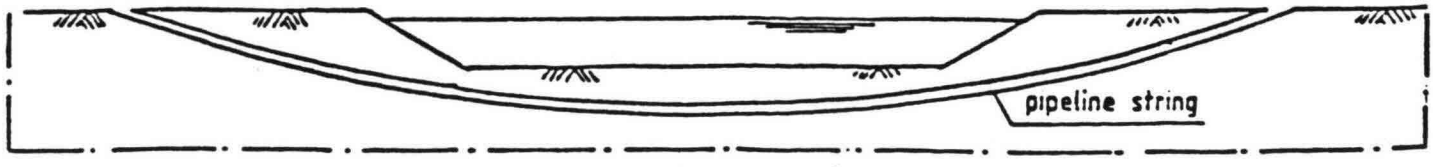
3. *Exit of the pilot string*



4. *Pull back of the pilot string*



5. *Preream operation and installing the pipeline string*



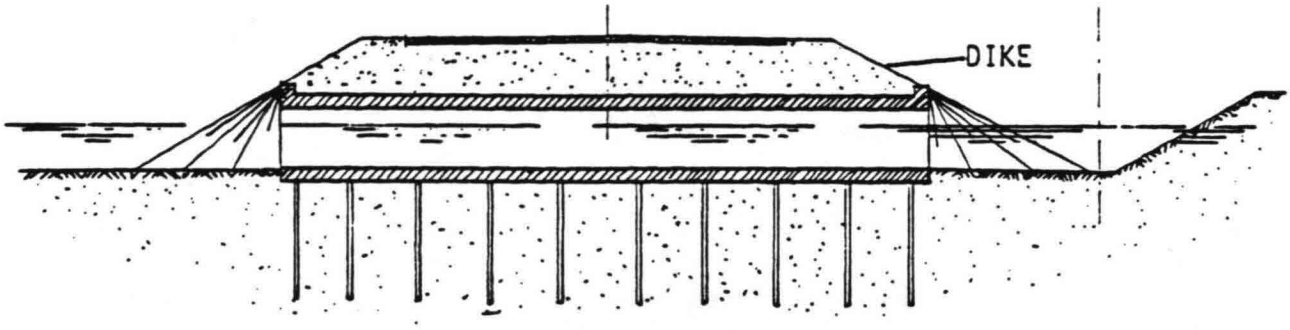
6. *Installation of the pipeline string completed*

Fig. 4.29. Sequence of the different phases.

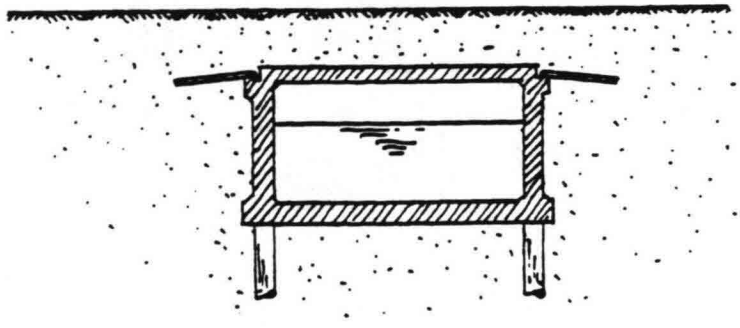
5. Culverts & siphons

Culverts are tubular structures, connecting two water-bodies or areas. The cross section may be rectangular or circular. Culverts are found in dams, under roads, etc. The water level in the culvert is free.

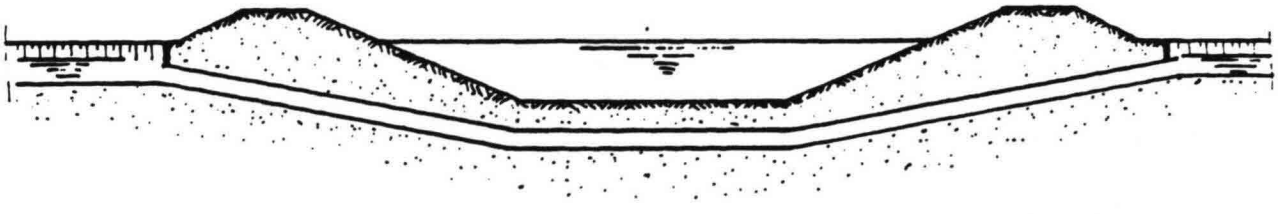
Inverted siphons (ground) culverts pass a flow under a river, are completely filled with water; the water is under pressure, no free water level. A culvert can be prefabricated of steel or concrete elements, and then transported floating to the site and sunk down in a specially excavated channel or within a dry building pit.



LONGITUDINAL SECTION (free waterlevel)



CROSS - SECTION



LONGITUDINAL SECTION (water under pressure)

Fig. 5.1. Culverts and inverted siphons.

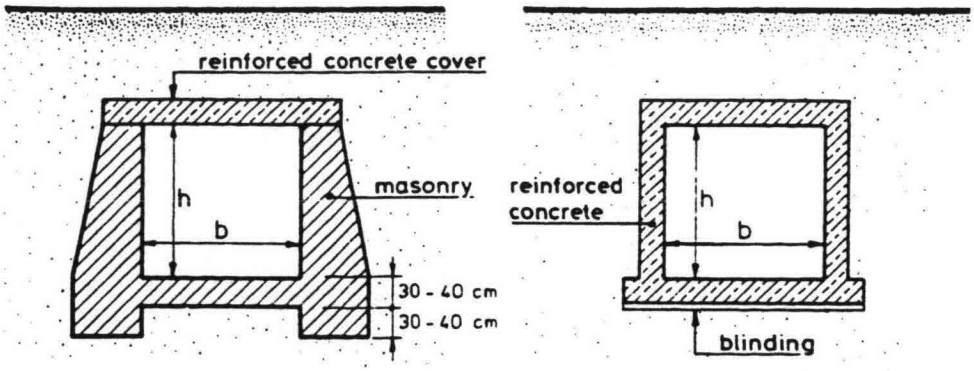


Fig. 5.2. Rectangular culverts.

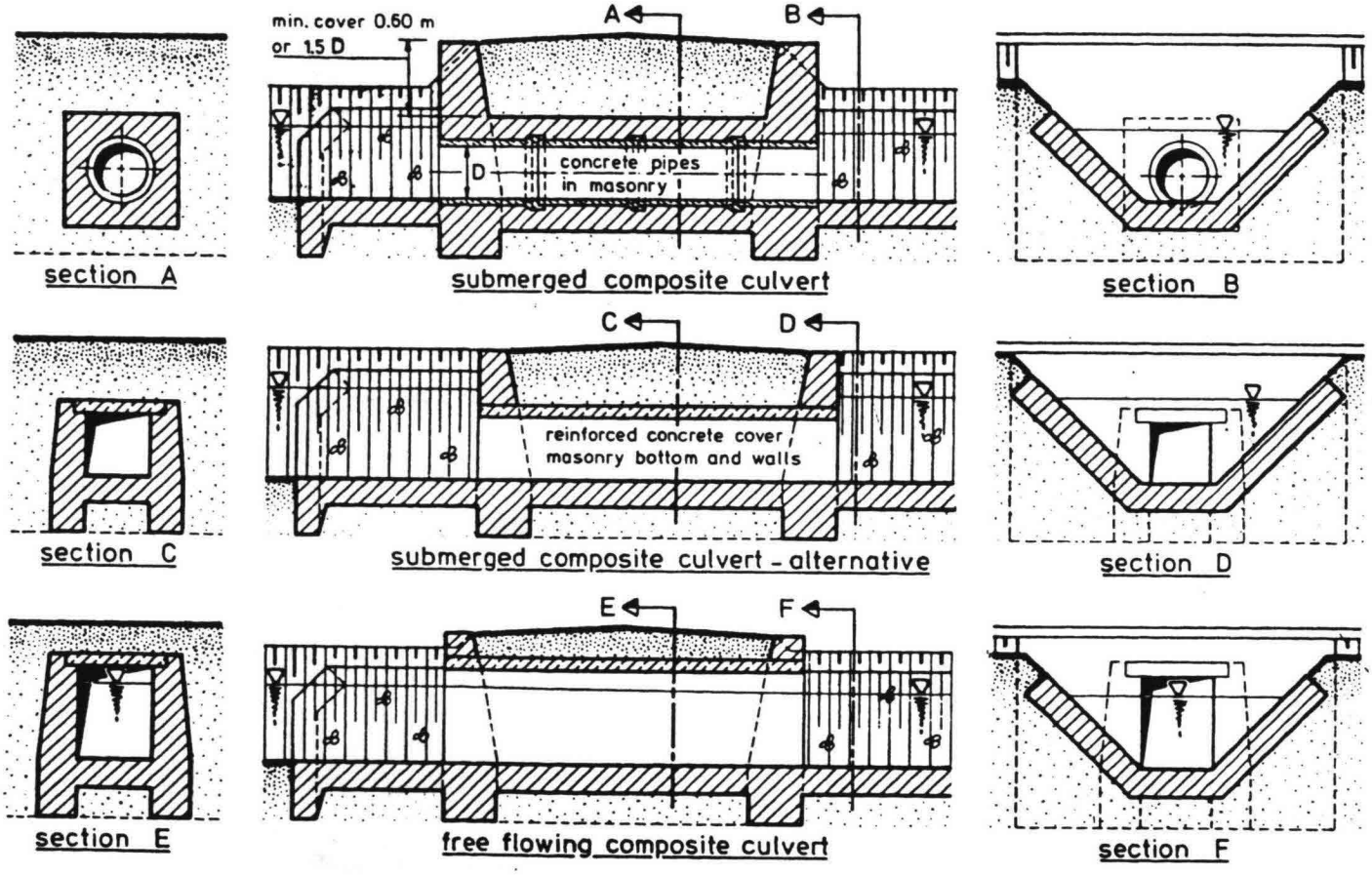


Fig. 5.3. Small road crossings (culverts).

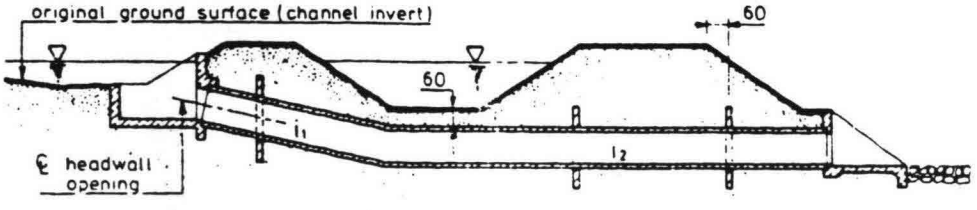


Fig. 5.4. Culvert-typical profile.

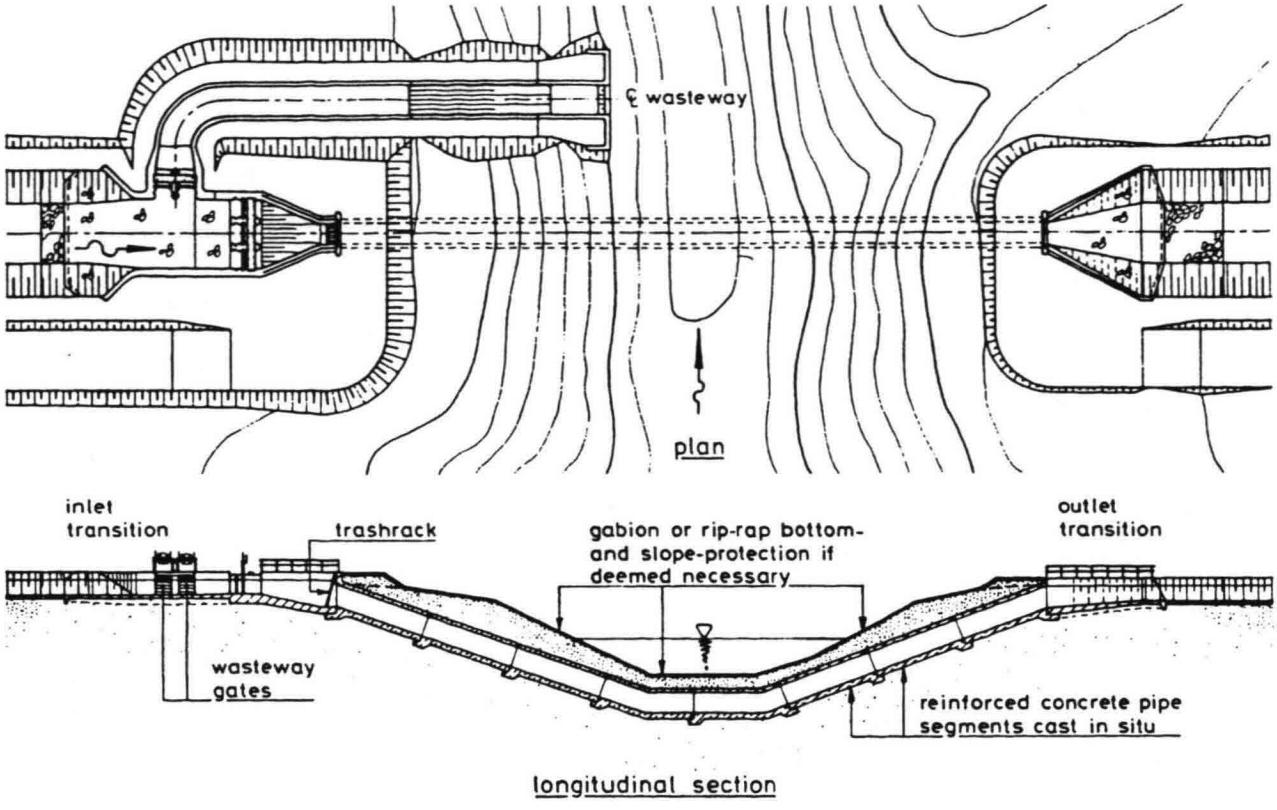


Fig. 5.5. Example of inverted siphon

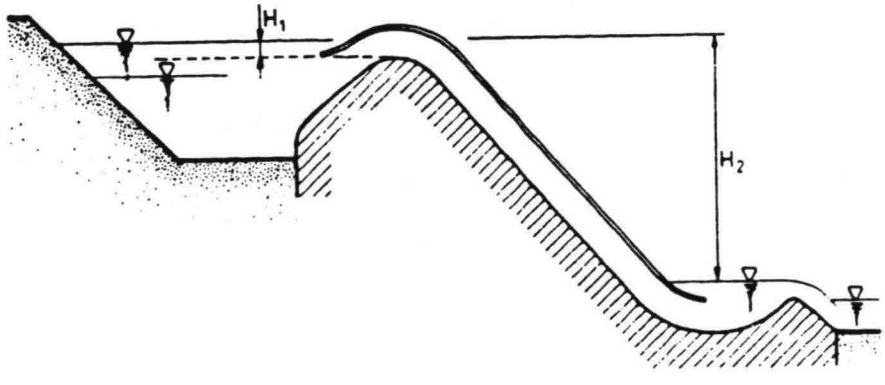


Fig. 5.6. Siphon spillway

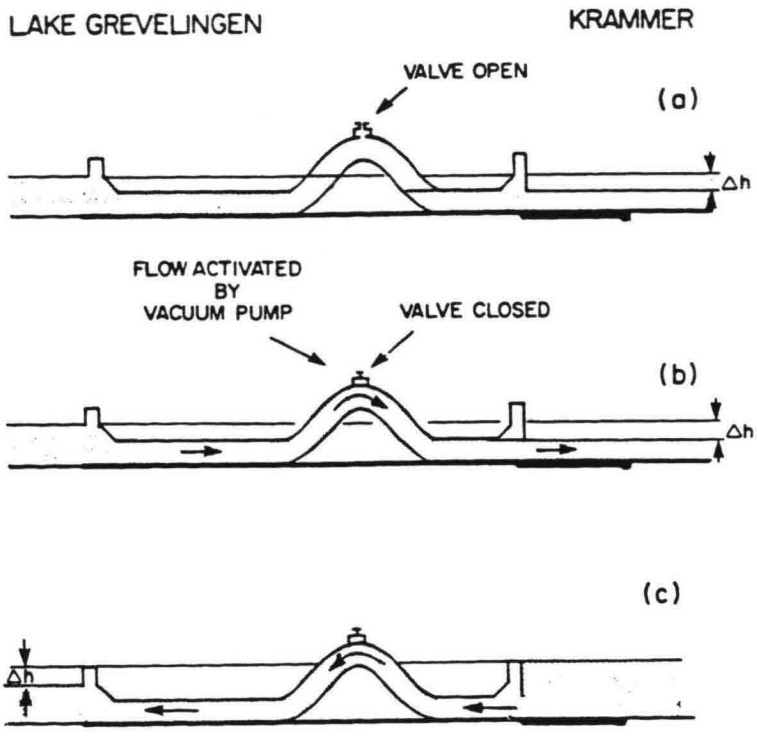
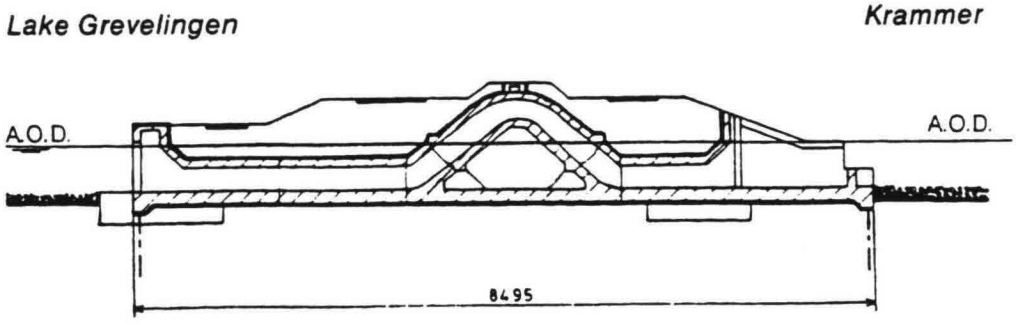


Fig. 5.7.
Siphon sluice
Lake Grevelingen, The Netherlands.
A cost-effective siphon sluice is part of the flushing system for Lake Grevelingen.
a) In an open-valve mode atmosopheric pressure prevents flow through the siphon.
b) In a closed-valve mode a vacuum pump may be used to lower atmospheric pressure and activate flow through the siphon.
c) Should it be decided to make Lake Grevelingen a freshwater lake, the sluice may be extended through Philips Dam and flow through siphon reversed (1987).



Longitudinal section of discharge sluice (syphon).

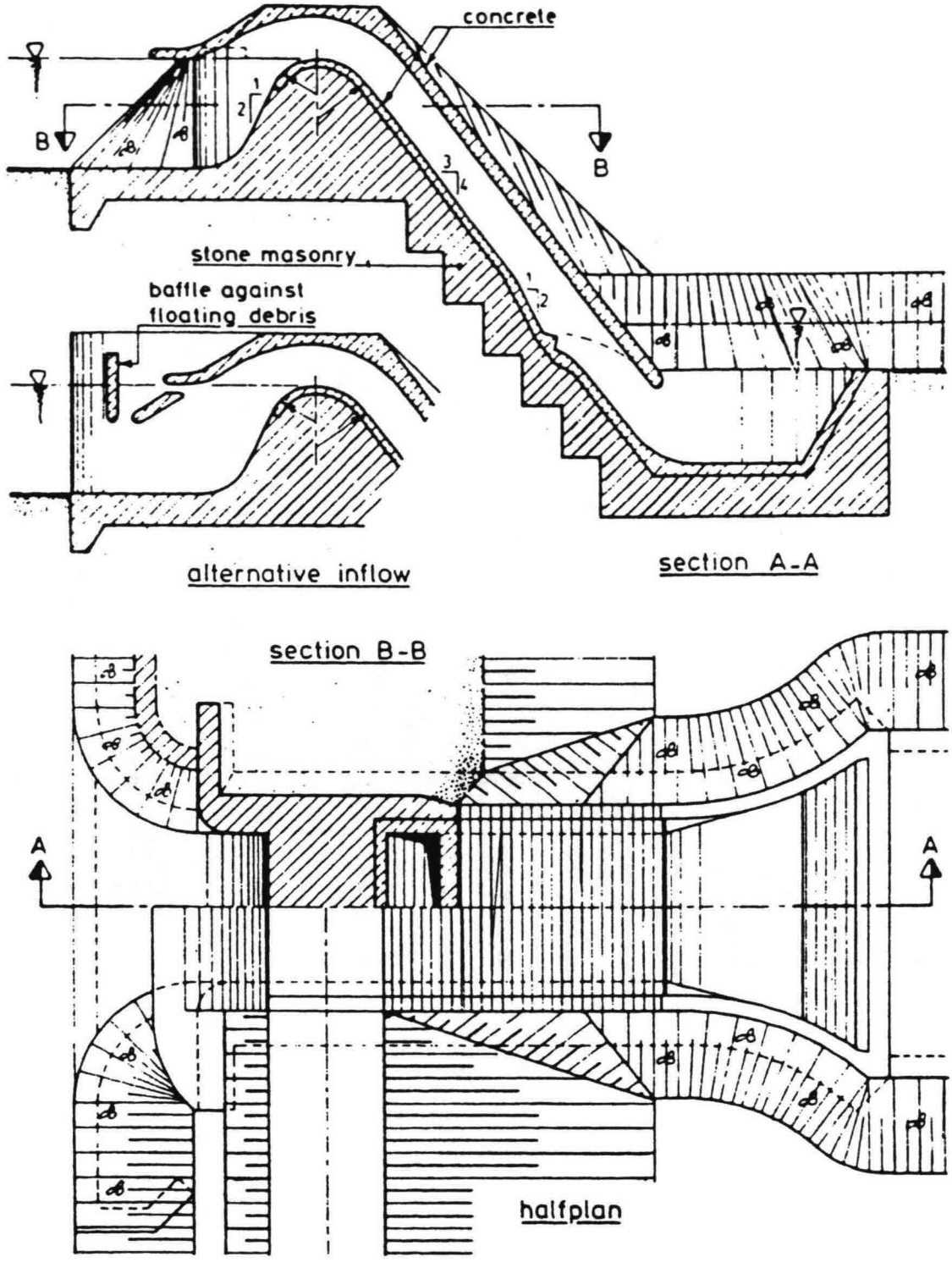


Fig. 5.8. Siphon in stone masonry combined with concrete.

6. Discharge or drainage sluices

A sluice is a hydraulic structure, as a part of a fixed flood barrage, to separate or connect two water-bodies with in general different water levels. These water levels are separated by means of movable locking devices, which can be opened or closed when necessary.

Discharge sluices are generally designed for a controlled discharge and supply of water. In Holland you will find this type of sluices in the sea- and river-dikes for the discharge of redundant rainfall-water.

At seaside the influence of the tide is significant: at low tide the sluice will be open to discharge redundant water, at high tide the sluice will be closed. An interesting possibility for a sluice is to remove sedimentation in front of or behind the sluices. This property is used e.g. to keep a discharge-canal at the desired depth; therefore a big difference in water level (=fall) is necessary in order to received enough velocity through the sluice (flushing). In Fig. 6.1. an existing example in Holland with the aid of a **sluice-basin** is presented. In periods of high river-discharge this basin is filled with water, and in a period of low discharge the sluice will be opened in order to remove the sedimentation in the discharge-canal.

In Holland there are a very special type of sluices : **inundation-sluices**, which are meant to inundate certain areas in time of war.

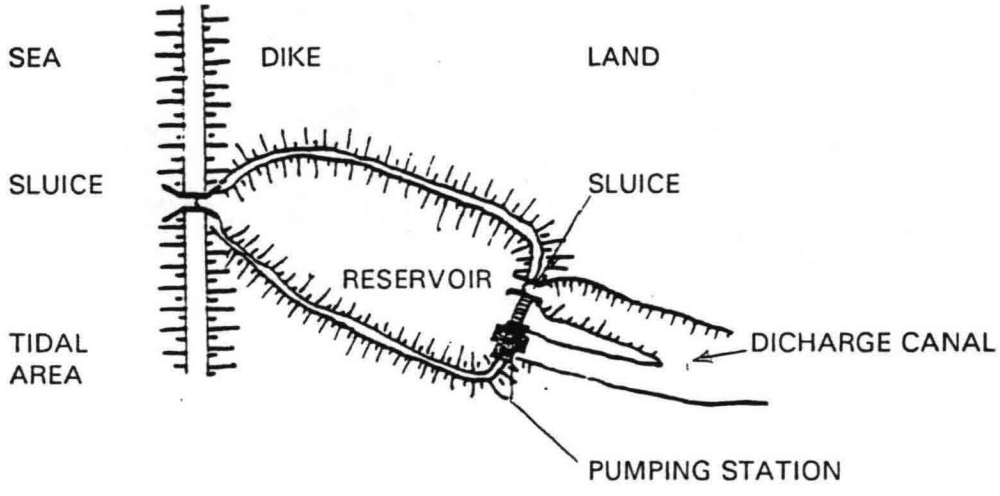
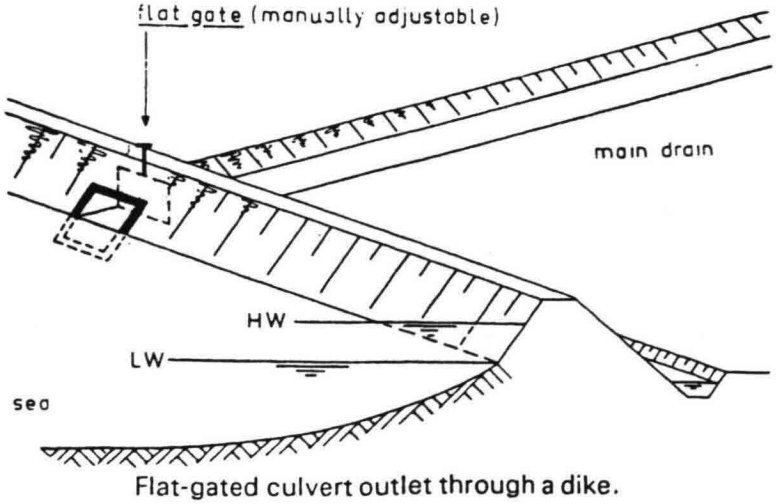


Fig. 6.1. Dicharge or drainage sluice.



Flat-gated culvert outlet through a dike.

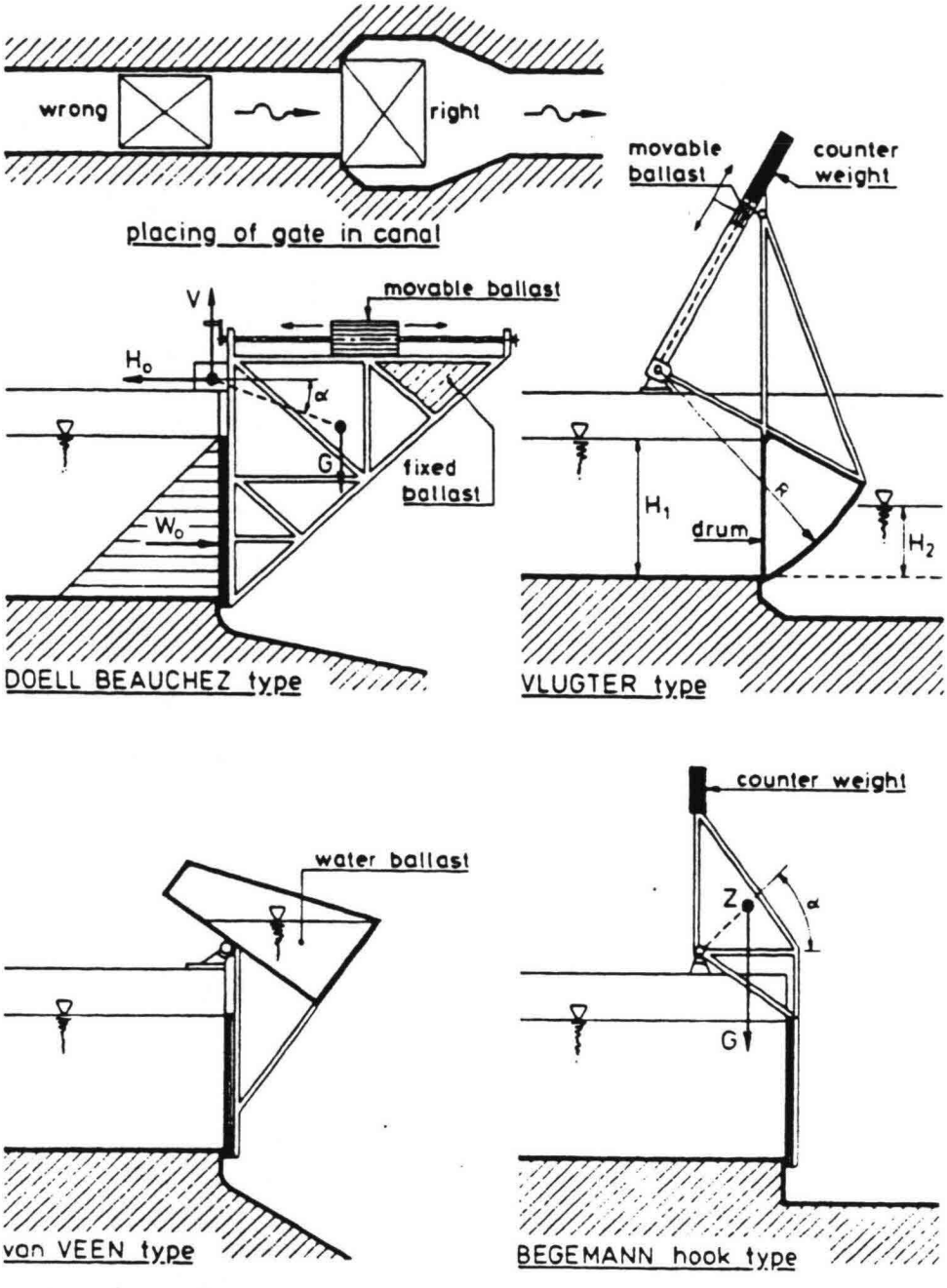


Fig. 6.2. Automatic gates used in Indonesia.

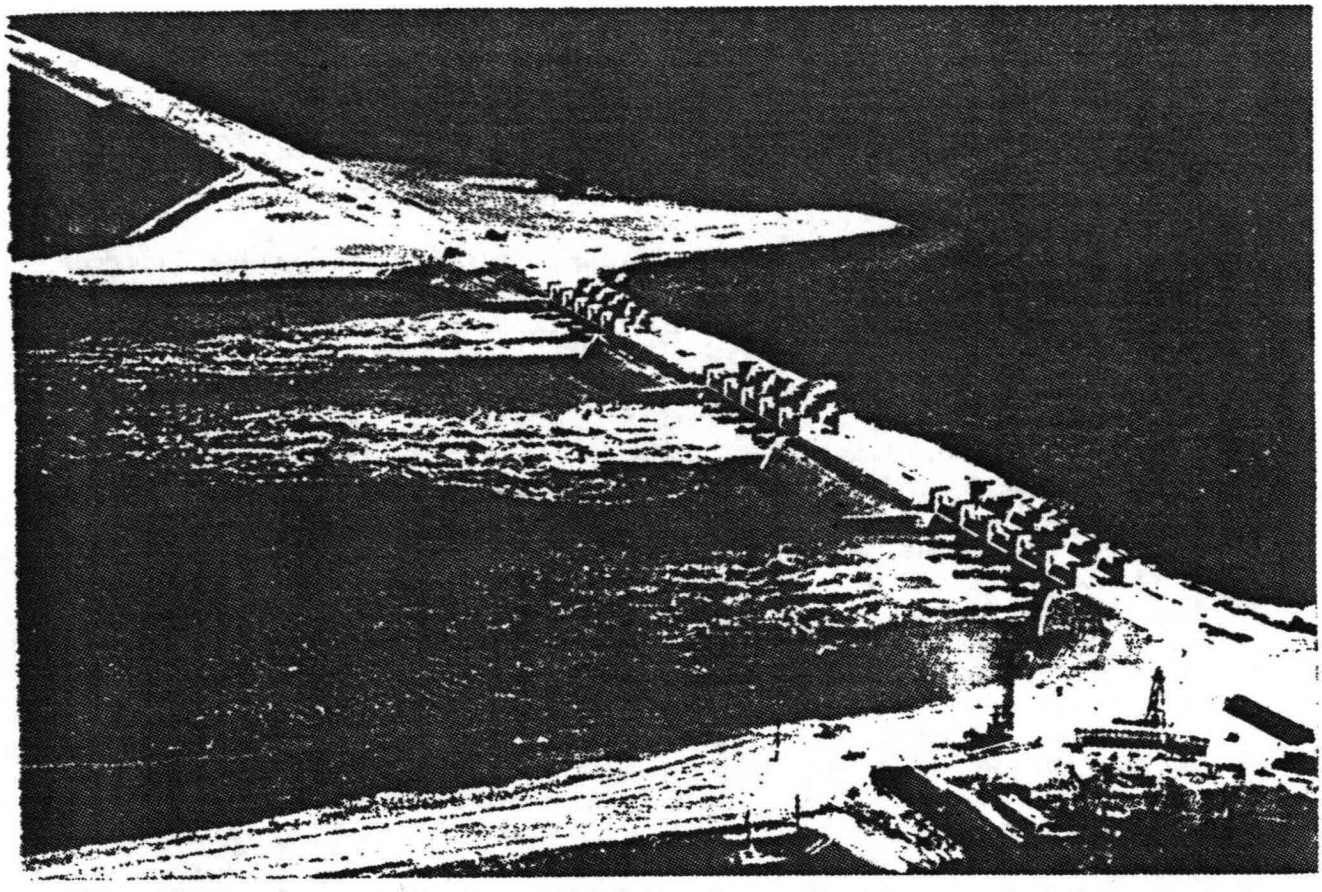


Fig. 6.3. Drainage sluices in the Zuiderzee dam. One of the two sets of sluices in the enclosure dam by which the Rhine water is passed into the North Sea; 25 sluices and 2 large locks were constructed in the open sea before the dam itself was made.

The "Lauwerszee"

The "Lauwerszee" is situated on the northern coast of The Netherlands between the provinces of 'Friesland' and 'Groningen'. It is surrounded by land on three sides and only connected to the 'Wadden Zee' on the north side. In 1611 the people of Groningen were already discussing the closure of this sea with the people living in Friesland.

The flood disaster in 1953 proved clearly that the seadikes in the two northern provinces of The Netherlands were too low. As an alternative to raising the 32 kilometer long seadikes around the Lauwerszee, the construction of a 13 kilometer long enclosure dam was studied. Other advantages of this plan were:

1. Improved drainage of the surrounding area
2. Reclamation of new agriculture land
3. Creation of a large 'nature area' with an inner lake
4. New recreation (beaches, marinas) possibilities.

In 1955 the department "Lauwerszee" - works was established and in 1960 the decision to opt for the closure dam was taken. A 'working' harbour in the 'Bootsgat' near Oostmahorn was made in 1961 and actual construction could start.
Bucket dredgers, suction dredgers, floating cranes with tugs, sandbarges and a fleet of inland ships for the transport of dike protection material were mobilized.

For the closure of the final gap 25 caissons were used. On May 23, 1969 the last opening of 70 meters wide between the caissons was closed with the last two caissons.
A new harbour was constructed in Lauwersoog for the fishermen (of shrimps) from the port of Zoutkamp, which became land locked by this enclosure dam.

In the enclosure dam a discharge sluice with a capacity of 1300 m^3/sec. is constructed to control the water level on the 'Lauwerszee'. The depth of the sill is at - 5 m below N.A.P.; the total width of the discharge is 100 m divided over 12 square tunnels of 8.5 m width.
A navigation lock for fishing boats is also provided for.

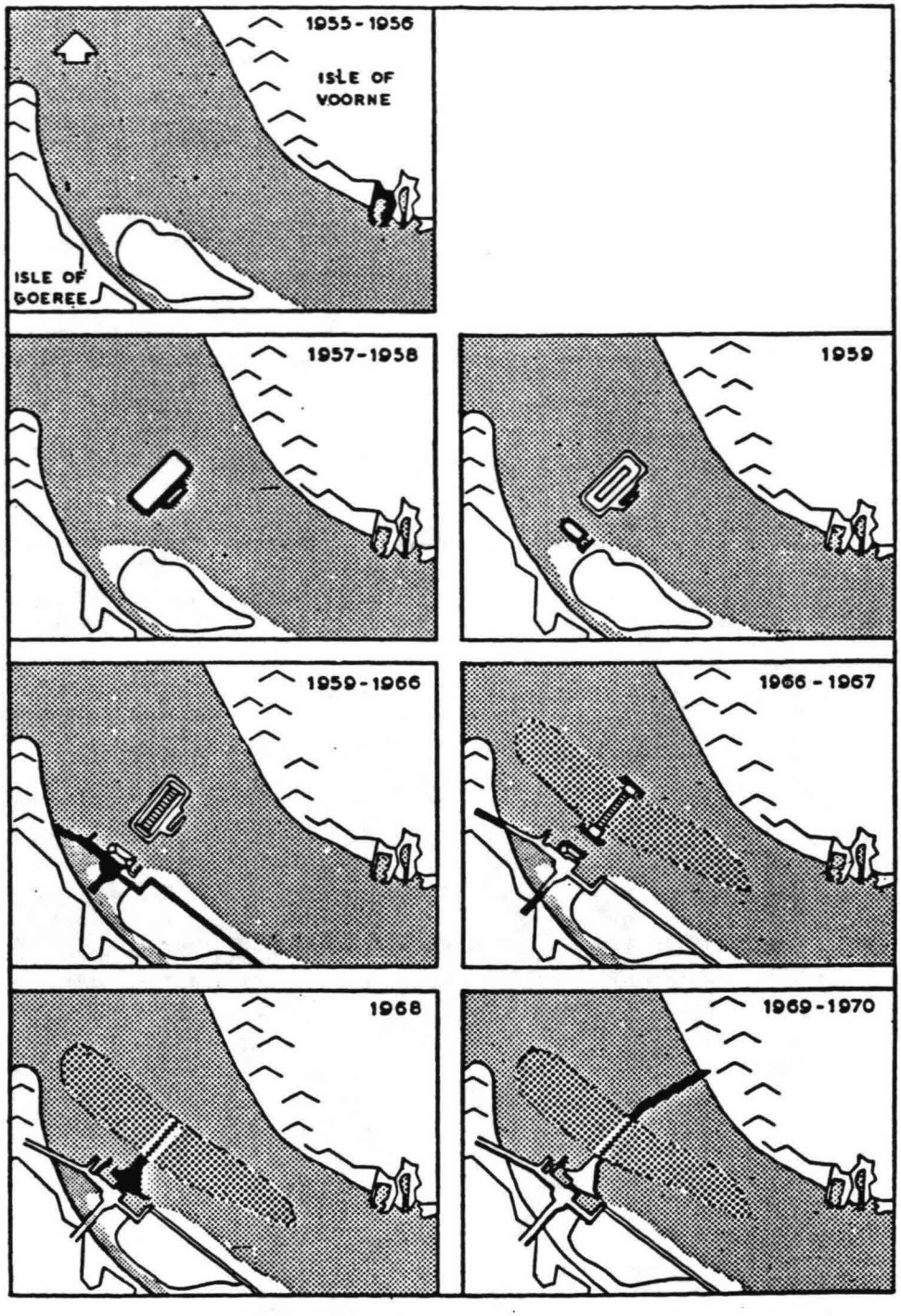


Fig. 6.5. Damming of the Haringvliet Estuary : sequence of works

1955 - 56 Construction working harbour near Hellevoetsluis

1957 - 58 Creation of artificial island by building a rectangular cofferdam (1400 x 600 m) to height of a 8 metres above sea level; pumping the building dry by, the end of 1958 the water level inside the cofferdam was low enough to allow construction of the sluice complex to begin.

1959 Building pit for shiplock construction.

1959 - 66 Construction of sluice complex.

1966 - 67 Making protective revetments by dumping specially imported huge blocks of stone each weighing 3,500 kilogrammes or more. When the complex was completed the cofferdam was dredged away and flooded; the completed sluice structure was left standing in the middle of the inlet.

1968 Construction of the earth dam joining the structure to the southern end.

1969 - 70 Closing the 1165 m wide channel at the northern end by way of cable way, followed by construction of the earthen dam.

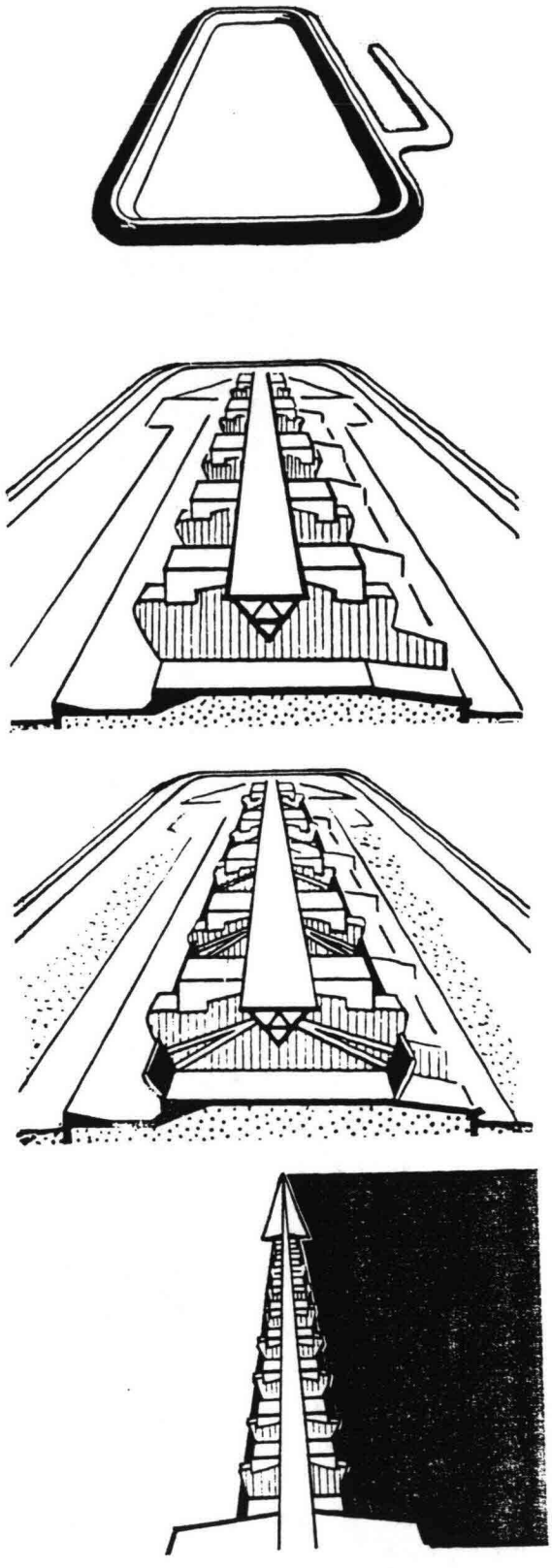


Fig. 6.6. Construction of the outlet sluices in the Haringvliet.

In the Haringvliet estuary an artificial island was made of sand from the surrounding bottom. The dikes of the island are protected against wave attack. Inside, the building pit was dredged out. At the wind protected side a work harbour was made to handle the necessary material transported by ship. A diesel-electric power station was built on this working area to provide power for the scheme and to drive the machinery used to pump the site dry for the duration of the project. (Capacity pumps : 3000 - 4000 cu. m. water per hour).

22,000 concrete piles varying in length from 6.3 to 24 m were driven into the dried up sea bed. On top of these a 3-metre layer of concrete was laid to form the floor of the complex 5.5 m below Amsterdam Ordnance Datum (A.O.D.). A series of 16 concrete piers (some of them containing fish passes) were then built on this foundation, 60 m apart. At either end of this framework for the sluice gates 300 m wide abutments were constructed. The backbone of the complex is formed by a series of specially built triangular beams ('nabla' girders) of prestressed concrete resting on top of the piers. The flat top of these triangular girders serves as the base of the road across the sluices.

The 'nabla' girders also form the supporting structure for the 34 (one on each side of the 17 sluice openings) steel gates which are attached by four steel arms to pivots set in the sloping underside of the girders. The gates, which vary from 56 to 58.4 metres in width, can be raised to 3.5 metres above A.O.D. On the seaward side and 5.5 metres on the landward side; a gate with its supporting arms weighs 550 tonnes. Heavy rubber strips are fitted to the edges of the gates to seal any gaps where they meet the piles and the cement floor of the complex.

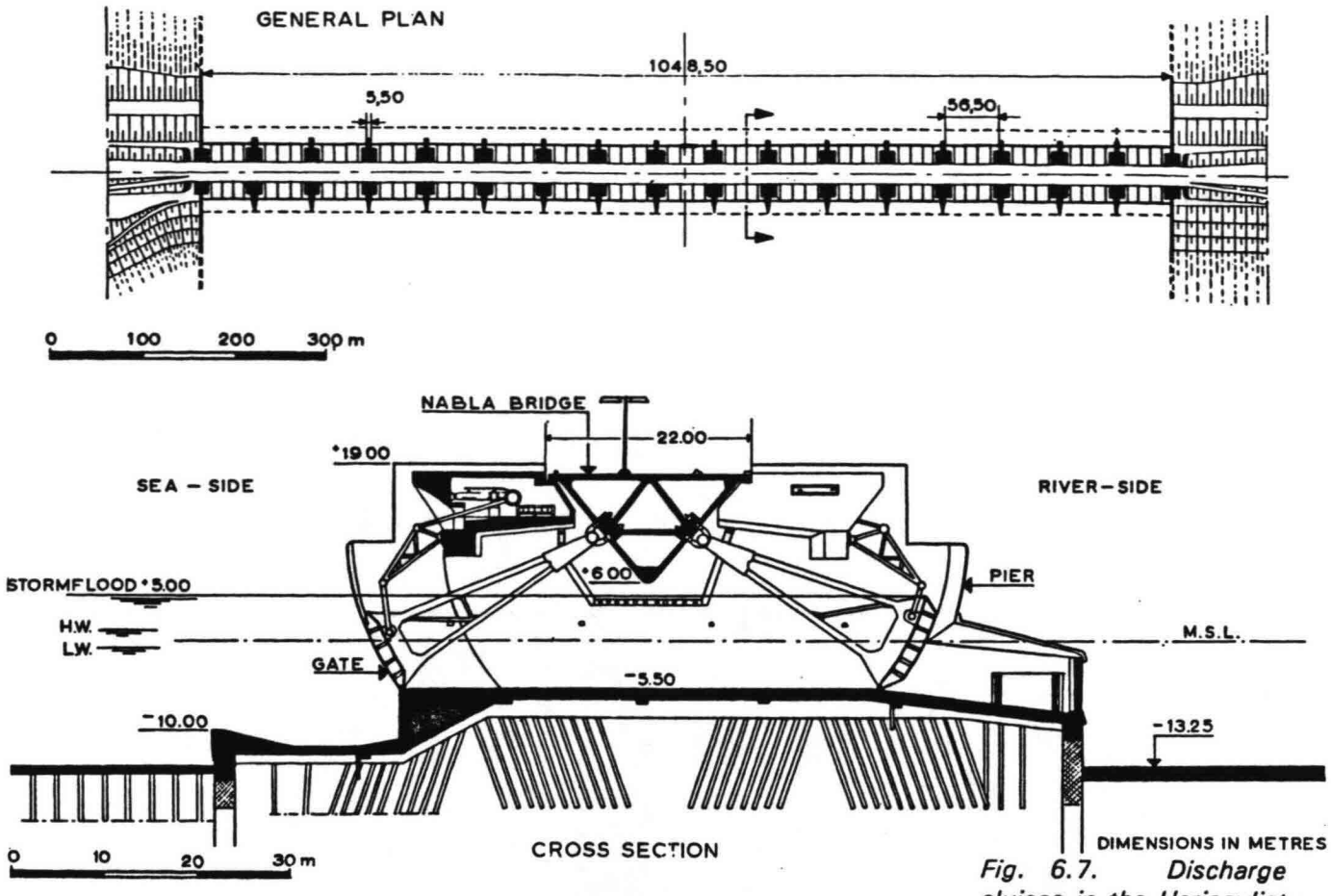
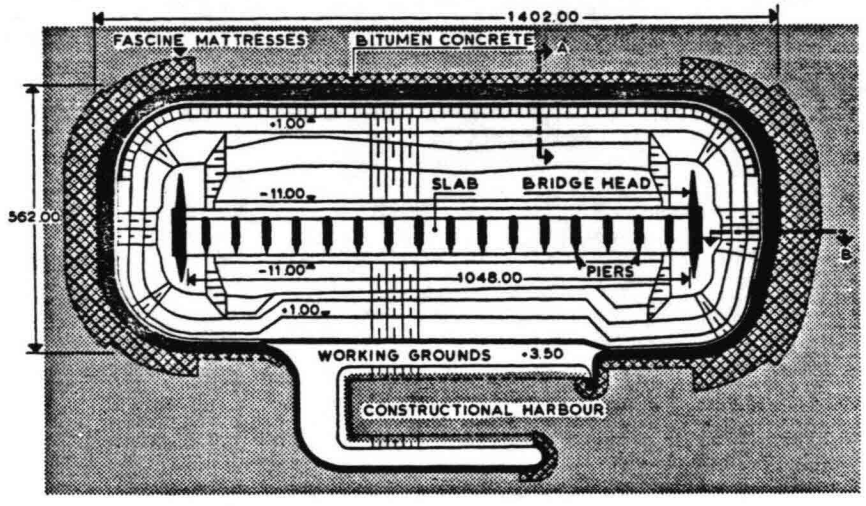
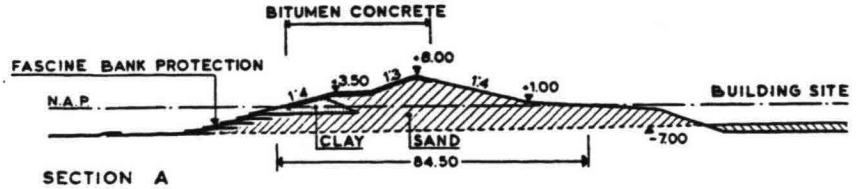
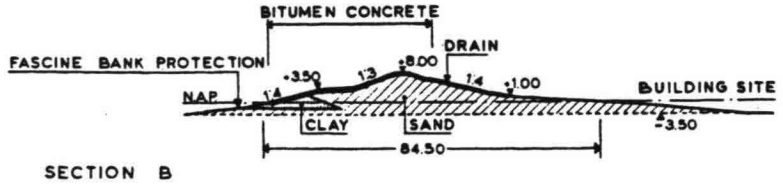


Fig. 6.7. Discharge sluices in the Haringvliet estuary.







Outside of the sluice the bottom at the seaside is protected with a concrete slab on piles over 6.5 m, followed by a filter bed. At the riverside there is a thick concrete slab on shallow foundation over 33 m followed by a filter bed. The outer gate can be seen as a breakwater and the inner gate to take the water difference during a storm flood. To reduce the forces from wave attack, the outer gate is made as low as 3ossible for summer water level. By means of 4 support arms with hinges the gates are every 15 m supported to the prestressed Nabla-beam. The 16 concrete piers contain the hydraulic machinery (cylinders) which operates the steel gates.

7. Irrigation structures

Irrigation-sluices convey river water, mostly raised up by a barrage or weir - a so-called **diversion structure** - to the irrigation-area. Fig. 7.1. gives a schematic view of an irrigation work. In the river weir A has been built with a fixed crest in order to raise the water level. Just in front of the weir the inlet-sluice B (**intake**) is situated. In the weir body just downstream of the intake a **scouring sluice** E is constructed to prevent coarse sediment to enter the canal system.

The Canal between B and C acts as a **sediment trap**. It causes sediment fractions larger than the fine sand fraction (0.06-0.07 mm) to settle. Finer material cannot be trapped in a normal sediment trap and must be transported through the canal system to the field. The material that has settled in the trap is periodically removed. This is done by **hydraulic flushing** (fast-flowing water) by opening the gates of the **flushing sluice F** or by **dredging**.

Through the intake gates C the water reaches the primary canal (**primary canal intake**). The flushing sluice (F) is often combined with the intake from the sediment trap to the primary canal (C) in one structure.

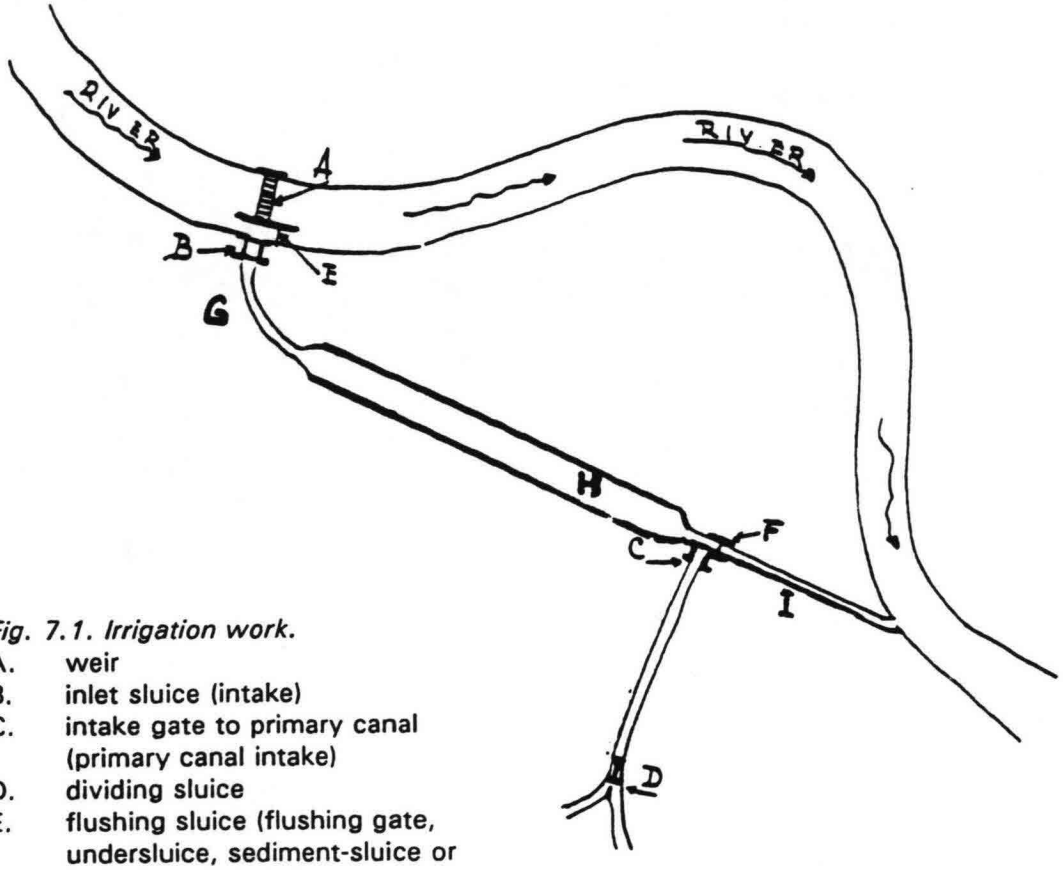


Fig. 7.1. Irrigation work.

A. weir
B. inlet sluice (intake)
C. intake gate to primary canal (primary canal intake)
D. dividing sluice
E. flushing sluice (flushing gate, undersluice, sediment-sluice or sediment excluder)
F. flushing sluice
G. feeder canal with guide walls
H. sediment or sand trap, settling basin
I. flushing canal.

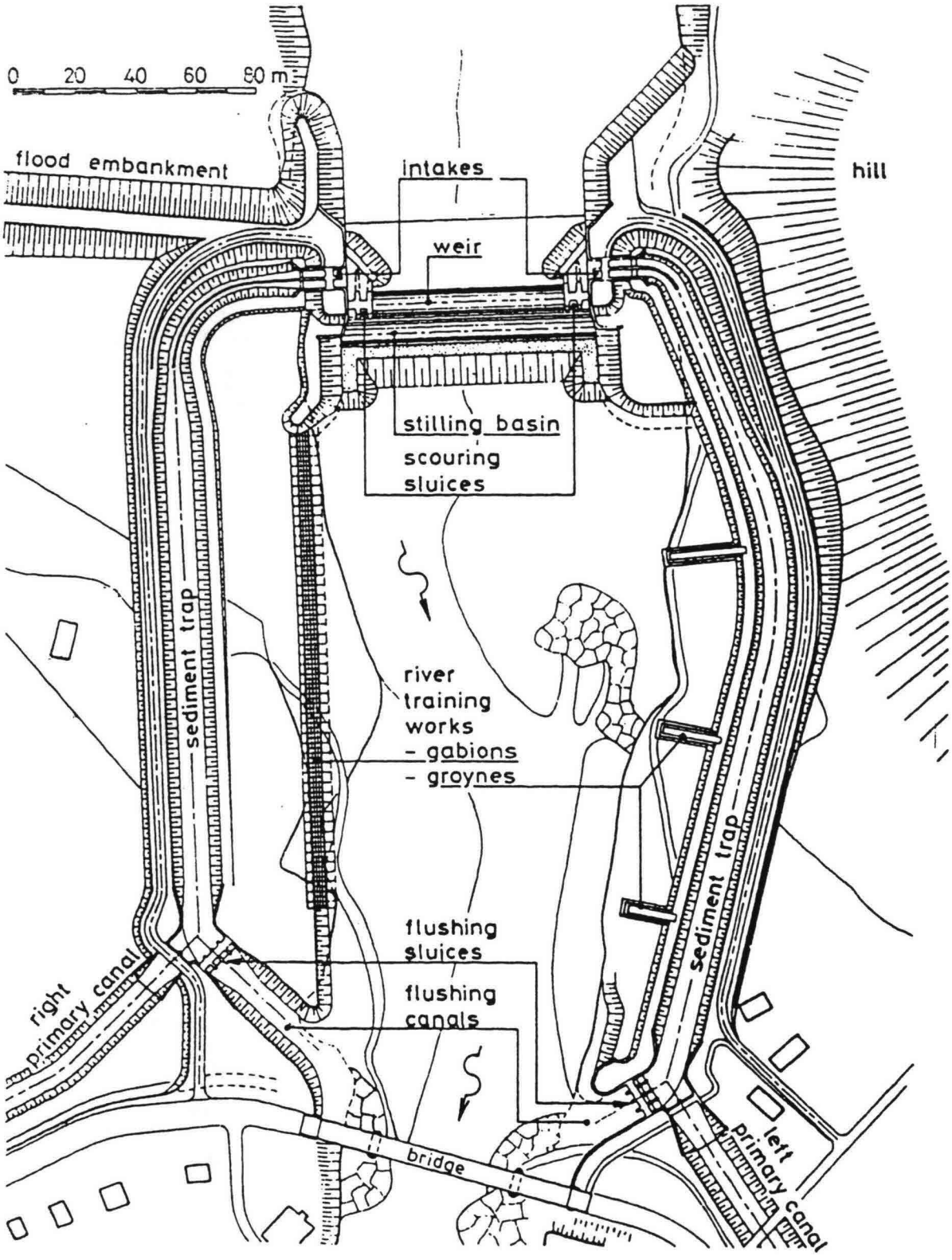


Fig. 7.2. Layout of headworks with diversion structure (weir, barrages), intake, scouring, sluice, sediment trap with flushing sluice and river regulation works. Intakes on both side of the weir is not typical, usually only one intake at the outer bend.

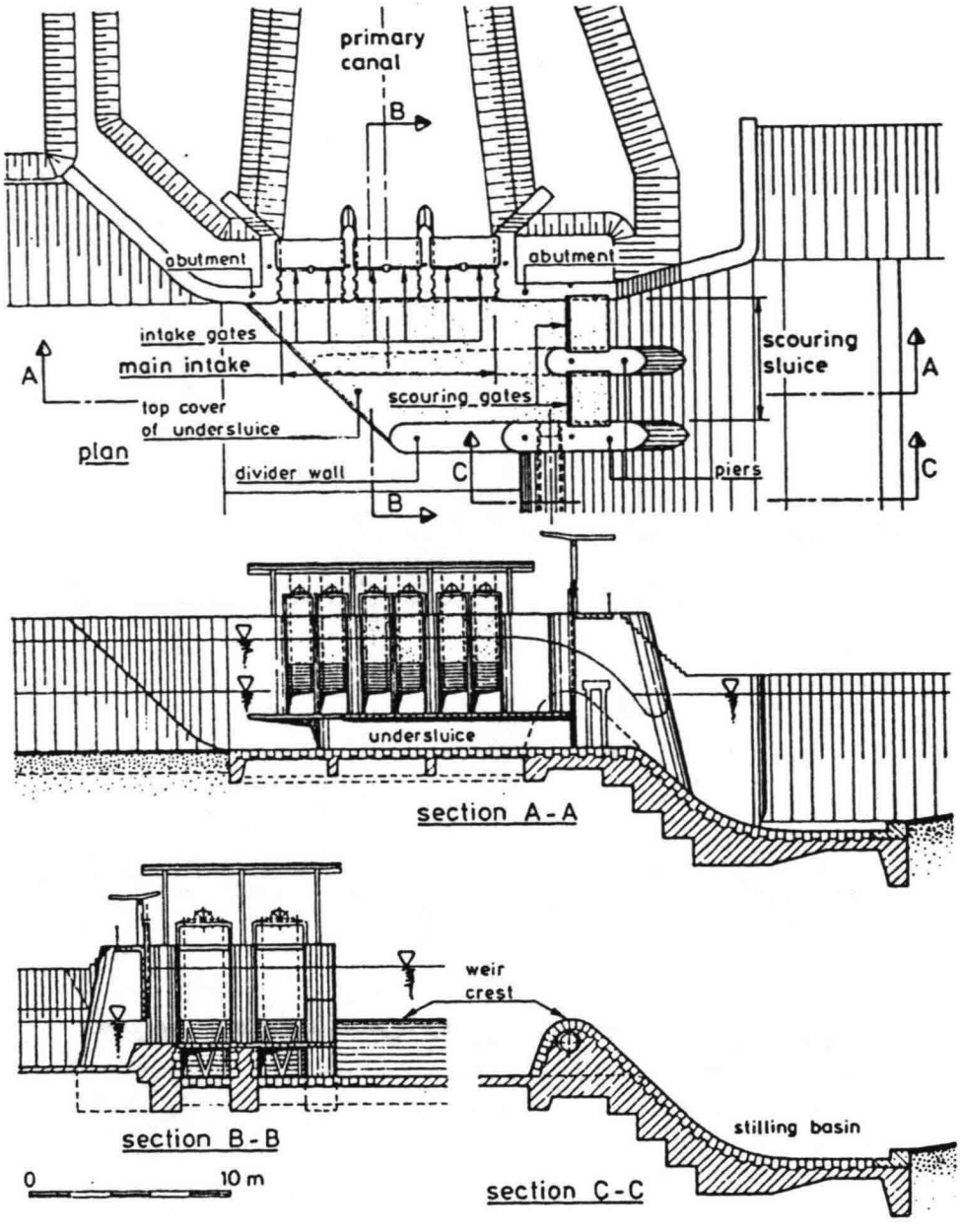


Fig. 7.3. Intake and flushing sluice.

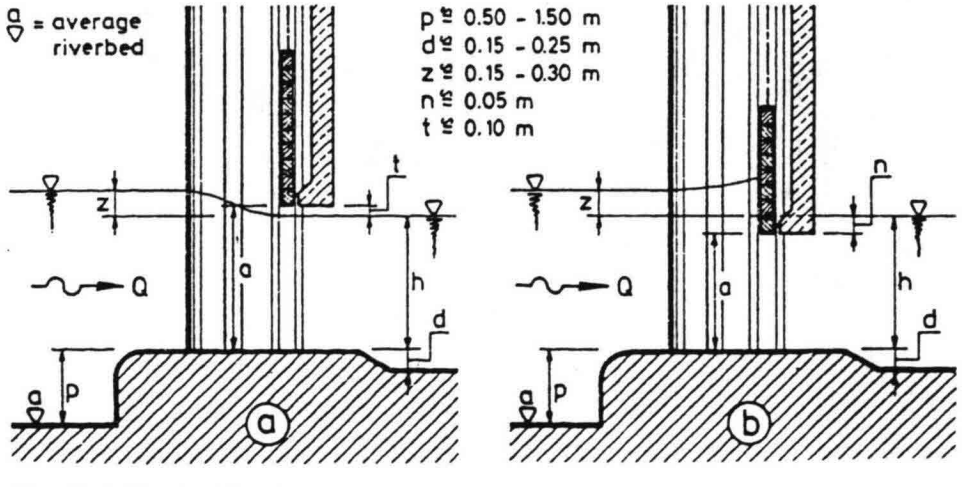


Fig. 7.4. Typical intake gate.

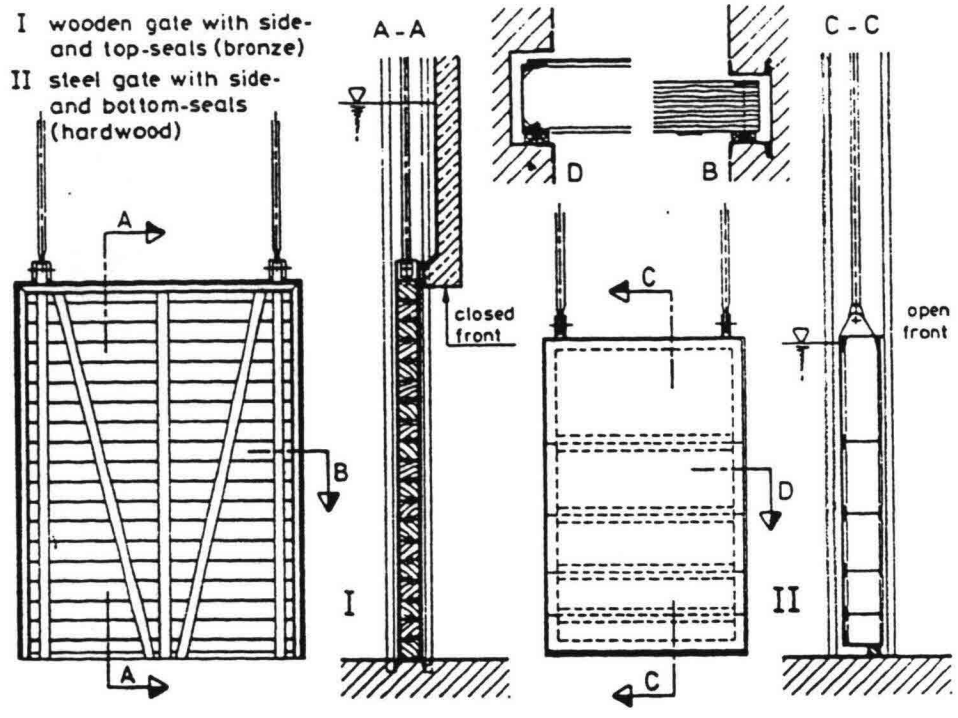


Fig. 7.5. Intake gate : wooden and steel sliding gates. The intake gates is normally a simple wooden sliding gate (for limited width) otherwise steel. If the water depth in front of the gate is considerable, operation of a sliding gate may become difficult. A segment gate or radial gate may then be advantageous.

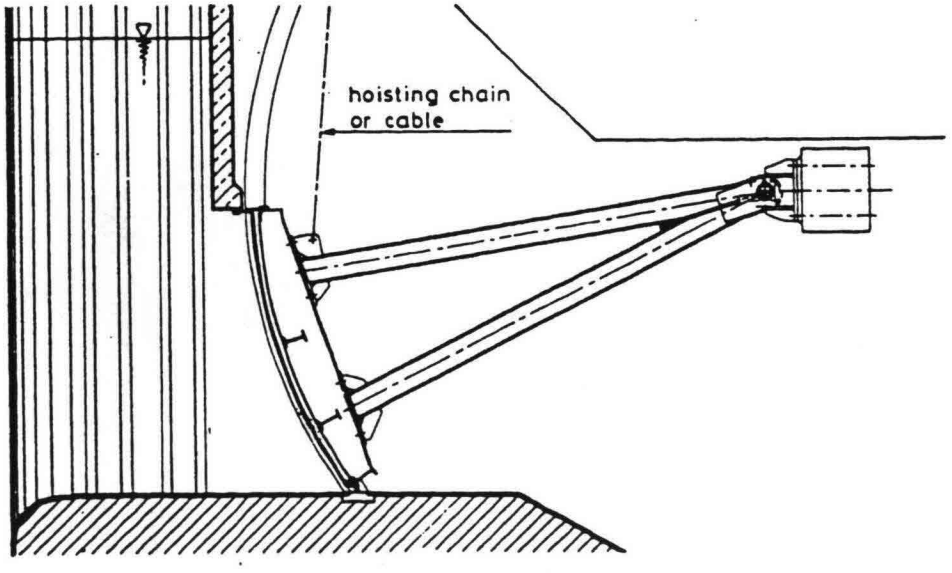
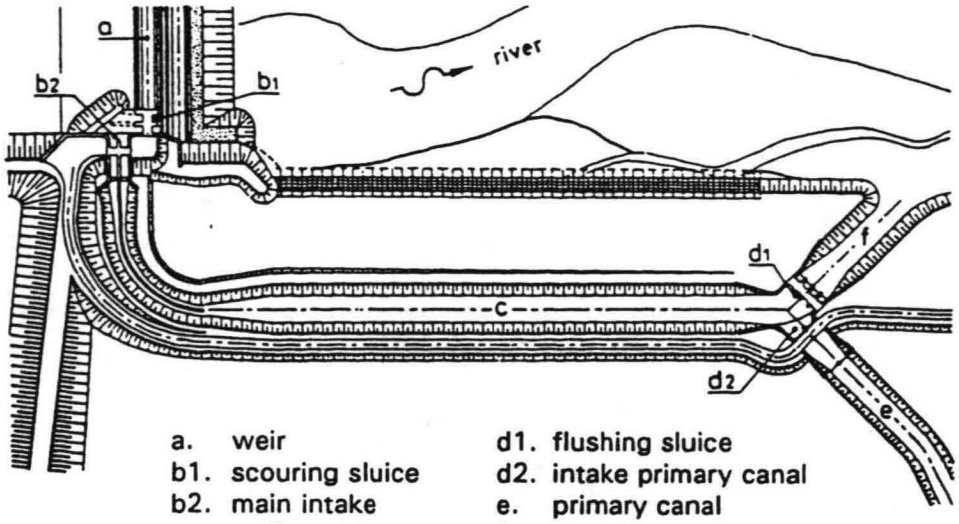
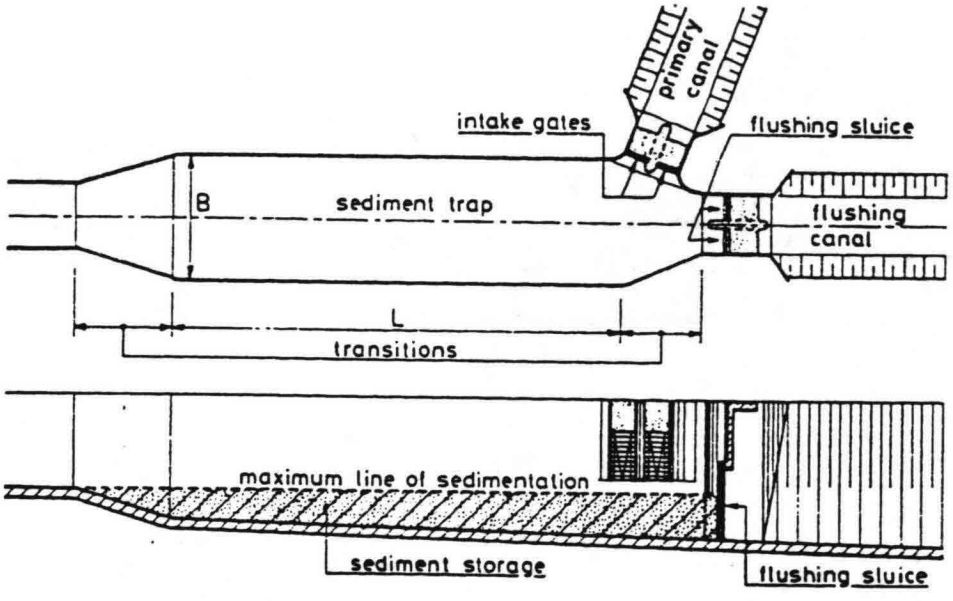
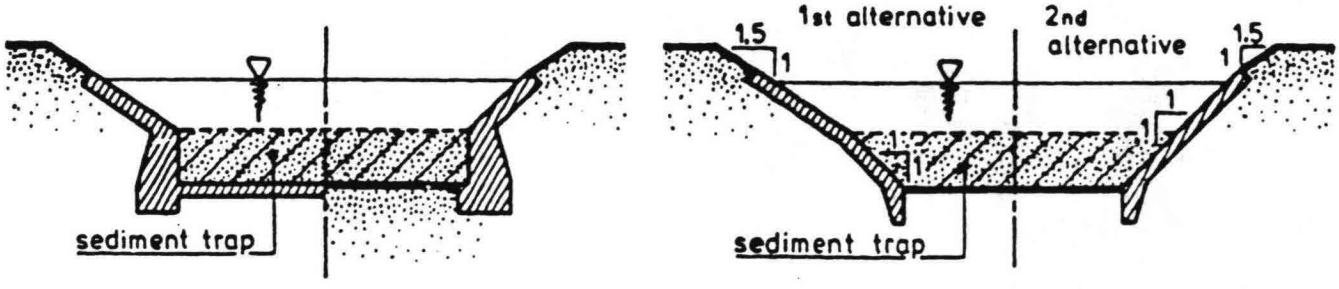


Fig. 7.6. Radial gate.



a. weir
b1. scouring sluice
b2. main intake
c. sediment trap
d1. flushing sluice
d2. intake primary canal
e. primary canal
f. flushing canal





a. sediment trap with vertical walls, with and without bed protection

b. side slopes can be steeper because of lining

Fig. 7.7. Sediment trap with feeder canal, guide wall, and flushing canal.

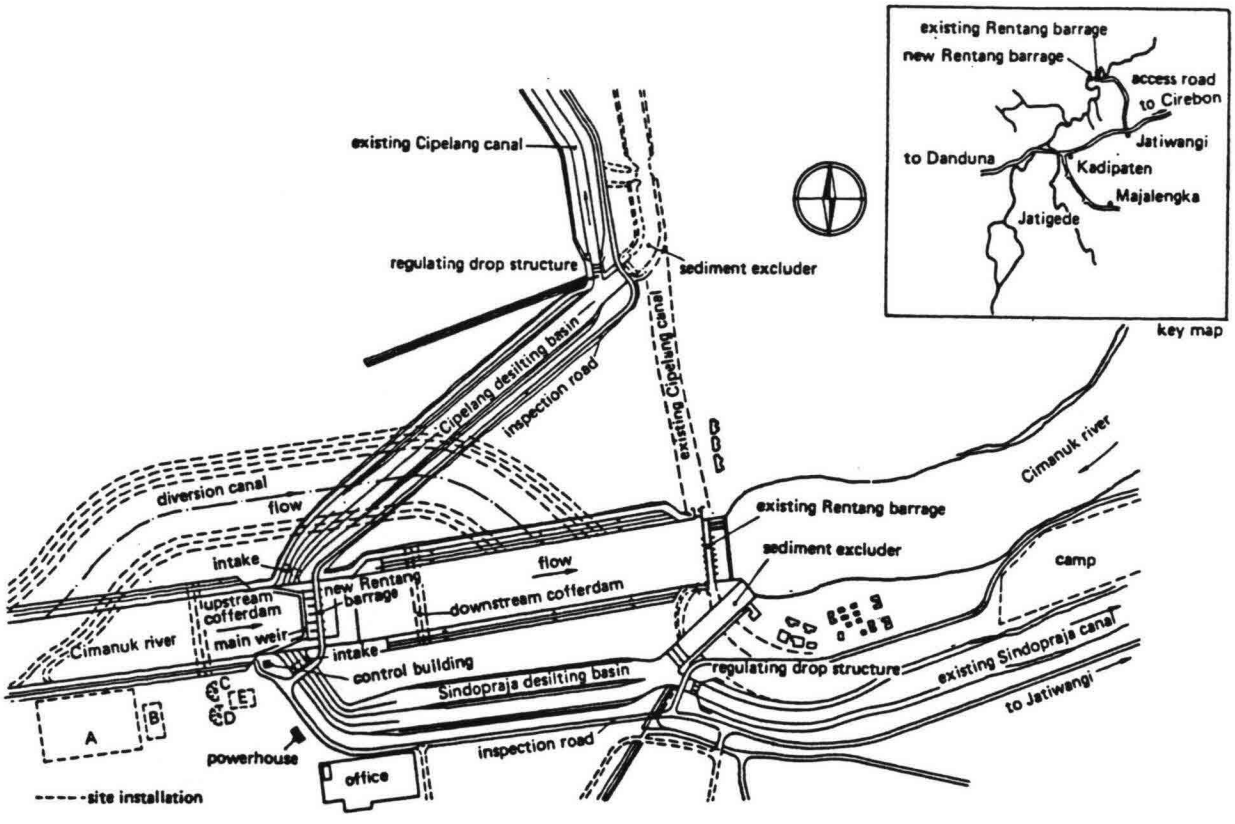


Fig. 7.8. Layout of the Rentang barrage.

In West Java province the muddy, slow-moving Cimanuk river irrigates a wide area of farmland, based partly on a barrage at Rentang originally built in 1916, which impounds the river waters, diverting the flow into two canals. On the right bank of the barrage is the Sindopraja canal, which serves 56,000 ha. of farmland on the left is the Cipelang canal, which serves 36,000 ha. The barrage has a weir crest of 14.7 m, and there are six bays, each 8.9 m. wide, with hand-operated stoplogs.

In 1979 work began with a new barrage 580 m. upstream of the existing one and appurtenant works built and incorporated into the irrigation facilities. The new barrage consists of a reinforced concrete weir with radial gates. This structure is 51.5 m. wide, with a crestlength of 94.1 m. and divided into ten bays by nine piers. There are six spillway bays equipped with radial gates and hoists and four sluiceway bays - two each side of the spillway - each having an invert or undersluice, equipped with fixed-wheel gates and hoists. Maximum storage level: 23.5 m. The barrage acts as a diversion weir for the Sindopraja and Cipelong canals.

Both canal intakes are located immediately upstream to the right and the left of the barrage. The intake of the Sindopraja canal is of reinforced concrete and consists of four bays with a total width of 31.2 m. and equipped with radial gates. The water diverted into the intake flows through an upstream feeding canal and into the desilting basin where particles greater than 0.06 mm. are deposited. Water is supplied to the existing canal through a regulator drop structure and the sediment deposited in the desilting basin is flushed through the sediment excluder to the downstream channel of the Cimanuk river.

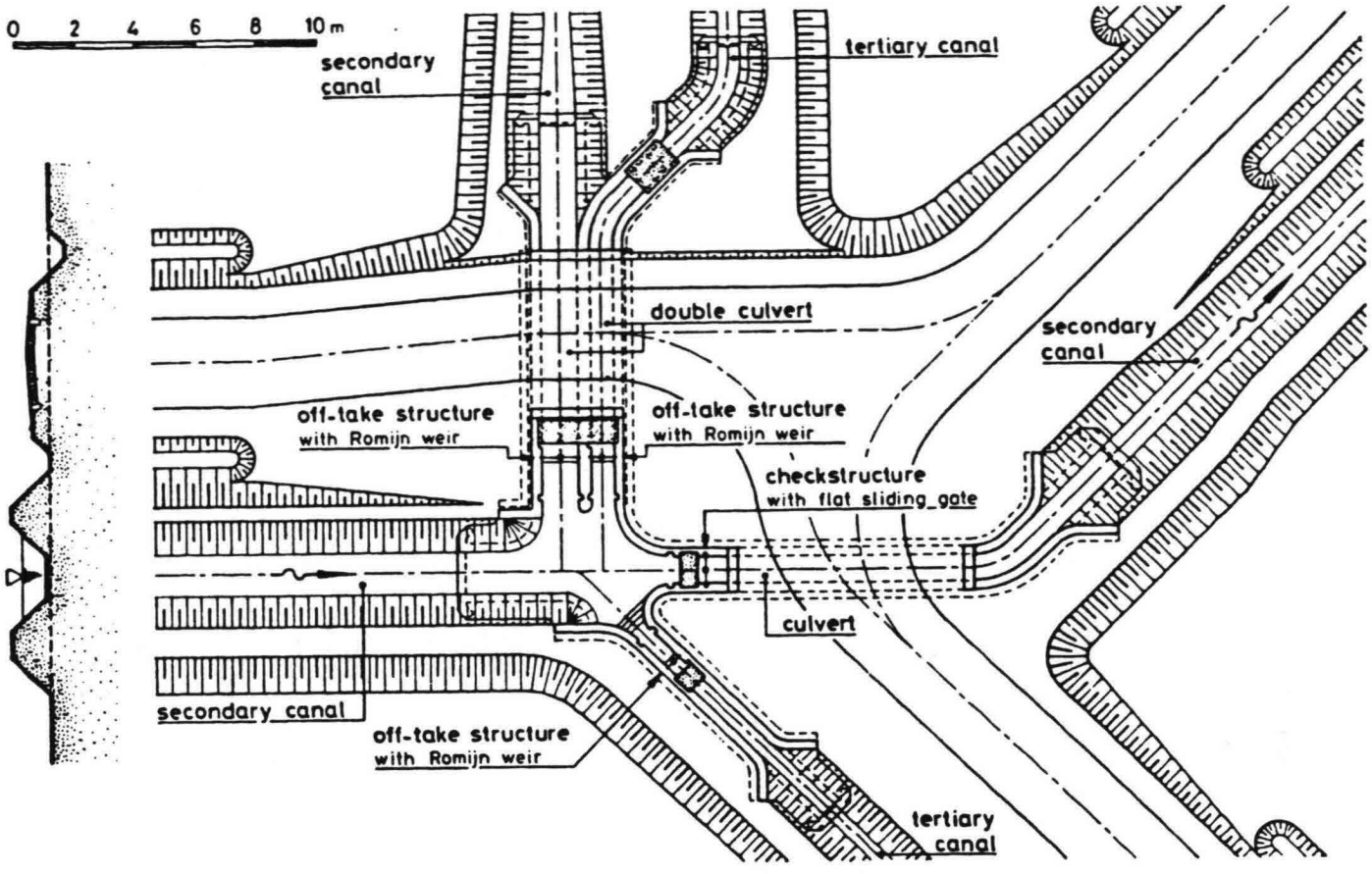


Fig. 7.9. Secondary canal with check structure and off-takes to different directions.

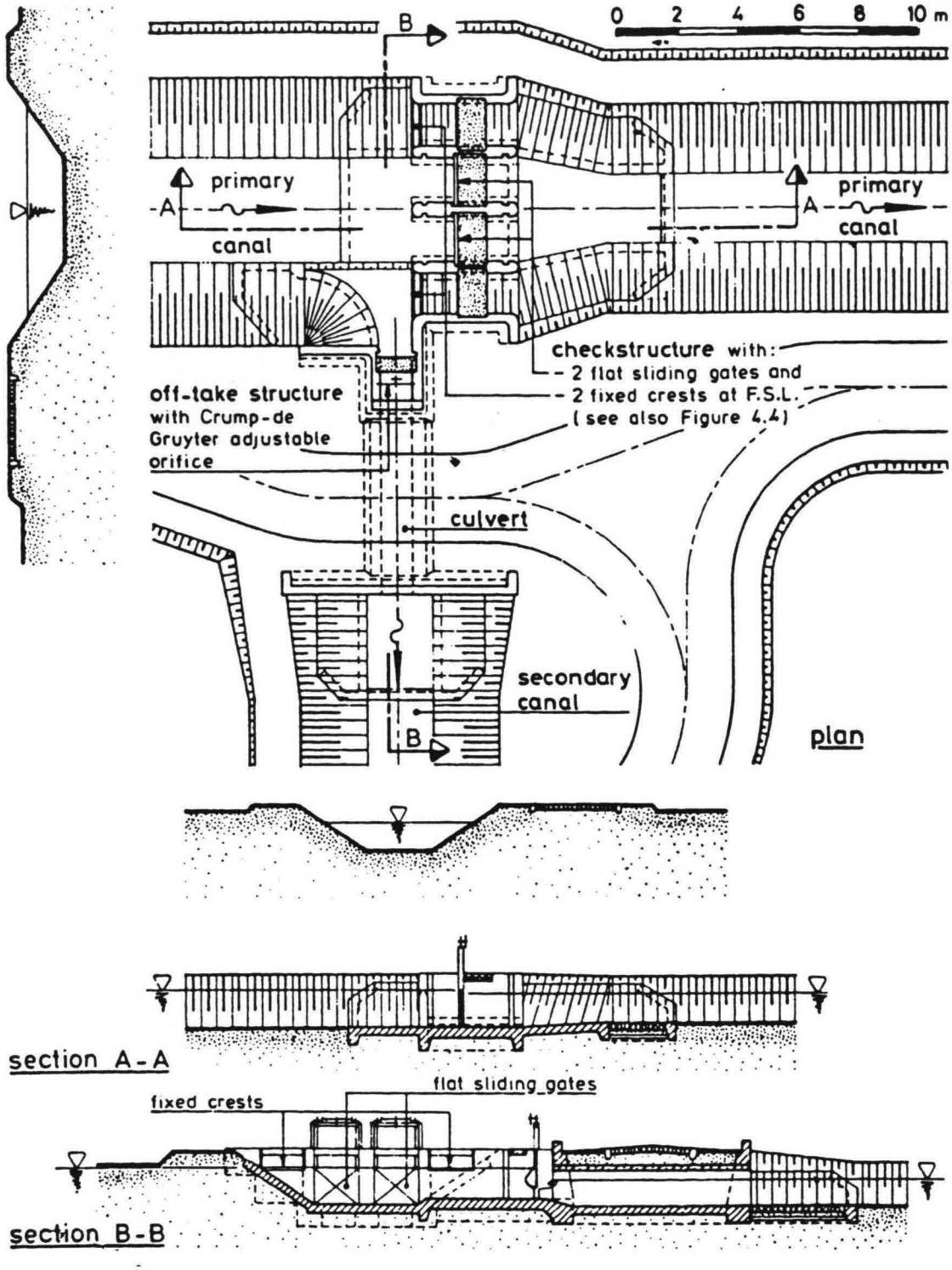


Fig. 7.10. Primary canal with check structure and off-take to secondary canal.

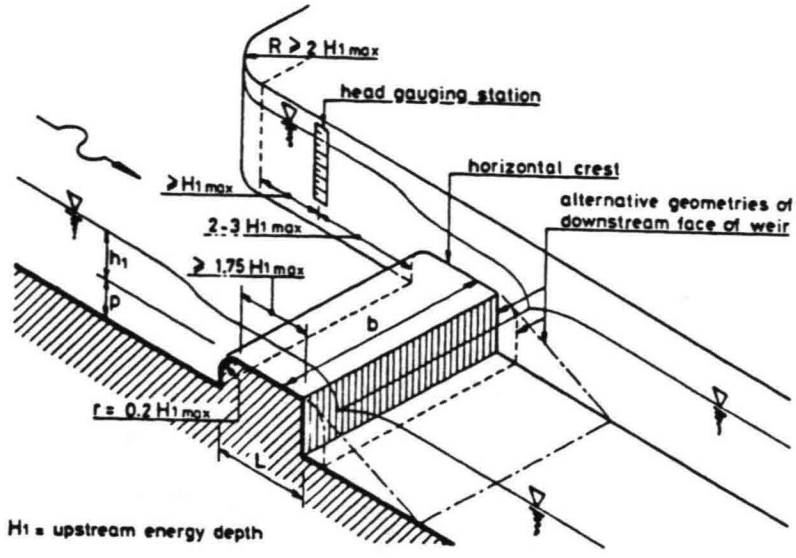


Fig. 7.11. Broad-crested weir with rounded entrance.

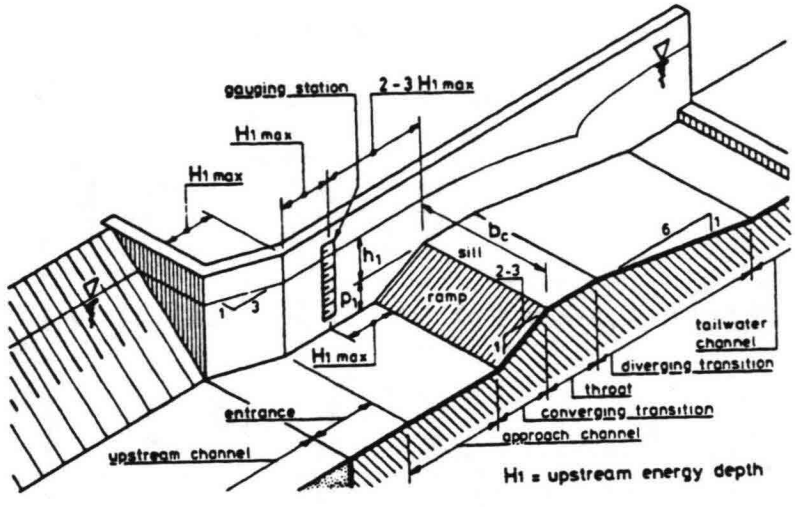


Fig. 7.12. Broad-crested weir with plane surfaced entrance and converging transition.

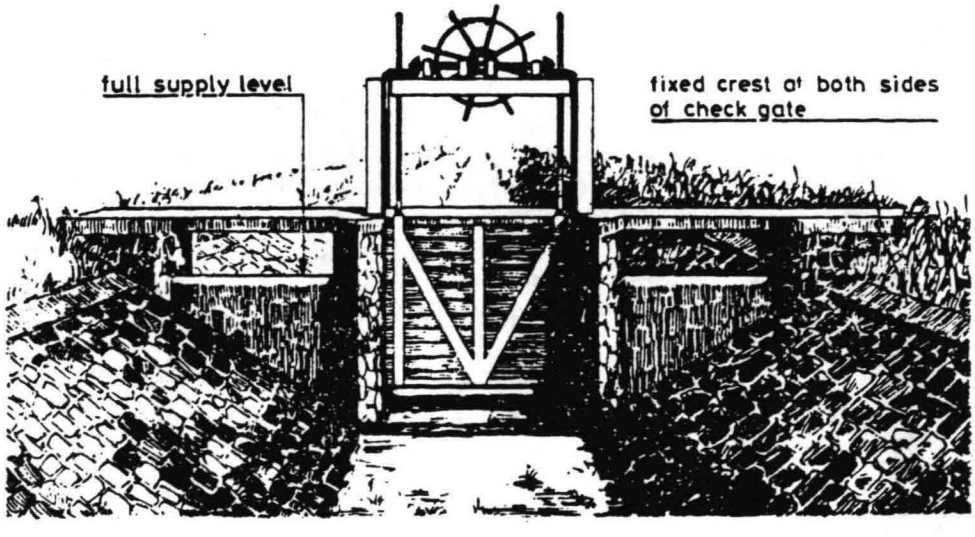


Fig. 7.13. Check structure : undershot gate with fixed crest.

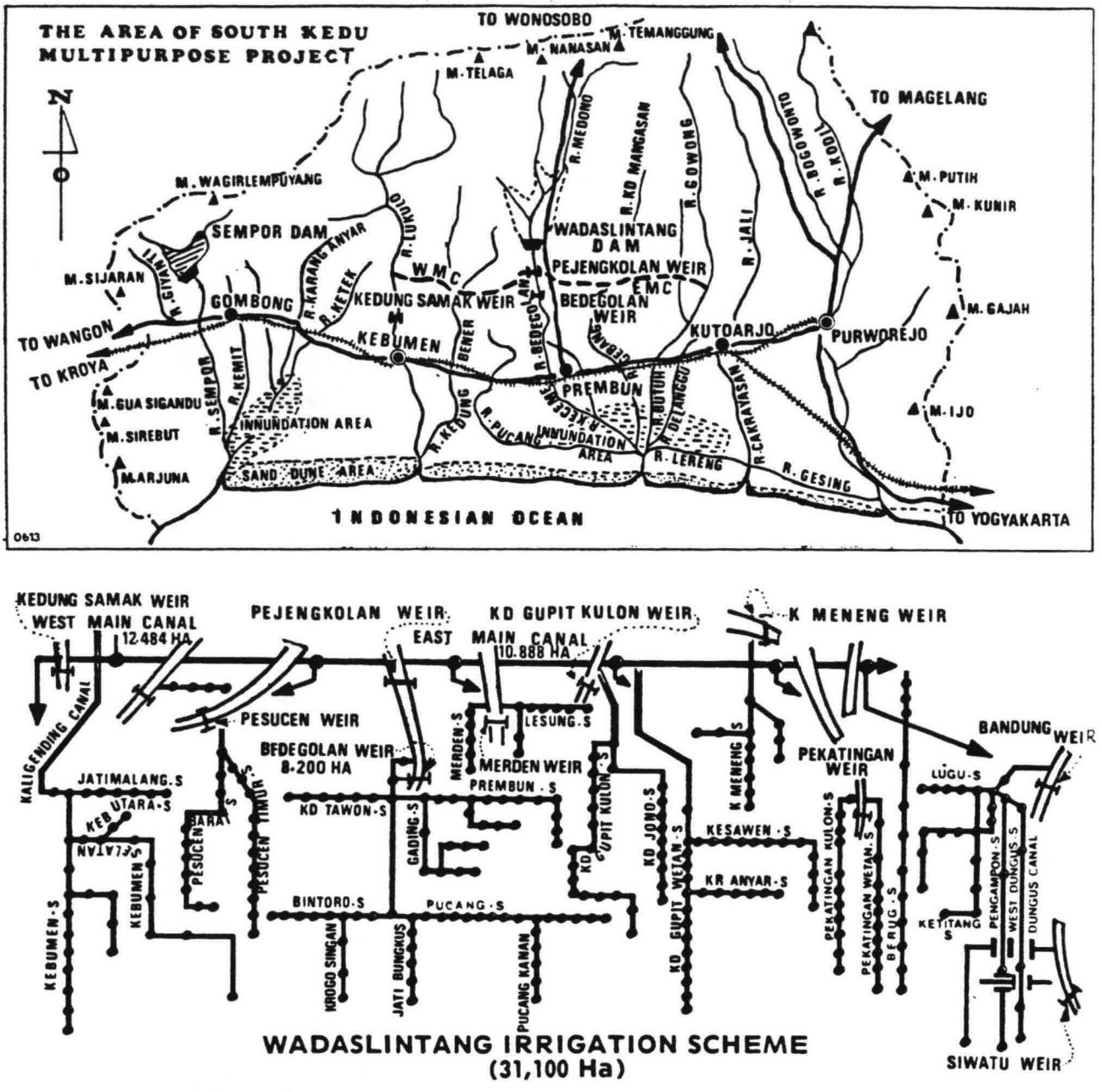


Fig. 7.14. Wadaslintang irrigation scheme, Indonesia.

The activities of the South Kedu multipurpose project (Java Island, Indonesia) commenced in 1969 with the construction of Sempor Dam in one of the five big rivers of the area. This was followed by the gradual development of irrigation, functioning since 1978. The main objectives of the scheme are to provide irrigation for the South Kedu area which covers either Sempor or Wadaslintang irrigtion area-total ricefield 37,000 ha, to support flood control efforts of 7000 ha inundated area and hydropower development. Construction of Wadaslintang dam started in 1982. The rockfill dam with impervious core is 116 m high, has a crestlength of 650 m and a volume of 7,200,000 cu.m. Two Francis turbines with a capacity of 16 MW produce 92 million KWH/year. Seven kilometre downstream of the dam the Pejengkolan diversion weir (construction 1984-86), height 19.5 m, length 76.5 m, ogee type crest 55,000 cu.m concrete, has two intake structures serving the Wadaslintang West (WMC) and East (EMC) Main Canal. WMC is 18.8 m long, 9 m bottom width, discharge 16 m^3/s; EMC is 22.6 km long, 8 m bottom width, 13.5 m^3/s. Total 163 appurtenant structures, such as aqueduct, siphons, tunnels and bridges are part of the scheme.

8. Navigation locks and other shiplifting devices

Navigation locks are to be constructed for a connection between two water bodies with different water levels. They are devices that raise or lower ships to different levels. When a ship enters a lock, great water tight gates swing shut behind it, then when water is let in, the ship rises with water level so that it can proceed at a higher elevation. Water is released from the lock to lower the water level and the ship.

The structures in which the gates are located, are called **lock heads**; the canal between the heads is called the **lock chamber**. The lock chamber is connected with the upstream level, as well as with the downstream level by means of **culverts or openings** in the lock heads.

In inland canals with practically constant water level difference is only in one direction. In that case, the heads are called **upper lock head** and **lower lock head**. Locks near to the sea (tidal variation) can have alternating water level differences. In that case, the heads are called **outer lock head** and **inner lock head**.

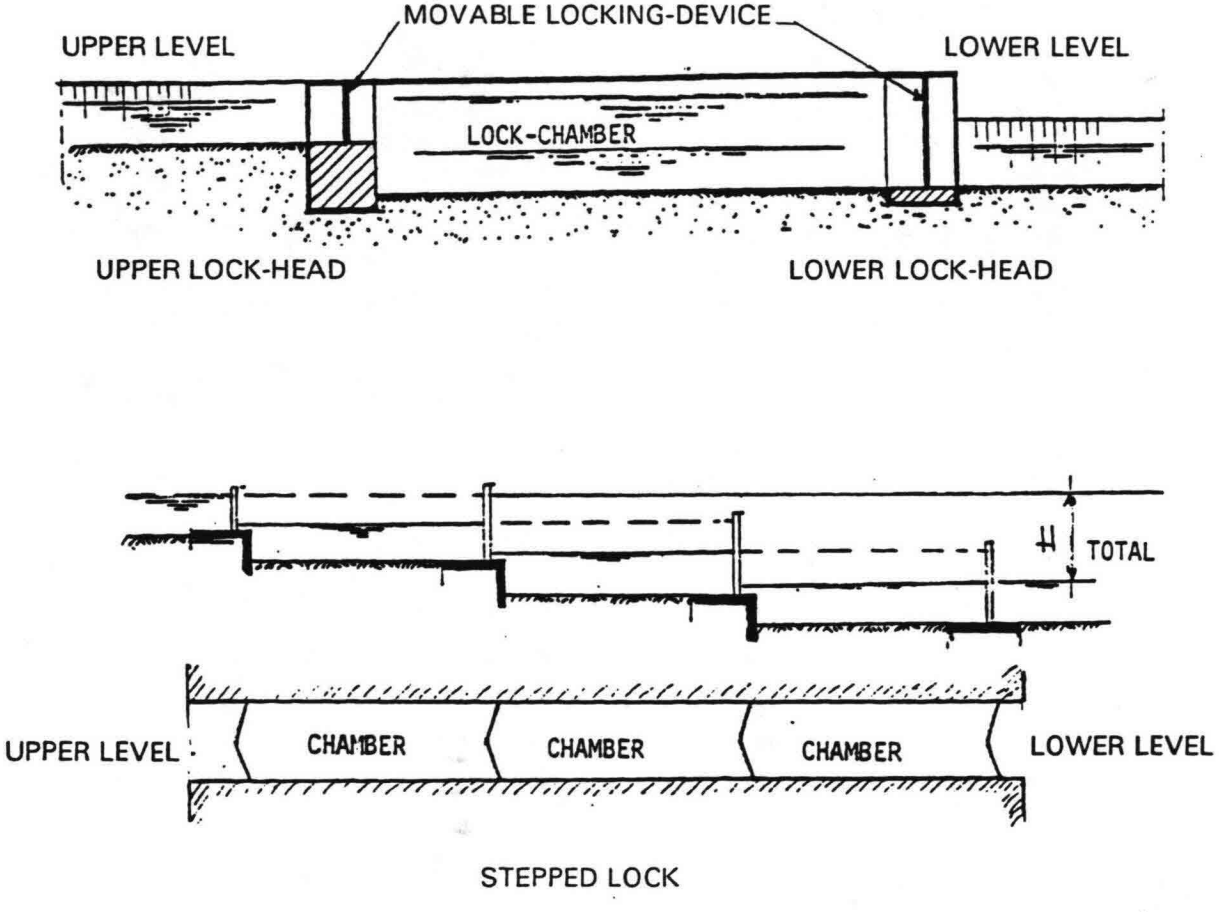


Fig. 8.1. Navigation locks.

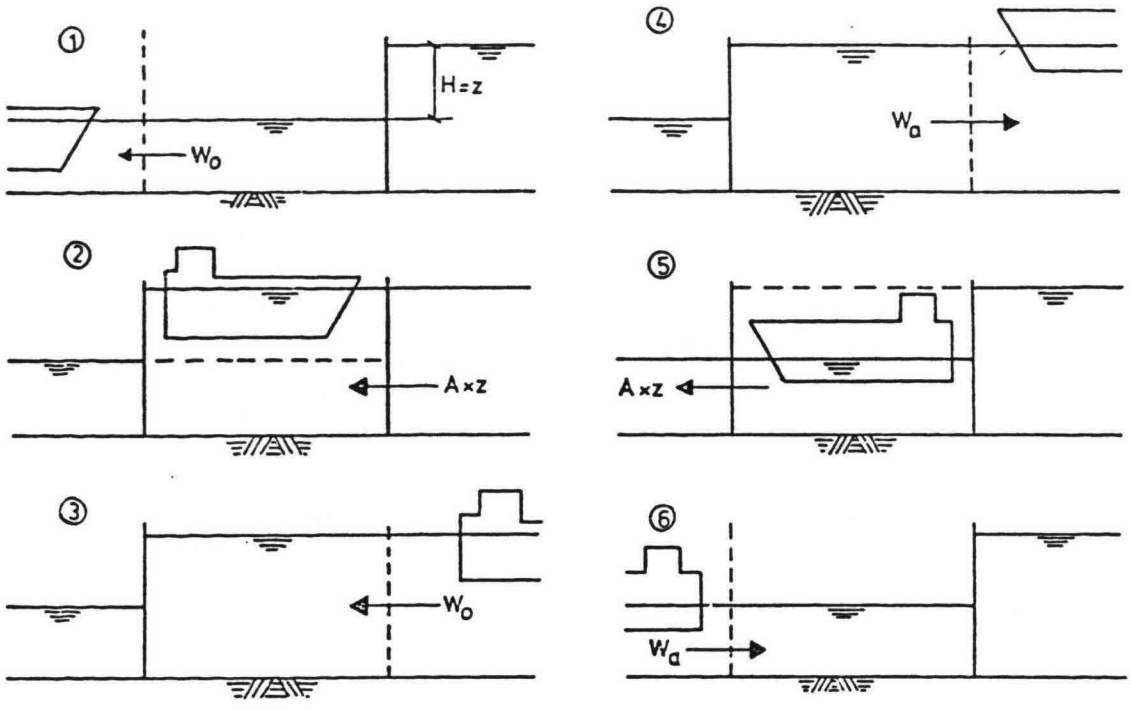
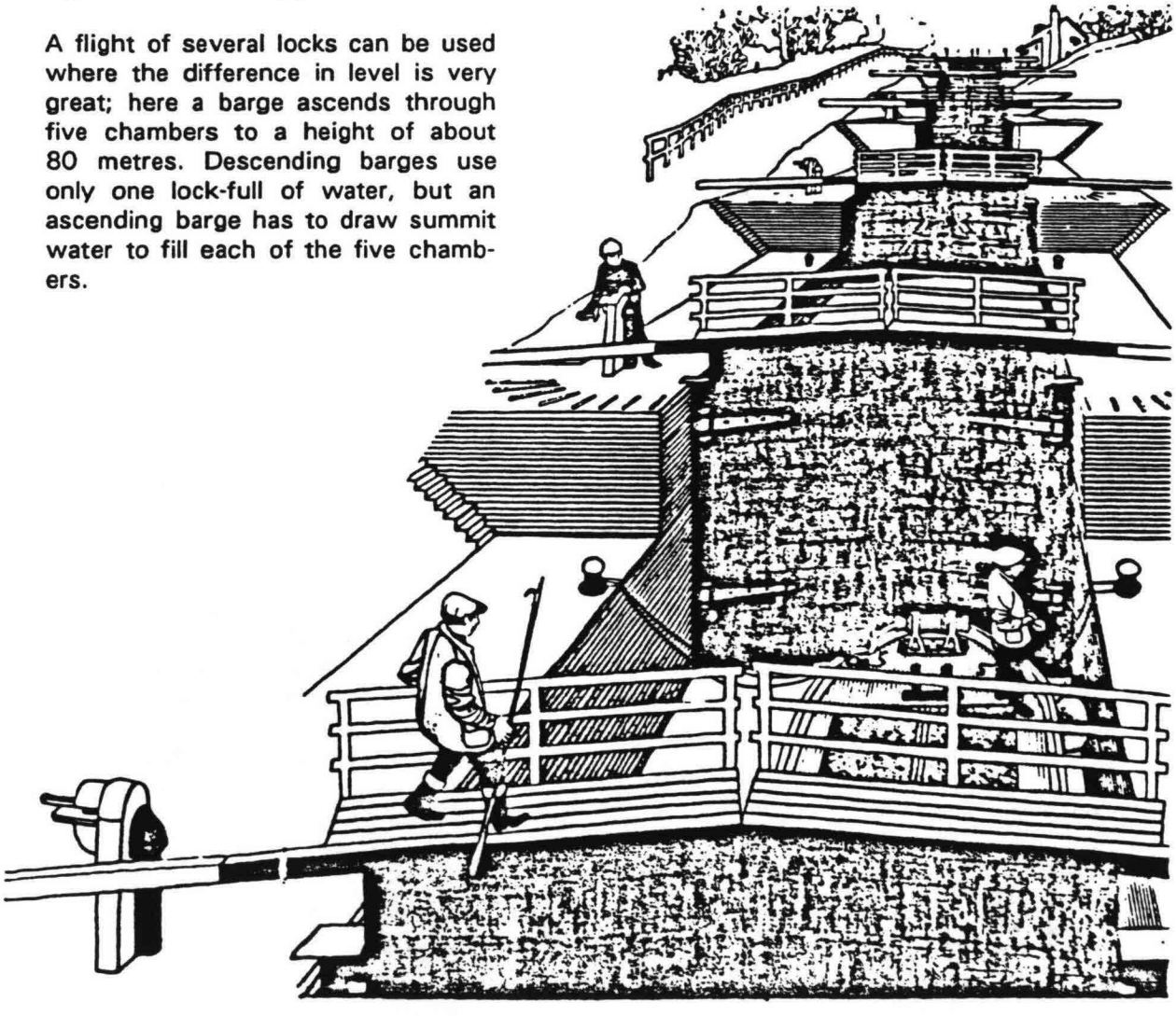


Fig. 8.2. The locking process

A flight of several locks can be used where the difference in level is very great; here a barge ascends through five chambers to a height of about 80 metres. Descending barges use only one lock-full of water, but an ascending barge has to draw summit water to fill each of the five chambers.



• **Navigation locks**

Structures located between two water planes with different levels, which make it possible for ships to move from one to the other by the operation of movable elements (gates and/or valves).

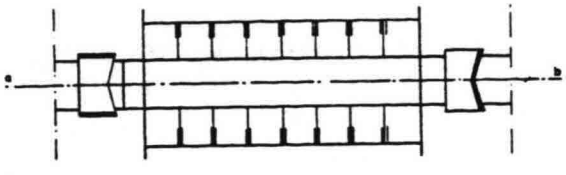
• **Inland navigation locks**

a = upstream

b = downstream

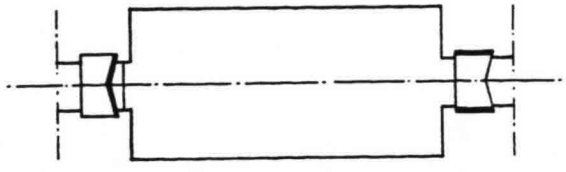
Lock

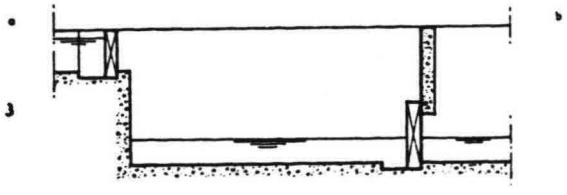
A navigation lock which includes, between its ends or heads equipped with gates, a chamber which can contain the ships to be locked through, and in which the water levels is increased or decreased to the upstream or downstream levels through a conduit usually controlled by valves.



Lock with a chamber having sloping sides

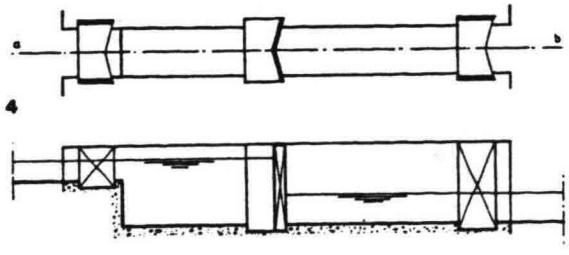
A lock, the chamber of which is bordered by two sloping sides and the heads of which are independent.





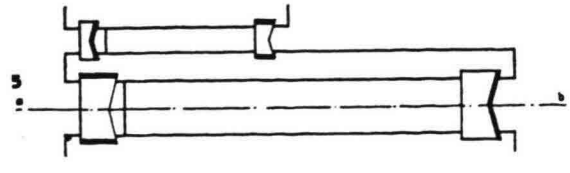
Lock with crosswall

High lift lock the chamber of which is closed, at the lower head level, by a cross wall leaving enough head room, for navigation purposes.



Lock with intermediate gate

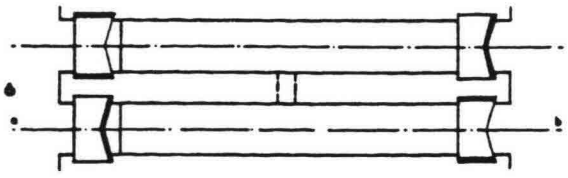
A lock which makes it possible to have three chambers with different lengths, which are used according to the traffic and the need to limit water consumption.



Group of locks

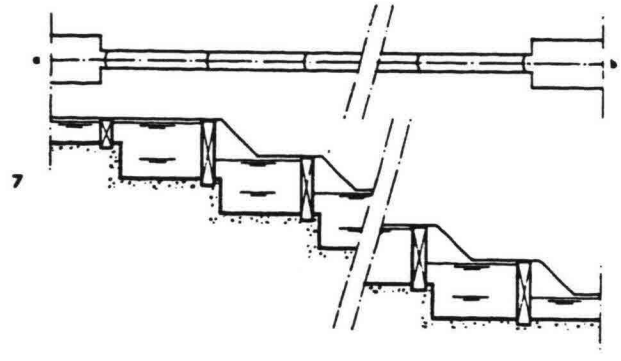
Group of locks having the same lift but usually of different sizes.

Inland navigation locks

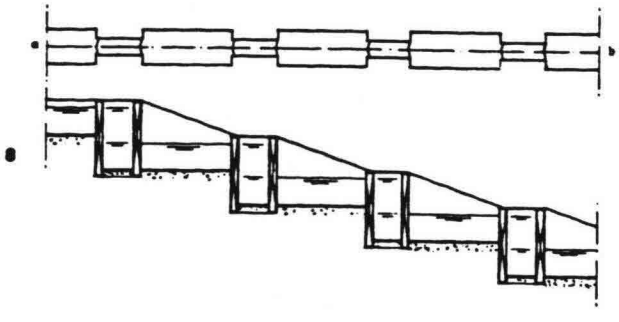


Twin locks
Side by side locks with identical dimensions.

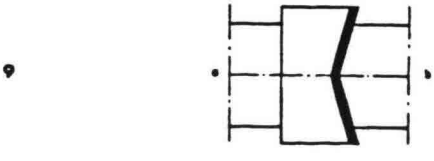
• Interconnected twin locks
Twin locks that interconnected in such a way that any of them may be used as a water saving chamber for the other one.



Flight of locks
Series of locks with identical dimensions where the downstream head of one is the upper head of the next. Also known as a ladder of locks.



Series of independent locks
Series of locks with identical dimensions separated by very short reaches which allow for two ships to pass one another.



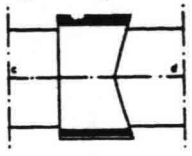
Guard lock
Single head lock located at the mouth of a canal river, which is normally open but can be closed in flood periods.

Maritime locks

c = inner (or upstream) side
= inner basin side
d = outer (or lower) side
= sea or tidal river side

In sketches 11 and 13, the heads are symbolically represented with ebb and flood mitre gates. In modern locks, opposed gates may be provided.

In sketches 12 and 13, the outer heads are in addition equipped with safety gates operated in case of strom tides. They should be integrated into the general protection system of the region against the effects of such tides.



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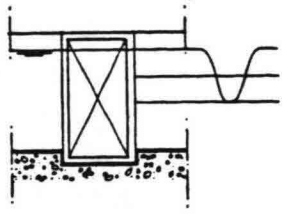

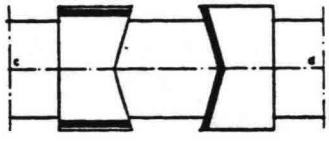

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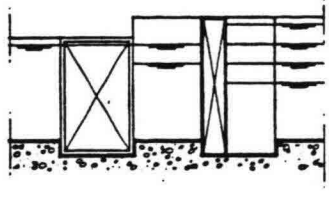


Safety gates are not needed:

a) If the gate(s) of the outer head is(are) designed in such a way that it (they) can be used for this additional purposes.
b) If there is no danger of such storm tides occurring.

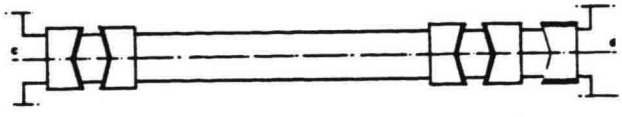
Usually, each head is equipped with two gates, one of which is kept in reserve.

Ebb tides gates

Single head lock for maintaining inner basin water level. It is open to navigation from the sea at high tide.

Ebb and flood tide gates

Single head ebb tide gate lock with the addition of a flood tide gate for use during exceptional spring tides.



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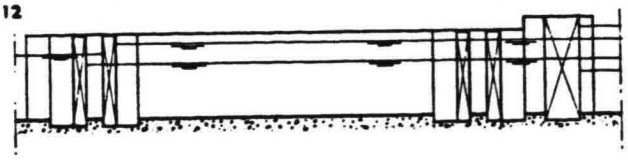
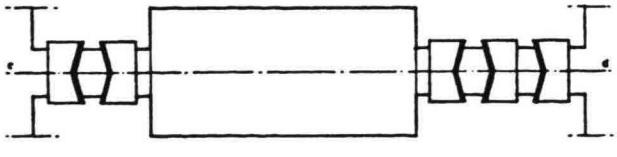


Lock with opposed gates

A lock allowing for the lockage of ships under any tidal level conditions excepting storm tides.

Lock with mid-tide basin

Large capacity lock the outer head of which remains open during ebb tide until mid-tide so as to allow small draught ships to have access to the chamber for as long as possible.



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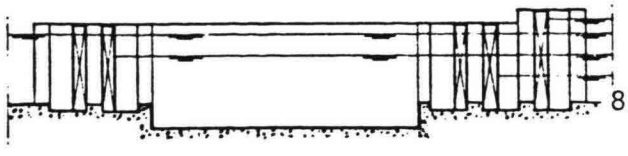


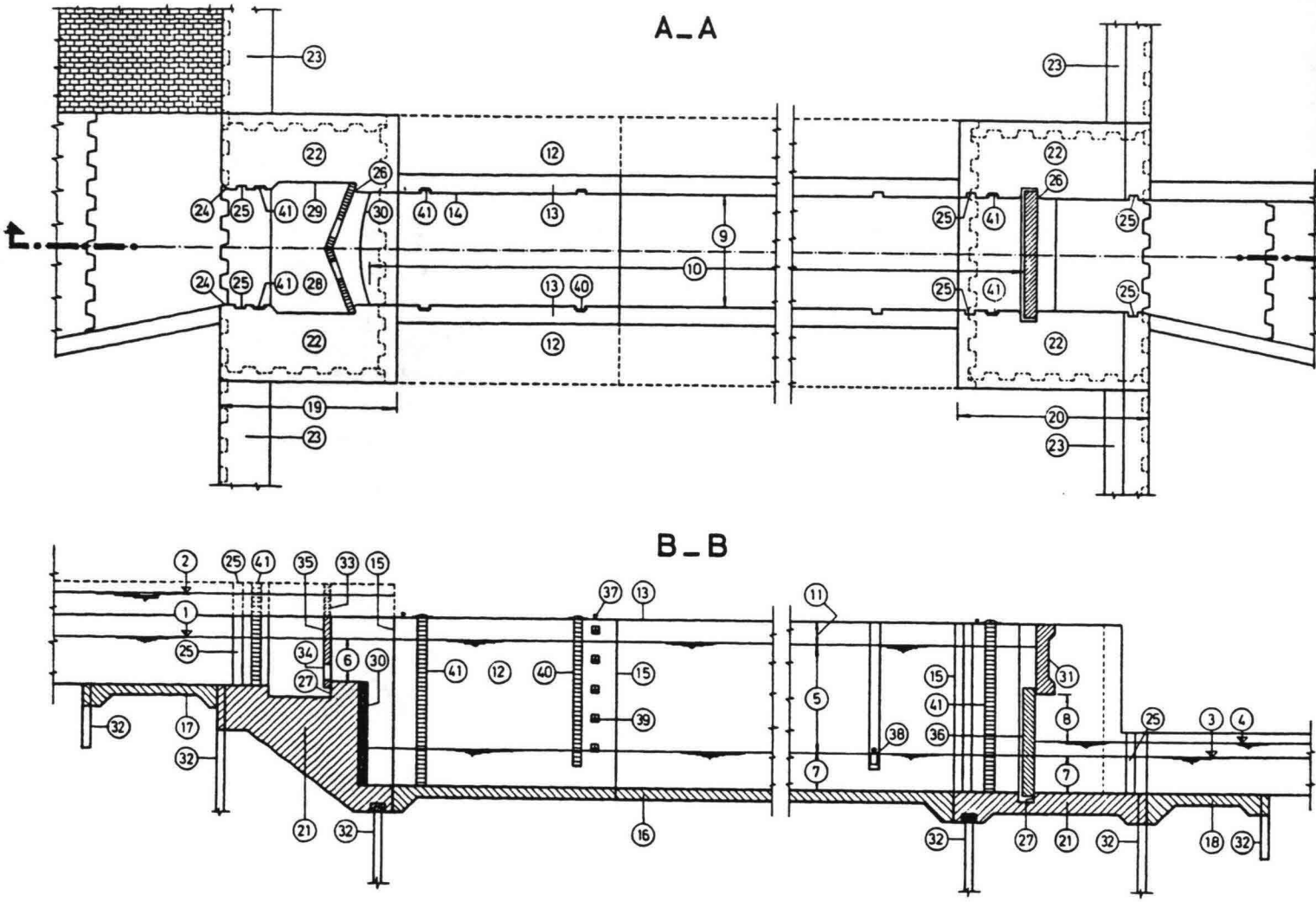


Fig. 8.3. Lock wit a chamber. AA - Plan; BB - Longitudinal section.

Characteristic data

1. Upstream water level
2. Max. upstream water level
3. Downstream water l.
4. Highest navigable downstream water level
5. Lift
6. Draft on upper sill
7. Draft on lower sill
8. Vertical clearance between H.N.D.W.L. and cross wall
9. Useful width of the chamber
10. Useful length of the chamber
11. Freeboard

Chamber

12. Wall of the chamber
13. Coping
14. Edge of coping
15. Expansion joint
16. Floor
17. Upstream apron.
18. Downstream

Lock head

19. Upper lock head
20. Lower lock head
21. Floor of lock head
22. Wall of lock head
23. Return wall
24. Bull nose
25. Stop log recess
26. Gate bearing surface in case of mitre gates: hollow quoin
27. Sill
28. Gate chamber
29. Gate recess
30. Breast wall
31. Cross wall
32. Cut-off wall

Gates

33. Upper gate.
34. Valve
35. Guard gate
36. Lower gate

Lock wall fittings

37. Cope bollard
38. Floating bollard
39. Recessed bollard
40. Ladder and ladder recess
41. Head ladder

The level of the sill of the downstream head and of the floor of the lock chambers are governed by the lowest water level of the downstream reach, while the level of the upper sill is determined by the lowest water level of the upstream reach. This means that it is possible to raise the floor of the upstream head.

Both under and on both sides of a lock structure, a groundwater flow will be generated by the difference in head between the upstream and downstream reaches.

Excessive flows must be prevented, as this can lead to movement of soil particles thus causing the formation of dangerous cavities under and alongside of the lock (piping). This will create an undermining and scouring action between the upstream and downstream ends of the lock. In general this can be prevented by lengthening the waterpath in tri-dimensional directions.

It is therefore necessary to provide locks with a **seepage cut-off** which acts as an impervious curtain perpendicular to the lock axis both under and on each side of the lock, usually both at the upstream and downstream end. In this manner, the seepage path is lengthened, thus reducing the groundwater flow and preventing movement of soil particles. The **cut-off walls** can be made from different materials such as steel sheet piles, concrete sheet piles either driven or cast in situ, mass or reinforced concrete, cement bentonite mixtures or timber sheet piles.

For locks with a high lift a second curtain is built at the lower end of the lock, while an almost similar solution is chosen for a lock located in a tidal region where there is reversal of head. It should be noted, that the second curtain is located either at the upstream end of the lower head (lock with high lift) or at the other end (lock in tidal region).

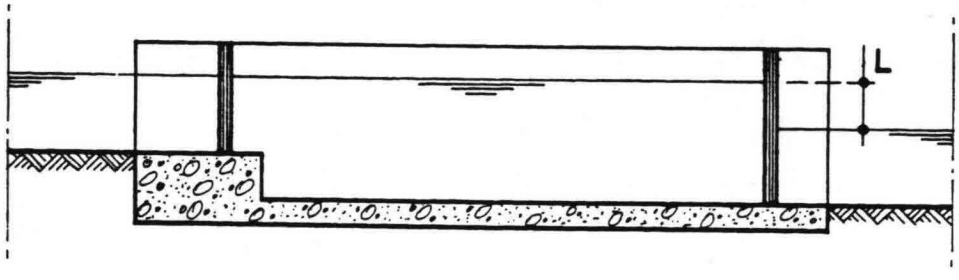


Fig. 8.4. Lock with raised upper sill. L = lift

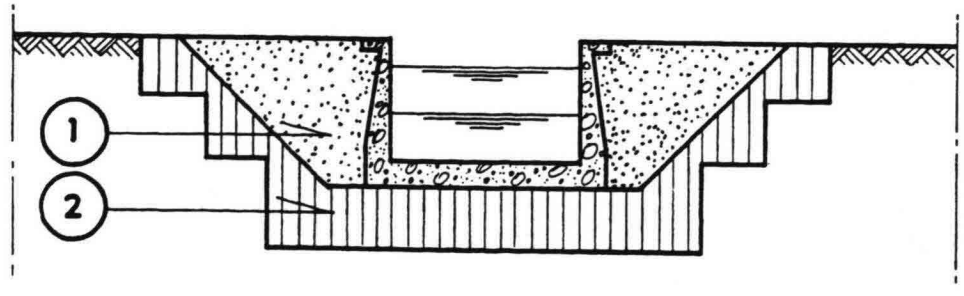
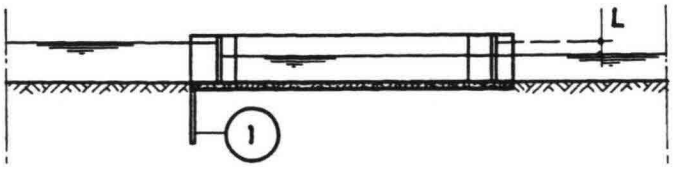
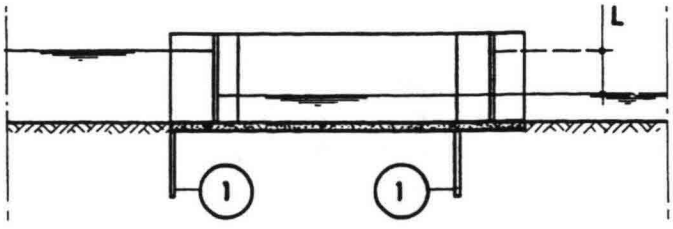


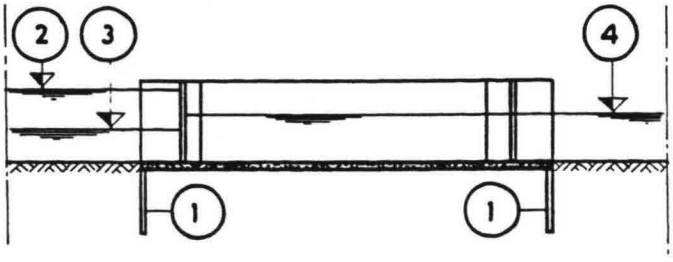
Fig. 8.5. Cross section of lock with seepage cut-off. 1. Backfill, 2. Sheet piling.



Seepage cut-off for lock with moderate lift



Seepage cut-off for lock with a high lift



Seepage cut-off for lock in tidal waters

Fig. 8.6.
Seepage cut-off.
Legend
L : lift
1. Seepage cut-off
2. High tide
3. Low tide
4. Canal level

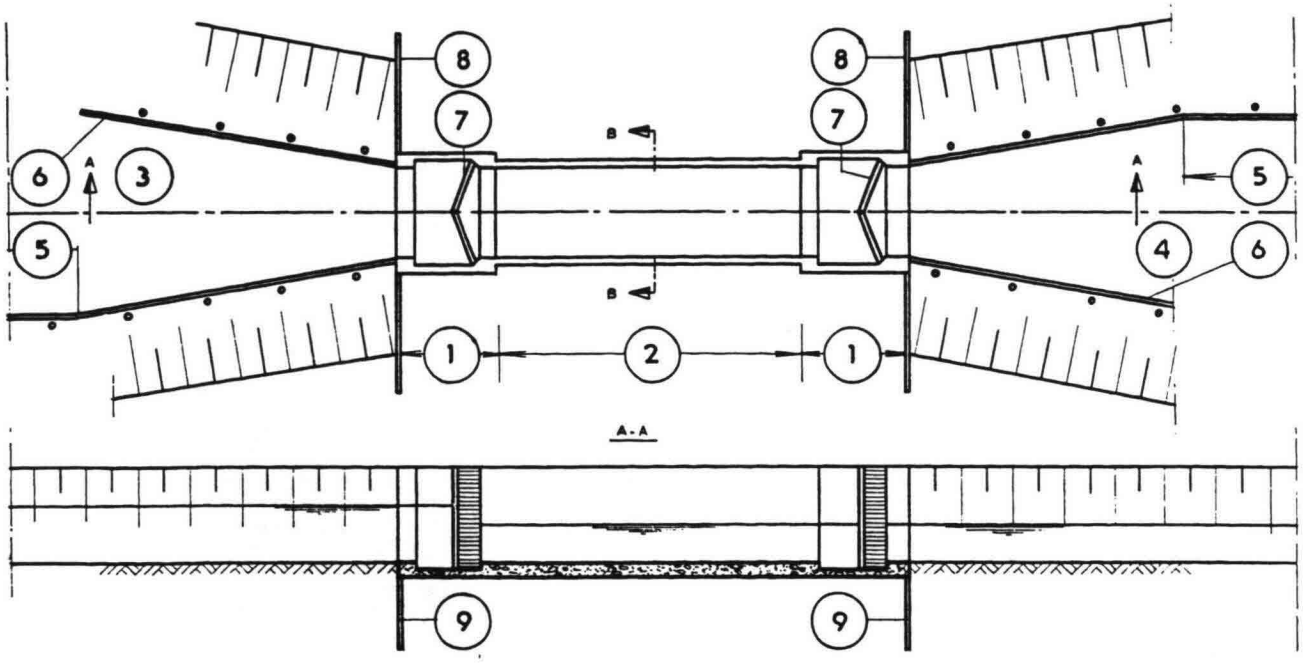
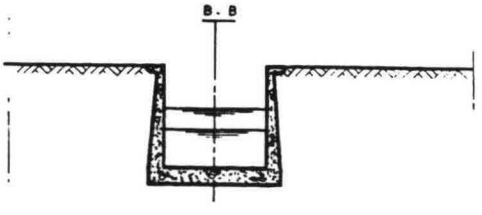


Fig. 8.7. Lock with a seepage cut-off.

1. Lock head
2. Lock chamber
3. Upstream canal
4. Downstream canal
5. Waiting place
6. Guide wall
7. Mitre gates
8. Retaining wall also used as seepage cut-off
9. Seepage cut-off.



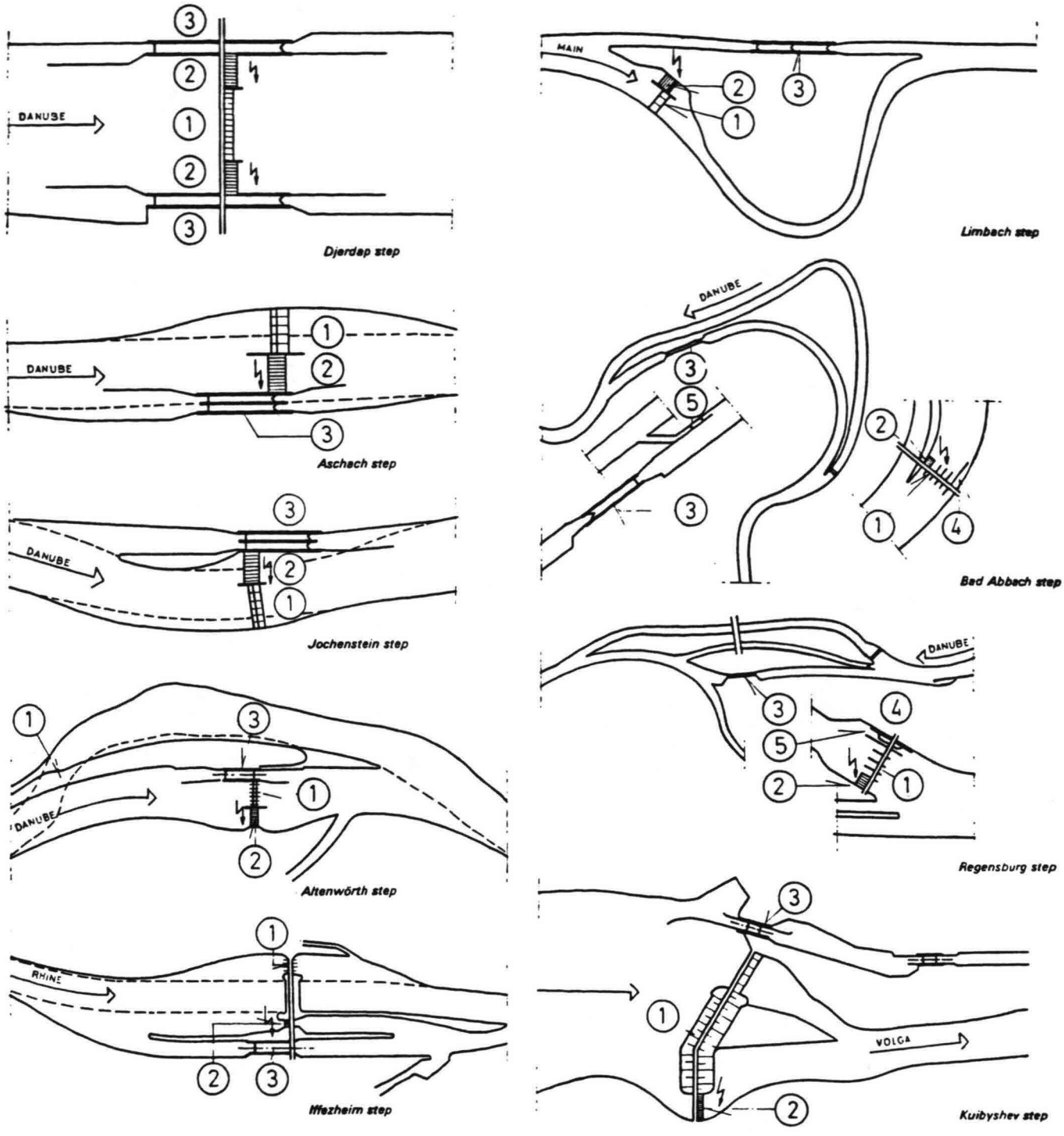
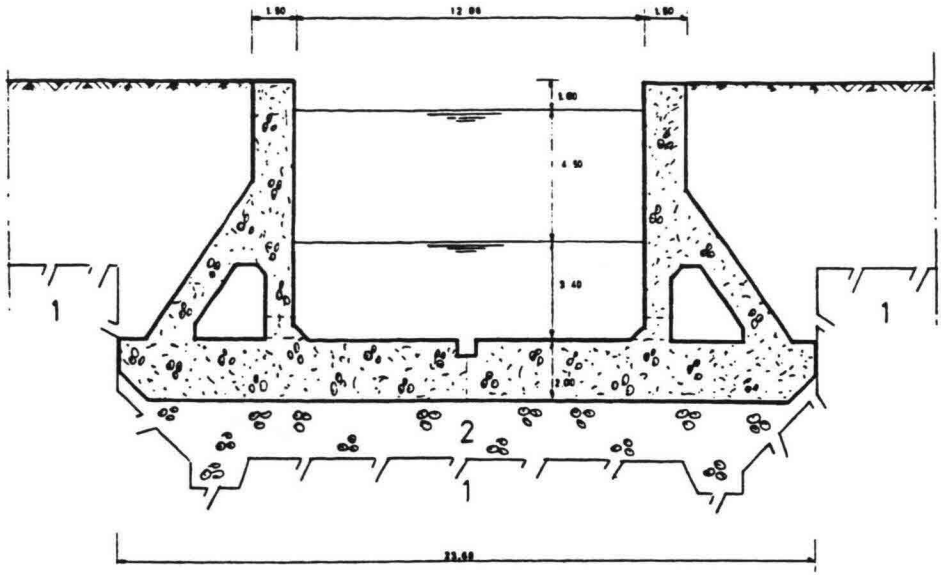


Fig. 8.8. Location of inland navigation lock (example).

1. Weir or dam
2. Power station
3. Lock
4. Channel for pleasure boats
5. Lock for pleasure boats

.....original bank



Lock-cross sections

Fig. 8.9.
Apach lock on River Mosel (France).
1. sandstone
2. weak concrete

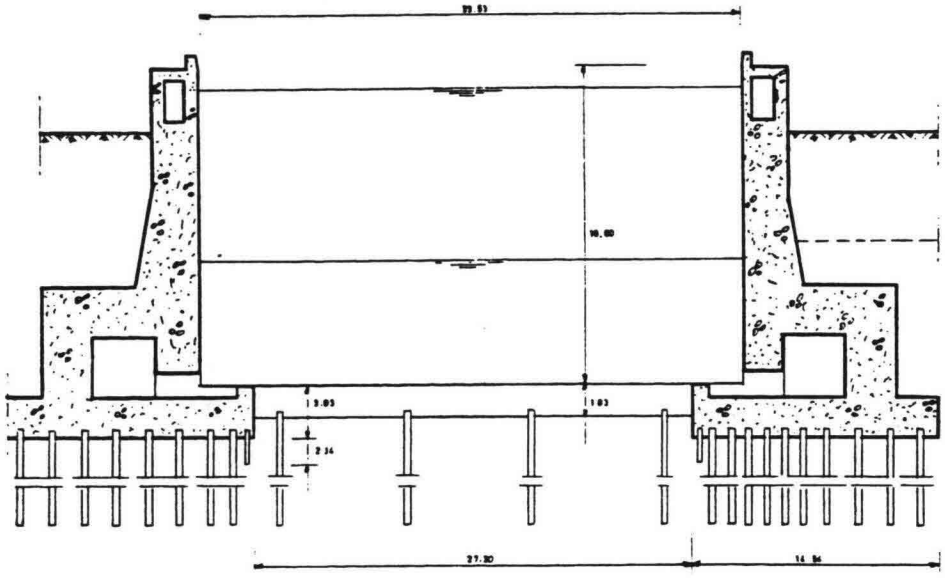


Fig. 8.10.
Lock on Arkansas river (U.S.A.).

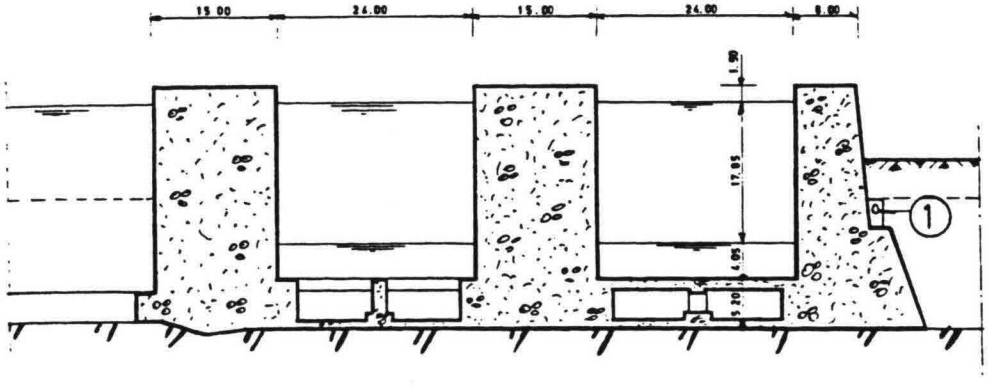
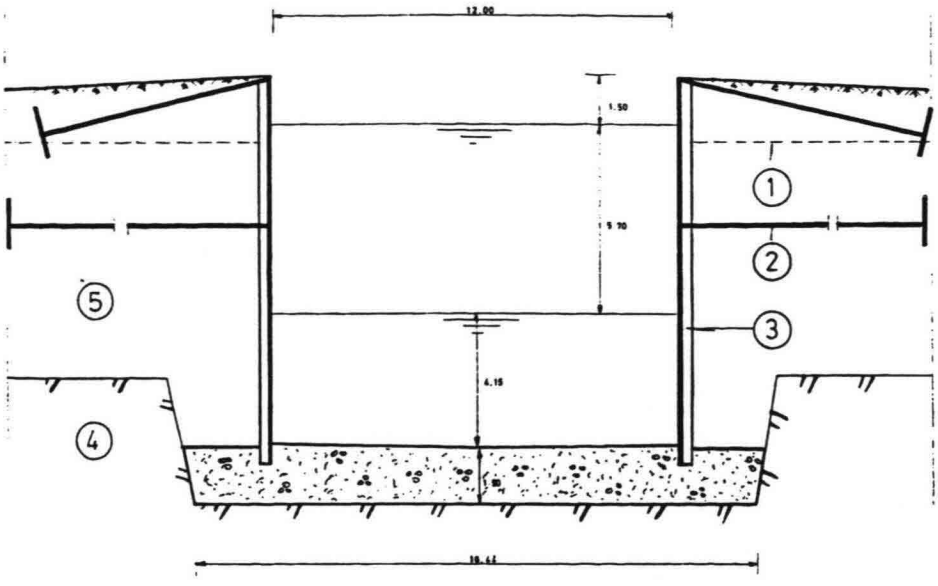


Fig. 8.11.
Aschach lock on River Danube (Austria).
1. drainage gallery

The structural design of the chamber cross section depends on the required dimensions, the hydraulics, the subsoil and conditions affecting construction work.



Lock-cross sections

Fig. 8.12.
Bad Abbach lock on River Danube (Germany).

1. Natural ground
2. Anchorage
3. Sheet piling
4. Limestone
5. Gravel

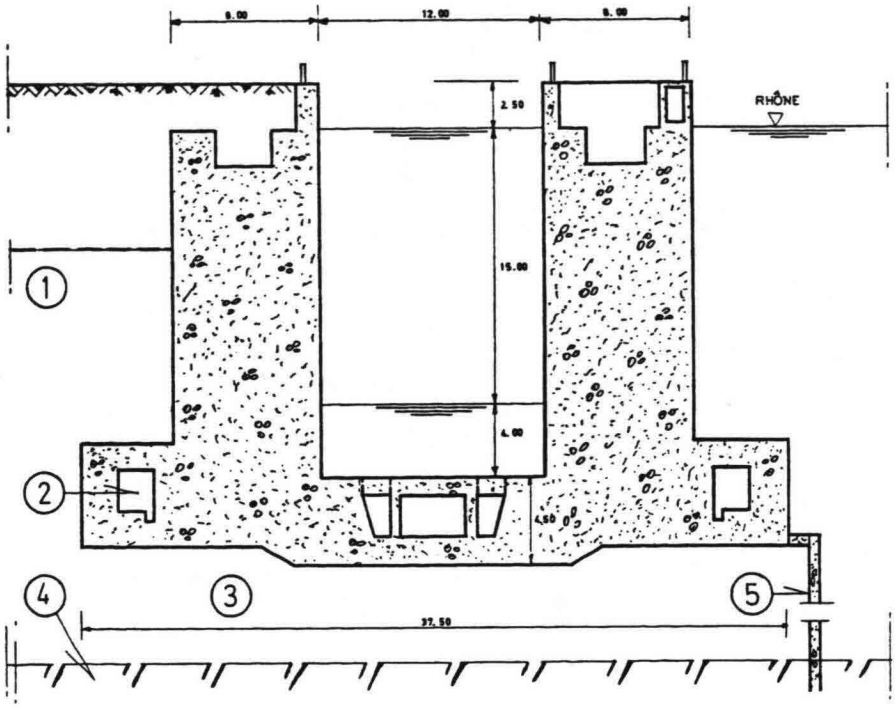


Fig. 8.13.
Vallabrégues lock on River Rhône (France).

1. Natural ground
2. Inspection gallery
3. Gravel
4. Clay
5. Impermeable wall

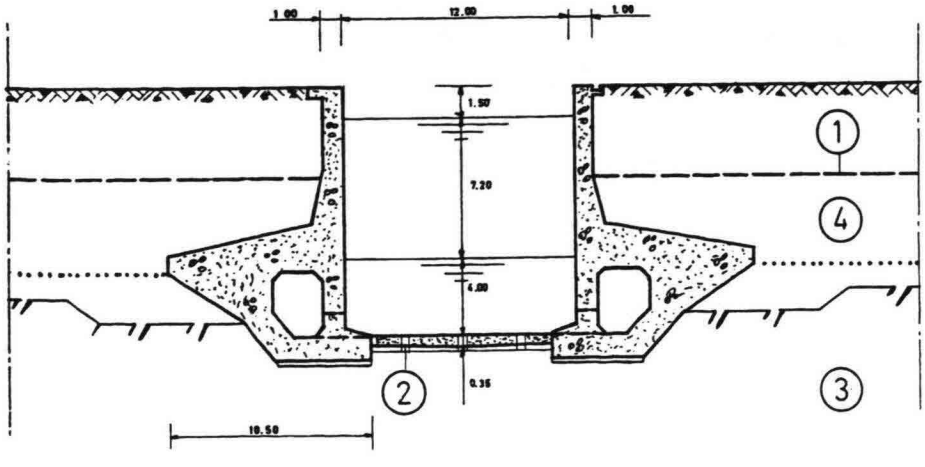


Fig. 8.14.
Villey le Sac lock on River Mosel (France).

1. Natural ground
2. Vent
3. Limestone
4. Sand, gravel, clay

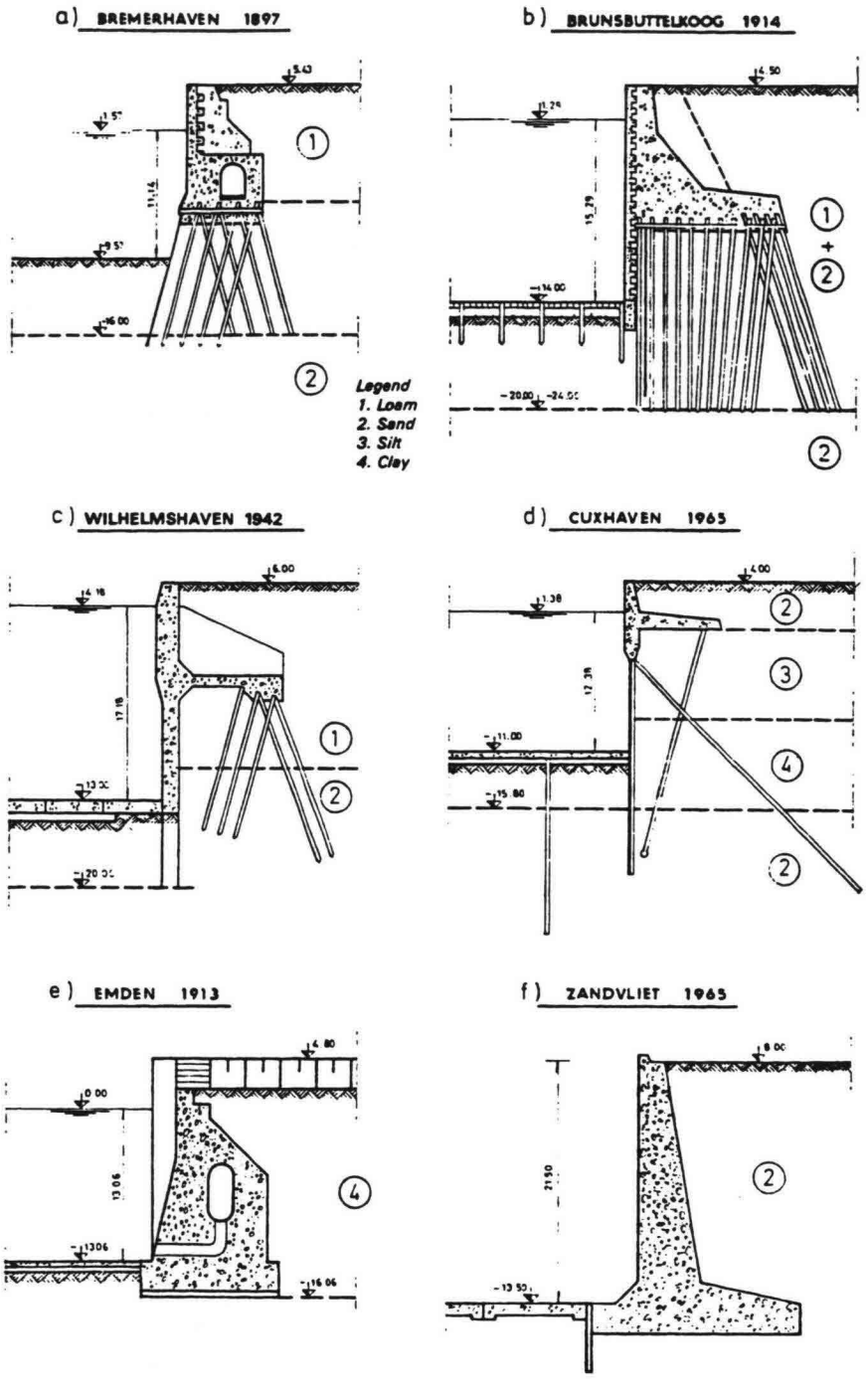
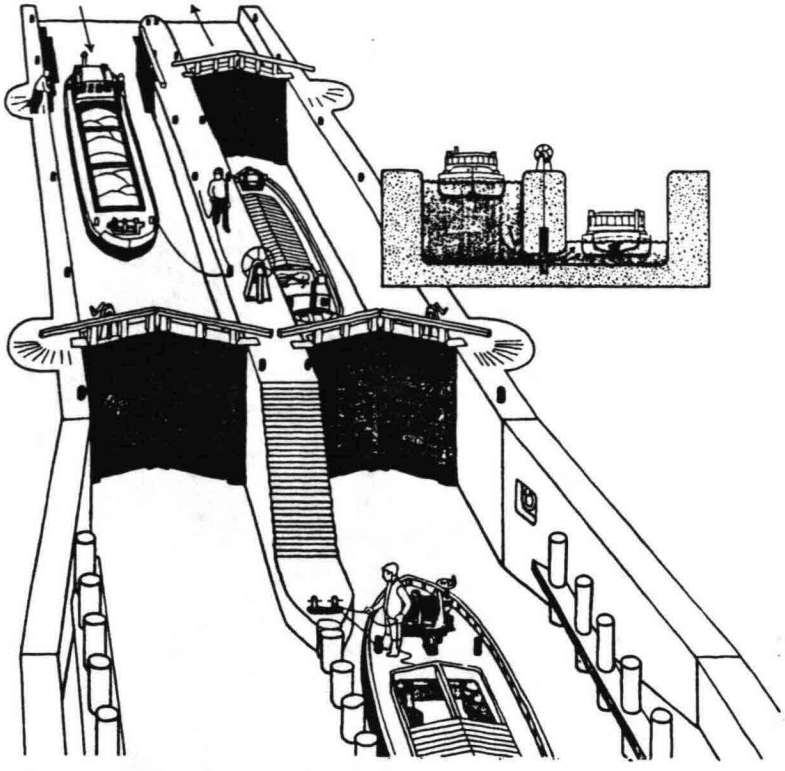


Fig. 8.15. Development of chamber walls (maritime locks).

Every time a barge travels through a lock, water is lost in the transfer from one level to another. When water is scarce, or where summit water has to be pumped up, a pair of locks or multi-reservoirs make a more economical use of the water.

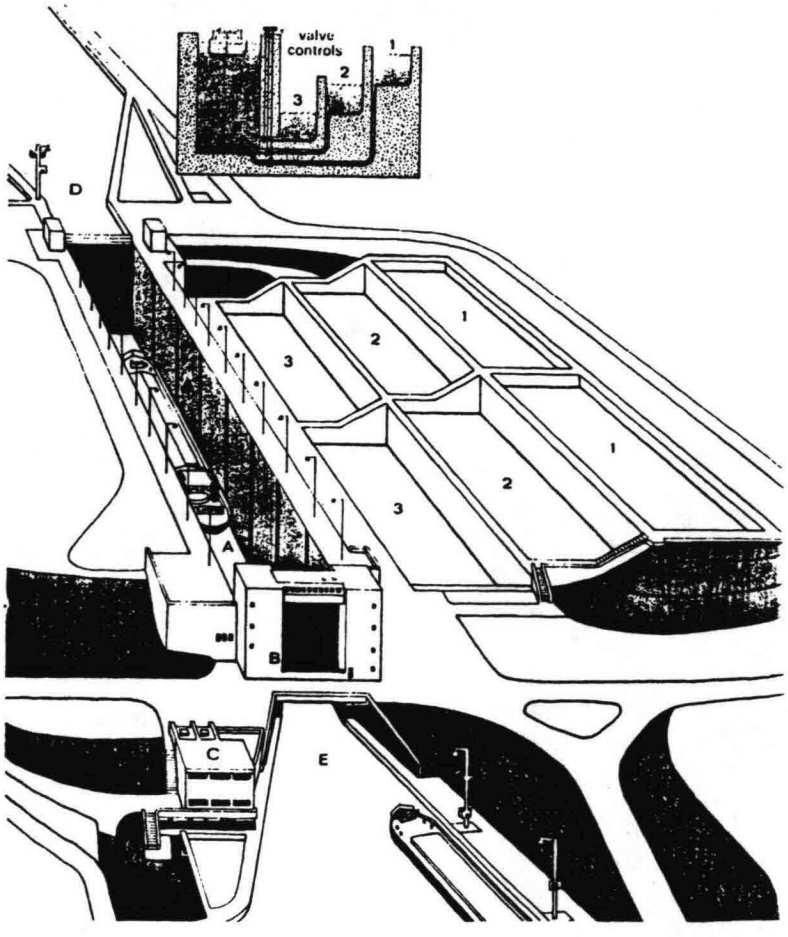
Fig. 8.16. Double Lock.



A double lock has two chambers side by side. Two barges enter, one going up, the other going down. Water from the full chamber flows through an extra culvert in the adjacent, empty chamber. When the water in both chambers is level, the flows stops. The rest of the water for the ascending berge is supplied in the normal way from the upper reach. About 50 % of summit water is saved.

Fig. 8.17. A multi-reservoir lock with 3 side tanks. Erlangen lock on the Europa canal.

1,2,3 Reservoirs
A Deep chamber
B Gates
C Pump house
D Upper reach
E Lower reach
F Culverts



Ascending barge has entered the deep chamber. After water from tank 3 has filled part of the chamber, water from tanks 2 and 1 follows. For every locking, a volume of water from each tank moves into the chamber and returns during a subsequent locking into the next lower tank. When the uppermost tank is empty, it is topped up from the upper reach. In this way, compared with a single chamber lock, only a quarter of the summit water is used.

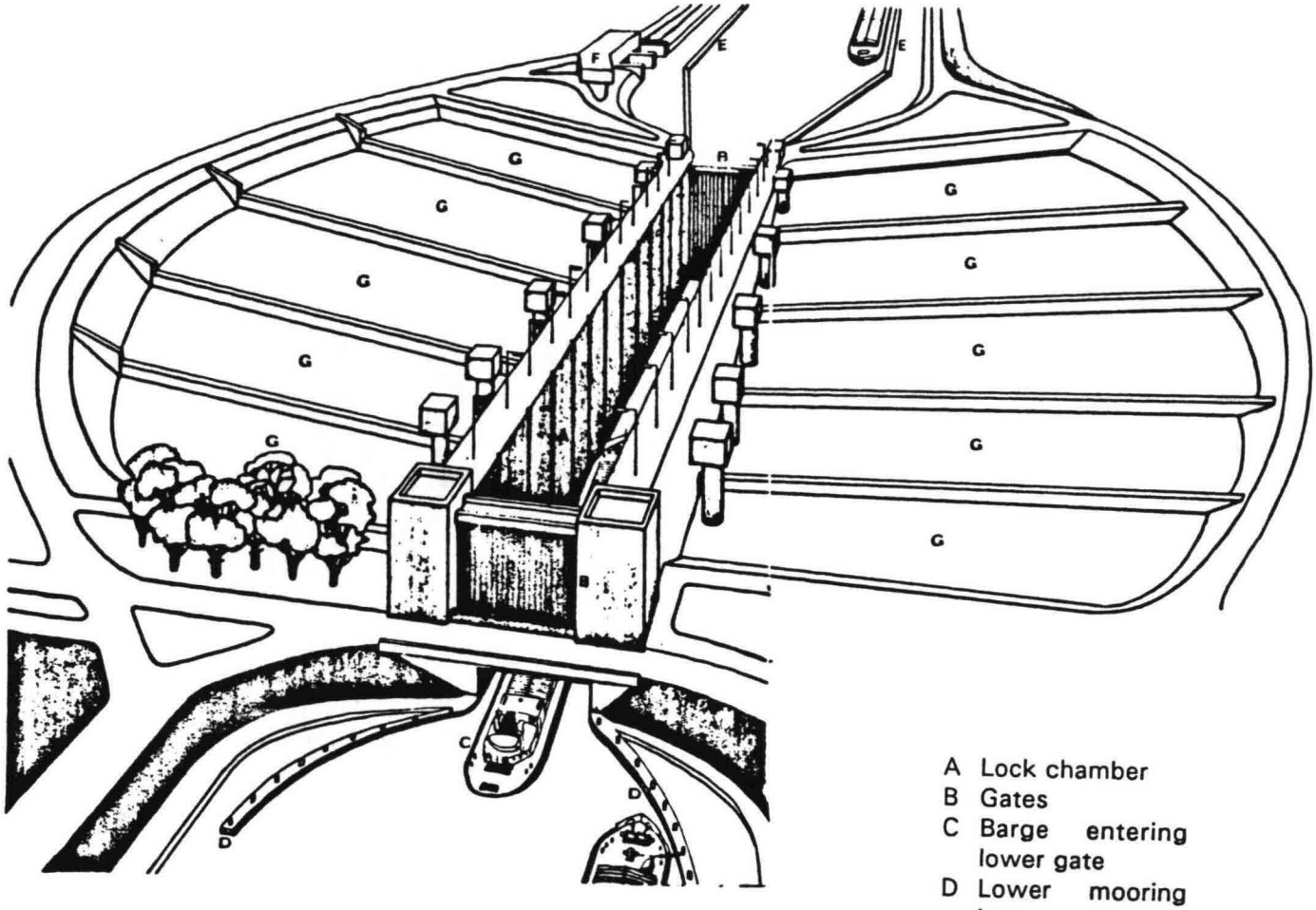


Fig. 8.18.
Deep multi-reservoir lock at a Heinrichenbrug, at a junction of the Dortmund Ems canal (Germany). It has five pairs of reservoirs. The lock chamber measures 95 m by 10 m and lifts vessels of 1500 tons, fourteen metres up. Its pumps lift 3 cubic metres of water per second to the uppermost reservoir.

Locks

Largest World
The world's largest singel lock is that connecting the Schelde with the Kanaaldok system at Zandvliet, west of Antwerp, Belgium. It is 500 m long and 57 m wide and is an entrance to an impounded sheet of water 18 km long.

Deepest World
The world's deepest lock is the John Day dam lock on the Columbian river, Oregon and Washington, USA completed in 1963. It can raise or lower barges 34.4 m and is served by a 998 tonne gate.

Philipsdam and navigation lock (The Netherlands)

The storm surge barrier has narrowed the mouth of the Eastern Scheldt, and less water now enters and leaves the estuary with tide. To prevent a serious reduction in the tidal difference the area of the estuary was reduced by building two dams, the Oesterdam and the Philipsdam. Both have tidal salt water at one side and fresh water with a permanent level at the other side. The Philipsdam was completed in 1987 and could be constructed entirely of sand.

The Philipsdam has an unusual lock complex with a salt/fresh water seperation system that prevents salt water entering the freshwater lake and fresh water entering the Eastern Scheldt whenever a ship passes through the locks.

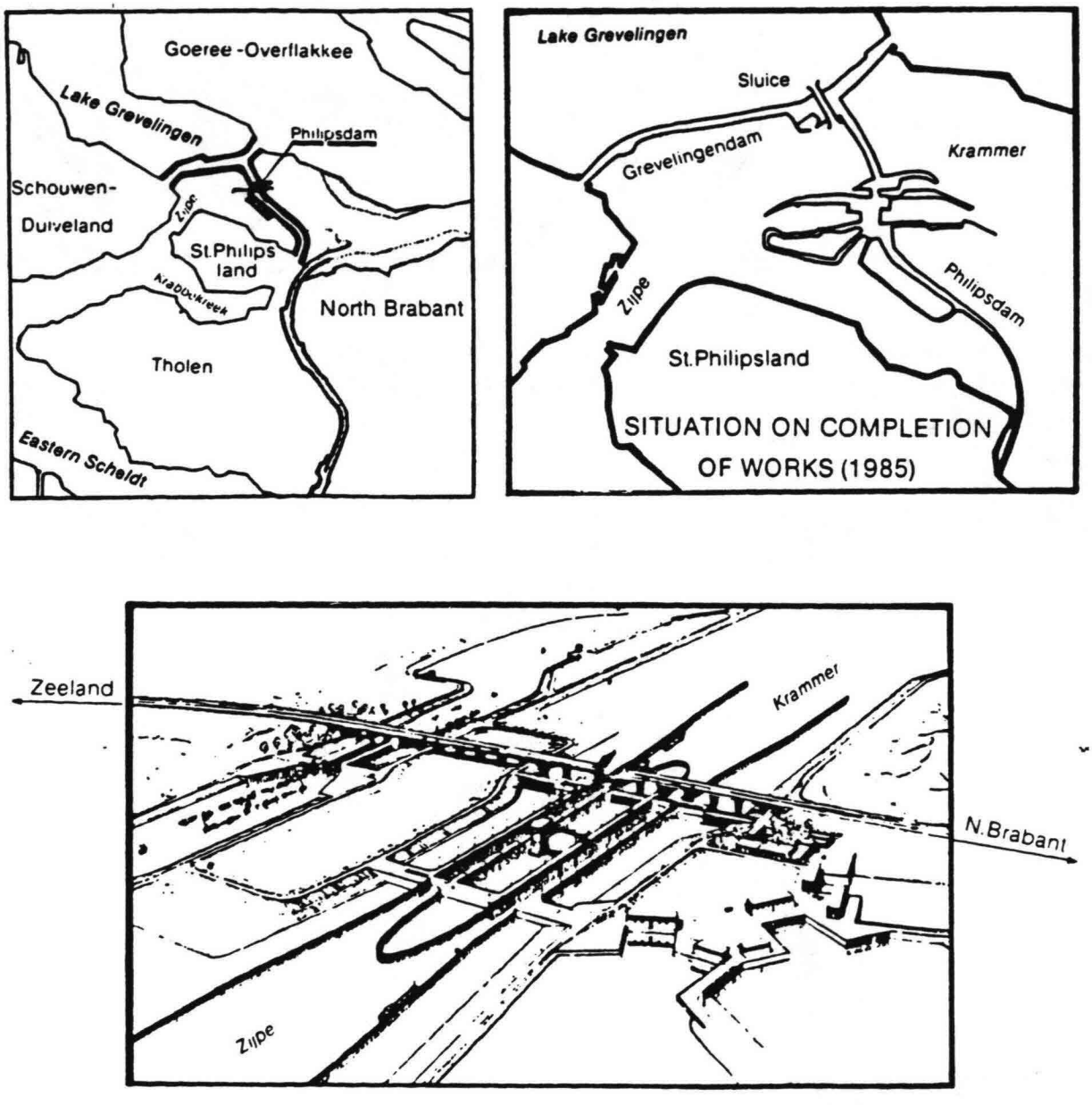


Fig. 8.19. The locks in the Philipsdam.

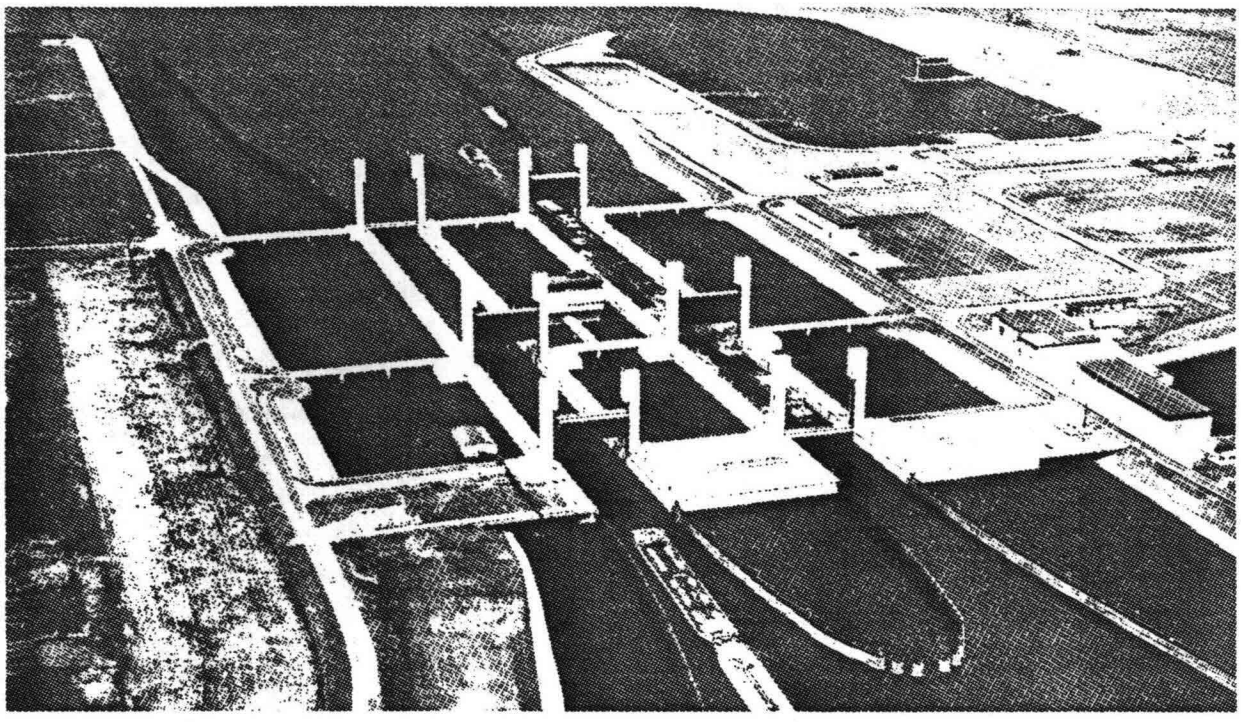


Fig. 8.20. Kreekrak Locks, The Netherlands.
Locks with simultaneous inlet of salt and fresh water.

8.1. Types of lock equipment

The most simple type is the **single- or revolving gate**, which turns, like a house-door, around a vertical axis. However, this type is only used for small locks because of the danger of twisting.

Usual **miter gates** are applied. When the gates are closed, they may be considered as a hinged girder that is symmetrically loaded by the water pressure.

In the case of **wing gates**, every mitre gate is provided with a wing, connected to the mitre gate and moving in the gate recess. The recess can be connected with the high water level and the low water level as well. In this way the gates stem to both sides and can be opened also with a difference in water level.

Sector gates can be completely turned into gate recesses. All mentioned types turn around a vertical axis.

Roller gates are resting on rails on the bottom of the lock. They can be moved in horizontal direction in an extended recess.

Lifting gates move in vertical direction, and suspended by suspension rods to lifting towers. The horizontal water pressure is transferred to the lock-body by wheels, nowadays more and more by sliding plates; in this way it is possible to move the gate also when the water level is not equal.

Segmental gates move also vertical, and turn around a horizontal axis with the aid of arms.

Flap gates turns around a horizontal axis near the bottom and are resting in a recess in the bottom when they are open.

1. Single swing-gate
2. Mitre gates
3. Flap gates
4. Wing gates
5. Sector-gates
6. Roller-gates
7. Lifting gates
8. Segmental gates

The types 1,2,3,4,5 and 6 have an unlimited clear headway, and therefore suitable for sealocks. The types 4,5,6,7 and 8 stem at two sides, which is an advantage for locks operating in tidal regions. In case of the types 5,6,7 and 8, filling of the lock chamber is possible by opening the gates a little, which means no extra slide-valves in the gates or culverts in the lock heads. (type 4 is difficult to control).

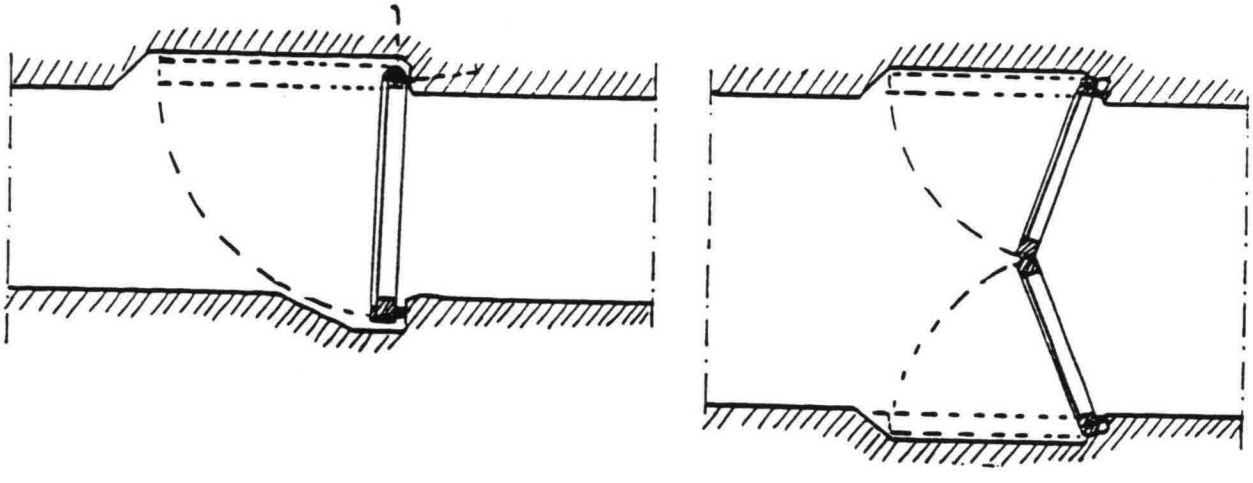
Types 7 and 8 are also suitable, and much applicated as locking-devices for weirs. The lifting-gates often are double, where the upper part slips along the lower part. The river-discharge takes place on top of the gate, which means a possibility for fine regulation and removal of floating obstacles.

Sometimes the gate is provided with a **movable flap**, which gives the same result.
In some cases **cylindrical gates** are applicated.

A special type of gates is the **visor gate**, used in the canalisation of the Lower Rhine (Netherlands). The shape is very favourable for water pressure : only tensile-forces appear!

Summary locking devices

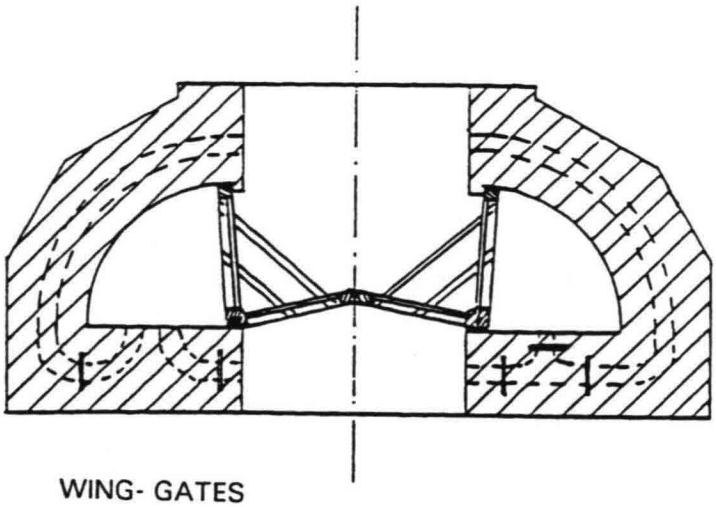
	Free clear-ance? (headway)	possible to move in case of water pressure	possible to stem at 2 sides
1. Single swing- gate	unlimited	not	not
2. Check gates	unlimited	not	not
3. Flap gates	unlimited	not	not
4. Wing gates	unlimited	yes	yes
5. Sector gates	unlimited	yes	yes
6. Roller gates	unlimited	not	yes
7. Lifting gates	limited	yes	yes
8. Segmental gates	limited	yes	yes



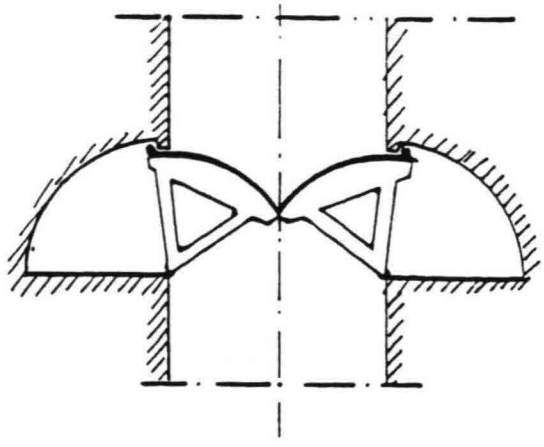
REVOLVING- GATE

MITER- GATES

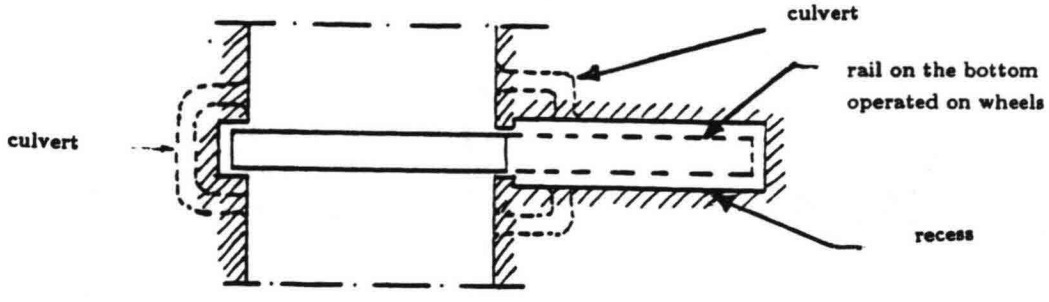
Fig. 8.19. Lock devices.



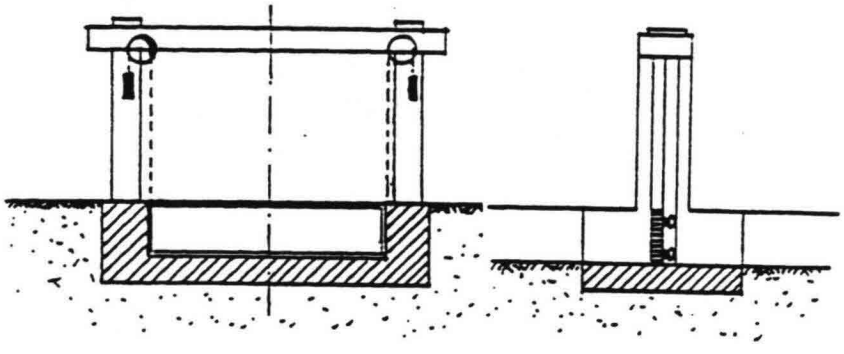
WING- GATES



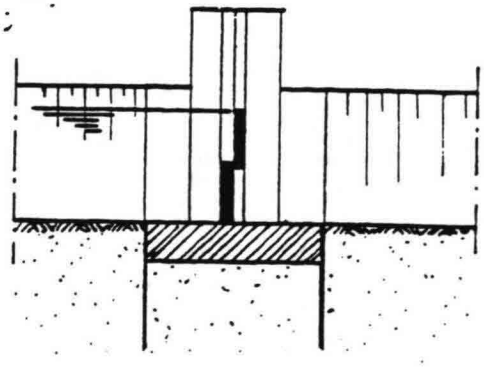
SECTOR- GATES



ROLLING GATE

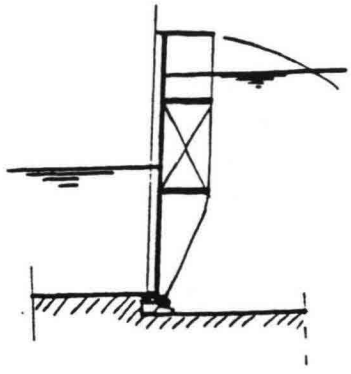


LIFTING- GATE (single)

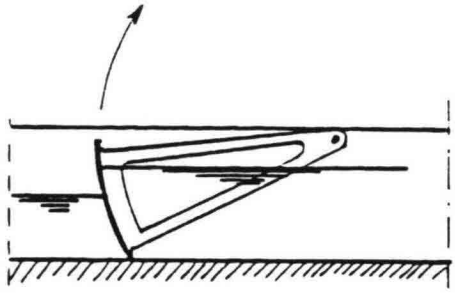


LIFTING GATE (double)

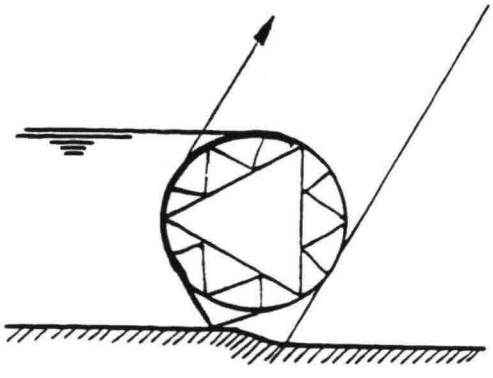
Fig. 8.22. Lock devices.



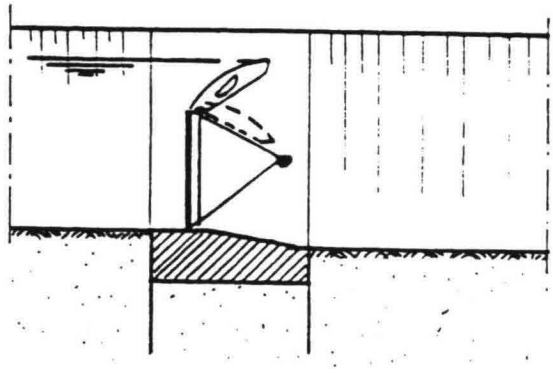
FLAP GATE



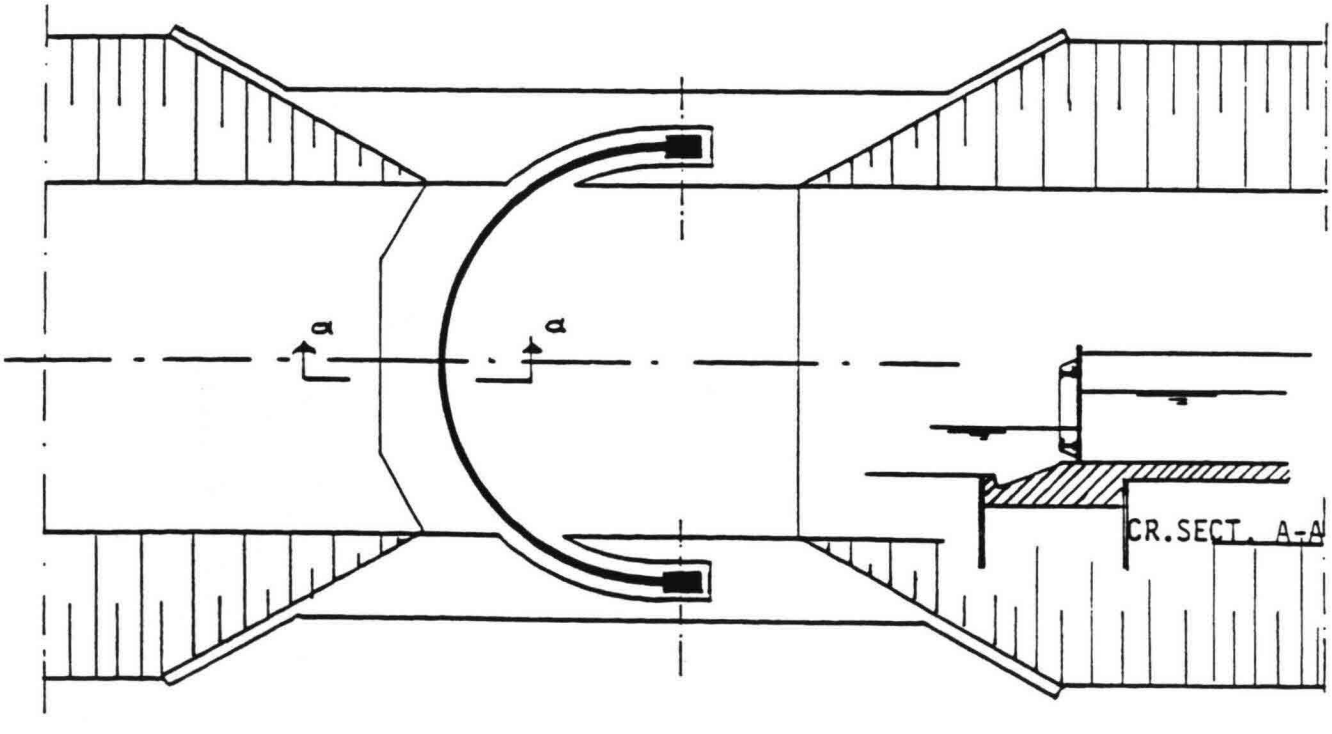
SEGMENT GATE



CYLINDRICAL GATE



GATE WITH MOVABLE FLAP



VISOR GATE

Fig. 8.23. Lock devices.

8.2. Cascade locks

A cascade of locks is a lock system with a number of successive steps. With a cascade system ships can only be transferred in one direction at a time. In this way large heads can be dealt with economically as a single dock might be very expensive.

The largest head of single locks are at the moment 34.5 m at **Carapatello** (Duoro, **Portugal**) and 42 m at **Ust Kamenogorsky** (Irtisch, **USSR**). A new single-lift lock overcoming a head of 39 m with a volume of lockage water of 200,000 m^3 has been built on rock on the Dnieper at Zaporozhie (USSR). The lock is filled at an average speed of 5.5 cm/sec.

A disadvantage of the cascade system is it is very time-consuming. Inland navigation from the opposite direction have to wait a long time. This disadvantage can be solved by constructing a twin lock system. This is of course costly but might be justified.

An advantage of the cascade system that the waterlosses per unit of cargo can be reduced. With 3 steps waterlosses are reduced by a factor 2/3, with four steps this factor is 1/2.

An example of a cascade of twin locks are the locks in the **Welland Canal**, part of **St. Lawrence Seaway** in **Canada**, where a head of 100 m is overcome is 8 steps. Another example of a cascade lift is the Bukhtarminsky four-lift lock with a head of 67 m and 7 m. fluctuations of the upper pool level. (constructed on rock). In the USSR a double-lift lock with a head of 27 m on the Volga at Volgograd has been constructed on soils.

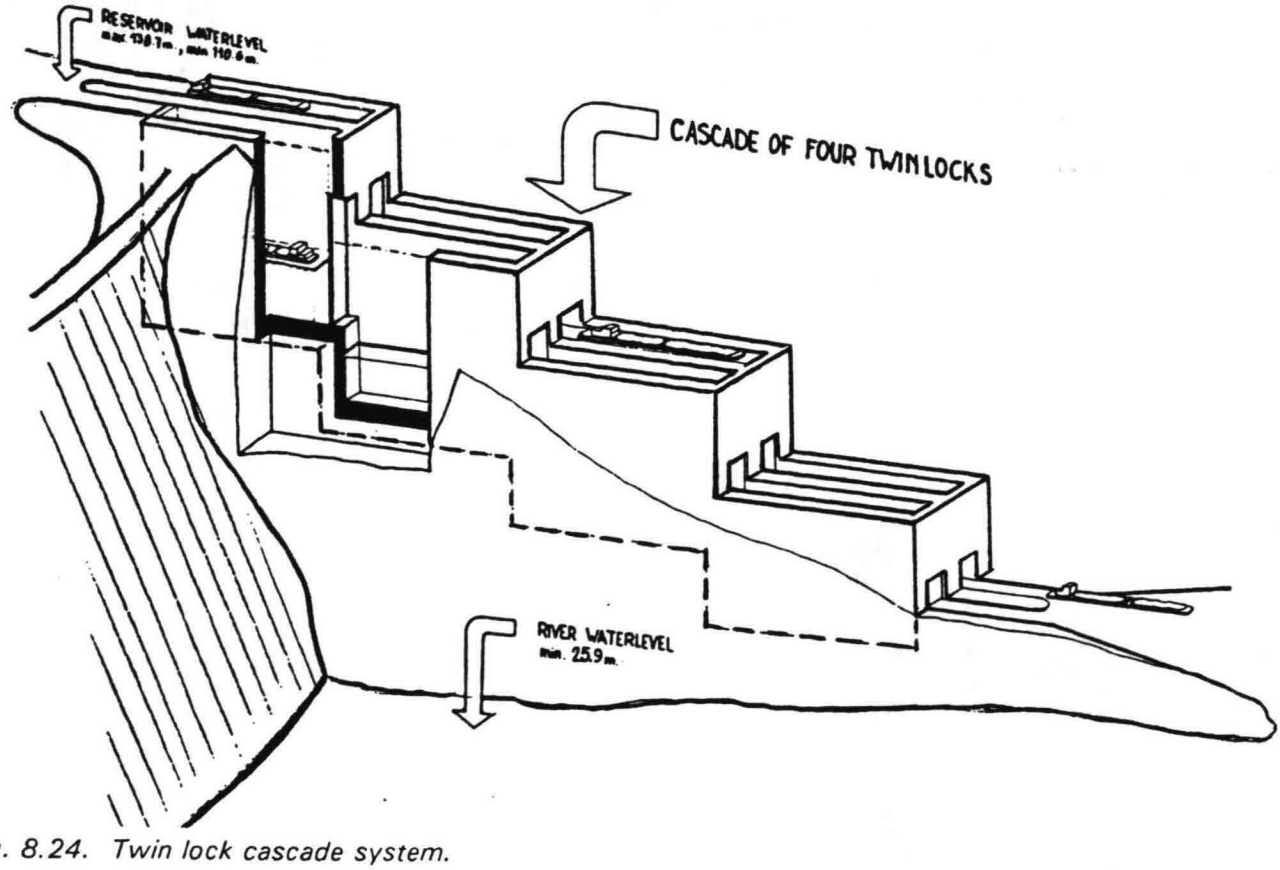


Fig. 8.24. Twin lock cascade system.

8.3. Shiplifts and inclined planes etc. ('boat elevators')

Locks are not the only method of lifting barges. Particularly where water is precious, other methods have to be used. Modern development to overcome difference in waterlevel in inland navigation instead of locks are shiplifts, inclined planes etc. In special cases the lock system, series of locks may become too expensive and other solutions have been sought for. The most important are :

- shiplift
- longitudinal inclined plane
- transverse inclined plane
- water slope system.

In a **vertical lift (shiplift)**, the boat to be raised enters a tank or caisson that has water-tight gates at either end. The gate is closed behind it and tank, water and boat are raised to the upper level. When the tank has reached the correct height, the opposite gate is opened and the boat leaves the tank, passing a second gate that seals the upper reach. One of the earliest lifts, the **Anderton lift** near Norwich, England, was openen in 1875. It has two tanks; whichever is the upper one is overfilled to counterbalance the lower one with the barge inside. Cast-iron weights, working via pulleys, produce the extra lift that raises the lower tank.

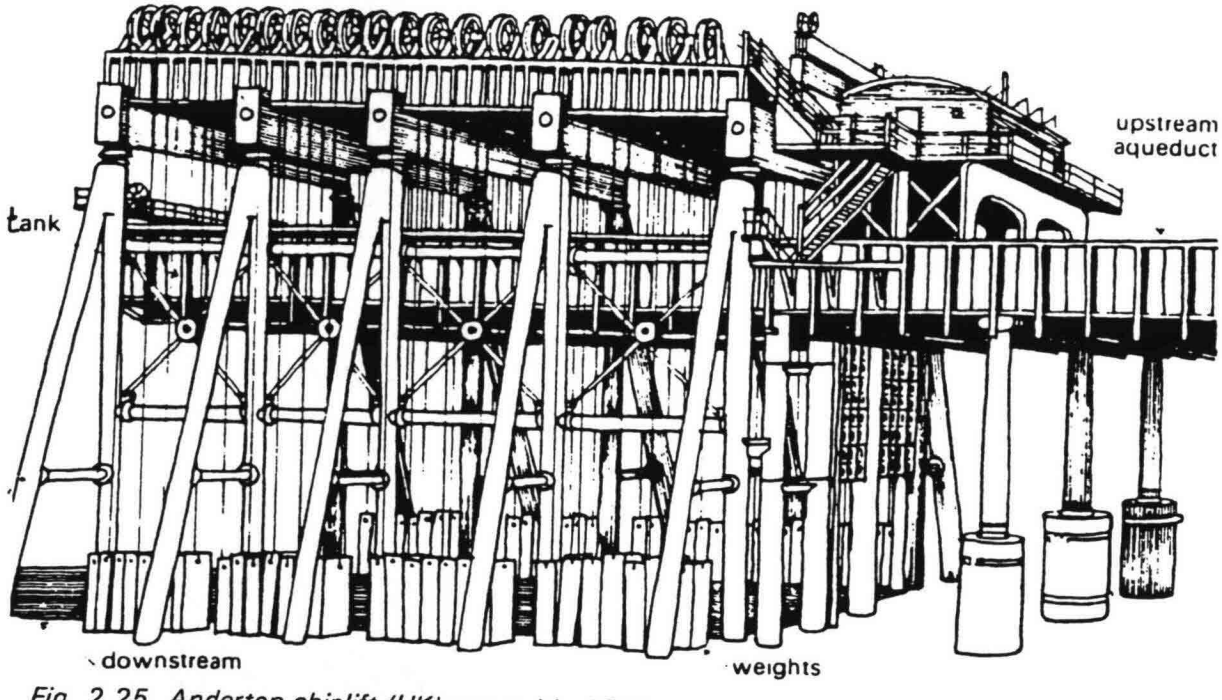
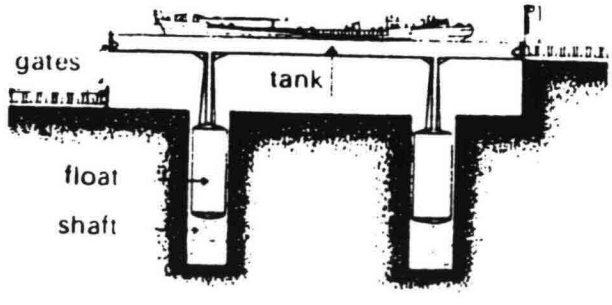


Fig. 2.25. Anderton shiplift (UK) opened in 1875.

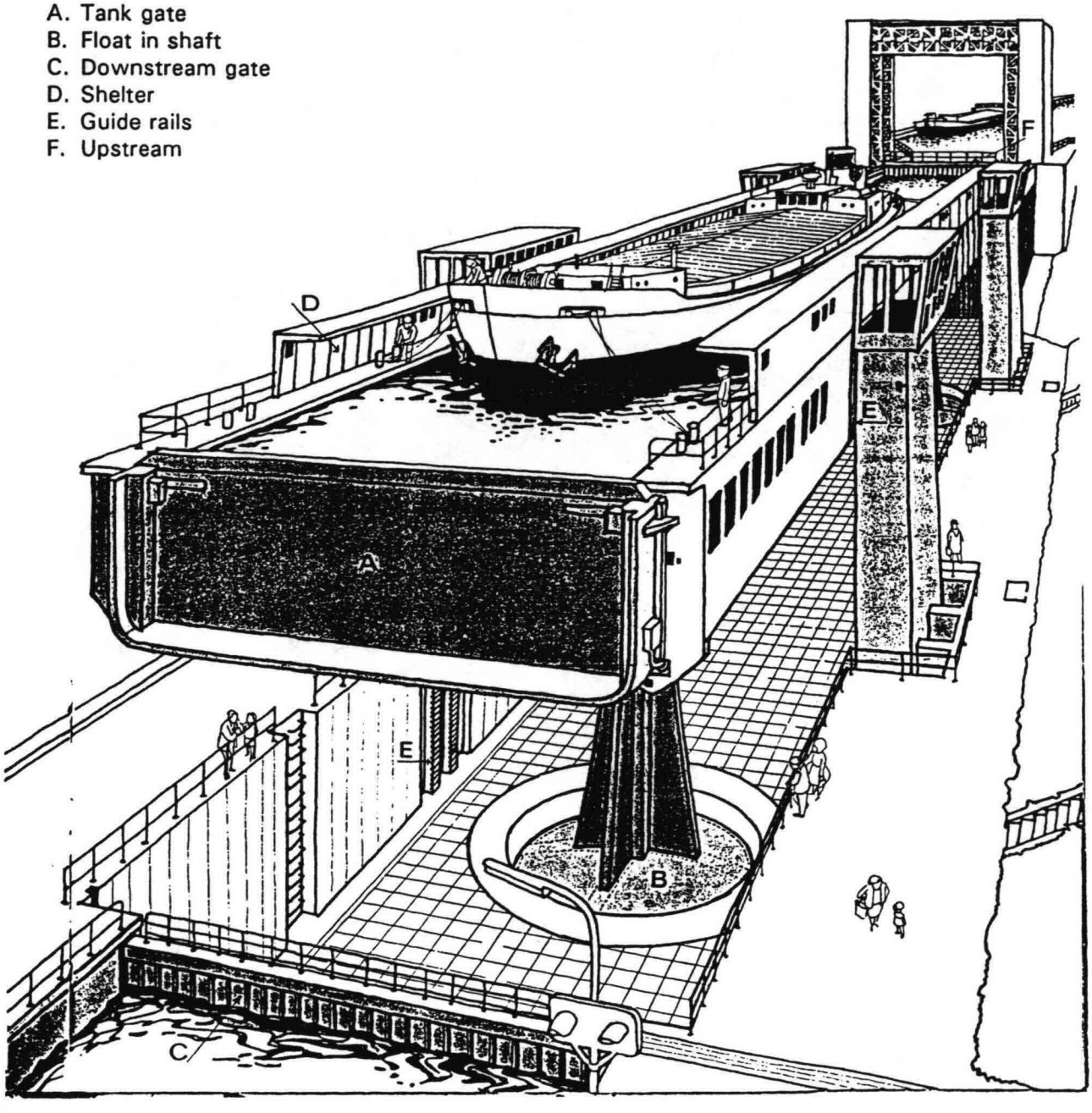
One would expect to see powerful engines, lifting the heavy tanks but in most modern lifts the canal supplies the energy hydraulic-ally. The tank is mounted on floats which are made to move inside shafts sunk deep into the ground. As water from the lower reach flows into the shafts, the floats rise under the pressure and lift the tank with the barge inside. To lower the lift, the water is pumped out of the shafts.



In a **shiplift** the barges or small ships are lifted vertically in a container filled with water. The mechanism to elevate the containers of water is crucial. This mechanism can either be mechanically or hydraulically controlled and driven. The mechanically system is based on counterweights. Shiplifts have been built in **Germany (Luneburg)** for 38 m difference and barges of 1350 tons. At **Strepy-Thieu (Belgium)** a shiplift is under construction in the Canal du Centre for 73 m head and also barges of 1350 tons (expected completion date: 1996). A special sealing arrangement is required with two watertight gates at each side. One gate seals the lift container and the other gate seals the approach channel. Both gates are connected to each other and lifted before the ships can enter.

Fig. 8.26. Large vertical lift on the Dortmund Ems canal at Heinrichenbrug (Germany). It raises and lowers barges of up to 1350 tons over a height of 14 metres. Its tank measures 90 m by 12 m and can take barges with a draught of 3 m.

A. Tank gate
B. Float in shaft
C. Downstream gate
D. Shelter
E. Guide rails
F. Upstream



Twin Ship Elevator - Lüneburg

The Twin Ship Elevator Lüneburg has been built in the years 1969 to 1975 as a part of the new Elbe Lateral Canal (115 km long). This canal connects Hamburg, the biggest sea harbour of Germany to the inland waterway network through the Mittelland Canal and partially shortens considerably the communications to Berlin, and Czechoslovakia through the Elbe as well as to the industrial area around Salzgitter and the Ruhr District. Between the Elbe and the Mittelland Canal a total difference of levels of 61 m has to be negotiated by means of two hydraulic structures, a twin ship elevator with 38 m height of lift near Lüneburg and a lock near Uelzen with 23 m height of lift. In the elevator near Lüneburg the ships are transported in two steel chambers filled with water, which can be moved independently from each other. The weight of the steel chambers including contents is compensated by counterweights in each of the four guide towers.

The chambers move up and down on toothed racks. In case of failure of balance between chamber and counterweights the load sets on the stationary spindles provided in the four towers. The upper and lower heads form the transition between canal and elevator. They contain the reach gates which allow together with the chamber gates to separate canal and chamber without water losses worth mentioning.

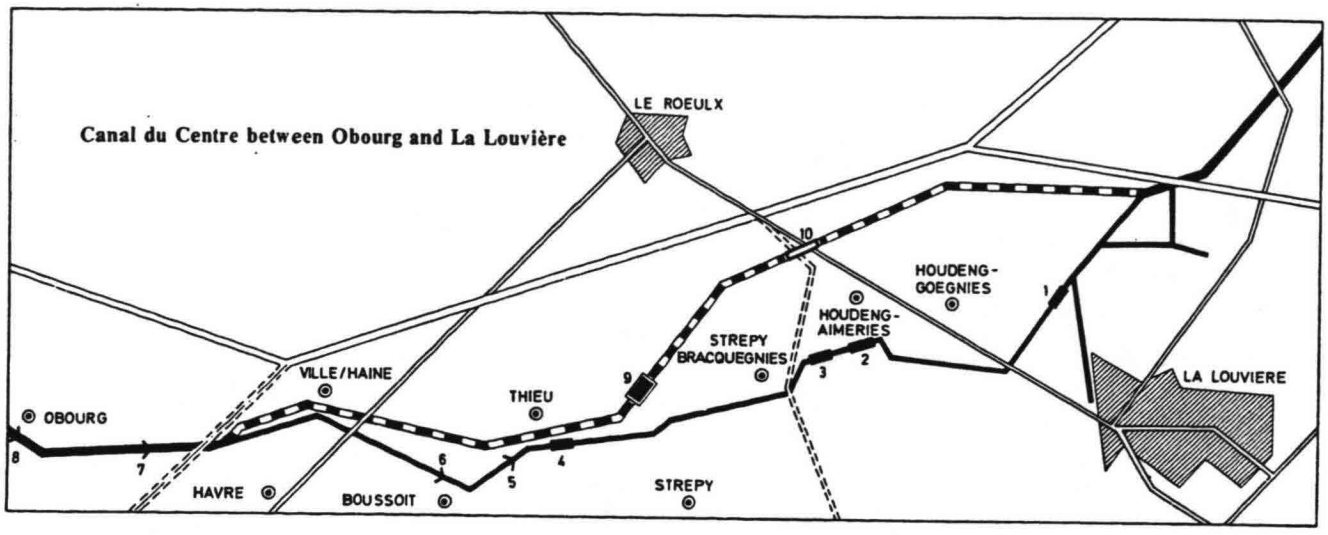
All movements of the twin ship elevator run off fully automatically. Operation of the entire construction and traffic control are made from a central control stand. As a vertical elevator it is presently the world's biggest twin ship elevator.

Fig. 8.27.
Longitudinal Section of Elevator (heights above datum referenced to datum level).

Shiplift in 'Canal du Centre', Belgium.

The world's largest ship lift - 110 m high, which will overcome a height difference of over 73 m in the 'Canal du Centre', is under construction near Strepy in Belgium. The shiplift will operate as follows. The vessel, including the crew, is piloted into one of the two water basins suspended on thick cables at the base of the structure. Ship, crew and basin are then winched upwards at a rate of 20 cm/s to the level of the 'Canal de Centre' 73.15 m above.

The basins, each 112 m long, 12 m wide and up to 4.15 m deep, are balanced with counterweights and can move independently, so that one of the two facilities will always be available even while maintenance work is going on. The two basins will be installed on either side of the central concrete section. A metal aqueduct will link the top of the lift with the canal. The building of the lift has also made it necessary to dig a new canal section. 19 km long.



Until 1963, the Canal du Centre which stretches from La Louvière to Mons over a length of 18.6 km had a 300 t capacity The unique feature of this canal was that it overcomes a 68 metre rise by using four hydraulic lifts of 17 metres rise each.
These lifts are succintly described in the following chapters. The work to enlarge the capacity to 1 350 t began in 1963.
Just as the engineers a century ago studied the Canal du Centre for a 300 t capacity, the public administration engineers were asked to modernize to 1 350 t and had to confront the problem of overcoming a 73 metre rise at Strépy-Thieu.

Several solutions were examined in order to solve this problem and they were compared from both technical and economic aspects:
- two lifts, each with a 36.50 metre rise;
- a 73 metre rise lift;
- an inclined plane with a 5% slope;
- an inclined plane with a 10% slope;
- a water slope of 3.5%.

The 73 metre rise lift, overcoming the rise in one step, was the solution which was chosen. It is described in the following chapters.

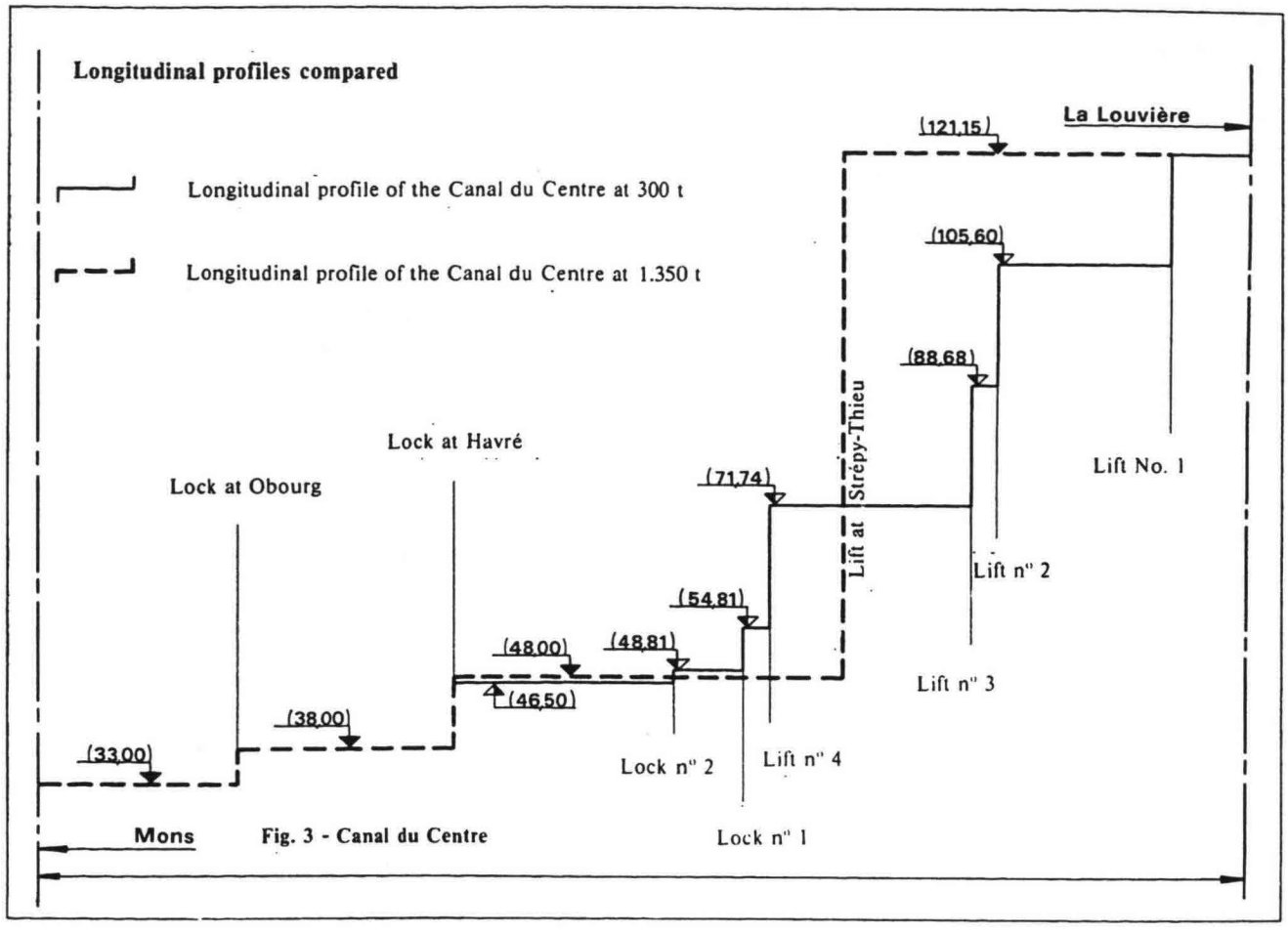
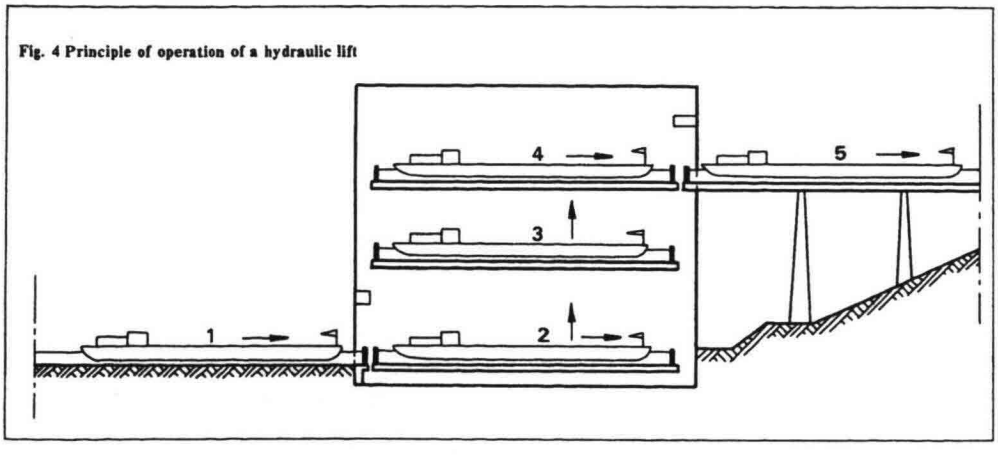


Fig. 3 - Canal du Centre



The series of operations used to overcome a rise by using a boat lift is shown in the plan in Figure 4:

1) The cage is stabilised in the lower position and the watertight joint between the cage and the pound is applied; the downstream gate of the cage and that of the downstream pound are raised; the boat enters into the cage;

2) the gates mentioned above are lowered, the water of the area between the gates is drained; the watertight joint is removed; the cage is released;

3) the cage is brought to the upper position;

4) the cage is secured and the upstream watertight joint is applied; the gate upstream of the cage and that of the upstream pound are simultaneously raised in a two step operation. In the first step, the gates are detached and slightly raised which fills the area between the gates and equalizes the pound and cage levels. In the second step, the gates are completely raised so the vessels can pass through.

5) The boat leaves the cage and heads to the upstream pond passing the canal bridge. For safety reasons, the boat lifts usually have two cages (twin caissons).

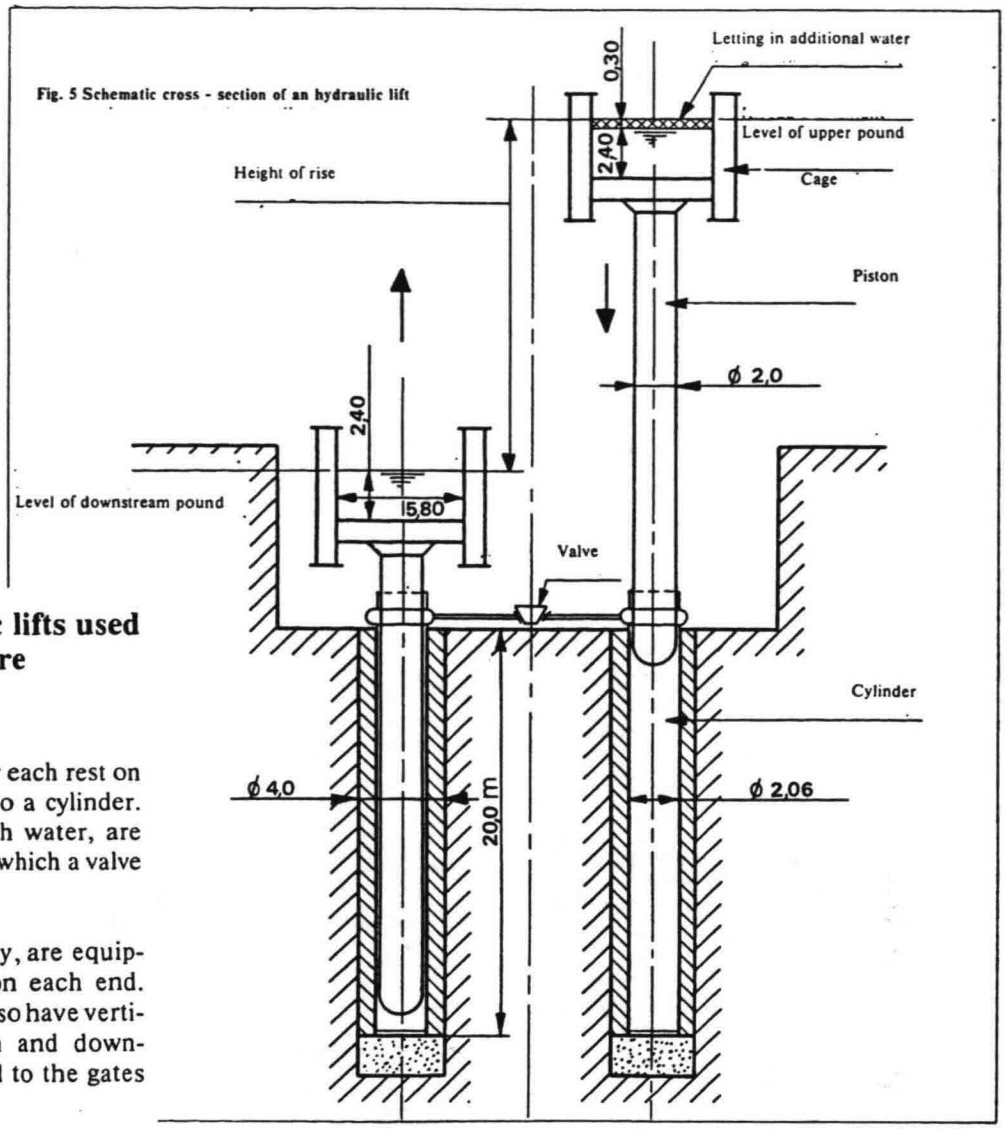


Fig. 5 Schematic cross - section of an hydraulic lift

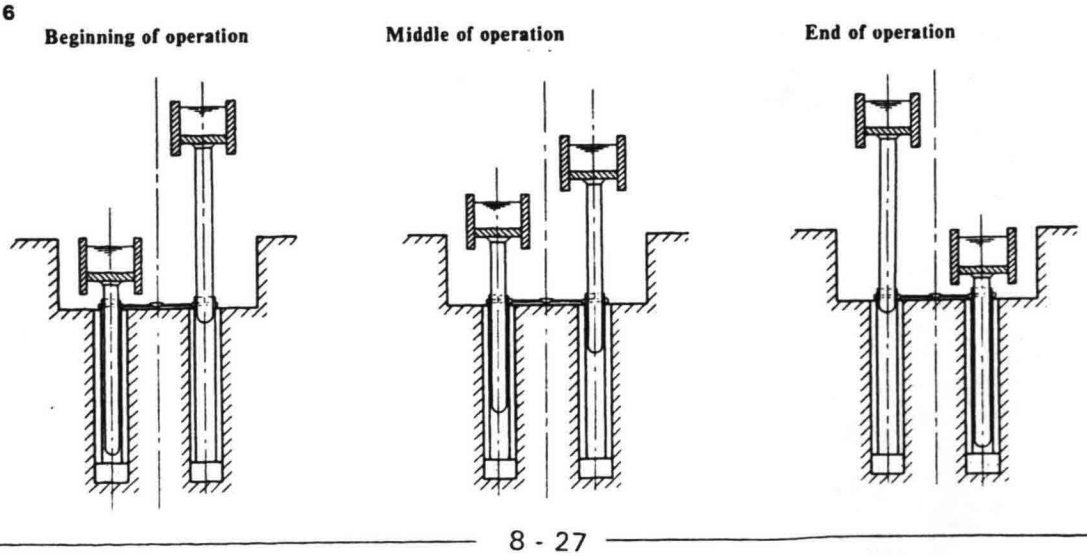
The former hydraulic lifts used in the Canal du Centre

Two cages filled with water each rest on a piston which extends into a cylinder. These cylinders, filled with water, are connected by a pipeline in which a valve is installed. (Fig. 5).

The cages, guided laterally, are equipped with vertical gates on each end. The pounds of the canal also have vertical gates, both upstream and downstream, which correspond to the gates of the cages.

If the cages, which are identical, are each filled with the same level of water, their masses would be the same (about 1 000 t); if the central valve is opened, the two cages would stop at the same height, the average between the level of the downstream pound and the upstream pound.

Fig. 6



To overcome the rise completely, an additional amount of water is added to the descending cage (the right cage in the illustration): 30 cm or 75 t of water) (Fig. 6). This additional water puts the two cages out of balance, just like the plates of a scale when one plate is loaded more heavily than the other. This imbalance brings the right cage into a lower position and the left cage in the upper position. The 75 t additional weight represent the power necessary to overcome the entire rise, this manoeuver is called balancing.

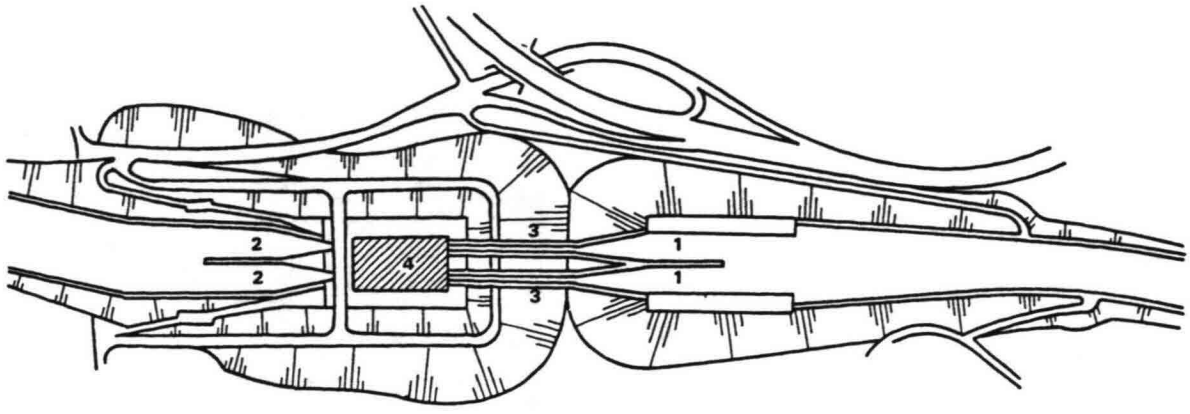
The valve is closed when the balancing is completed. The additional 75 t of water is released into the downstream pound as soon as the gates are opened; this represents the minimum water consumption of the system.
This hydraulic lift operates only with hydraulic energy.

Some general characteristics:	
Length of a cage	43.00 m
Width between lateral walls	5.80 m
Height of the cage	3.15 m
Height of the water without addition	2.40 m
Including additional water	2.70 m
With boat-Spits type (300 t)	38.50 m x 5.00 m x 1.90 m
Masses of the frame of a cage	396 t
Masses of 2.40 m water in cage	599 t
Masses of 2.70 m water in cage	674 t
Masses of a piston	80 t
Total weight of cage (frame + piston + water): 2.40 m	1 075 t
2.70 m	1 150 t
Piston diameter	2.00 m
Cylinder diameter	2.06 m
Duration of balancing of cages	2 min 45 sec
Average speed	0.10m/sec
Duration of manœuver (entry and exit of boats included)	15 minutes

	Rise	Date put into operation
Lift No 1 at Houdeng-Goegnies	15.40 m	1888
Lift No 2 at Houdeng-Aimeries	16.934 m	1919
Lift No 3 at Strépy-Bracquegnies	16.933 m	1919
Lift No 4 at Thieu	16.933 m	1919

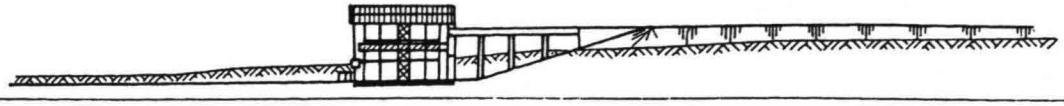
Fig. 8

Overview



1. Downstream garage 2. Upstream garage 3. Canal-bridges 4. Lift

Longitudinal profile



THE LIFT AT STREPY-THIEU

The work as a whole

This work is part of a serie of works (Fig. 8) which include from upstream to downstream:

- a section of rockfill canal 800 m long, including safety installations for the immediate isolation in case of dike rupture;
- the upstream garage 200 m;
- two canal-bridges linking the upstreams garage to the two lift cages;
- the lift itself;
- the downstream garage 200 m;

The works are to permit the passage of:

- a self-propelled boat of 1 350 t (80 m x 9.50 m x 2.50 m)
- or a 2 000 t barge (76.50 m x 11.40 m) with a pushtow 20 to 23 m long;
- two 600 t ««kast campinois»» (50 m x 6.60 m x 2.50 m) accompanied by two 300 t spits (38 m x 5.00 m x 2.20 m);
- or 4 spits.

To meet these conditions, the useful dimensions of the cage are 112 m x 12 m.

In the upstream pound, the theoretical floating level is 121.15 m. This level can exceptionally be brought down to 120.60. The upstream level can vary between 120.60 and 121.40.

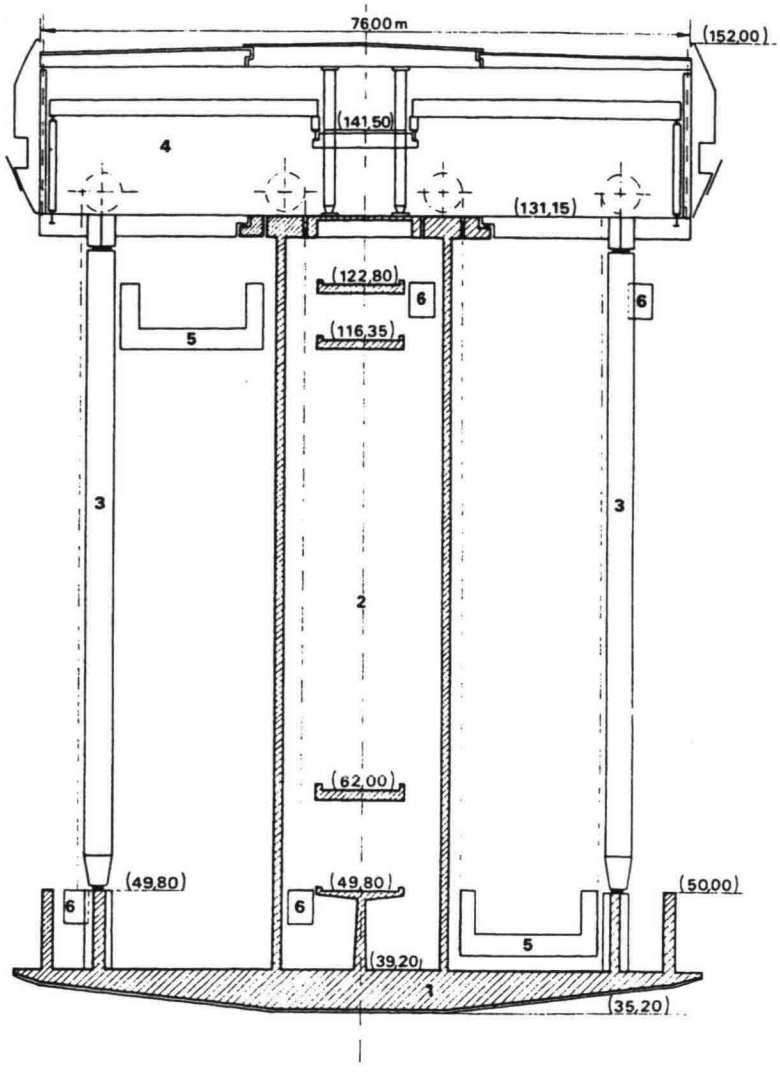
In the downstream pound, the theoretical floating level is 48.00 m. There are fluctuations of +/- 0.20 m which are caused by the supply of the downstream pound by the pumping station at the Havré lock and waves of +/- 0.20 m caused by the use of this lock. The downstream floating level can thus vary between 47.60 m and 48.40 m.

The rise is between 72.2 m and 73.80 m

The duration of the cycle of the work was chosen to act in harmony with tha of the adjacent works. One cycle inclu des: the entry of the vessels, carrying them across, the exit of the vessels, and the same operations in the othe direction.

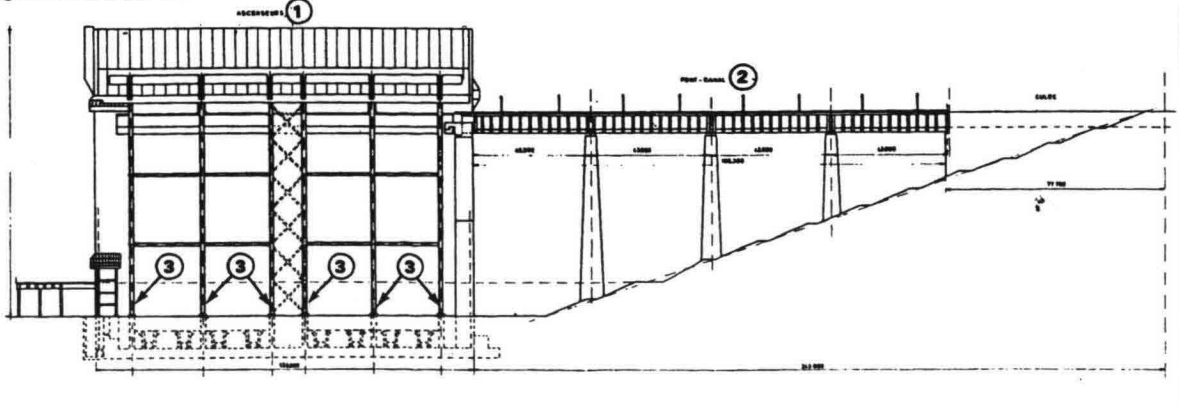
The duration of the cycle of the close work, the Havré lock, being 60 minute and the work at Strépy-Thieu was dou bled, a 120 minute cycle could hav been chosen. However, to limit th stoppage of navigation in case one o the doubled works was not operatin an 80 minute cycle was chosen, or duration of 60 minutes plus 30%.

Fig. 10 a Cross section of lift



1. Lining
2. Central Tower
3. Metal columns
4. Machine room
5. Cage
6. Counterweight

Fig. 10 b Elevated view



1. Lift
2. Canal bridge
3. Metal columns

Fig. 16 Perspective view

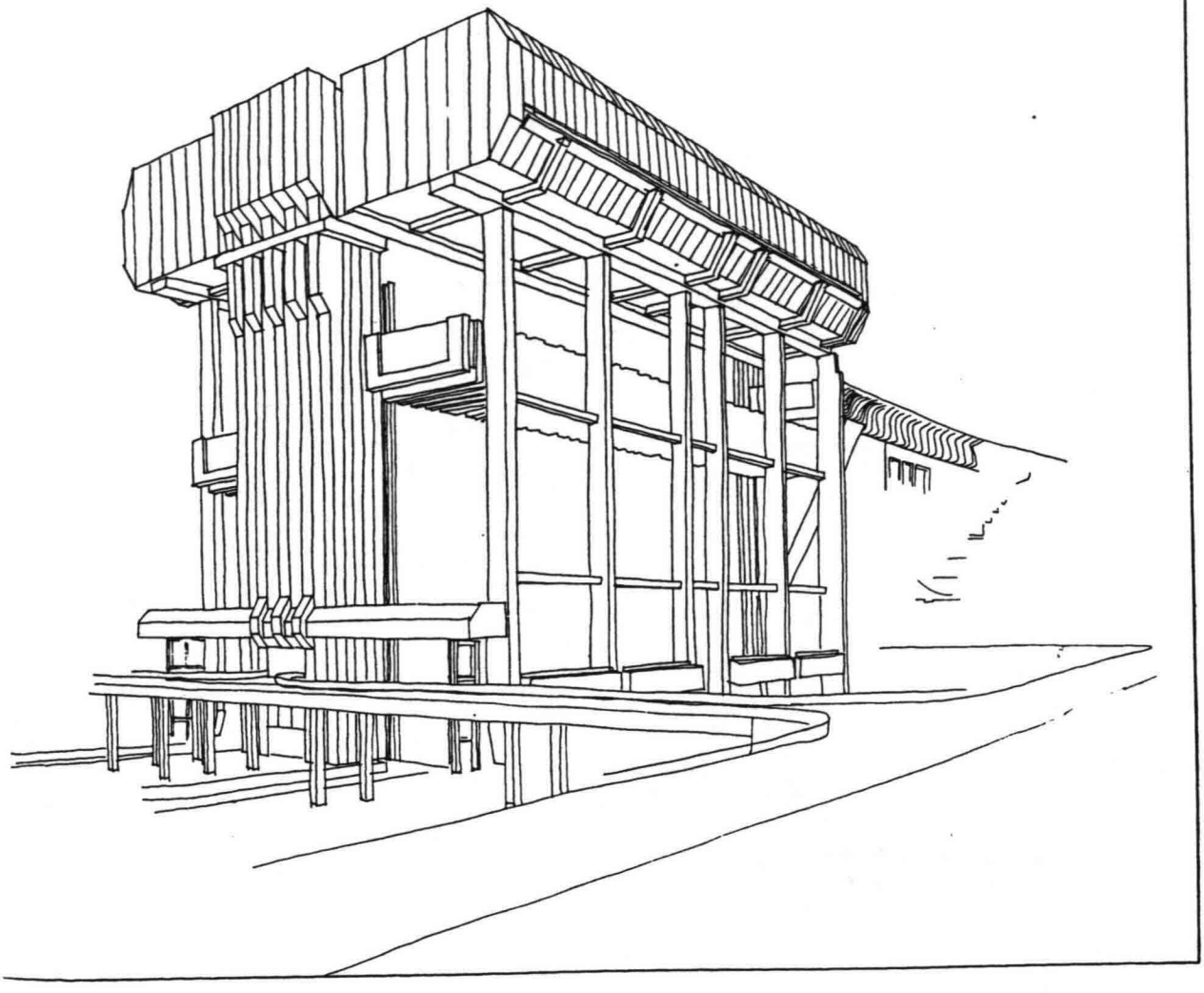
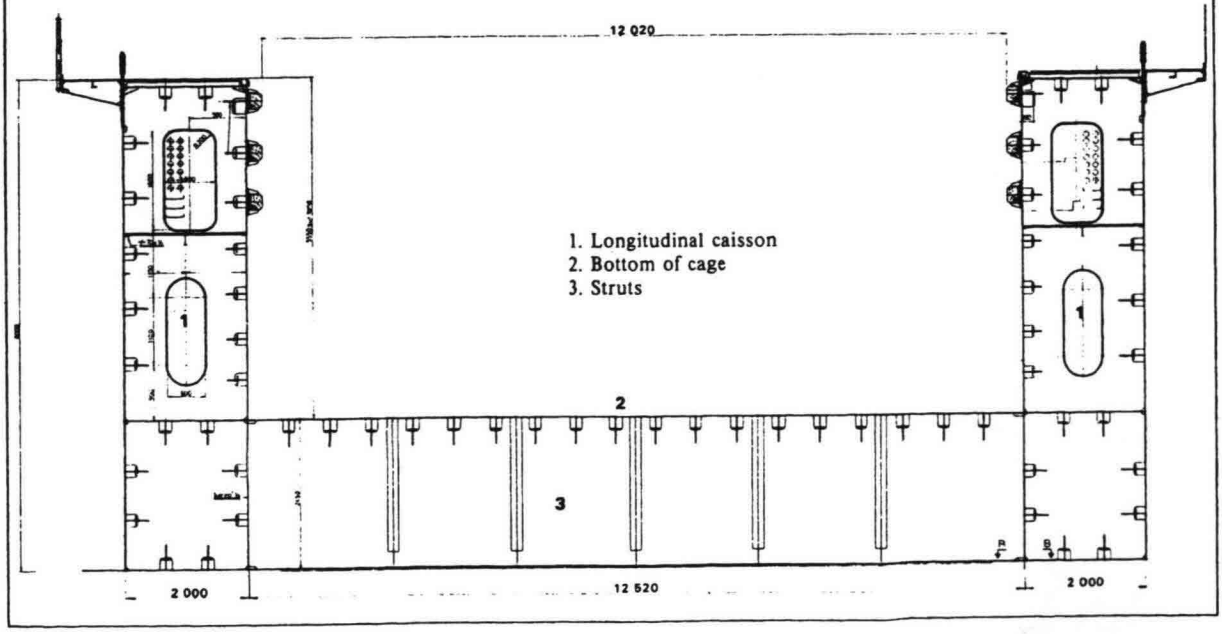


Fig. 11 Cross section in a cage



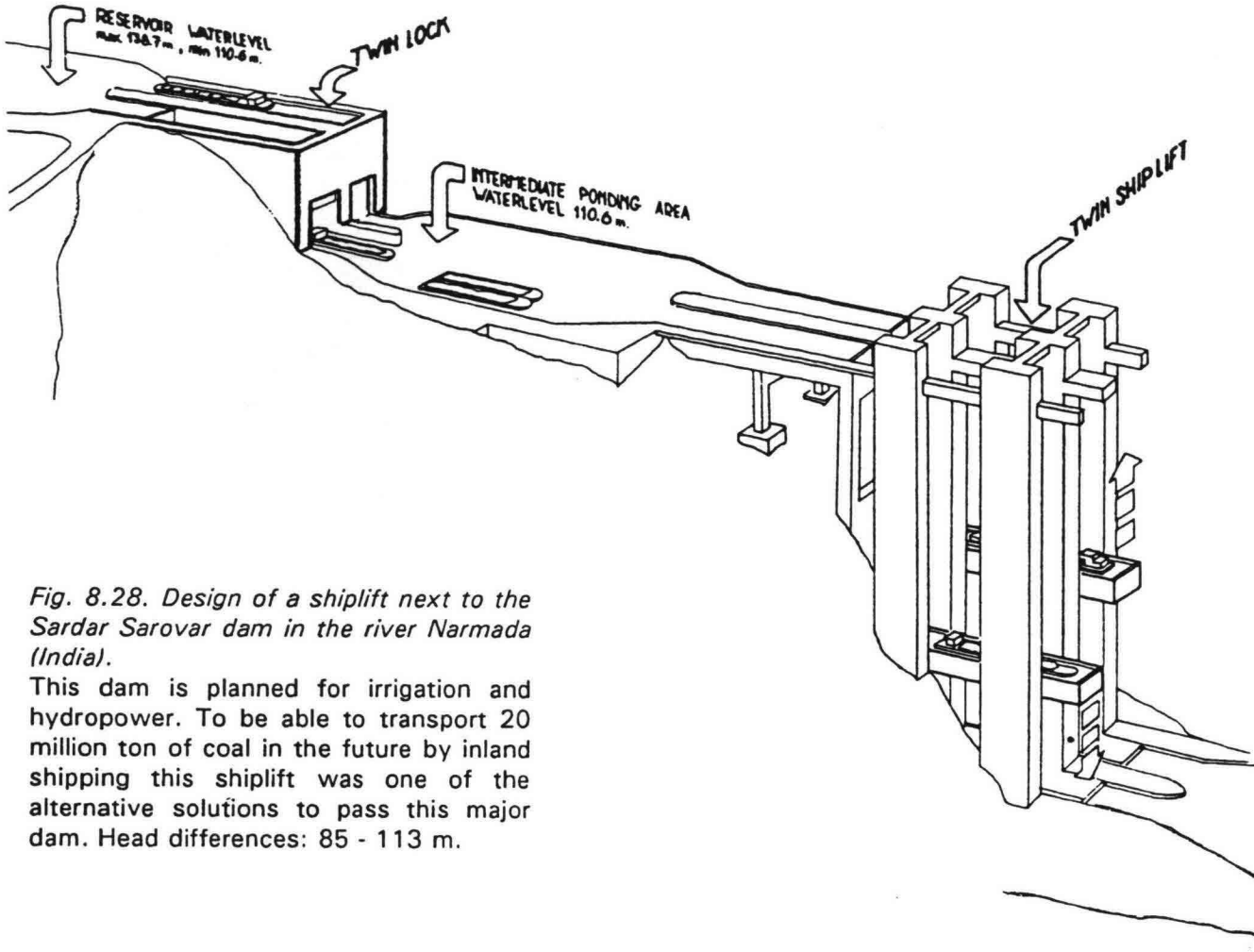


Fig. 8.28. Design of a shiplift next to the Sardar Sarovar dam in the river Narmada (India).
This dam is planned for irrigation and hydropower. To be able to transport 20 million ton of coal in the future by inland shipping this shiplift was one of the alternative solutions to pass this major dam. Head differences: 85 - 113 m.

Longitudinal inclined planes

The container (with water + ship) is moved along a slope in a longitudinal sense. In **Belgium (Ronquieres)** (1968) a mechanical longitudinal inclined plane has been built with a head of 68 m, that is able to transport barges of 1350 tons. It consists of two parallel planes on which containers (87 x 12 m, with a water depth of 3.70 m) on wheels are operated by means of enormous counterweights (also on wheels). Slope 5% - weight container : 5.5 x 10^4 kN. It takes 22 minutes to cover the 1432 m ramp.

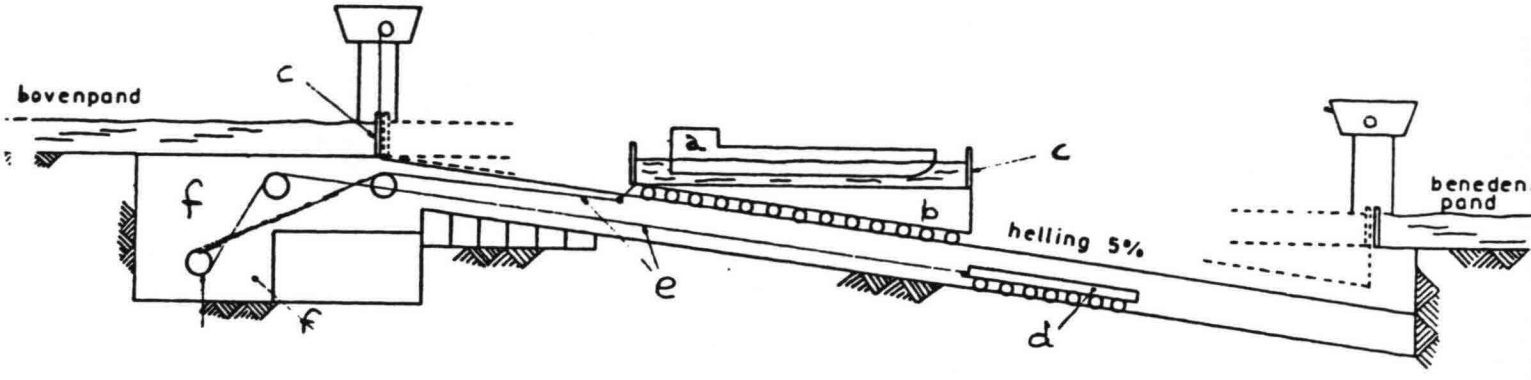


Fig. 8.30. Inclined plane at Ronquieres (Belgium).
a. barge; b. container; c. door; d. counterweight; e. cables; f. engine room; g. lowerhead;
h. upperhead.

In **Russia (Krasnojarsk** - 1976) the container is operated by the own propulsion of the wheels. Head: 102 m; slope: 1 : 20; containers: 90 x 18 x 3.30 m.

Transverse inclined plane

The container is moved along a slope in transversal sense. This system is based on the same principle as the longitudinal inclined plane with the exception of the direction in which the ships are lifted. The slope is steeper because the container weight with barge/ship is moved in a transverse direction.

The largest transversal inclined plane is built at **Arzviller (France)** (1969) in the Marne-Rhine Canal, with a head of 45 m and a capacity of barges/ships up to 350 tons. The dimensions of the containers are 41 x 5.2 m, with an available waterdepth of 2.5 m. Total weight: 9000 kN. Slope: 41 %

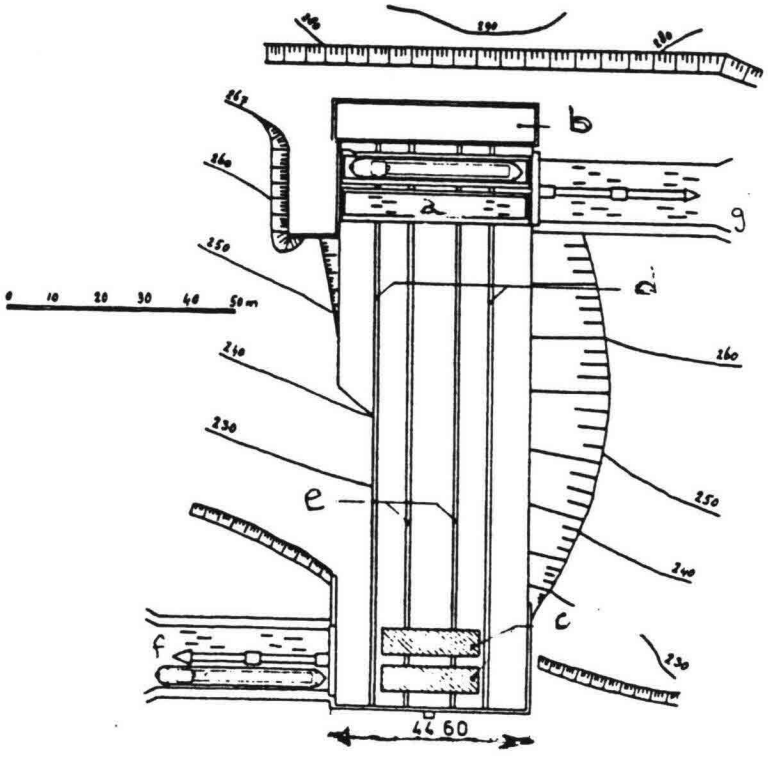


Fig. 8.31. Planview of the inclined plane at Arzviller (France).

A. Barge
B. Engine room
C. Counterweights
D. Conveyor track container
E. Conveyor track counterweights
F. Lower head
G. Upper head.

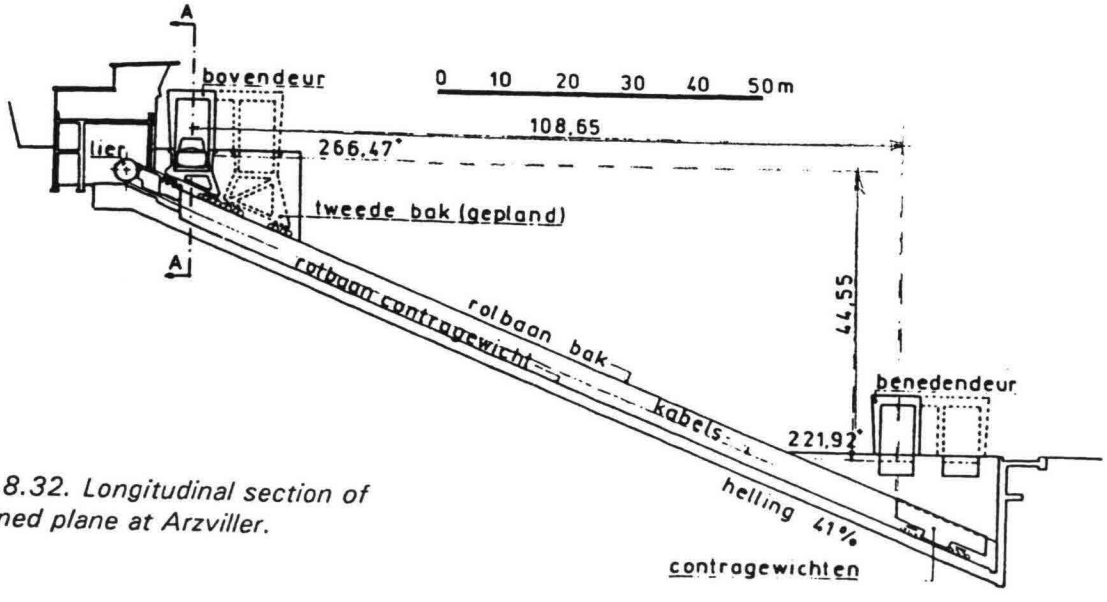


Fig. 8.32. Longitudinal section of inclined plane at Arzviller.

Water slope system (Pente d'eau)

A locomotive pushes a shield with a wedge of water behind it on an inclined plane. There exists only two water slope systems in the world :

Montech, lateral canal of the Garonne, France (1974) and

Fonserannes, Canal du Midi, France (1984).

Ships up to 350 tons, slope: 3 %, head: 13.5 m. The problem to make the shield watertight have been solved by using rols of neopreen that are pressed against the wall with a jack-system.

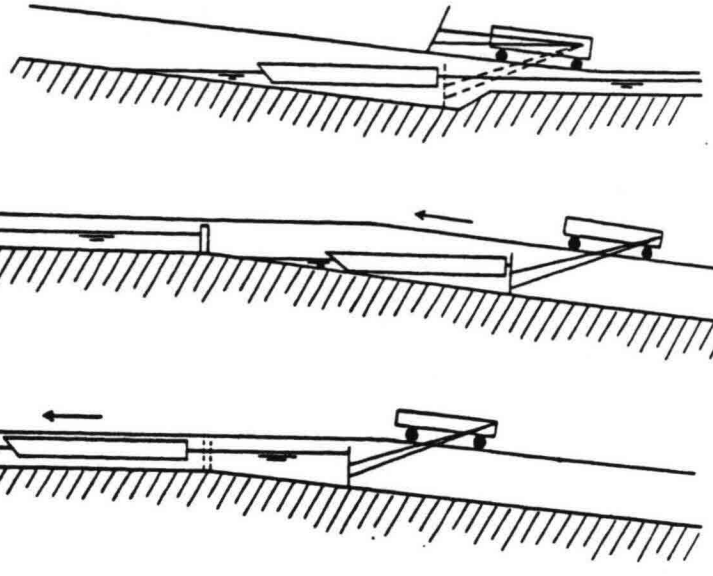
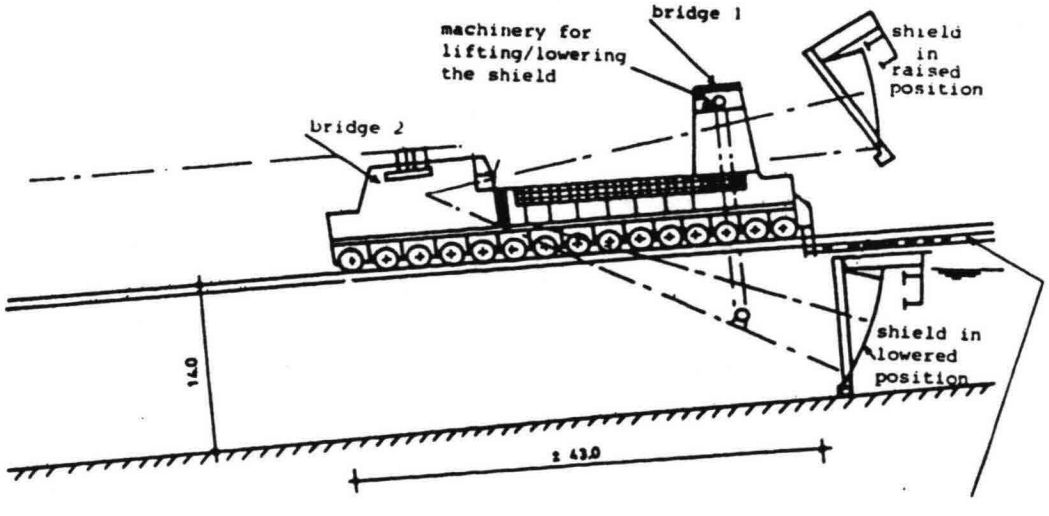


Fig. 8.33. Functioning of the long water slope system.



9. Barriers

The risk of serious flooding from tidal surges penetrating inland via estuaries and tidal inlets has led to several major schemes for tide-excluding barriers. These are in the form of gates, perhaps single gates for schemes of modest size but multiple gates for major estuaries.

Navigation is often the controlling feature determining the necessary span, the elevation of the sill and the clearance height under any structure spanning over the waterway.

Storm surge barriers are built nowadays in order to control dike raises. Under normal circumstances the gates are open but during storms when high water levels might be expected they are closed and form part of the protecting dike system.

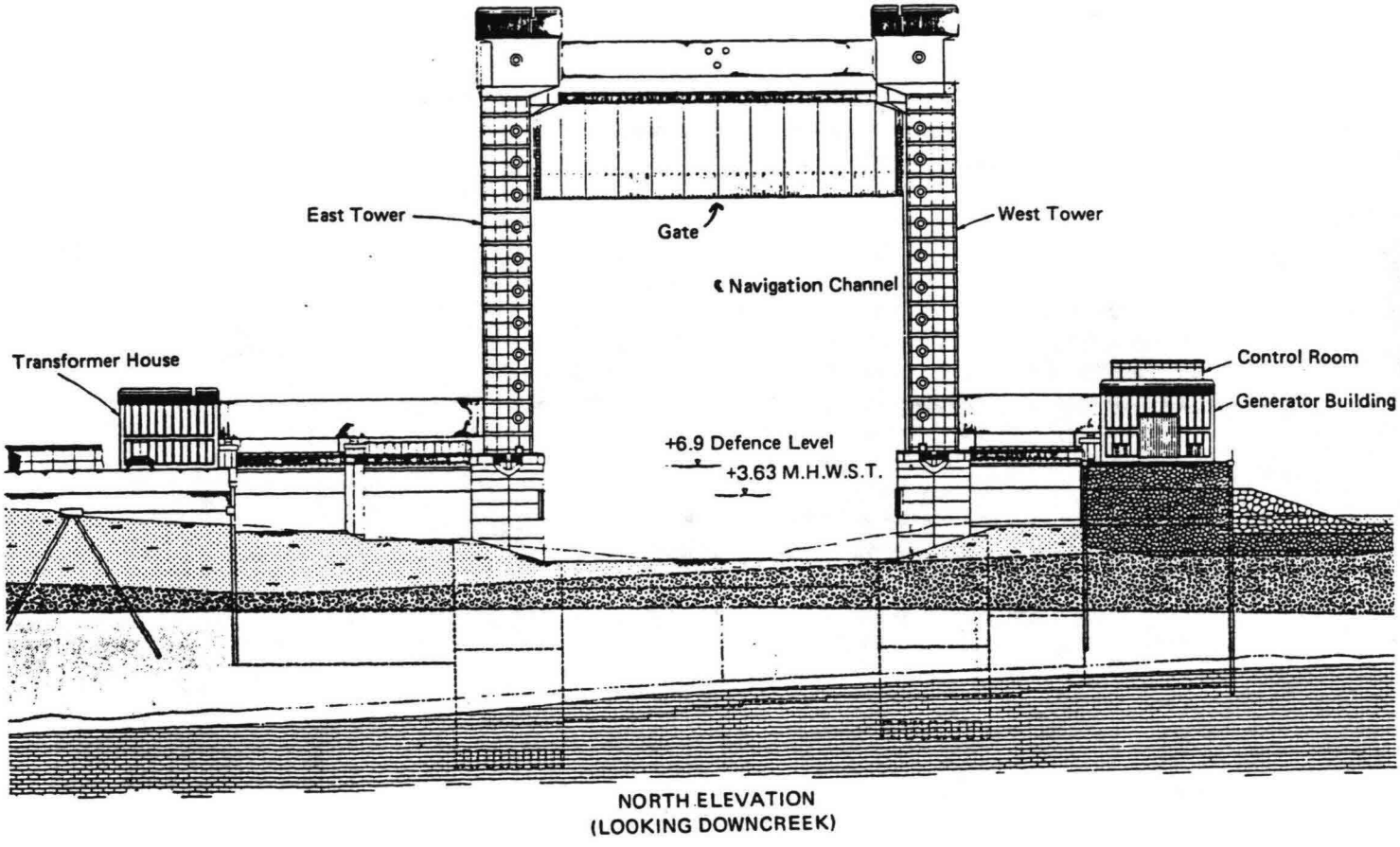


Fig. 9.1. Tidal barrier, Barking Creek (U.K.). Very large vertical lift gates have been used as tidal barriers as for example at Barking Creek in the Thames estuary. The gate normally rest at the top of its support towers, thus providing clearance for navigation by medium-sized ships.

Storm surge barrier - Krimpen a/d IJssel (The Netherlands)

A storm surge barrier was built at Krimpen to protect one of the lowest lying and most densely populated areas of The Netherlands. Under normal conditions, the gates of the barrier, which was completed in 1958, are raised 12 metres above the water level, so that there is no obstruction to shipping. If the tide rises to a dangerously high level, the gates can be lowered to dam the river. A lock allows ships to travel up or downstream when the barrier is closed.

The gates are about 12 m high, 80 m wide, and weigh 653 m. The two sliding gates are positioned between two pairs of towers. Counterweights in the towers keep the sliding gates in position. Two electro-engines, each 25 hp, can lower the gates with a velocity of 2 cm per second. The first gate was installed in 1958, the second in 1976. for the second gate better steel quality was used and a sharp underside was provided to reduce vibrations during lowering in flowing water. The 2nd gate will be closed first; the first gate acts as reserve (since 1976).

This structure could also be used for a bridge to the Krimpenerwaard.
The navigation lock at the west side of the river 'Hollandse IJssel' has a chamber length of 120 m and a width of 24 m.

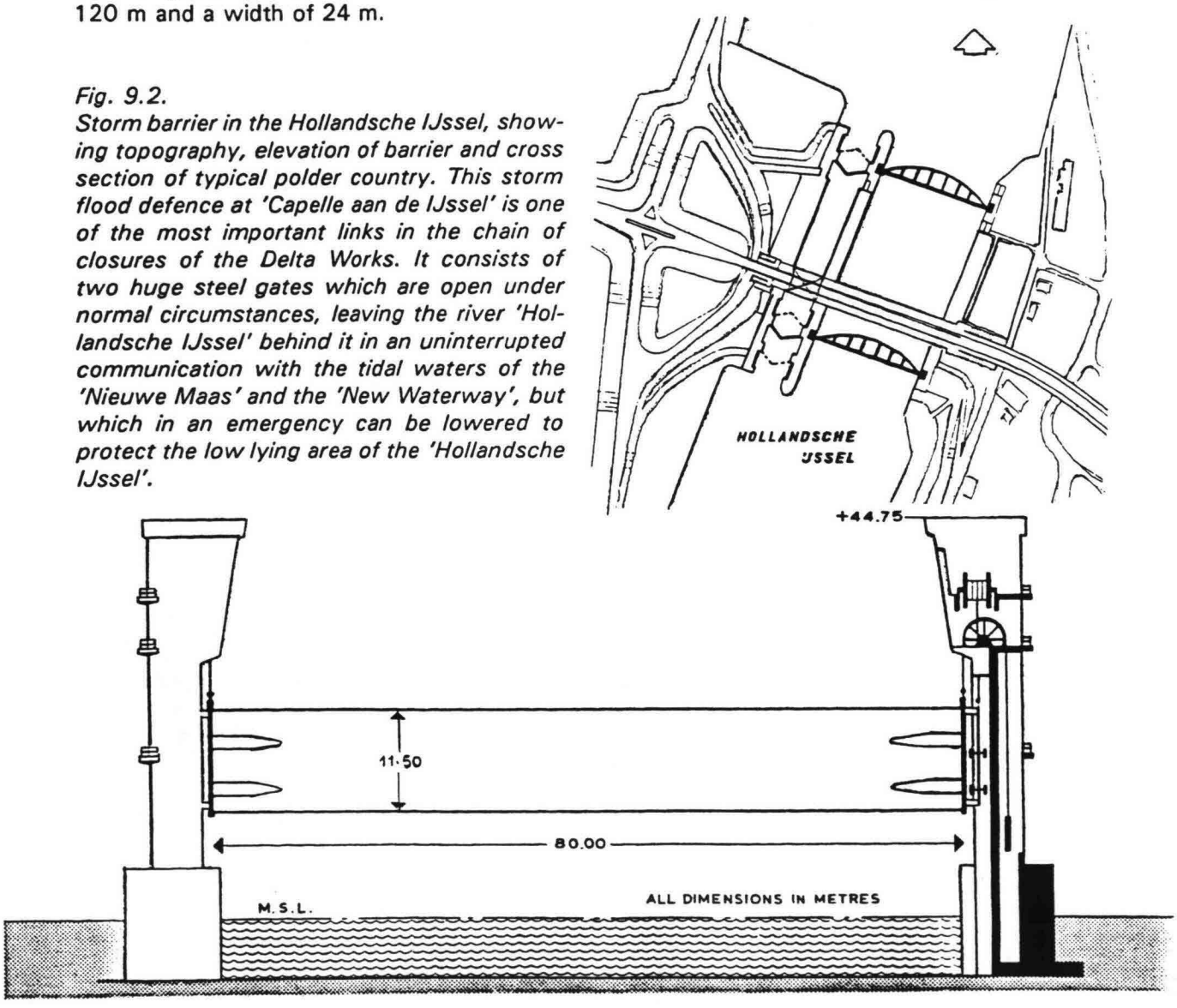
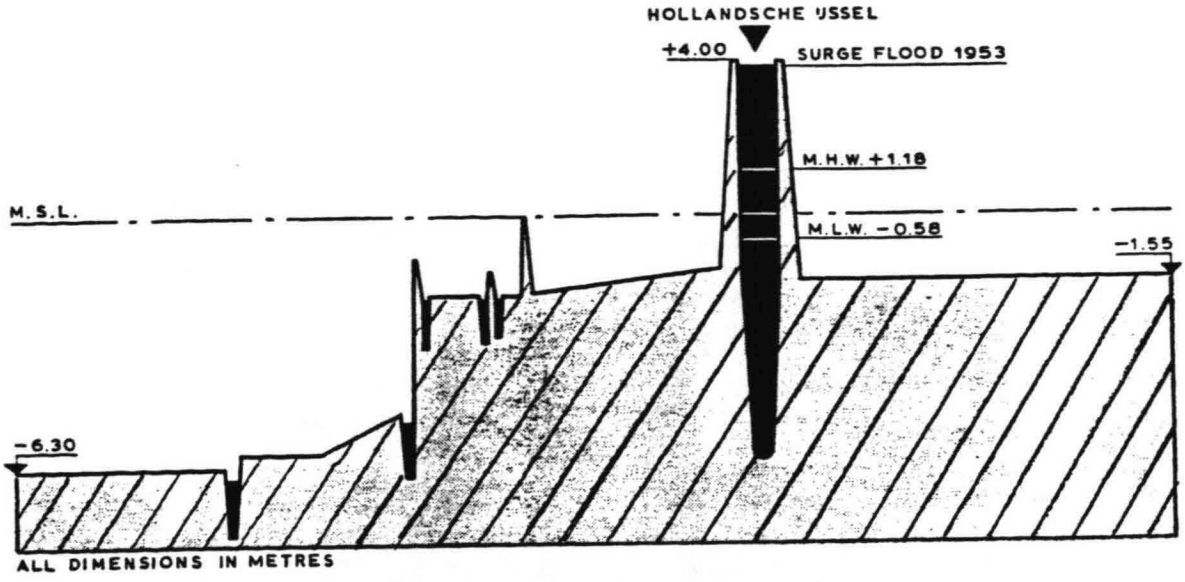
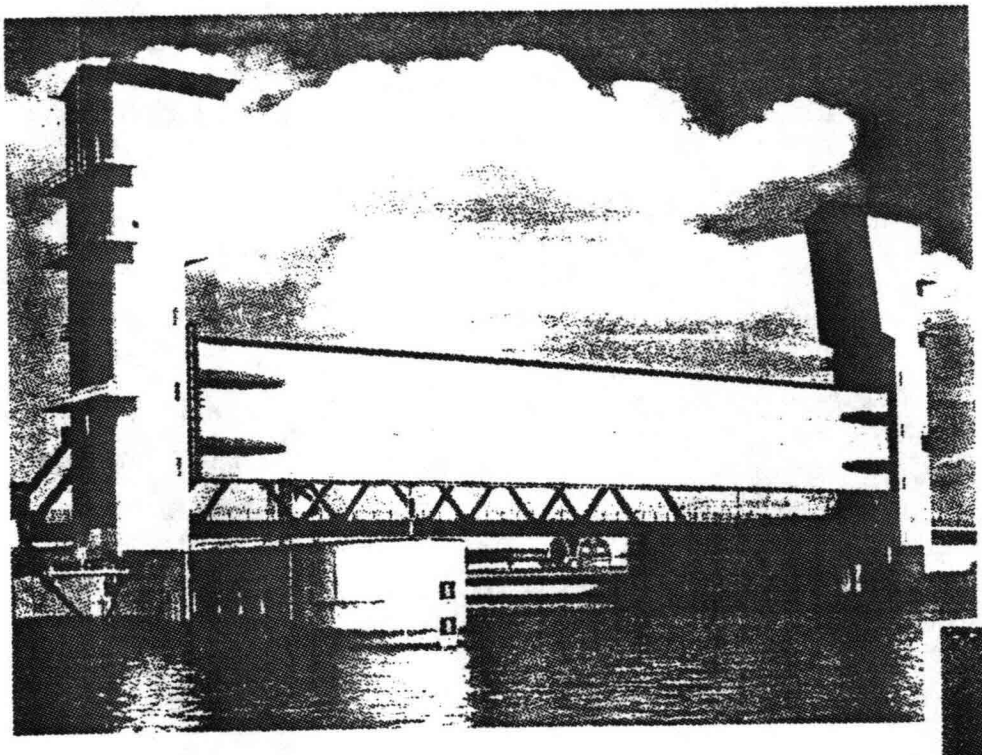


Fig. 9.2.
Storm barrier in the Hollandsche IJssel, showing topography, elevation of barrier and cross section of typical polder country. This storm flood defence at 'Capelle aan de IJssel' is one of the most important links in the chain of closures of the Delta Works. It consists of two huge steel gates which are open under normal circumstances, leaving the river 'Hollandsche IJssel' behind it in an uninterrupted communication with the tidal waters of the 'Nieuwe Maas' and the 'New Waterway', but which in an emergency can be lowered to protect the low lying area of the 'Hollandsche IJssel'.





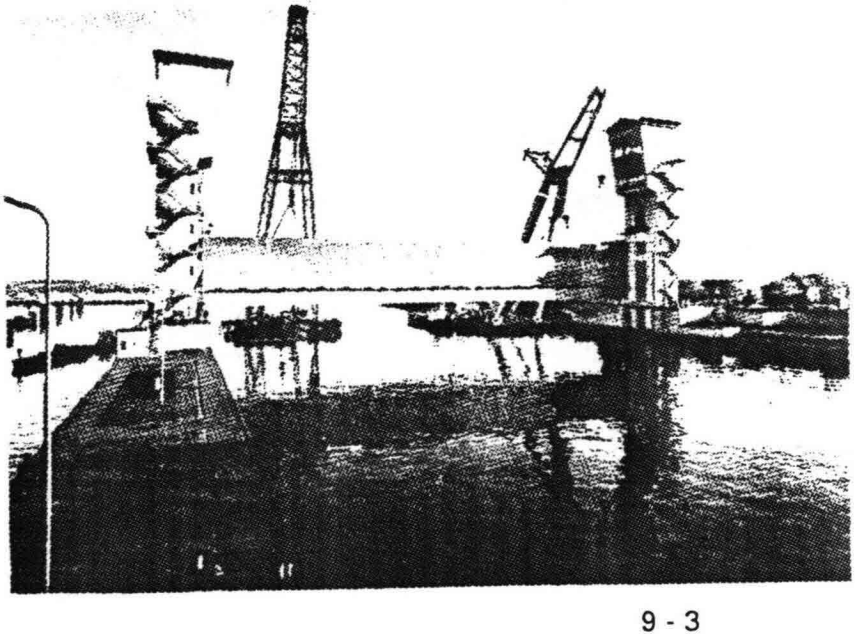


Fig. 9.3. Storm surge barrier in the Hollandsche IJssel; normally open, closed under unfavourable conditions. (Storm, high water, etc.).

Thames Barrier. A structure that is capable off closing off the Thames estuary at high flood and so preventing the sea surging inland to flood the London area. The main gates of the Thames Barrier are of the rising sector type gates. This type does not require a high supporting structure because they normally rest below the bed of the navigation channel. Their operating mechanism is such that they can be rotated through 180 degrees from their normal level in the sills on the estuary bed to raise them above water level for maintenance. They turn through 90 degrees to close the barrier against the tide. The rising sector gates of the four main spans of the Thames Barrier have a span of 61 m and effectively they form box-girders between their end wheels. There are six subsidiary gates with spans of 31.5 m.

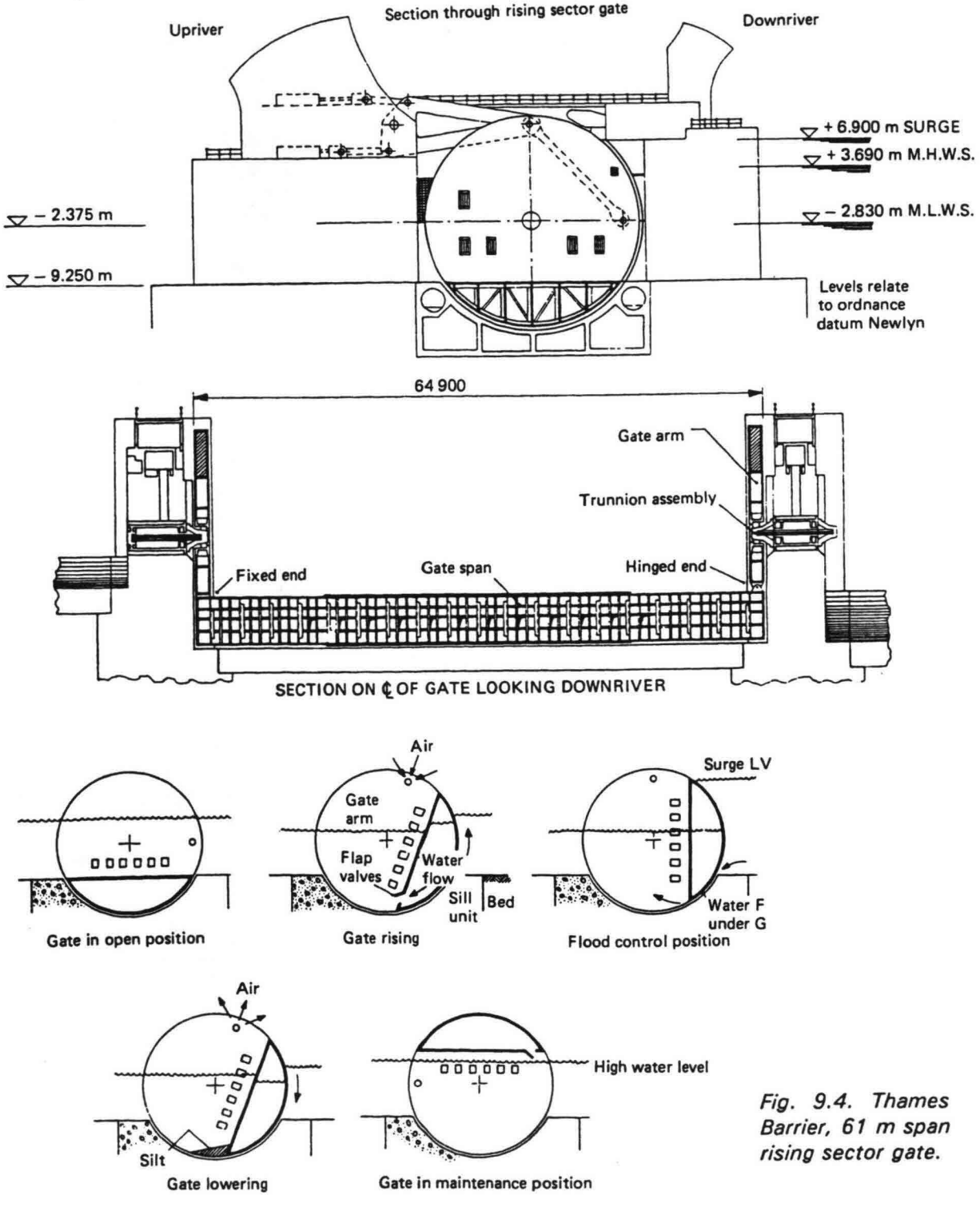


Fig. 9.4. Thames Barrier, 61 m span rising sector gate.

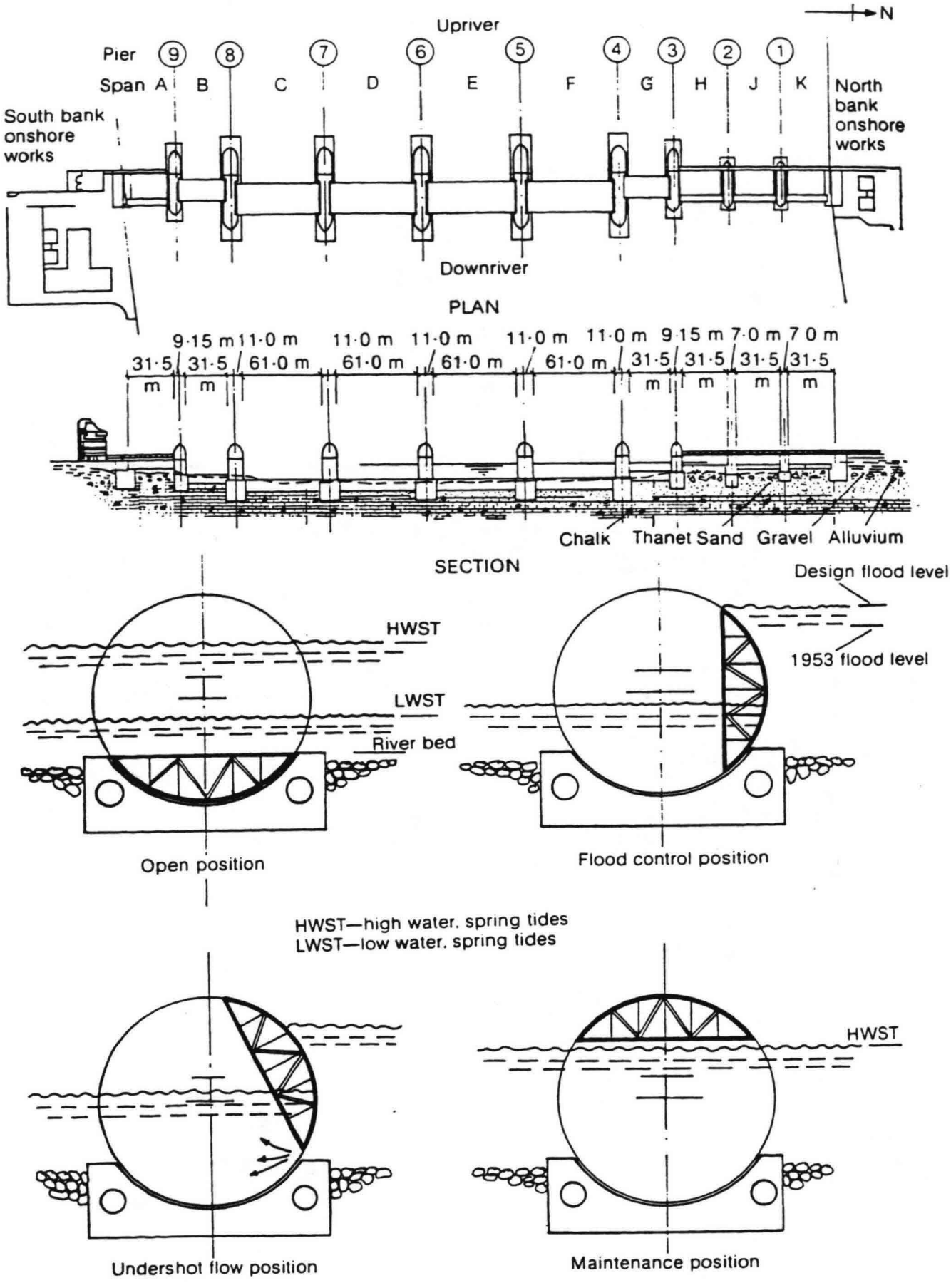


Fig. 9.5. The Thames Barrier : Barrier gate in four positions. The barrier consists out of 9 piers and 10 gates, 4 rising sector gates of 61 m wide and 6 falling radial gates 31.5 m wide.

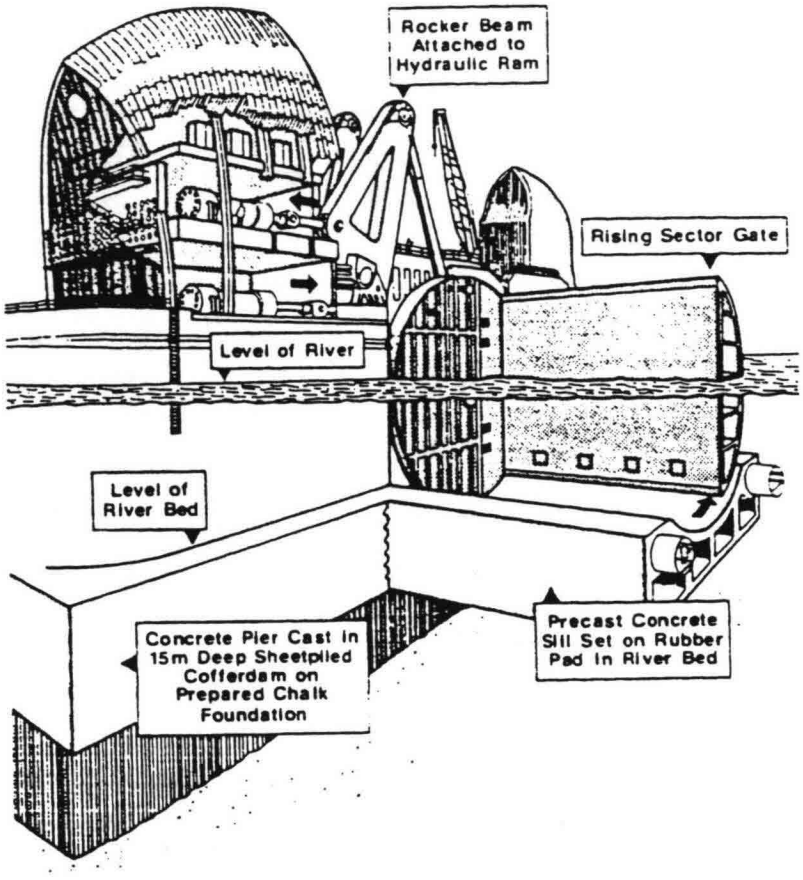


Fig. 9.6.
Thames barrier at Woolwich, East of London.

Eastern Scheldt Storm Surge Barrier

While the Haringvliet dam and the Brouwers dam were nearing completion, preparations had already begun for the construction of the dam across the mouth of the Eastern Scheldt, the last, largest and also most complex part of the Delta Project. Three islands were constructed: Roggenplaat, Neeltje Jans and Noordland. A pumped sand dam was built between the latter two. In the remaining channels the first steel towers were built for the cableway, as it was planned to dam the Easter Scheldt using this well-tried method. Its completion date was set for 1978.

At the end of the 1960s however protests were voiced about the project. Scientists became aware of the special significance of the flora and fauna in and around the Eastern Scheldt. The sandbars and mud flats exposed at low tide are important feeding grounds for birds, and the estuary is a nursery for fish from the North Sea. Fishermen and action groups made sure that the scientific findings were heard by the government and parliament. A heated debate flared up. Opponents of the dam believed that the safety of the region could be guaranteed by raising the height of the dykes along the

Eastern Scheldt. The inlet would then remain open and saline. The equally vigorous supporters of the solid dam, for example agricultural and water boards, appealed to the emotions of the Zeelanders, asking whether the consequence of the flood disaster of 1953 had already been forgotten.

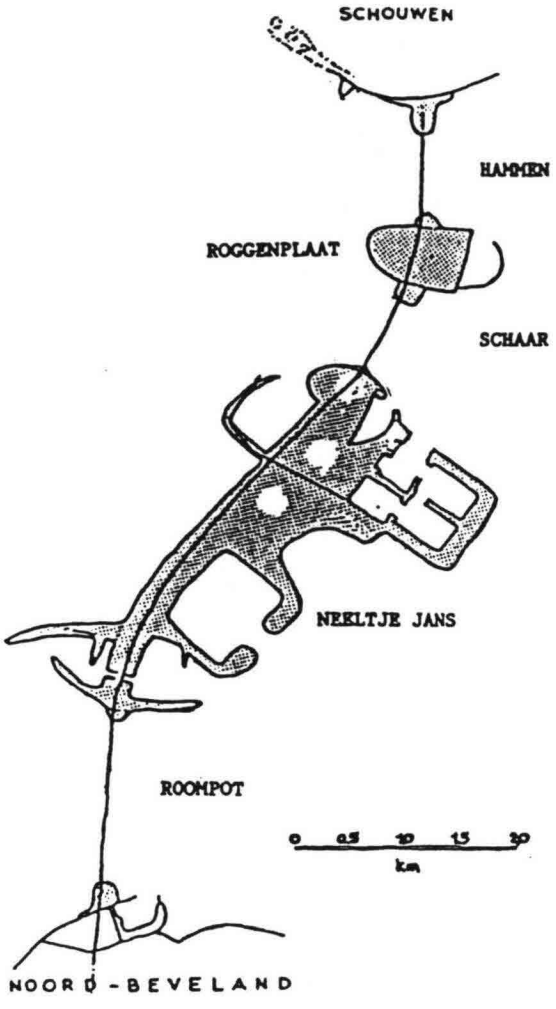
A compromise was reached in 1976: a storm surge barrier, which would stay open under normal conditions but which could be closed at very high tides. The construction of the storm surge barrier meant a break with the policy that the Public Works Department and the hydraulic engineering contractors in the Netherlands had pursued in working form small to large and from relatively simple to complex. The storm surge barrier needed expertise that had yet to be developed and experience that yet had to be gained. Extensive research was carried out to determine the feasibility of building the storm surge barrier, taking full account of the interests of the environment, flood protection, and the fishing and shipping industries. The actual construction of the storm surge barrier also had to be thoroughly studied.

The solution was a barrier consisting of pre-fabricated concrete and steel components that were assembled in the three channels at the mouth of the Eastern Scheldt. Sixty-five colossal piers form the barrier's backbone. A stone sill and a concrete sill beam were placed between each of the piers, and the openings could be closed with steel gates. Concrete box girders were placed on top of the piers to form a road deck.

The seabed also needed special consideration. A new technique was required to prevent the strong current in the mouth of the river from washing away the sand on which the piers were to stand. The solution was to place the piers on mattresses filled with graded layers of sand and gravel which would allow water to flow through but trap the sand.

The construction of the storm surge barrier also required the development of special equipment. The 'Mytilus' made its appearance in the estuary to compact the seabed, followed by the 'Jan Heijmans' which laid asphalt and dumped stones, the 'Cadmium' which positioned the mattress, the 'Ostrea' to lift, transport and position the piers and the mooring and cleaning pontoon 'Macoma'. These are very special ships designed for just one purpose: to construct the storm surge barrier. New measuring instruments and computer programs were also developed, so that engineers working 30 to 40 metres below the surface could position components with such precision that the maximum error would be just one centimetre.

The Cardium laid the first mattresses in November 1982 and the Ostrea placed the first pier in august 1983. Work progressed quickly. There were virtually no technical setbacks; only the cost turned out to be higher than expected. The storm surge barrier was 30% more expensive than estimated. On 4 October 1986 Her Majesty Queen Beatrix officially opened the storm surge barrier. The Eastern Scheldt has remained open and flood protection has been achieved. On average the barrier has to be closed once a year because of storms.



The Storm Surge Barrier in the Oosterschelde marks the last and most difficult part of the Dutch Delta Plan. The original Delta Plan involved the construction of a water-tight dam across the Oosterschelde estuary, which has a width of 9 km. This would have eventually led to the disturbance of the unique salt water tidal environment and the plans were scrapped in the 1970's.

In their place came a new Storm Surge Barrier which would protect the environment and fishing, as well as guarantee the safety. The Storm Surge Barrier was commissioned on October 4th 1986 by Queen Beatrix.

The Storm Surge Barrier consists of 62 movable steel gates suspended between 65 piers. Each of these gates is 42 m wide. To maintain the natural equilibrium of the channels the piers and gates vary in height.

Fig. 9.7. Plan of Storm Surge Barrier.

In normal circumstances the gates remain open, allowing the tides to flow. Should the water level increase to a dangerous level all 62 gates would close and it is generally expected that the gates will have to be closed about once a year. This is done centrally from the Ir J.W. Topshuis and takes up to one and a half hour. The gates are hydraulically controlled. The oil pumps and electronic controls are all protected from the damp and salt in the road box girders. Maintenance and testing procedures are done once every three months.

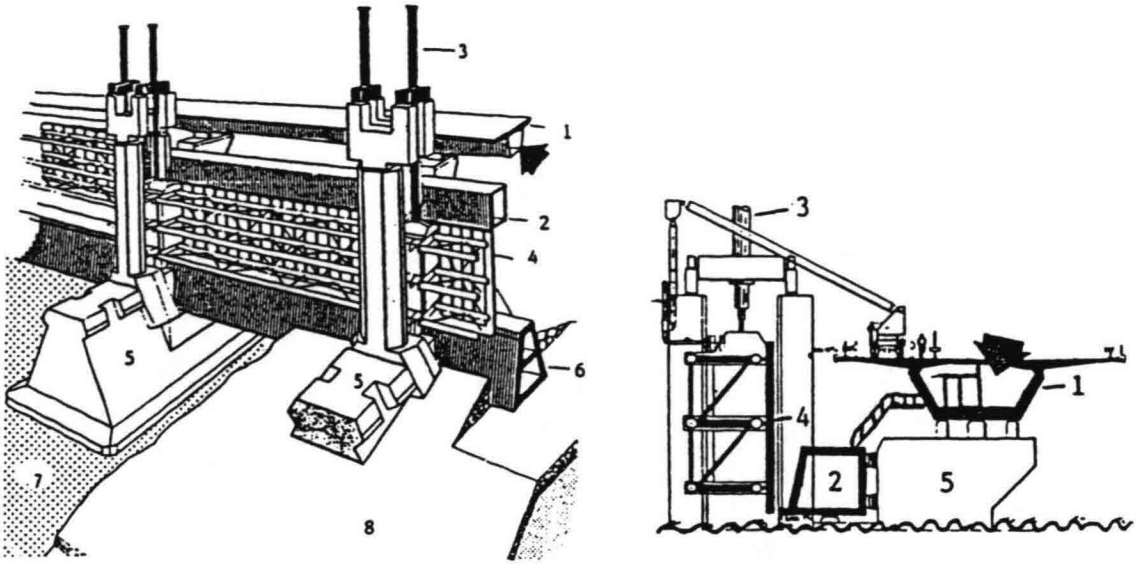
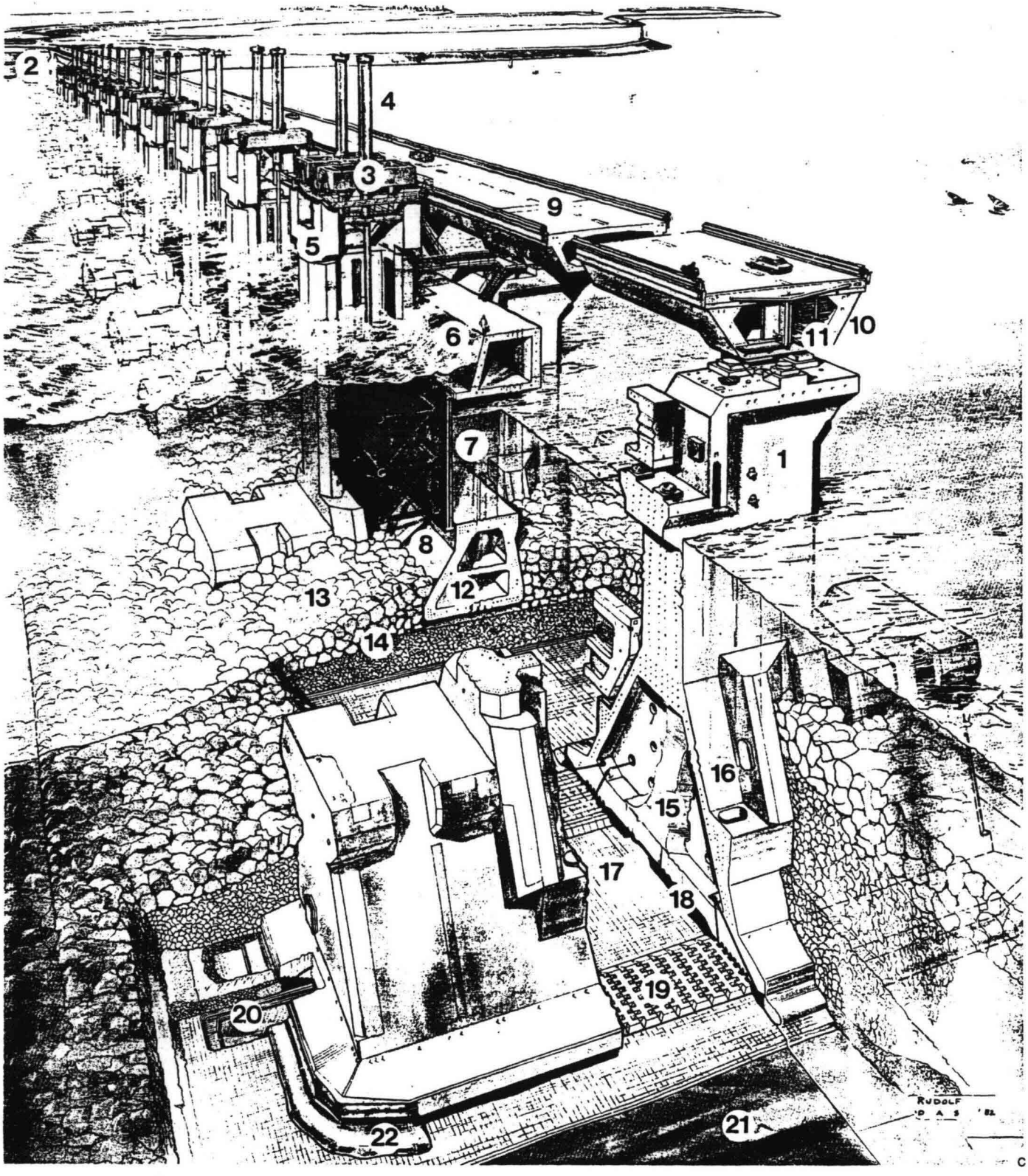


Fig. 9.8. Cross-section.

1. Road box girder
2. Upper beam
3. Hydraulic cylinder
4. Gate
5. Pier
6. Sill beam
7. Foundation mattress
8. Stone sill.



1 pier
2 quarry stone dam for land abutment construction
3 beam supporting operating equipment
4 hydraulic cylinders
5 capping unit
6 upper beam
7 gate
8 sill beam
9 road
10 road box girder and machinery for gate operation
11 power supply duct
12 sand filling of sill beam
13 top layer of sill
14 core of sill
15 sand filling of pier base slab
16 sill beam stops/bearings
17 upper mattress
18 grout filling
19 block mattress
20 bottom mattress
21 compacted sand under the bed of the Eastern Scheldt
22 gravel bag

Fig. 9.9. Eastern Scheldt Storm Surge Barrier.

While the Haringvliet dam and the Brouwers dam were nearing completion, preparations had already begun for the construction of the dam across the mouth of the Eastern Scheldt, the last, largest and also most complex part of the Delta Project. Three islands were constructed: Roggenplaat, Neeltje Jans and Noordland. A pumped sand dam was built between the latter two. In the remaining channels the first steel towers were built for the cableway, as it was planned to dam the Easter Scheldt using this well-tried method. Its completion date was set for 1978.

At the end of the 1960s however protests were voiced about the project. Scientists became aware of the special significance of the flora and fauna in and around the Eastern Scheldt.The sandbars and mud flats exposed at low tide are important feeding grounds for birds, and the estuary is a nursery for fish from the North Sea. Fishermen and action groups made sure that the scientific findings were heard by the government and parliament. A heated debate flared up. Opponents of the dam believed that the safety of the region could be guaranteed by raising the height of the dykes along the

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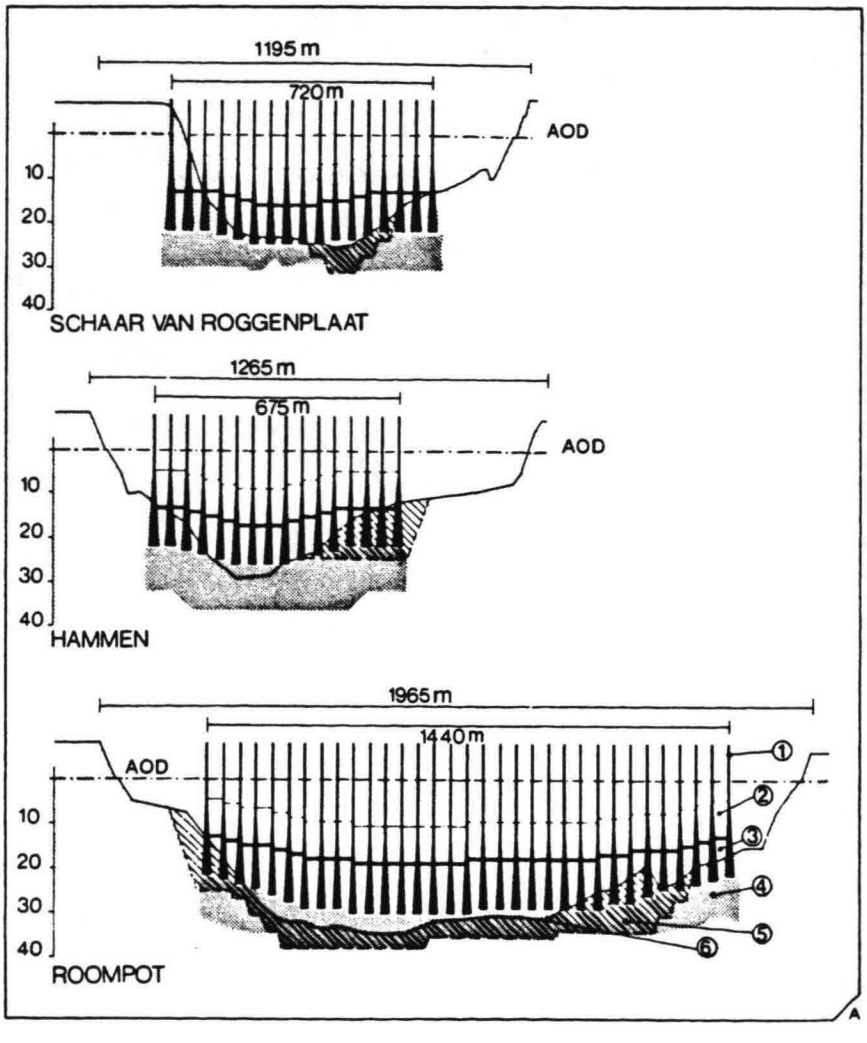
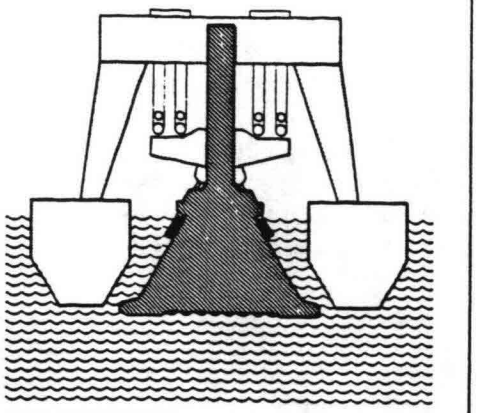
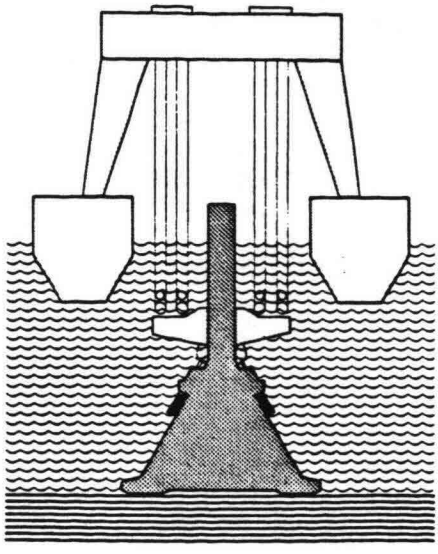


A. Cross-sections of the three tidal channels in which the piers are being positioned.
1 piers
2 sill beam
3 underwater sill
4 depth compaction
5 seabed improvement
6 original bed profile

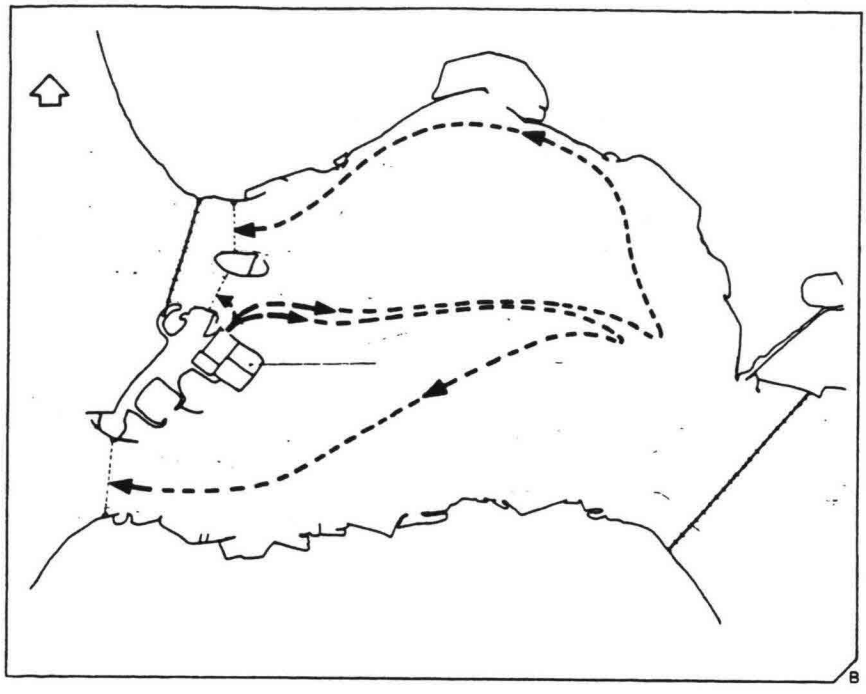


PIER TRANSPORT



PIER INSTALLATION

A



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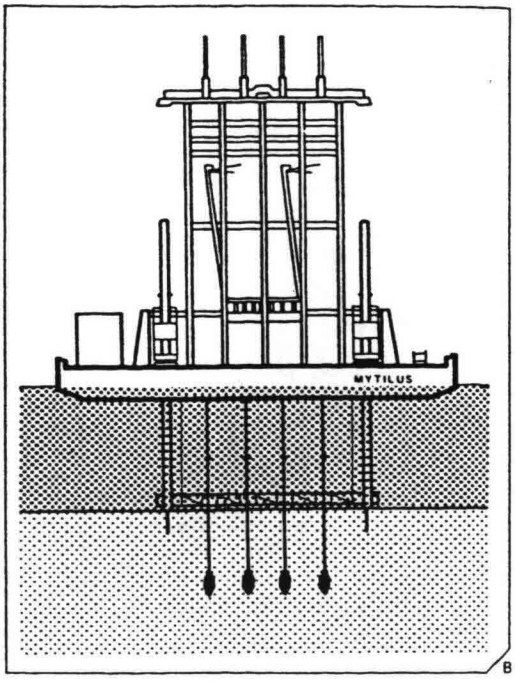

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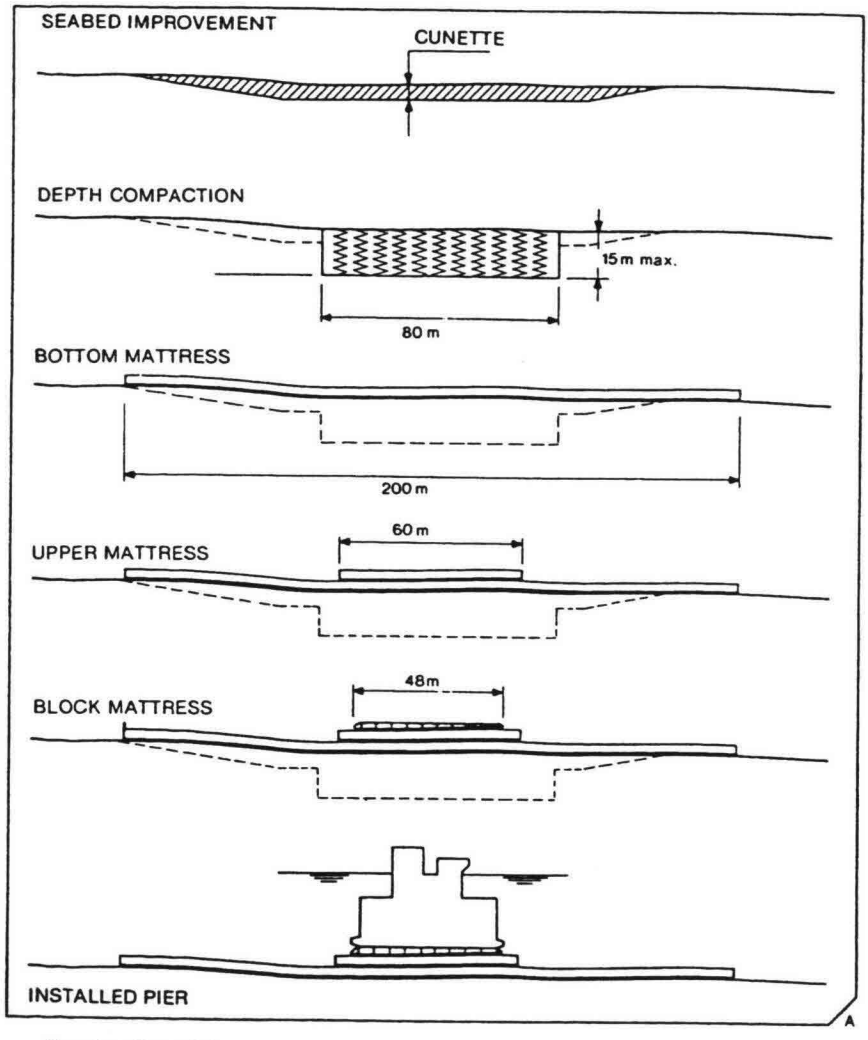


A

Construction of the foundations

Diagramatic representation of seabed compaction by the Mytilus

Eastern Scheldt Storm Surge Barrier (The Netherlands)

This barrier forms part of the Delta Plan that briefly provided for the closure of the four main tidal estuaries and inlets in the south-western part of The Netherlands.

The original intention was to close off the largest estuary, the Easter Scheldt, with a fixed dam as in the other closures, but environmental pressures led to this being radically changed, in order to preserve the tidal action.

The design chosen in 1976 was for a storm surge barrier consisting of monolithic piers, the seabed between them being raised by a sill construction of quarry stone and threshold beams. The piers, sill and beams together would form the frame within which steel sliding gates would be raised and lowered.

During normal weather conditions, the gates were to kept in the raised position to enable water to pass freely through the barrier and thus preserve the tidal environment. During severe weather conditions the gates were to be lowered.
The barrier was completed in 1986.

Figures :

Tidal channels (3)	:	1800 m (max. depth 30 m) 1200 m (max. depth 25 m) 2500 m (max. depth 45 m)
Piers	:	total 65, max. height 53 m, base 25 x 50 m, max. weight 18.000 tons
Steel gates	:	Total 62, length 43 m, thickness 5.40 m, height 5.90 - 11.90 m, weight 300 - 535 tons.
Actual cost (1986)	:	Dfl. 7.600 million.

The barrier was constructed in the open sea on the leeside of the North Sea, on a distinctly unfriendly coastline with shifting sands. Construction of the barrier on a construction dock was not permitted so as not to disturb the hydraulic estuary. The seabed, consisting of loosely-packed sand with insufficient load-bearing capacity, had to be compacted and prefabricated filter foundation mattresses had to be placed to prevent the deformation of the pier foundations. These piers with an underwater weight of 10.000 tons were prefabricated in a construction dock and transported and placed by one self-propelled vessel.

Colne Barrier (U.K.)

The Colne Barrier is a surge barrier across the River Colne to improve the tidal flood defence of the upper estuary, particularly the commercial and industrial areas of Colchester. The defence is being built on behalf of the Anglian section of the National Rivers Authority at a cost of 12 million pounds as lower-cost alternative to raising the height of the upstream river defences.

The barrier site, chosen from three possible locations, was the attractive village of Wivenhoe, which imposed on the designers the task of creating as low-profile a structure as possible. It also inhibited access to the building site to a degree that the contractor had to bring in 95 per cent of the building materials by river.

The barrier is 130 m long with a 30 m opening for the passage of Colchester-bound cargo vessels. The opening is bounded by two 40 m long piers, each housing a 130 tonne, 15 m mitre gate, which will be closed when unusually high tides threaten flooding up-stream. A 40 x 30 m concrete sill within steel sheet pile walls has been built between the piers to prevent seepage below the mitre gates when closed.

Approach structures from the banks to the two piers house 13 radial sluice gates to control the flow of water. These sluices will be hydraulically operated from a shoreside control building.

In an extremely complex building programme which involved working within tidal constraints and always giving precedence to shipping movements, the contractor first tackled the project by building temporary steel-piles jetties either side of the proposed approach structures of the river to the sites of the piers. Once the temporary jetties were in place, a bascule bridge, which at 31.5 m is the longest in the U.K., was erected which enabled a constant flow of trucks carrying sand from the quarries to the north bank. It also enabled concrete to be trucked from a batching plant on the north bank to the south site.

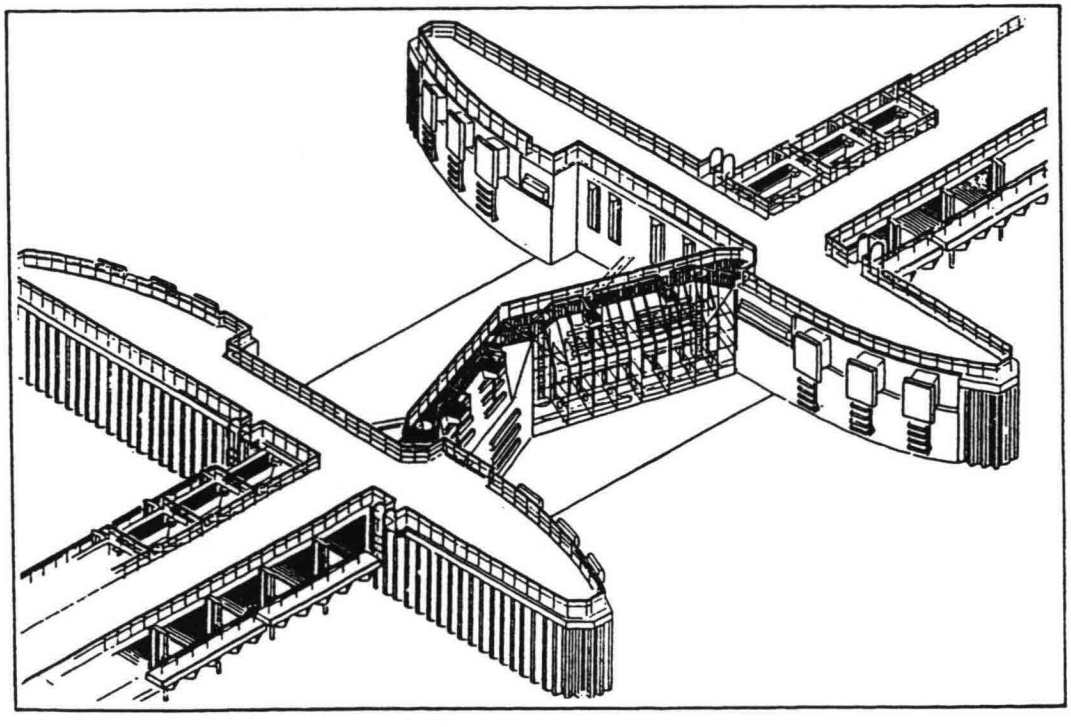


Fig. 9.10. The Colne Barrier's mitre gates fitted into the piers.

10. Weirs and barrages

Weirs or headworks are built in rivers to raise the upstream water level and to regulate the discharge. In dry periods : storage and in wet periods : discharge. Raising the upstream water level may be required for reasons of navigation, intake works (irrigation), etc.. Sometimes a water power plant is also installed (low pressure plant). When the embankment is high enough, a weir with a fixed crest is possible.

In case of low embankments, **weirs with movable gates** or also **barrages** called are used in order to prevent flooding by high discharges. A barrage consists of a continuous sill of concrete, at the river-bed level, with piers and abutments and in between movable gates. The gates are regulating the river discharge and the water level, which means that the gates must be moved also with running water. In case of high discharges the gates are completely opened and the river can flow unobstructed through the weir.

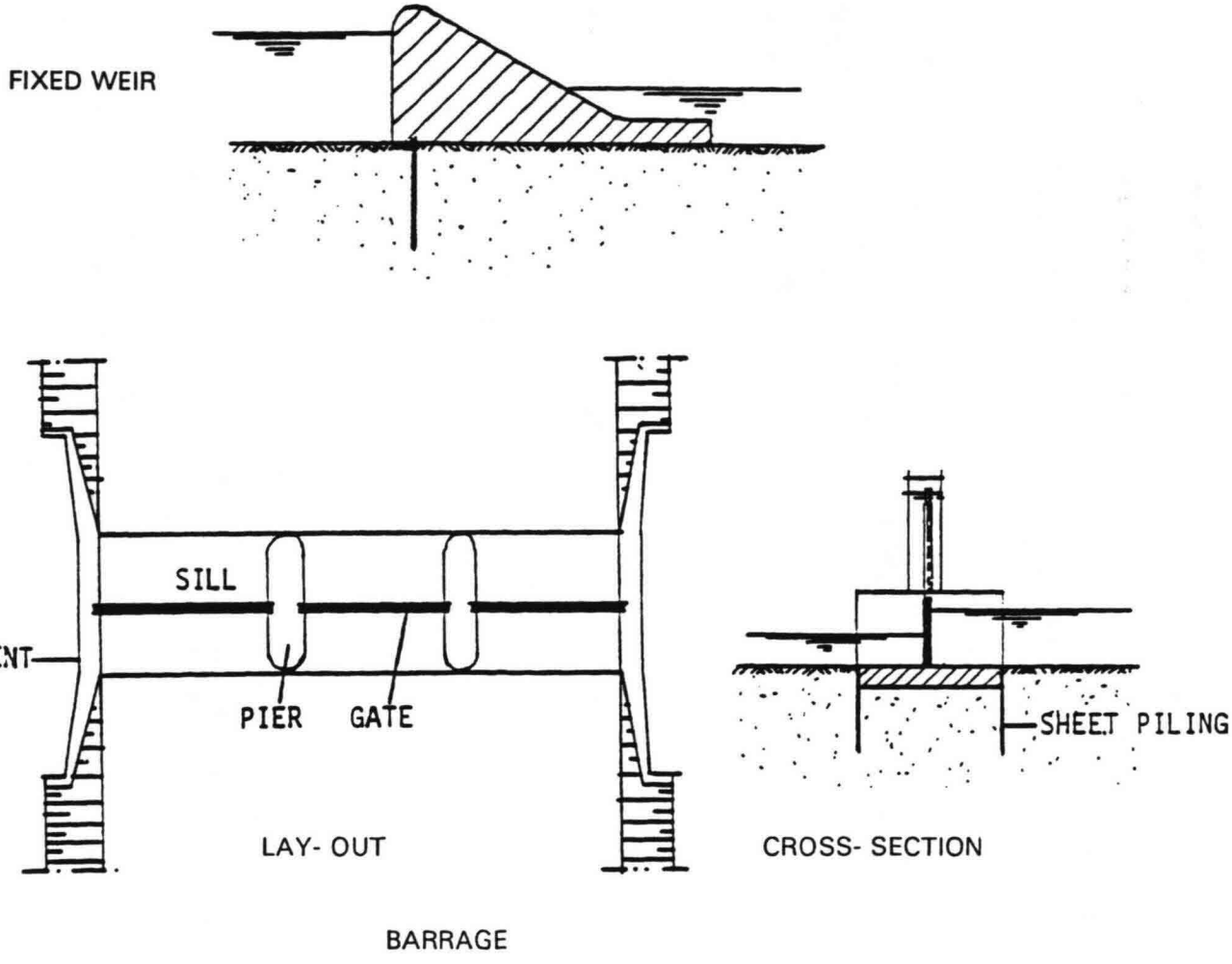


Fig. 10.1. Fixed weir and weir with movable gates (barrage).

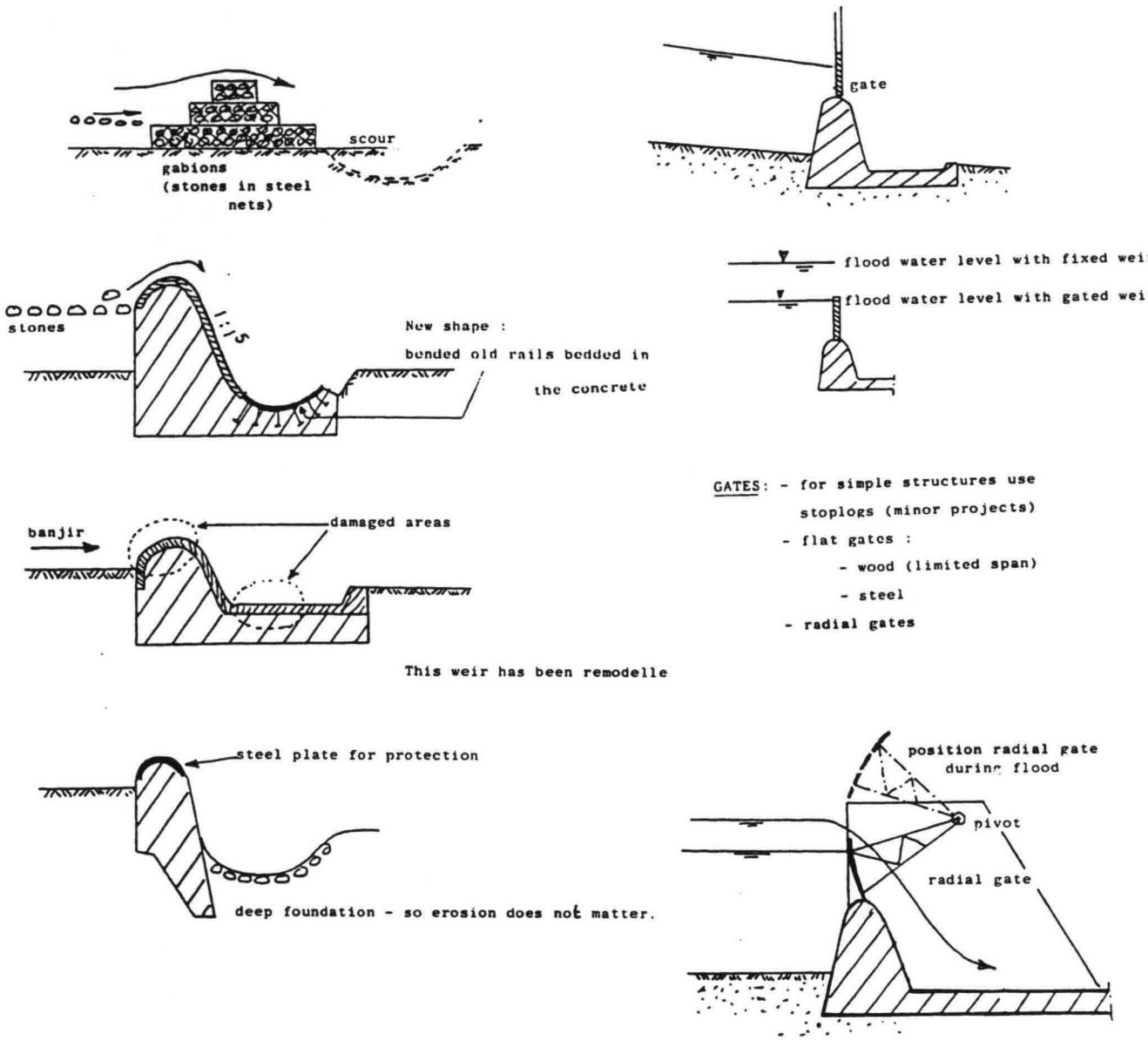


Fig. 10.2. Examples of ungated and gated weirs.

Gates. For simple structures stoplogs are used (minor projects). Otherwise flap gates, wood (limited span) or steel; and radial gates etc.

10.1. Weirs

The most common type of diversion structure in Indonesia is the solid overflow weir, a structure with a fixed crest and where the water passes over the crest. It is constructed across a river to fix the minimum water elevation needed for diversion.

Topside considerations
- head across weir as small as possible during floods (crest design)
- scouring downstream should be limited and not endanger stability (stilling basin, energy dissipators)
- topside of weir should have sufficient resistance to withstand abrasive forces of transported material

Bottomside considerations
- weir body must be stable under all loading conditions
- seepage forces should be acceptable
- piping or liquefaction of non-cohesive bed material should be avoided

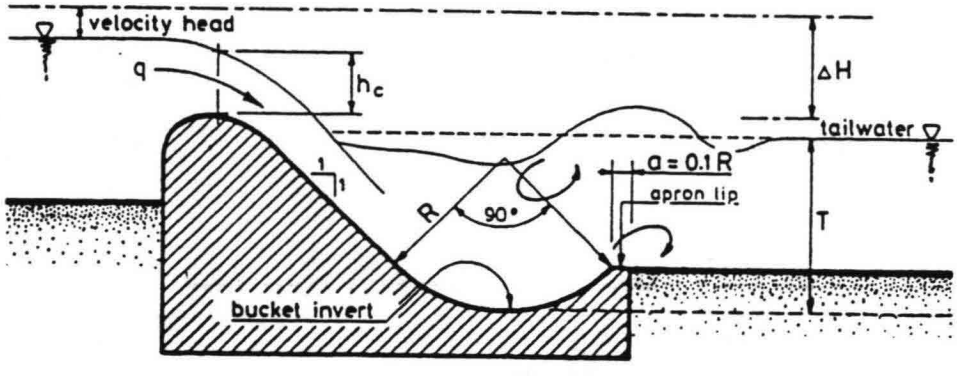


Fig. 10.3. Weir with submerged bucket dissipation

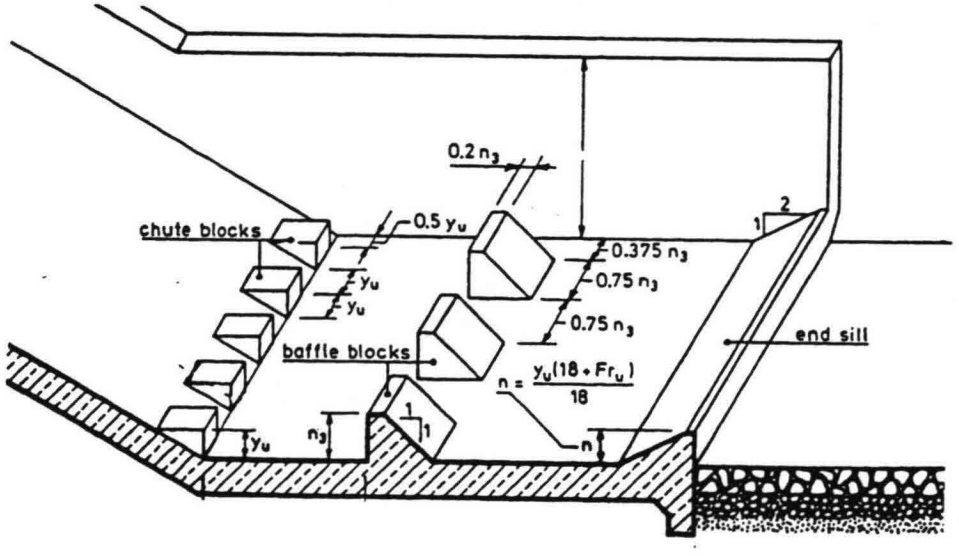


Fig. 10.4. Typical stilling basin with baffle blocks and chute blocks for a weir

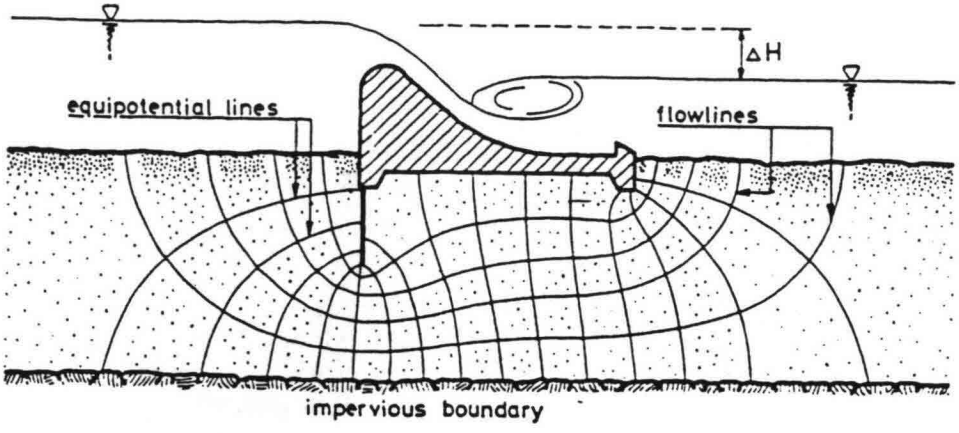


Fig. 10.5. Examples of a flownet under a masonry dam and sand (uplift forces, seepage).

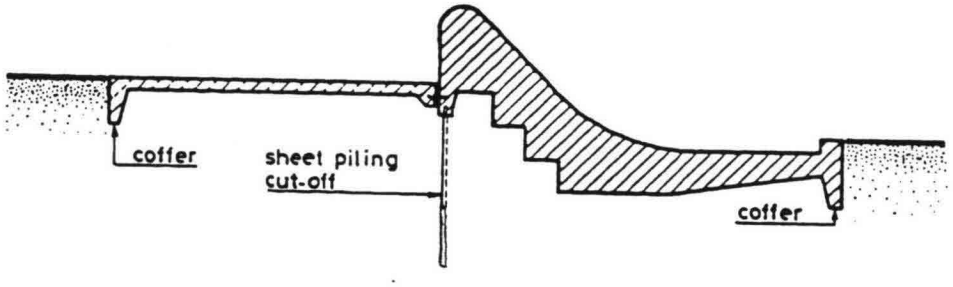


Fig. 10.6. Cut-off walls under upstream floor or weir body. To reduce the uplift pressures on the structure as much as possible the best place for a cut-off wall is at the upstream end of the weir. A cut-off can be constructed as a reinforced concrete or stone masonry wall, as a compacted earth or puddle core or by means of steel or wooden sheetpiling.

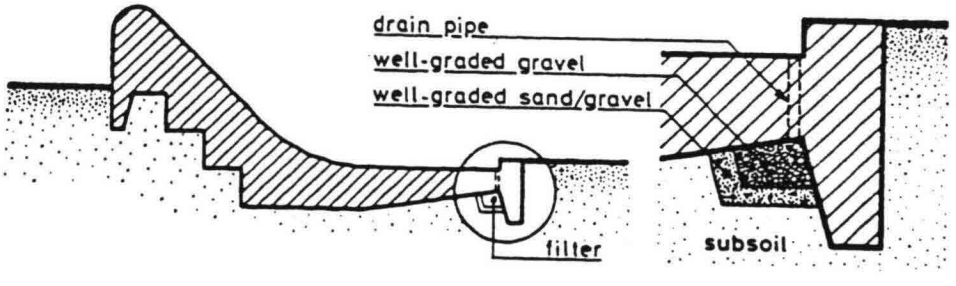


Fig. 10.7. Drains/ filters under stilling basin. Drains are provided to reduce the uplift pressure under the stilling basin of overflow weirs as at these places there is no counterweight of the weir body. To prevent loss of solid material through these drains they must be constructed with reverse filters made from well-graded gravels and sand or from systhetic filter material.

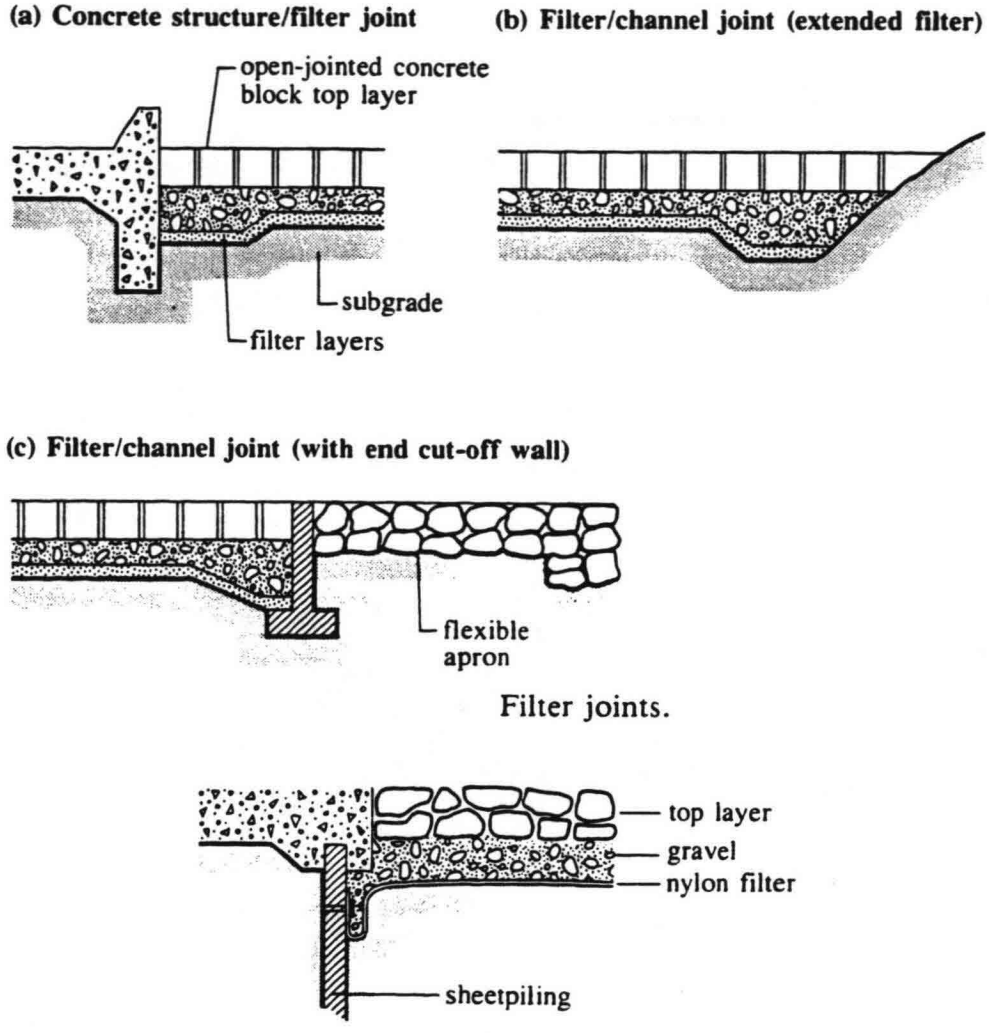


Fig. 10.8. Filter construction with nylon fabric.

10.2. Barrages (gated weirs)

A barrage is a structure with gates, which will be opened during high flows. A barrage can regulate the water level in front of the intake to suit the irrigation needs. It poses some operational constraints as its gates (flap gates, radial gates, etc.) must be well- maintained and operated under all circumstances.

In the flat alluvial areas where raising of the water in the river during floods has far reaching consequences (long flood dikes) the construction of a barrage is justified. Because of the use of movable parts like the gates with their hoisting devices, the barrage is an expensive structure.

In case the river is used for navigation purposes a navigation lock has to be built next to the barrage. The main purpose of the barrage is in that case to ensure a minimal waterdepth for shipping. The lock is used when the gates are closed.

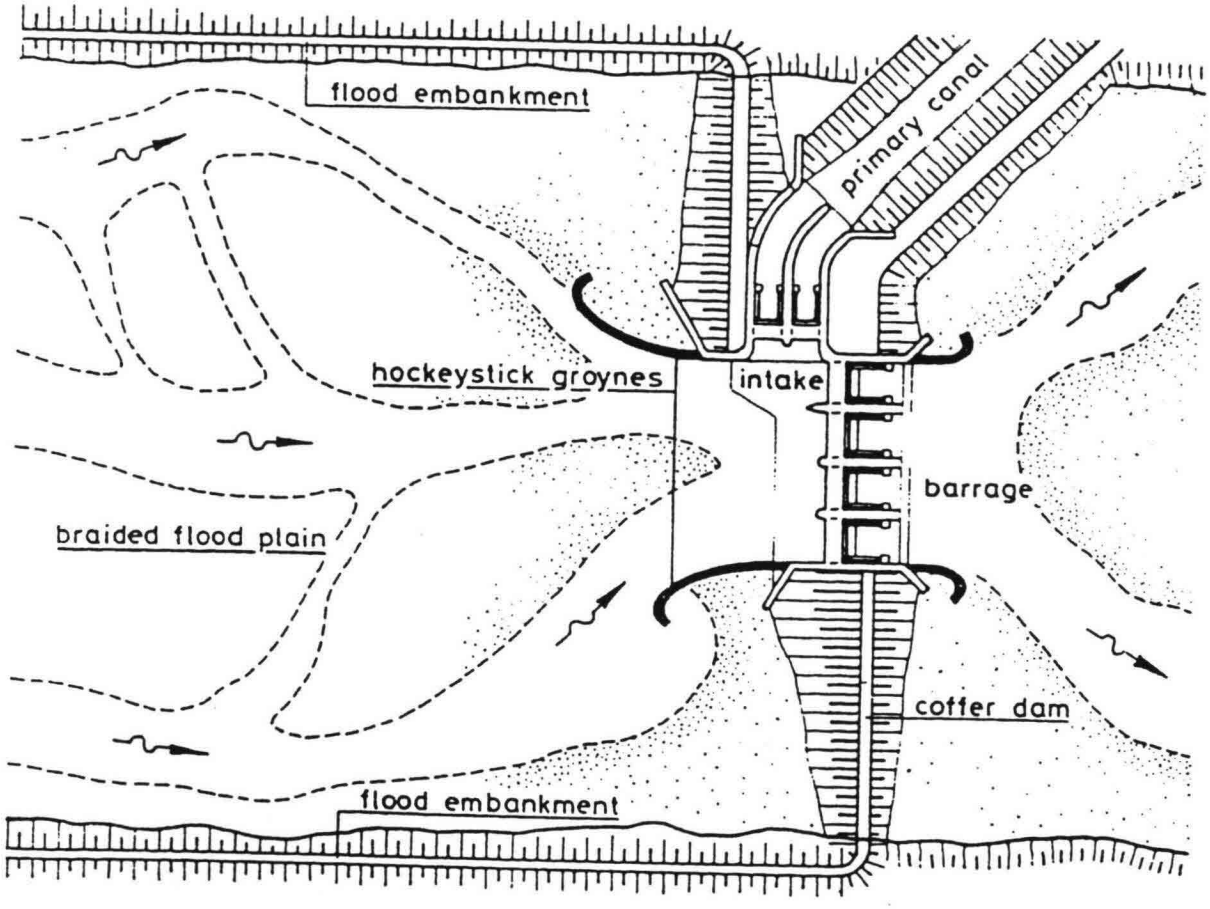


Fig. 10.9. Situation of a barrage in a river. River training works may be required to stabilize the flow upstream of the barrage. Downstream of the headworks the danger of scouring of the riverbanks is usually greater as turbulence and velocities are higher. In relatively wide and deep rivers groynes may be an economical solution.

Gates - main types

1. **Sliding gates** - are used up to maximum 3 meter height and not more than 3 meter wide. For larger openings the hoisting device becomes too heavy to overcome the friction in the grooves.

2. **Roller gates** - used larger openings and high pressures. They have less friction and can be hoisted by means of steel cables, steel chains or hydraulically-driven cylinders.

   Two types : a. **Stoney gate** (wheels fixed to the gate, but running in a separate frame).

   b. **Normal roller gates** (wheels fixed to the gates).

3. Double gates - Sliding/roller gates composed of two gates than can, unrelated from each other, be lowered or raised.
   These gates, therefore, can have overflowing discharge and bottom discharge.

4. **Segmented or radial gates** - as they have little friction the hoisting devices can be small and light. It is normal to give radial gates a possibility of discharging water over the top (to flush floating debris), either by lowering the gate or by fitting the gate with a movable flap at the top.

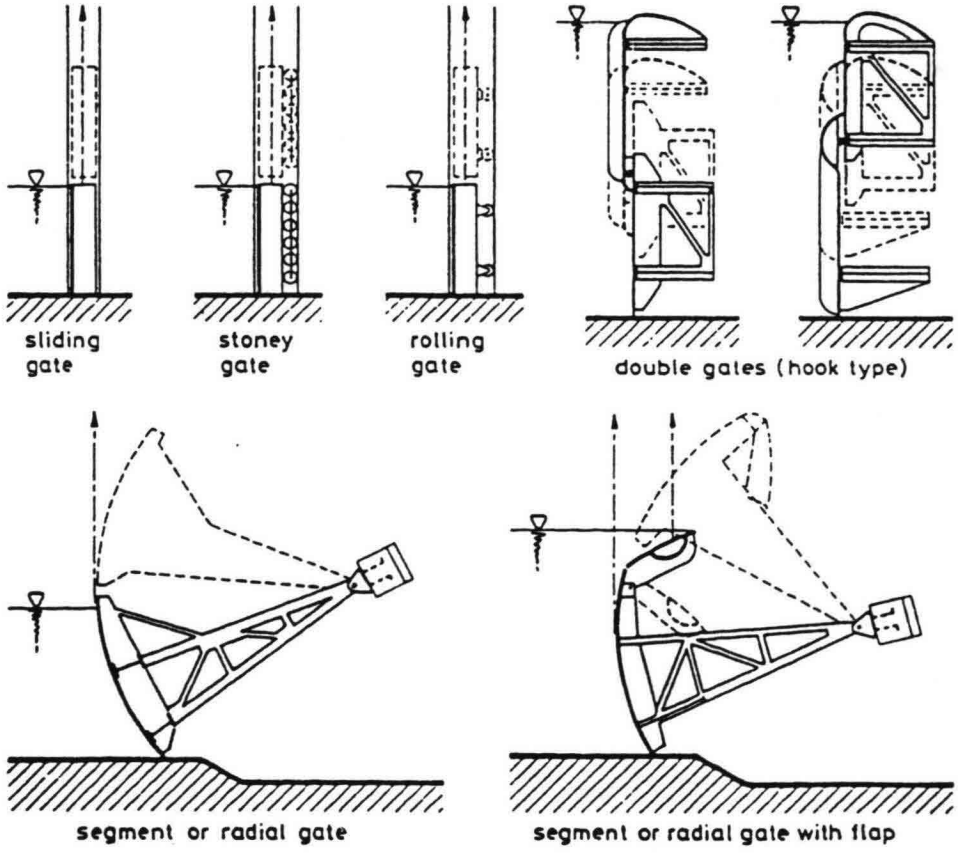


Fig. 10.10. Different types of gates.

Regulate weirs (barrages) in the Lower Rhine (The Netherlands)

Seventy kilometres of the Lower Rhine and the Lek have been canalized, thanks to the construction of three practical identical weirs and lock complexes at Driel, Amerongen and Hagestein. A weir complex consists basically of two land abutments and a central pier between which there are visor gates. It also has a lock which when the gates are closed enables ships to negotiate a head of about 3 m.

The most important part of a weir is the visor gate, a semi-circular "valve" which by reason of its shape and the way it is raised recalls the visors on the helmets of medieval knights. Each gate weighs 200 tons. The gates move on massive pivots in the abutments and the central pier. With the aid of cables they can be raised to an angle of 60°C (in 2.5 hour time). 2.2 Kw electric motors (a total of 4 per gate) drive the cable drums around which the cables are slowly wound. Both the electric motors and the drums are installed in the engine rooms at the top of the concrete arches.

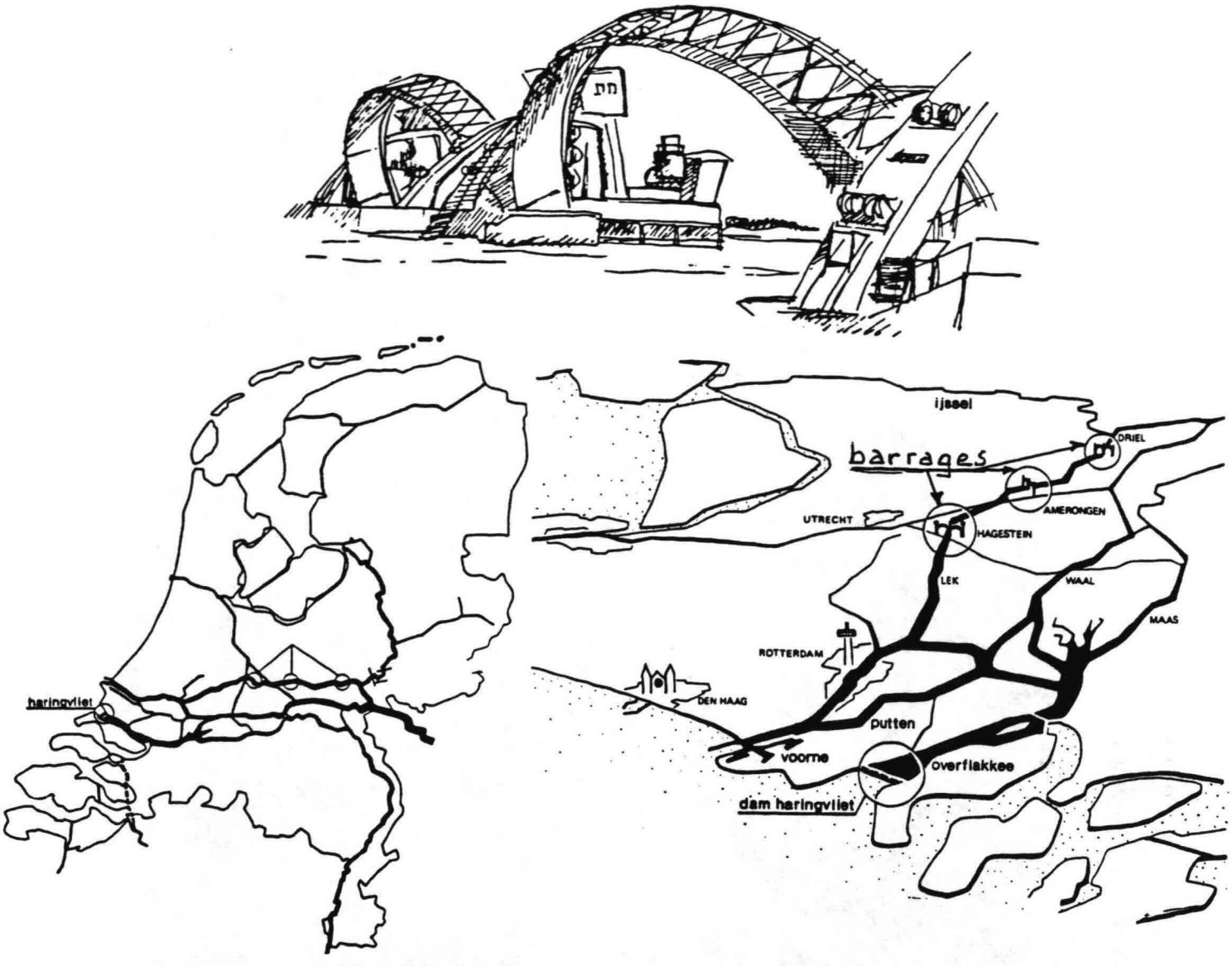
Total horizontal clearance	:	48	m
minimum vertical clearance (at highest recorded water level):		9.10	m
Height of visor gate	:	9	m

The central pier has a height of 12 m above the weir floor, a width of 12.5 m and a length of 59 m. The pier contains an outlet channel and a valve which, when the visor gates are closed, allows a maximum flow of 90 m^3/s to pass through. Like the central pier, the land abutments have a concrete arch with an engine room built above it, almost 32 m above the weir floor. Each abutment has a fish lock.

The floor of the weir between the abutment consists of large concrete slabs. Under the upstream part of the weir floor there is a tunnel which can be reached by lift from the abutments and the central pier. When the gates are raised the tunnel is the only link with the other bank of the river.

Improved water management was a major objective in canalizing the Rhine. To increase the discharge of the river IJssel during periods of low water in the Rhine the gates of the weir complex at Driel are lowered. A quantity of water from the Rhine is forced to flow into IJsselmeer (Ijssel Lake) via the river Ijssel. The barrage at Driel thus serves as a "tap", making it possible to distribute the available water between the Lower Rhine and the IJssel.

Navigation on the IJssel is possible with a flow of 250 m^3/s, which gives the river a navigable depth of 2.70 metres, this being the lower limit. In order to be able to maintain this level, the barrage at Driel must be closed completely for about fifty days per year.



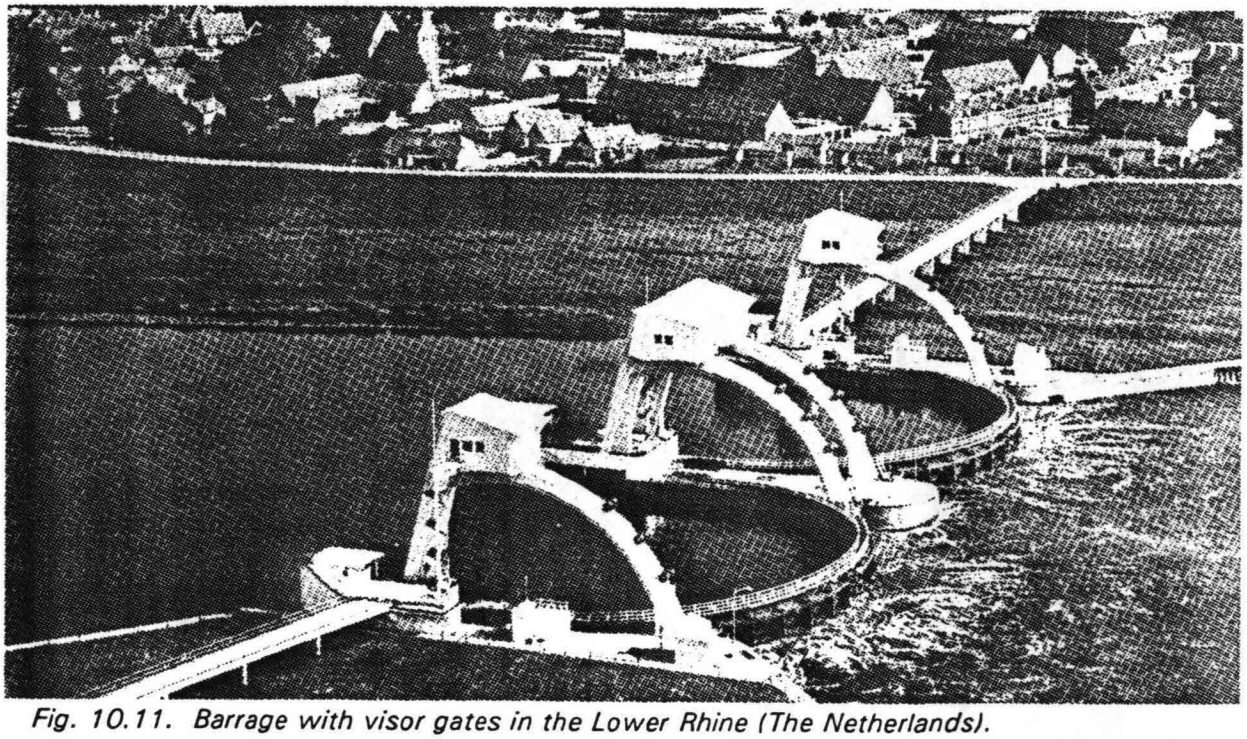


Fig. 10.11. Barrage with visor gates in the Lower Rhine (The Netherlands).

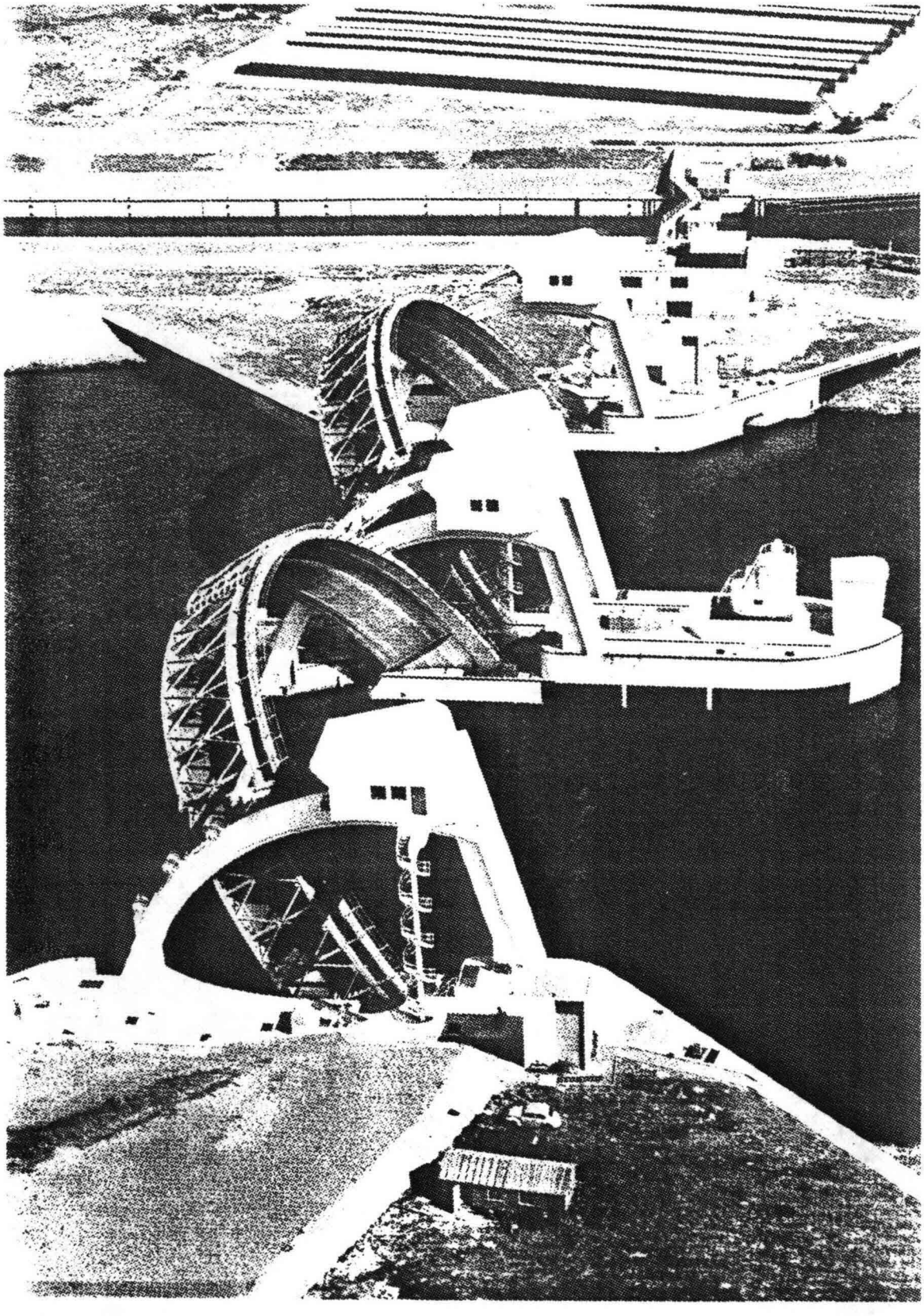


Fig. 10.12. Barrage in the Lower Rhine (The Netherlands). The visor gates in raised position; minimum vertical clearance 9.10 m; total horizontal clearance 48 m.

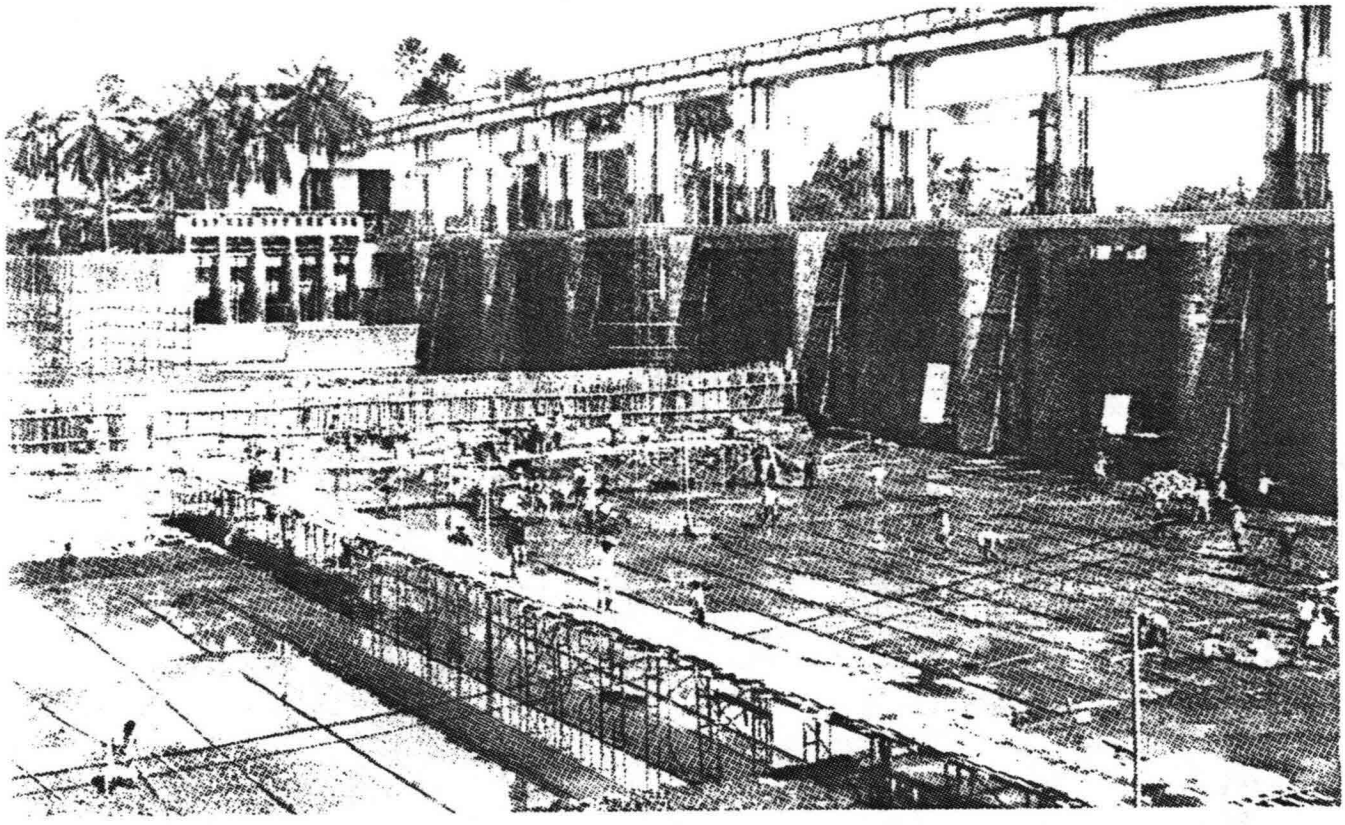


Fig. 10.13.
Repair and rehabilitation of the Pamarayan Barrage on the Ciujung River, Indonesia.

10.3. Bottom rack or Tyroller weir

The bottom rack weir is a type of structure which can extract water from a river without interfering with the water level. It consists of an open ditch running perpendicular to the flow across the river. Large boulders and stones are prevented from entering the ditch by means of a grating of steel bars.

The structure is used in the upper parts of a river where the river transports only large sized materials. It is not suitable for rivers with large fluctuations in material transport. The structure must be equipped with a suitable gravel trap/sediment excluder that can be flushed with sufficient velocity to remove all particles.

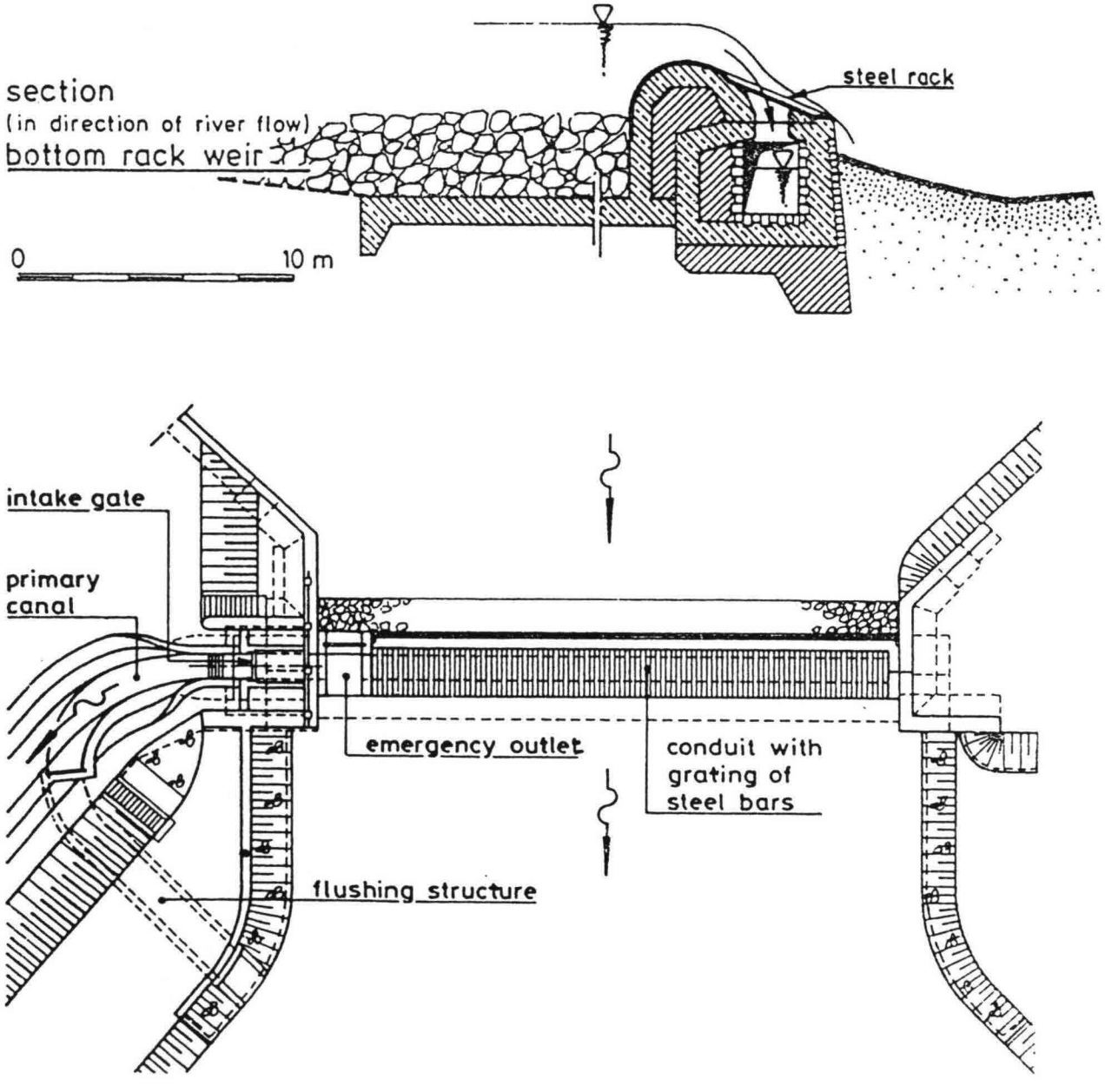


Fig. 10.14. Bottom rack weir.

11. River inprovement works

The objectives of river improvement works are to aid navigation, to prevent flooding, to reclaim or protect land, or to provide water supply for irrigation, hydropower development, or domestic and industrial use. For navigation purposes the main river improvement are those which provide sufficient depth and/or stabilize the river channel in a suitable form, and provide bank protection against wave action, particularly on constricted waterways.

Table River uses and measures to achieve them
1 Measure to achieve the required use
2 Measure of secondary importance for the use
3 Measure which might be necessary as result of the operation

	Bed regulation								Discharge and water level regulation		Quality control
Measure	repeated dredging	temporary construction in river bed	fixation of bottom	elimination of obstacles from low-water bed	channel rectification and fixation	channel constriction	revetments and groynes	rectification in the flood plains	reservoirs	weirs	quality control works
Use of river											
Flood control					1		3	1	1		
Navigation	1	1	2	1	1	1	3		1	1	
Hydropower									1	1	
Irrigation and water supply			3				3		1	2	
Waste discharge											1
Bank protection			2		1		1				
Cooling water						3	2				1
Commercial sand dredging	1		3								
River crossings		1					3	2			
Control of sea water intrusions			2						2	2	

The principal methods used to improve river channels are **river regulation and dredging**; on navigable rivers **canalization**, construction of **lateral canals**, and flow improvement by reservoir construction and operation has also to be considered. Flood protection works include high-water **river training** (mainly by dikes), **diversion** and **flood-relief channels** with or without control structures and flood-control reservoirs. In river regulation or training the river may be encouraged to pursue its natural course or it may be straightened (the latter requires great sensitivity). In the middle and lower reaches it is often necessary to raise **river banks**. In estuaries **dredging** may be necessary together with the construction of works reducing the channel width, e.g. **groynes, longitudinal training walls**, etc.

Groynes are small jetties, solid or permeable, constructed of timber, sheet piling, vegetation and stone rubble, etc. They usually project into the stream perpendicularly to the bank but sometimes are inclined in the upstream or downstream direction. The main purpose of groynes is to reduce channel width and remove the danger of scour from the banks.

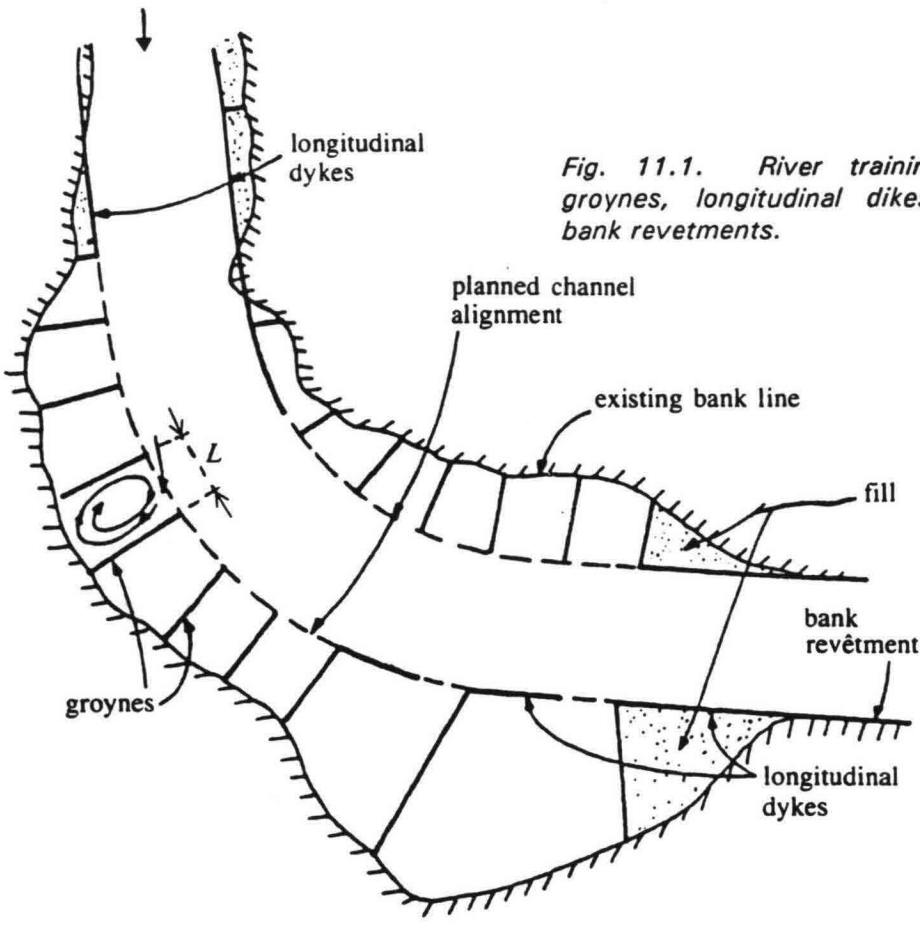


Fig. 11.1. River training by groynes, longitudinal dikes and bank revetments.

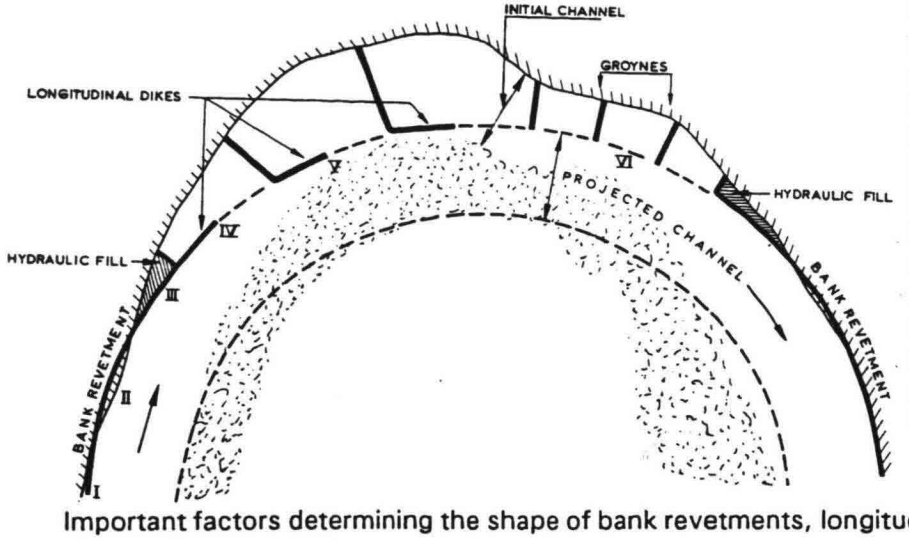


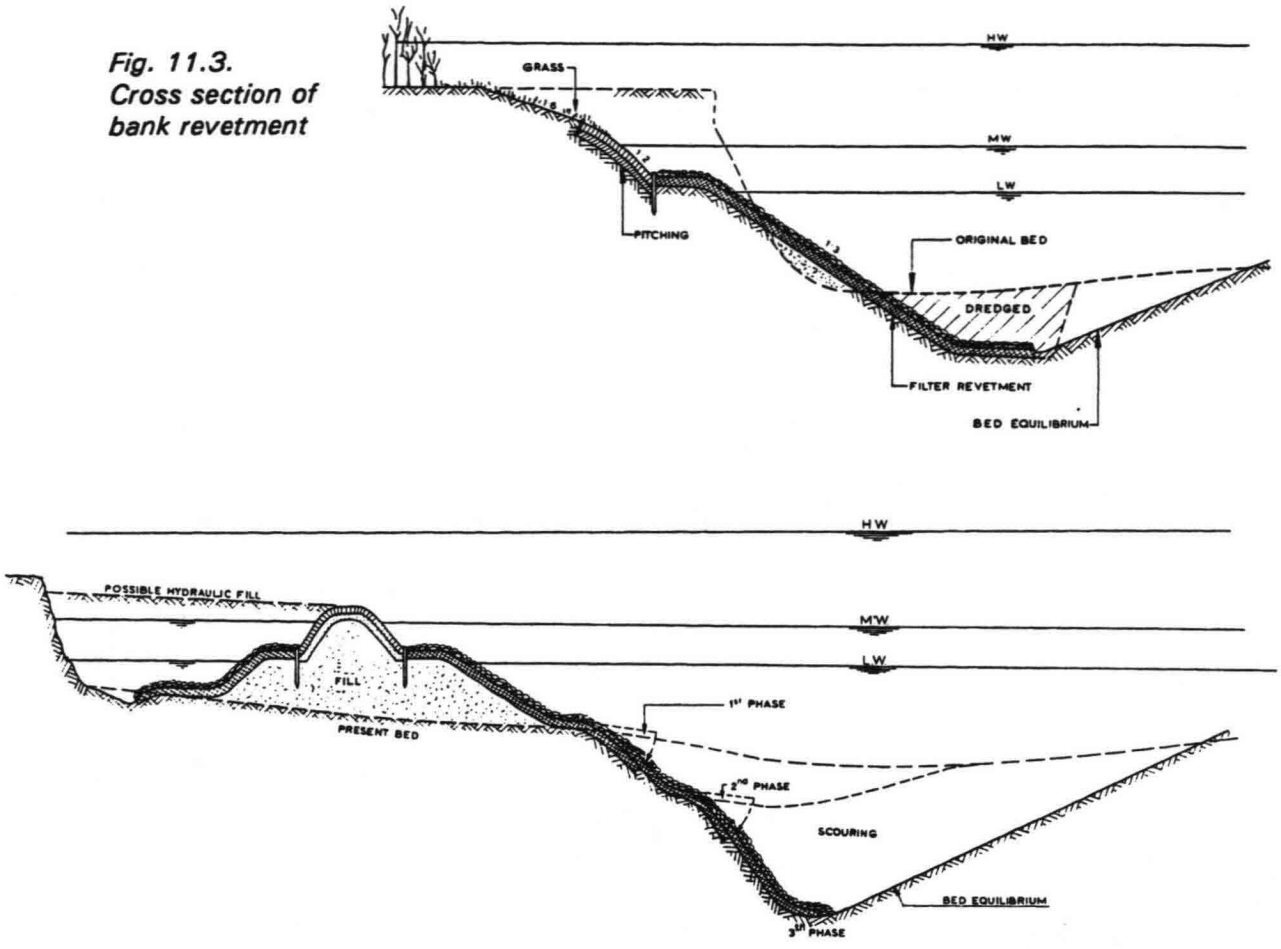
Fig. 11.2.
Types of permanent regulation structures.
Section I and II : fixation by a bank revetment
Section III: longitudinal dike with hydraulic fill
Section IV: longitudinal dike with a permanent inner revetment and without fill
Section V : longitudinal dikes with spurs
Section VI: groynes.

Important factors determining the shape of bank revetments, longitudinal dikes, groynes and closure dams, are:

- the toe level
- the crest level
- the steepness of slopes and the level and width of berms

Estimation of the equilibrium depth at the toe of the structure is important as it determines the dimensions of the revetment. The best solution is dredge a cut in front of the bank revetment right down to the anticipated equilibrium depth and then construct the revetment to that depth (Fig. 11.3.).

Fig. 11.3.
Cross section of bank revetment



There are also uncertainties in the calculation of the crest height of regulation structures. For economical reasons the crest level will be kept as low as possible. However, in most cases the crest needs stone ptiching, which requires execution above the waer level. This normally leads to a crest height which is not far below mean water level. A maximum crest height of the structures which aim to guide the river flow is determined by the flood plain level, since at high river stages current concentration and erosion behind the structures should be prevented (flow across the groynes during floods).

The slopes applied to bank revetment, groynes etc. depend on the quality of the subsoil, the groundwater flow and the type of revetment. Normally underwater slopes are between 1:2½ and 1: 3½, while slopes constructed in the dry are somewhat steeper (1:1½ to 1:2½). In some cases the slopes of the head of groyne at the river side is constructed to be more gentle (1:5 to 1:10) in order to form a more gradual transition between eddy and main flow. Berms are often designed so that they facilitate execution, improve stability or form a good transition between two types of revetment.

Core; for economic reasons the core of bank revetments, groynes and longitudinal dikes will, where possible, be made of the river bed material. When strong currents at the location of the proposed structure prevent such a solution the core material may consist of stone or of mattresses or sausages loaded with stones, and filled in with sand.

Longitudinal dikes (training walls) are usually more economical than groynes and - if properly positioned - equally or even more effective. The material used is again rubble, stone or fascine works (on soft river beds). Training walls may be single-on one side of the canal - or double. An example of the combined used of groynes and dikes in river training is shown in Fig. 11.2.

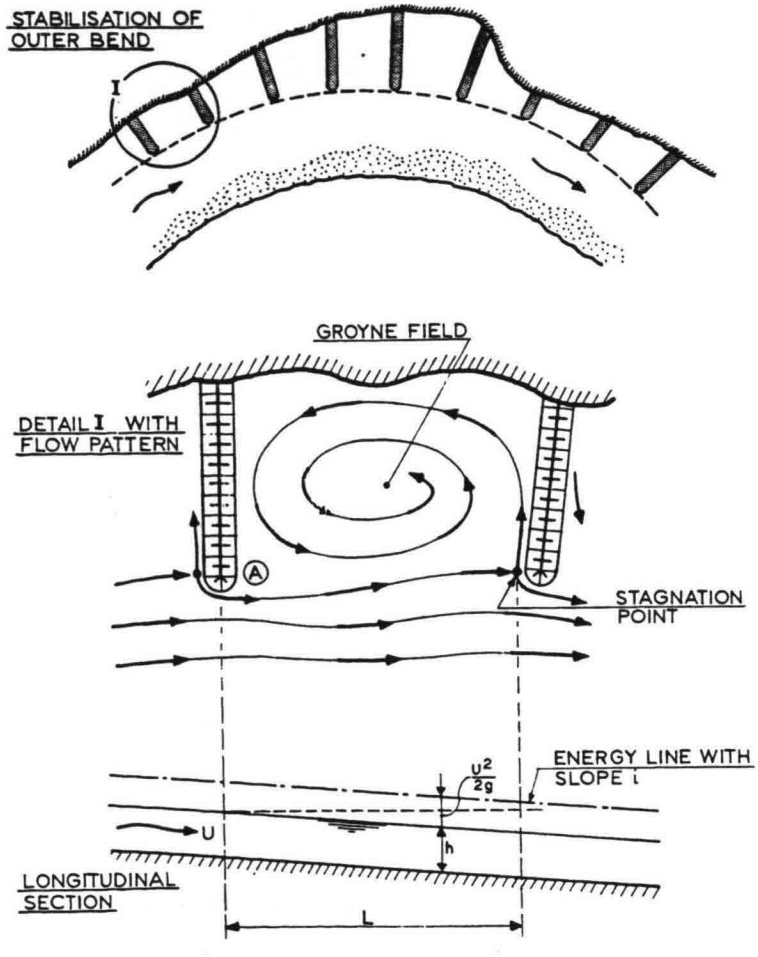


Fig. 11.4.
Bank stabilization with local supports (groynes).
The stabilization of an outer bend with series of groynes rather than longitudinal structures is only economically justified if the distances between the groynes are large, yet they retain their guiding function on the main flow of the river. This guiding function will be best fulfilled if there is one strong eddy between each pair of groynes. This wish restricts the length L of the distance between the groynes because the stability of one eddy is governed by the factor
$2\ gL/c^2h$ (should be < 1).

Hardly any theoretical background is available to help answer the question of the distance between groynes in a river section. Obviously the eddy between the groynes plays a dominant role in guiding the current. Only a single elliptical eddy can give efficient support.

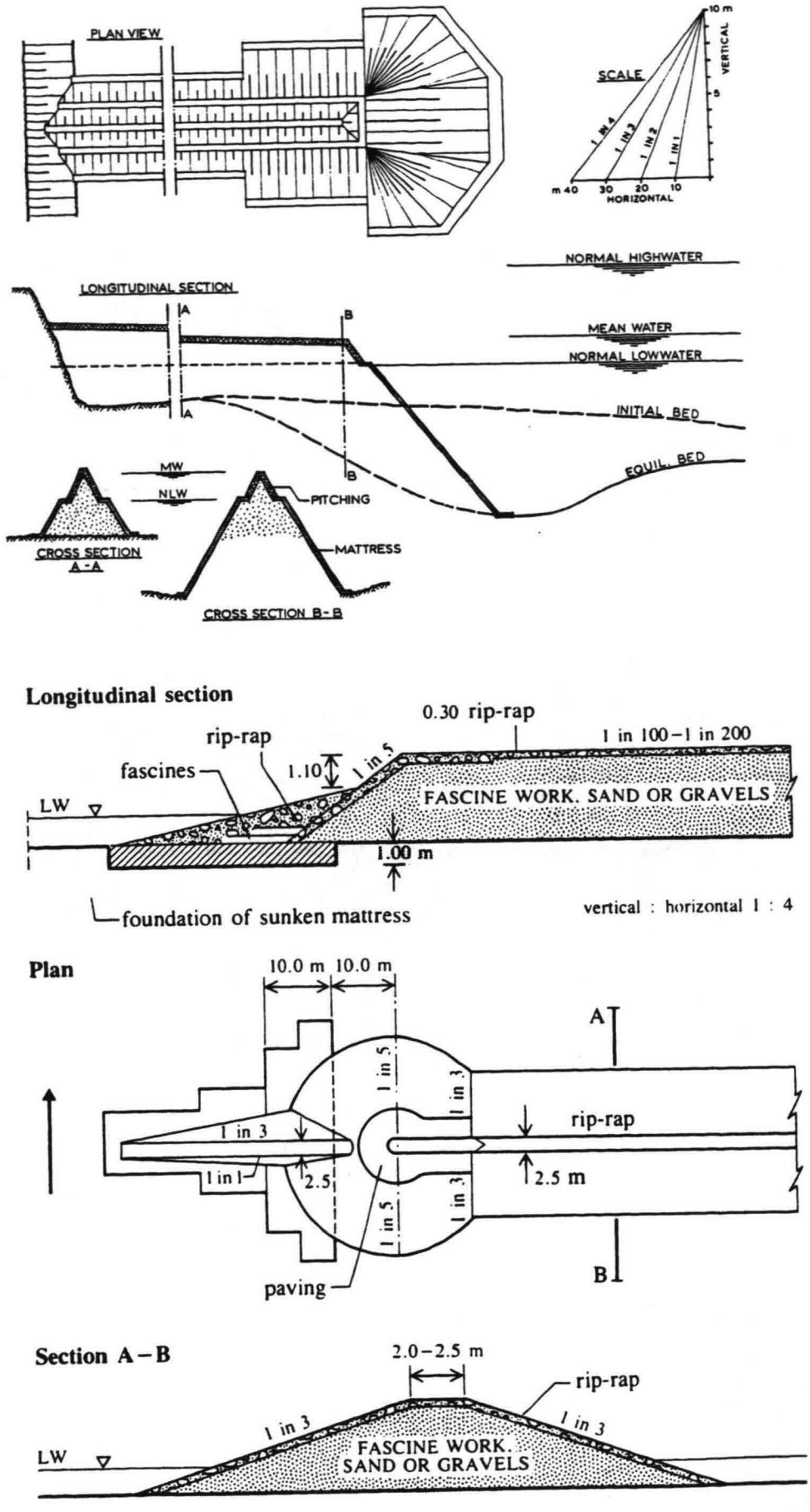


Fig. 11.5. Examples of groyne construction.

Details of groyne design strongly influenced by economic factors, and a cost-benefit analysis for the determination of their height, spacing, length and material is usually necessary.

Open spur dikes

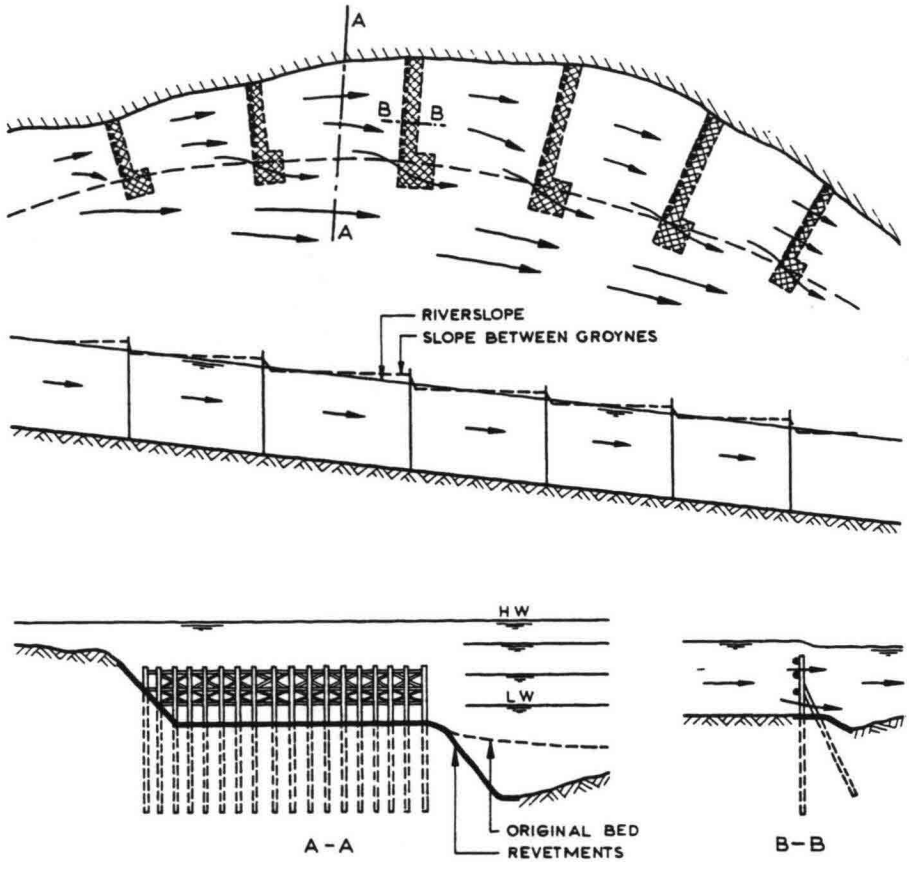


Fig. 11.6.
Open Spur-dikes.
Sometimes an open structure is used. They permit part of the flow to pass through at all water stages. A well-known example is the row of timber- or pile-clumbs. By absorbing part of the energy of the flow through the open structure, the flow velocity and consequently, the sediment transport capacity are locally reduced, thus inducing sedimentation. In this way an eroding bank may be stabilized or a river branch closed.

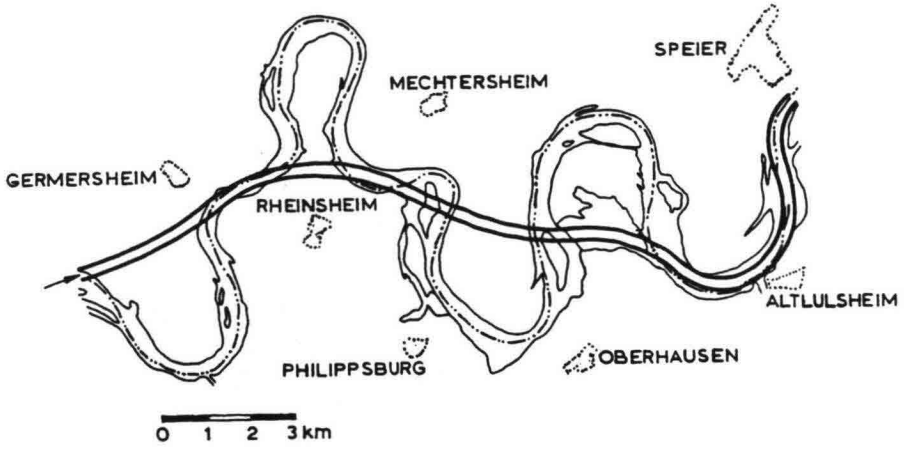


Fig. 11.7.
Channel regulation on the Rhine upstream of Mannheim (Germany, 19th Century).

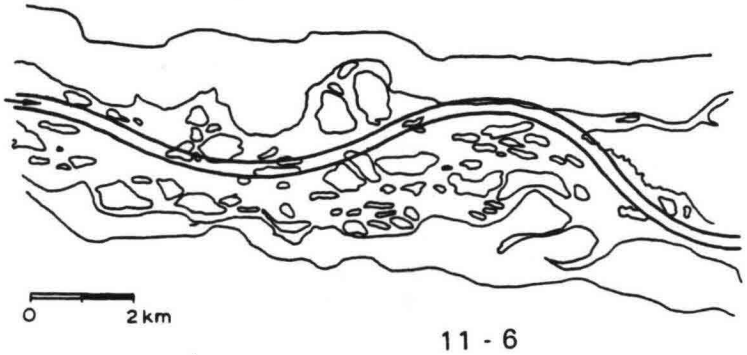


Fig. 11.8.
Channel regulation on the Rhine downstream of Basle (Germany, 19th Century) with groynes and training wall.

Ground sills

Channel beds liable to substantial erosion can be stabilized by ground sill or- more extensively (and expensively) by a series of drop structures. Ground sill usually span the whole width of the river channel with the greatest height at each bank and a gentle slope to the stream centre. Rubble mounds, cribs filled with rubble and concrete are some of the materials more frequently used for ground sills.

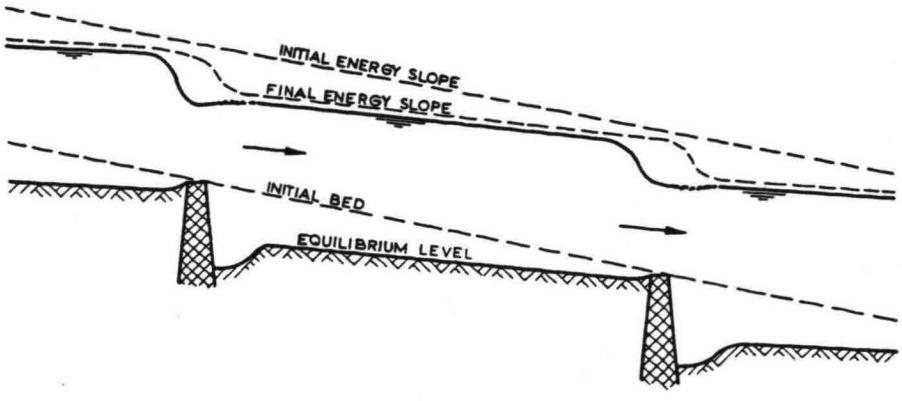


Fig. 11.9. Bed fixation by ground sills.
Degradation of a river bed can be stopped by fixation. The construction of fixed weirs also offer a possibility for stopping degradation but is not recommended on rivers used for shipping because they need costly bypasses with locks. Then a reasonable solution might be the application of a series of submerged sills perpendicular to the direction of the current.

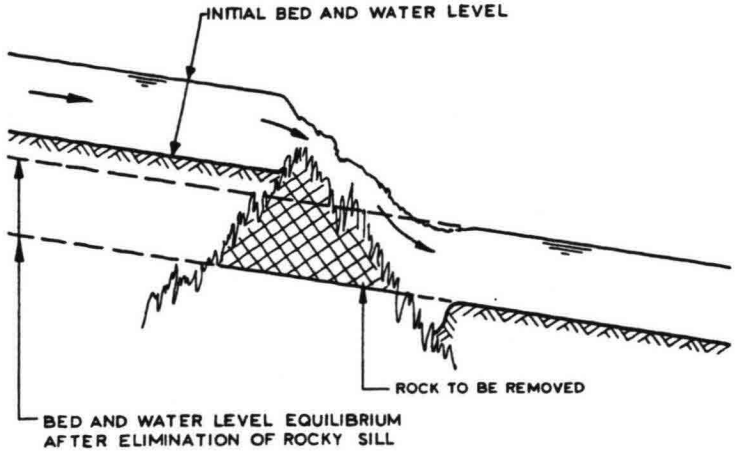


Fig. 11.10.
Elimination of rocky sill.
The reverse problem of unwanted degradation of the low water bed is the elimination of unwanted resistance bodies, such as rocky sills, by explosives combined with dredging. In exceptional cases it may be possible to avoid the resulting morphological problems by cutting narrow channels through the rocky sills with sufficient width and depth for navigation hardly lowering the water level at the upstream end. Well-known examples of such a solution (as a first step before a more radical one) are the 'Iron Gate' in the River Danube where it breaks through the Carpates and the 'Bingerloch' in the River Rhine in the 'Gebirgstrecke' reach in Germany.

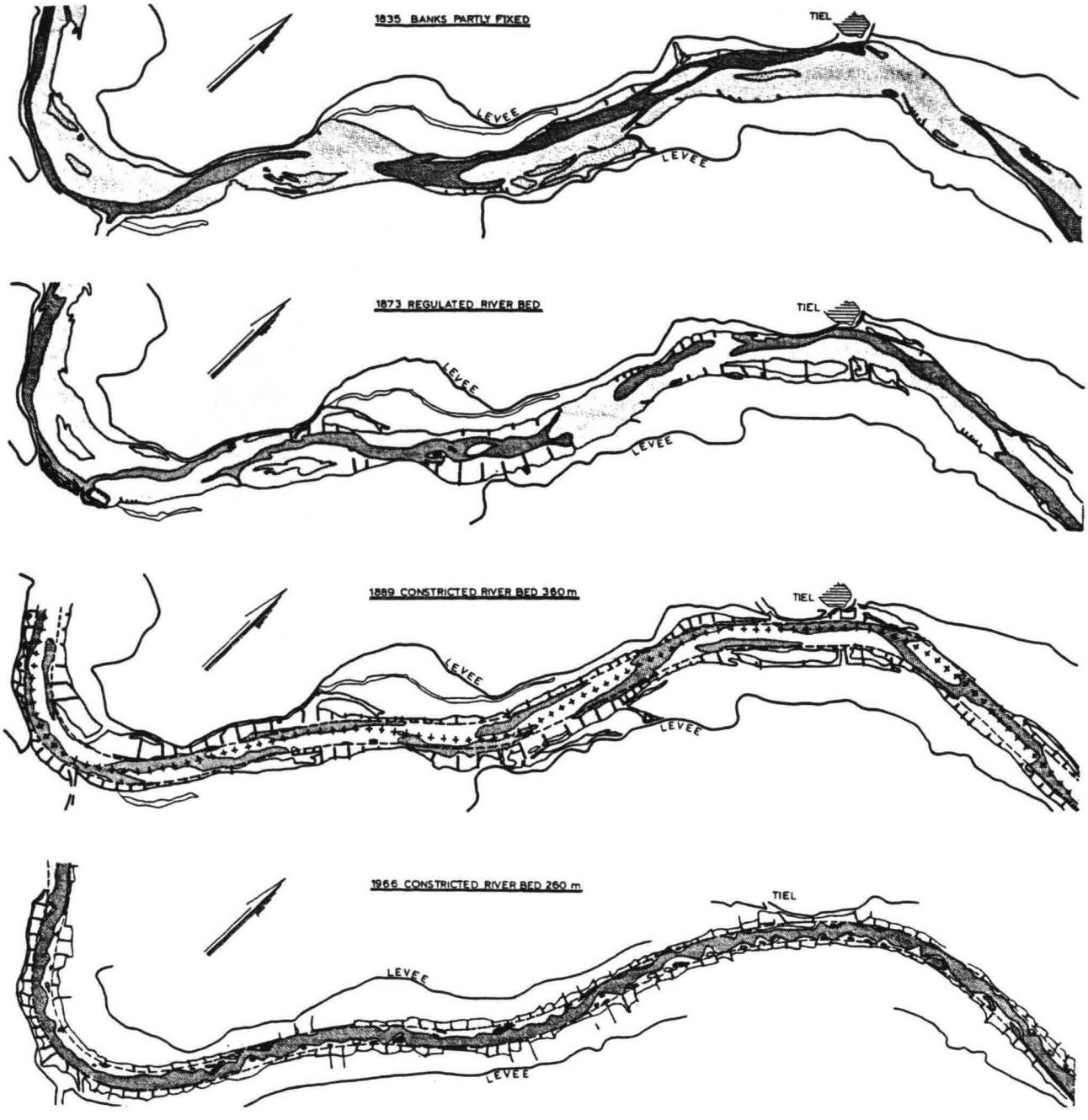


Fig. 11.11. History of the improvements on the section km 910-930 of the River Waal, the main branch of the Rhine in The Netherlands. Downstream to the left. There are only a few examples of river normalization in the world. River normalization is to constrict the width of the low water bed over a long river reach for navigational requirements. One of the best known is the normalization of the lower part of the Rhine in Germany and of the Rhine branches in the Netherlands. Normalization is the last step in bed regulation. It often starts with some local fixations and rectifications, followed by more extensive regulation. An example of successive steps in river improvement is shown above.

The first fixation structures were built by land-owners in order to protect their land against erosion. Later, dike boards and municipalities continued this work. Then the federal authorities started the regulation of large river reaches. Their aim was not to improve navigational conditions but to prevent formation of ice jams which might cause flooding of the adjacent land. The risk of jamming was reduced by the elimination of sand banks, islands and obstacles along the banks. In this century further constriction has been carried out, resulting in a width of 260 m for a large part of the River Waal. The navigational channel at low water has a depth of 2.5 m over a width of 150 m (normalization of the low water bed for navigational reasons).

River Maas improvement in The Netherlands

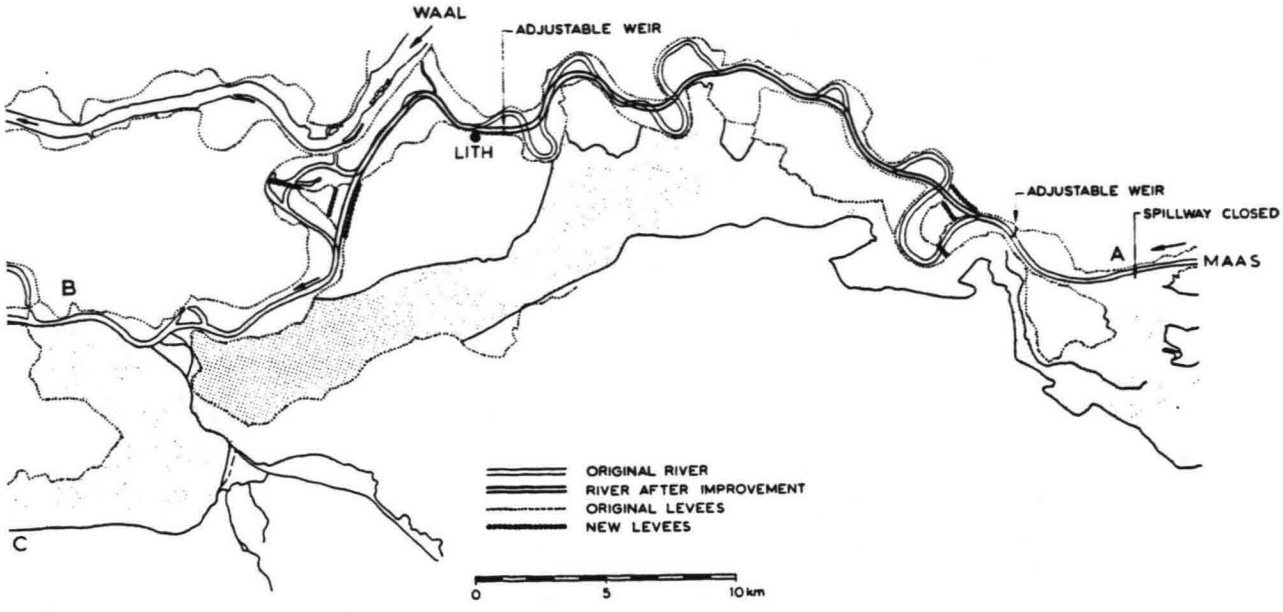


Fig. 11.12. Improvements of River Maas, The Netherlands, by shortening the river channel

An example of local improvement of the discharge capacity, in which a number of aspects were combined, is given by the river works carried out between 1930 and 1940 in the lower reach of the River Maas in The Netherlands.

Upstream (location A), a spillway in the levee on the left bank diverted flood water into a vast area (hatched in the figure), a part flowing into estuary waters (location C) and another part returning to the Maas (location B). In order to create good conditions for the development of the inundation area, the spillway was closed, after the discharge capacity of the river had been drastically improved between locations A and B. The engineering project was based on a design flood with a maximum discharge Q_{max} = 3200 m³/s having a return period of some hundreds of years. Previously the crest of the spillway was overtopped at a discharge Q = 1300 m³/s having a return period of about two years. Thus, in order to allow for the closure of the spillway, the discharge capacity of the river had to be greatly enlarged.

To achieved this goal the following improvements were carried out:

1. the discharge capacity of the flood plain was enlarged by locally lowering the ground level and by clearing many obstacles. However, the flood plain discharge capacity was of minor importance in comparison with the capacity of the low water bed.

2. the discharge capacity of the river channel was enlarged by widening it from 100 m to 135 m and by deepening it by about 5% of the maximum water depth. These improvements could be achieved because of the small sediments supply from the canalized river upstream, and because the dredged material could be used for industrial purposes.

3. the discharge capacity of the river channel was also enlarged by the river bend cut-offs which resulted in a reduction in length of the reach 50 km to 37 km. This gave a direct increase of 50% in the bottom slope. To prevent a gradual decrease of this steeper slope, an adjustable weir was built near Lith. This had the added purpose of maintaining navigational depth during the low-water stages.

Bifurcations

MEAN VALUES	Q m^3/s	S m^3/day	D_m mm
RHINE	2140	550	6
WAAL	1490	700	4
PANN. CHANNEL	650	300	4
NEDERRIJN	390	270	2
IJSSEL	260	130	5

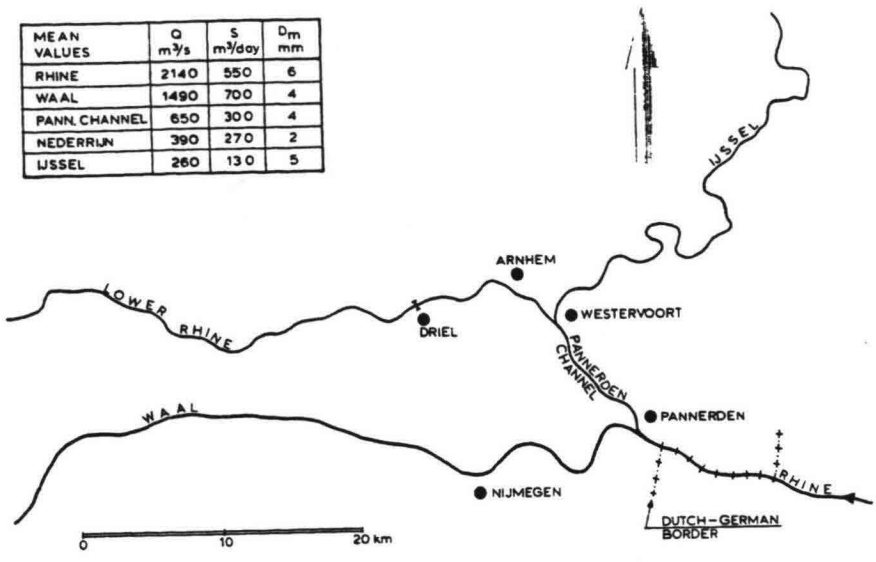


Fig. 11.13. The River Rhine bifurcates into the River Waal and the Pannerden Channel just downstream of the German-Dutch border near Pannerden.

Another bifurcation is situated near Westervoort, a distance of 11 km downstream of Pannerden.
Three aspects seem to determine the characteristics of these two bifurcations:

1. all the banks are fixed
2. all the branches are used for navigation
3. the morphological time -scale of the Rhine branches is relatively large.

The present geometry was fixed around 1930.

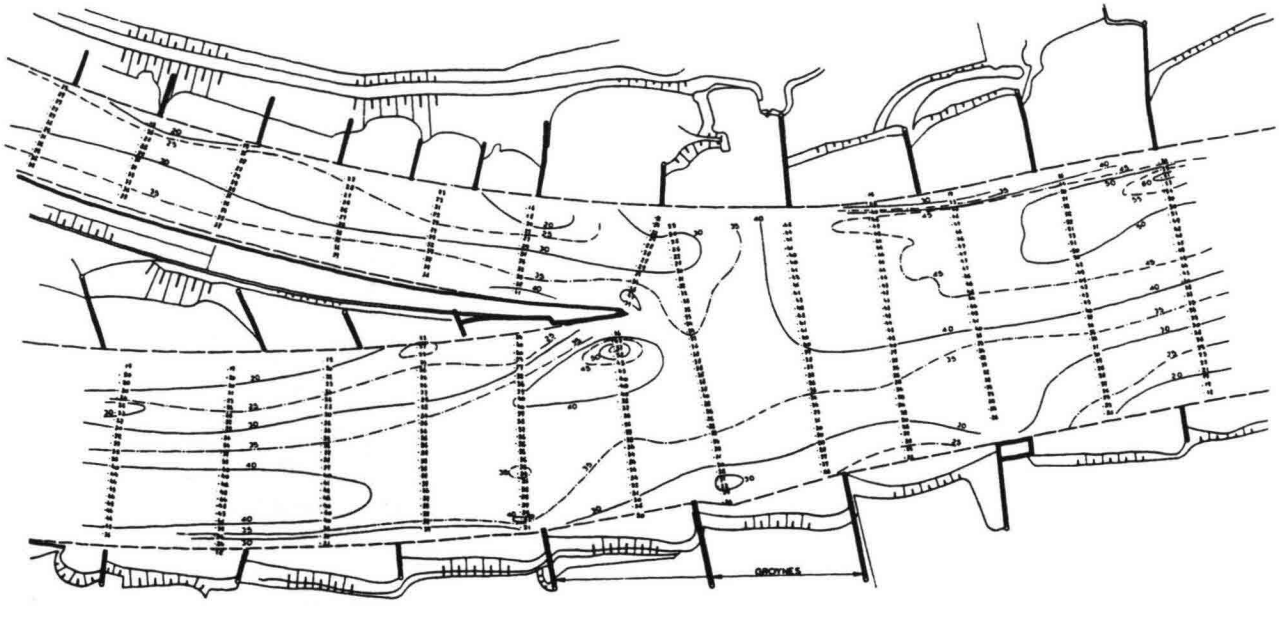


Fig. 11.14. Sounding chart with contour lines at a river bifurcation.

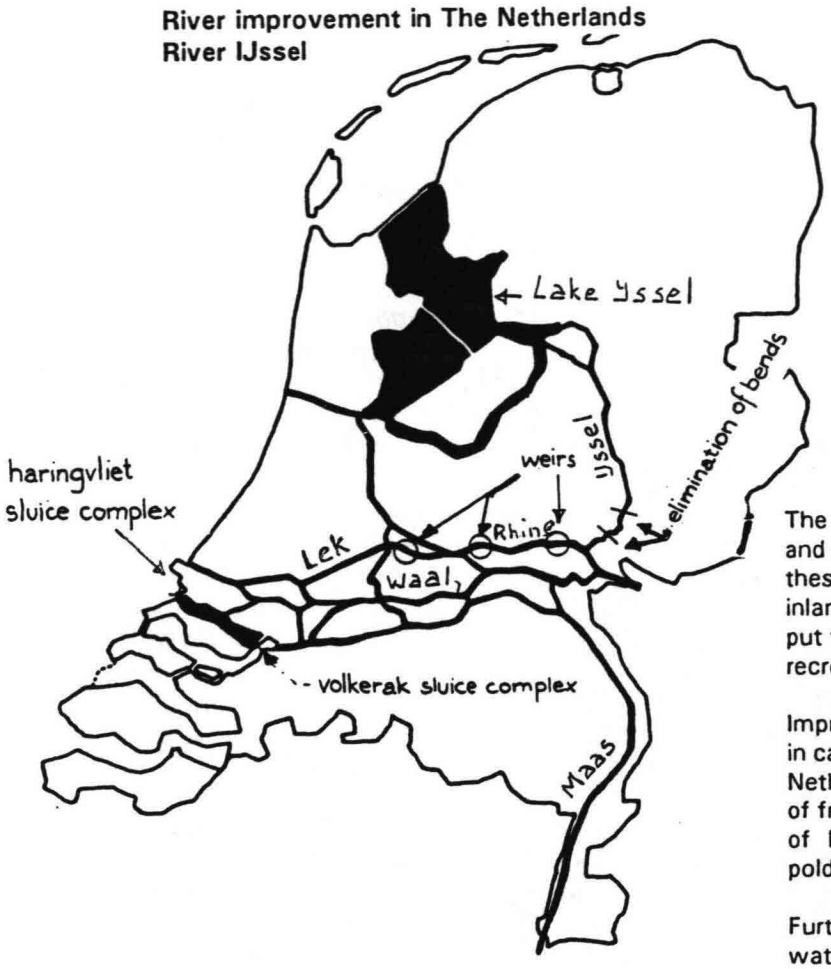


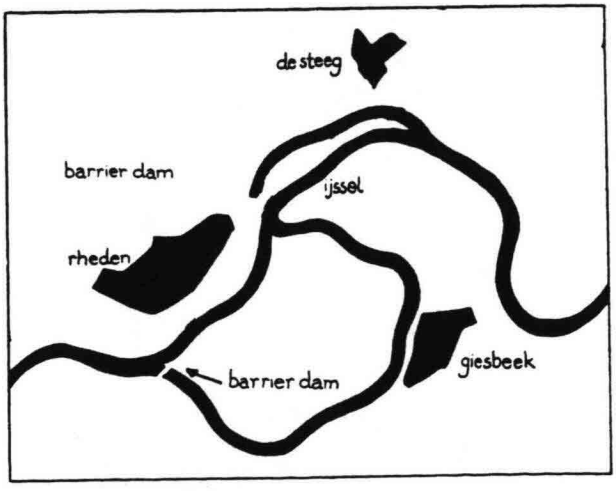
Fig. 11.15. The original wayward courses of the IJssel, the Lower Rhine and the Lek are no more. The need for space for the construction of the new IJssel highway and the need to improve the navigability of the IJssel as well as its effectiveness as an artery draining water into Lake IJssel have made it necessary to eliminate entirely two picturesque bends in the river and even to divert the tree-lined bend at De Steeg.

The same can be said for the Lower Rhine, the Lek and the Pannerdensch Canal. Along with the IJssel, these waterways are now better suited for use by inland shipping. The cut-off bends, too, have been put to good use; they are now intensively used for recreational purposes.

Improved water management was a major objective in canalizing the Rhine, for the northern part of The Netherlands depends on Lake IJssel for its supply of freshwater. It was realised that the surface area of Lake IJssel would be reduced as the new polders were completed.

Furthermore the IJssel does not supply enough water to cover consumption requirements in periods of drought. To improve this state of affairs the discharge capacity of the IJssel during periods of low water in the Rhine had to be increased, a task which is now performed by the weir complex at Driel. When the gates in the weir are lowered, a quantity of water from the Rhine is forced to flow into Lake IJssel via the IJssel. The weir at Driel thus serves as a "tap", making it possible to distribute the available water between the Lower Rhine and the IJssel.

In an average year 12% of the water discharged by the Rhine flows off through the IJssel. Navigation on the IJssel is possible with a flow of 250 cum/s, which gives the river a navigable depth of 2.7 m. In order to be able to maintain this level, the weir at Driel must be closed completely for about 50 days per year.
In order to able to cope adequately with all the dammed-off water the IJssel must be able to flow in a relatively unrestricted fashion, it must have sufficient "suction power," For this reason not only was a section of the river bed deepened but shortened too. This was done by eliminating two bend, which shortened the river course by 8 km. The sand which became available was used for road construction.



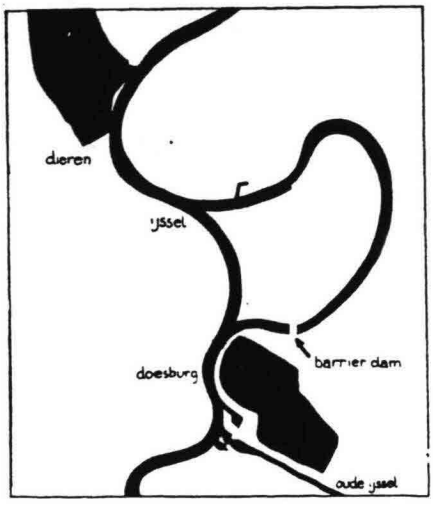


Fig. 11.16. Elimination of bends in the IJssel at Rheden and De Steeg in 1969 and at Doesburg in 1954.

Flood diversion

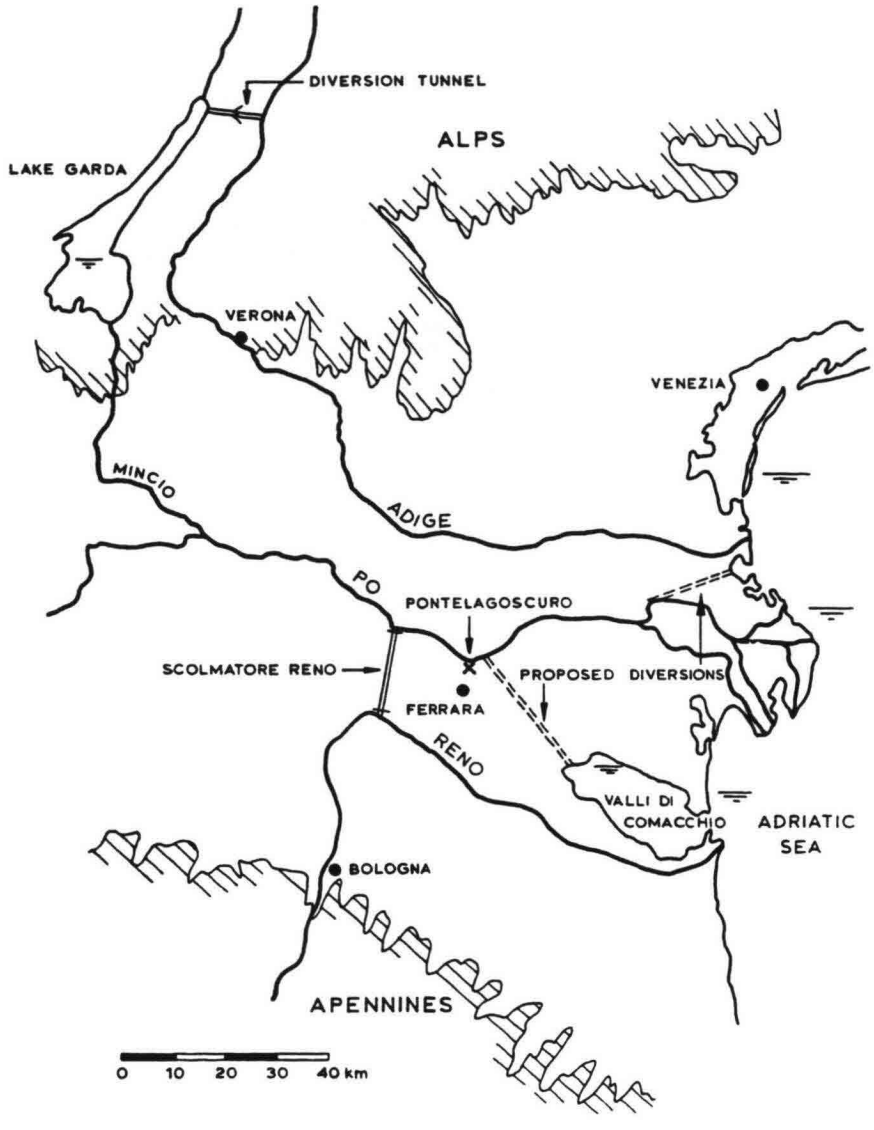


Fig. 11.17. Flood diversion to the River Po (Italy).
The diversion of flood water is a direct method to lower the top level of the flood waves. It has frequently been applied to protect reclaimed areas. The River Reno, flowing from the Apennines to the Adriatic Sea is a notorious river because of its high sediment load which results in a high bed level and dangerous flood levels.

In the time of Napoleon a project was carried out to divert flood water from the Reno to the Po River. The diversion canal, the so-called Scolmatore Reno, connected the Reno with the Po, with spillways at both ends. The increased probability of inundation of the Po area as a result could not be accepted and the spillway near the River Po was closed.

Another problem was the diversion of surplus flood water from the River Adige (which flows from the Alps into the Adriatic Sea) to lake Garda to avoid the inundation of towns like Verona. A successful solution was the drilling of water discharge tunnels through the mountains. These were provided with control gates and had a total discharge capacity of 500 m^3/s.

During the last decades the delta of the Po has subsided relative to the Adriatic Sea level. The crest of the levees along the river branches have subsided too. Heightening of all levees, built on a soft subsoil would be very expensive. A less expensive solution would be the construction of diversion canals. Two proposals are shown, but have been rejected for other reasons.

Lateral canals

In countries with substantial river transport like Germany and the Netherlands this has lead to the extension of the network of waterways by navigation canals. The function of an inland navigation canal is often to form a connection between a navigable river and a seaport, a centre of population or industry, or another navigable river. Sometimes, however, a navigation canal is constructed parallel to a section of a river, taking traffic away from that section. Especially if a certain river stretch is too wild or shallow, and improvement costs are too high, a lateral canal system might be considered.

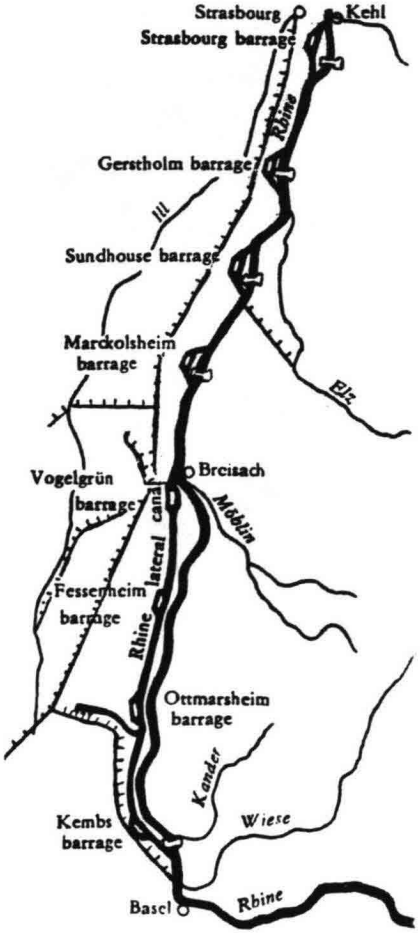
In Holland some examples exist of lateral canals along difficult riverstretches, but they are partly abandoned, partly remnants of the past, partly not really successful or require continuously modifications. Typical examples of lateral canals:

"Zuid Willems Vaart : mid-19th century, along river Maas, from Maasbracht via s'Hertogenbosch to Wehl, approx. 83 km.

Juliana Canal : opened 1935 along River Maas, between Maastricht and Maasbracht (Province of South Limburg), 35 km. long.

These two canals are shipping connections by-passing a difficult river stretch.

On both ends of the **Juliana lateral canal** hydraulic structures were built. Near Borgharen, just North of Maastricht a weir was built in the river Maas, so enough water could be diverted through the Juliana Canal. The result was that the river Maas itself, which forms the border between Belgium and The Netherlands up to the village of Stevensweert is very shallow on most places, through which with the help of gravel banks it is easy to cross. At the down end of the canal there is the navigation lock of Maasbracht, which has been renewed in 1966 (3 lock chambers of 16 x 142 m, maximum lift of 12.25 m, minimum depth on sill 3.60 m, mitre gates). Furthermore there are navigation locks in the canal at Limmel and Born. The lock at Limmel is usually open. The total difference in water level between both ends is 23 m.



The navigation route was shortened by 16 km by this lateral canal. The width of the canal at water level was originally 46 m but has been widened to 70 m in the sixties. The canal is suitable for ships up to 2000 tons.

The **Apeldoorn Canal**, 2nd half 19th century, was a shipping connection along the shallow IJssel river but was abandoned in the sixties after the improvement of the river IJssel.

Nowadays all the canals in The Netherlands used for navigation have at least a double function. Not only shipping considerations but also water management aspects (or "irrigation") play a role.

Fig. 11.18. The French lateral canal on the Rhine has caused the Rhine almost to dry up between Basel and Breisach. Further work in the form of a "looping" arrangement has been proposed, by which water that has flowed through the power stations is returned to the Rhine.

Levees and canal dikes (embankments)

A canal embankment is a structure constructed for an artificial watercourse serving one of various purposes (fig.11.19), such as transport, irrigation, water power. It guides water as we choose and prevents water loss. Considering their aims, they can be designated as **hydraulic structures** and divided into five groups :

(a) hydroelectric plant canal embankment - taking water to a power station;
(b) embankments of irrigation canals - agricultural purposes;
(c) waterway embankments - transport purposes;
(d) canals for fish;
(e) multi-purpose canals.

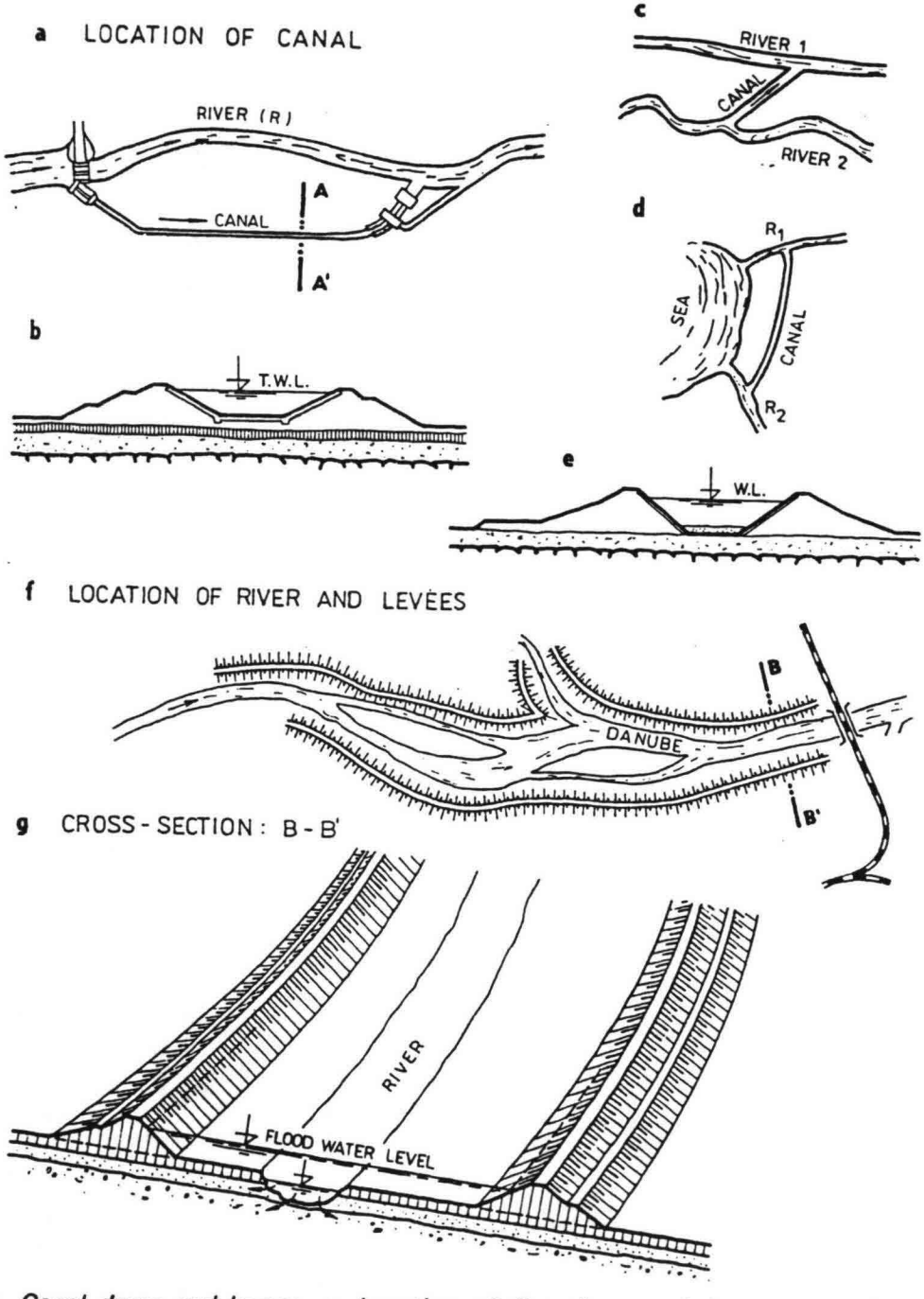


Fig. 11.19. Canal dams and levees. a. Location of diversion canal; b. cross-section; c. location of canal connecting two rivers; d. connecting canal near the sea; e. enabling navigation; f. location of river and levees, levees protecting land but enabling communication for groundwater; g. cross-section of river and levees.

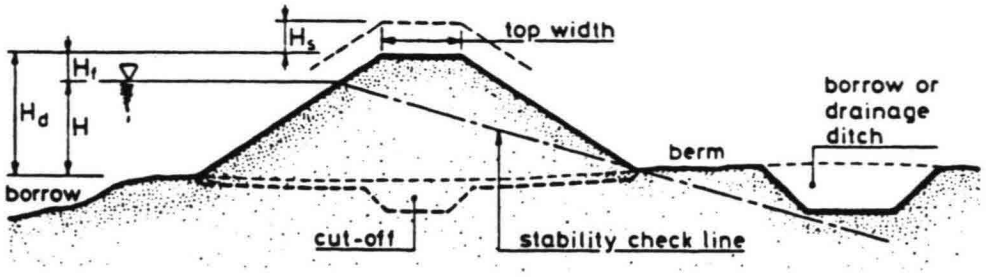


Fig. 11.20. Section through dike.

12. Revetments (bank- and bottom protection)

A revetment is placed at the boundary between water and soil. The function is to protect the soil from eroding by water (current, waves). It is used when non-acceptable scour (erosion) is expected of non or not enough cohesionsive bottom and bank materials (sand, silty sand).

Permanent applications :	**type of attack**
* rivers and shipping canals	currents (by discharge) currents and waves induced by ships
* protection of coasts : basis of groins	(tidal) currents, wind waves
* breakwaters and harbour dams	(tidal) currents, wind waves
* protection of marine pipelines	(tidal) currents, wind waves
* downstream of discharge sluices or navigation locks	(tidal) currents with high turbulence and large energy losses
* downstream of weirs, barrages, dams with an overflow	currents

Temporarily application :	
* during construction stage of an estuary closing or a dike breaching	(tidal) currents with high turbulence and large energy losses

The **classification** of revetments can be done in many ways. One method is according to **permeability** and **flexibility.**

Flexibility \ Permeability		High ⟶ Low
High	Loose material	Filter construction
	'caged'	Gabions
	loose ballast	Willow mattresses —— Matt. + polyprop. cloth
	Mattresses fixed ballast	in situ mattresses —— Polyprop. + concrete blocks ———— polyprop. + gravel
	Cohesive materials	Sand asphalt —— Fix stone - - - - - - clay
Low	Monolithic constr.	———— grouted rubble ———————— asphalt concrete ———————————— concrete

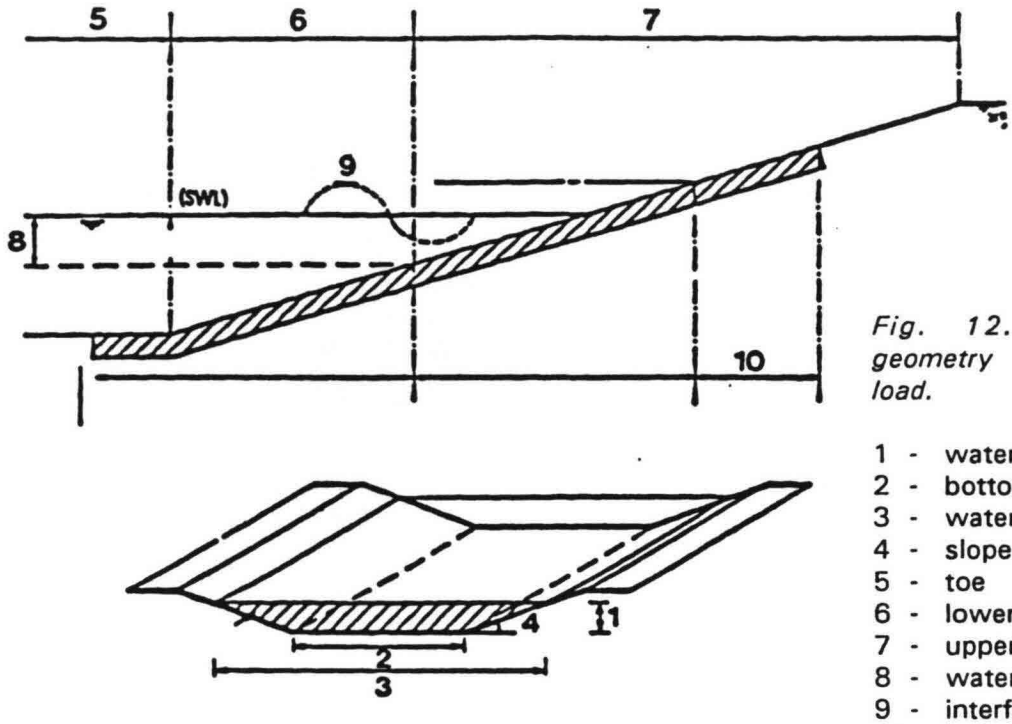


Fig. 12.1. Waterway geometry and hydraulic load.

1 - water depth
2 - bottom width
3 - water line width
4 - slope angle
5 - toe
6 - lower revetment
7 - upper revetment
8 - water level depression
9 - interference peak
10 - wave run-up zone

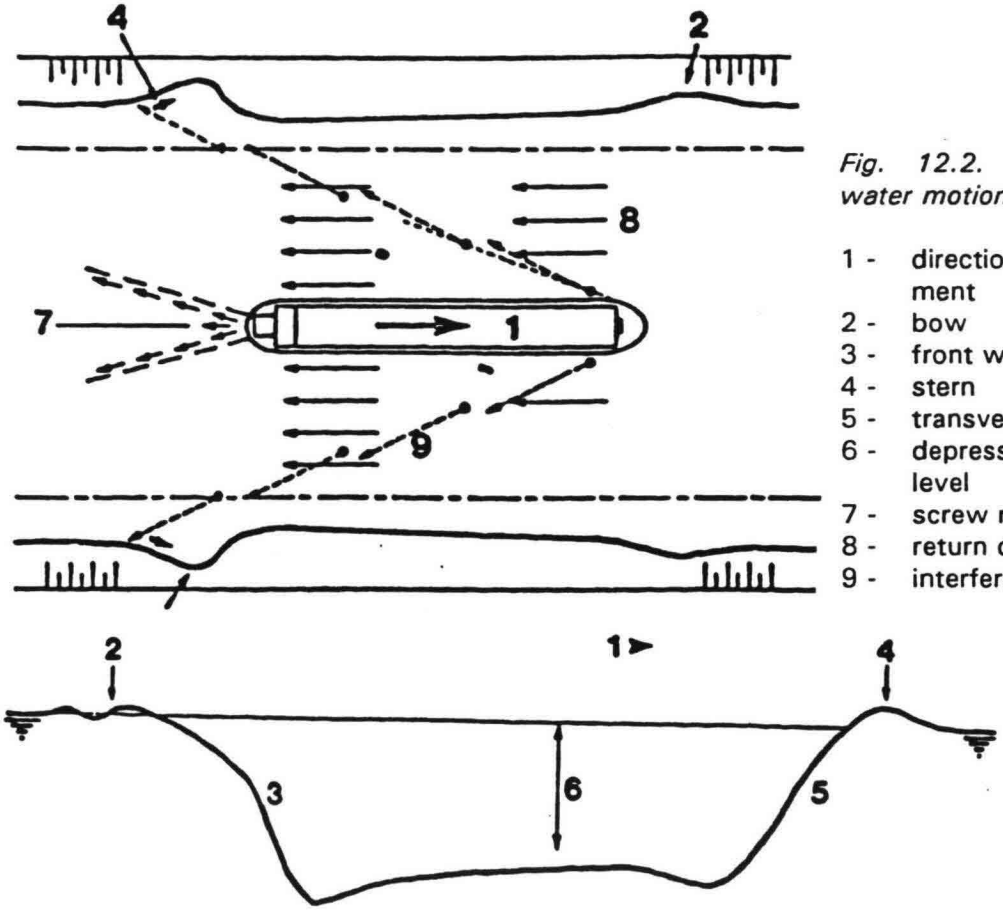


Fig. 12.2. Ship induced water motion.

1 - direction of ship movement
2 - bow
3 - front wave
4 - stern
5 - transversal stern wave
6 - depression of water level
7 - screw race
8 - return current
9 - interference peaks

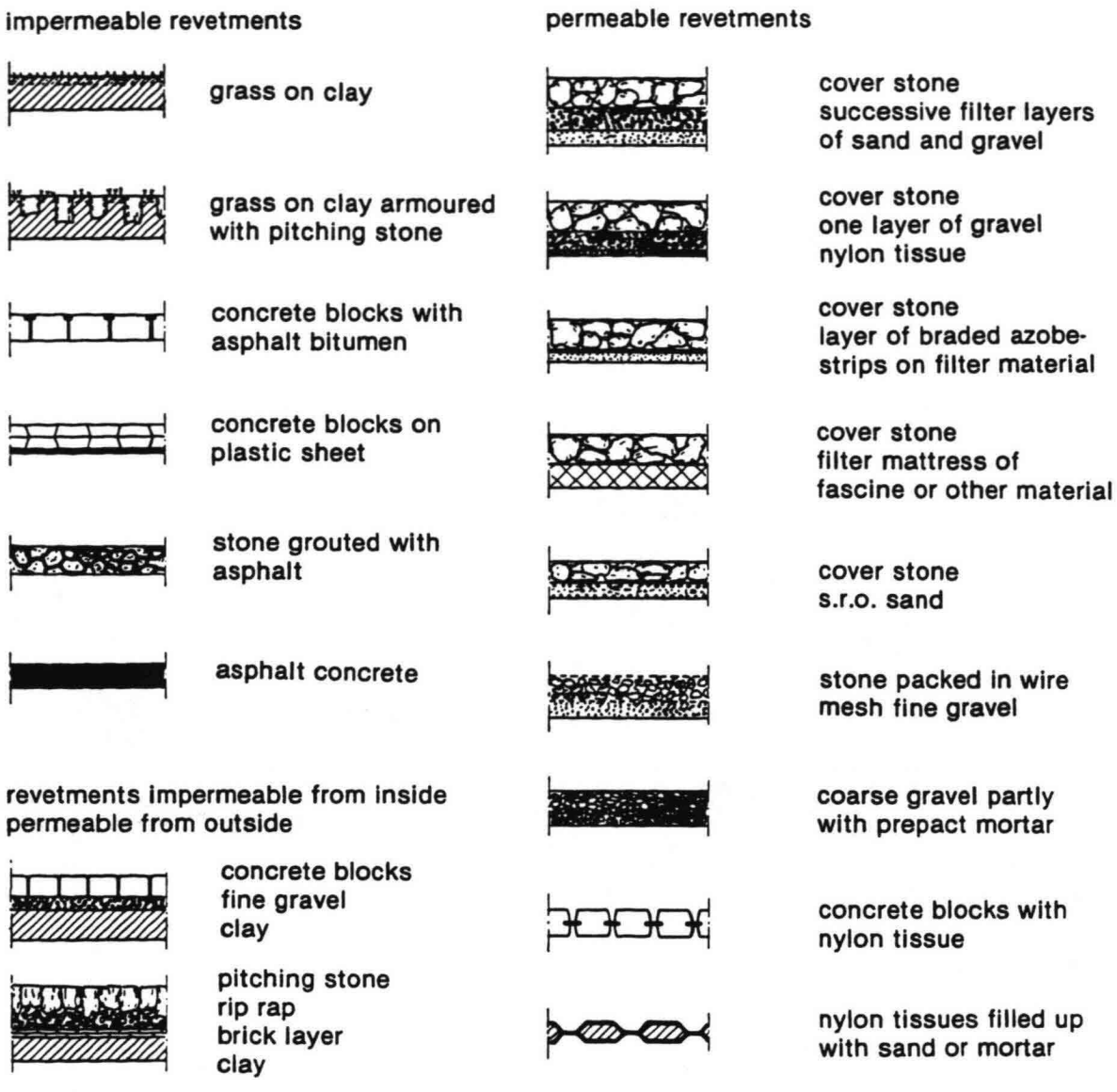


Fig. 12.3. Examples of revetment constructions.

In general, two main types of revetments can be distinguished: namely permeable and impermeable. The permeable types have a (more of less) "open" surface whereby water may flow into and out of the structure. Permeable types generally require one or more layers of filter material to retain the soil particles (sand underlayer). Following principles types of permeable revetments:

- grassmats on a permeable sublayer
- open filter revetments
- stone pitching
- artificial block revetments

The impermeable types prevent water from flowing through the structure, so that hydraulic pressures may build up behind (important factor in the design). Types:

- glassmats on an impermeable clay-layer
- bitumen grouted stone, block or slab revetment
- asphalt revetments

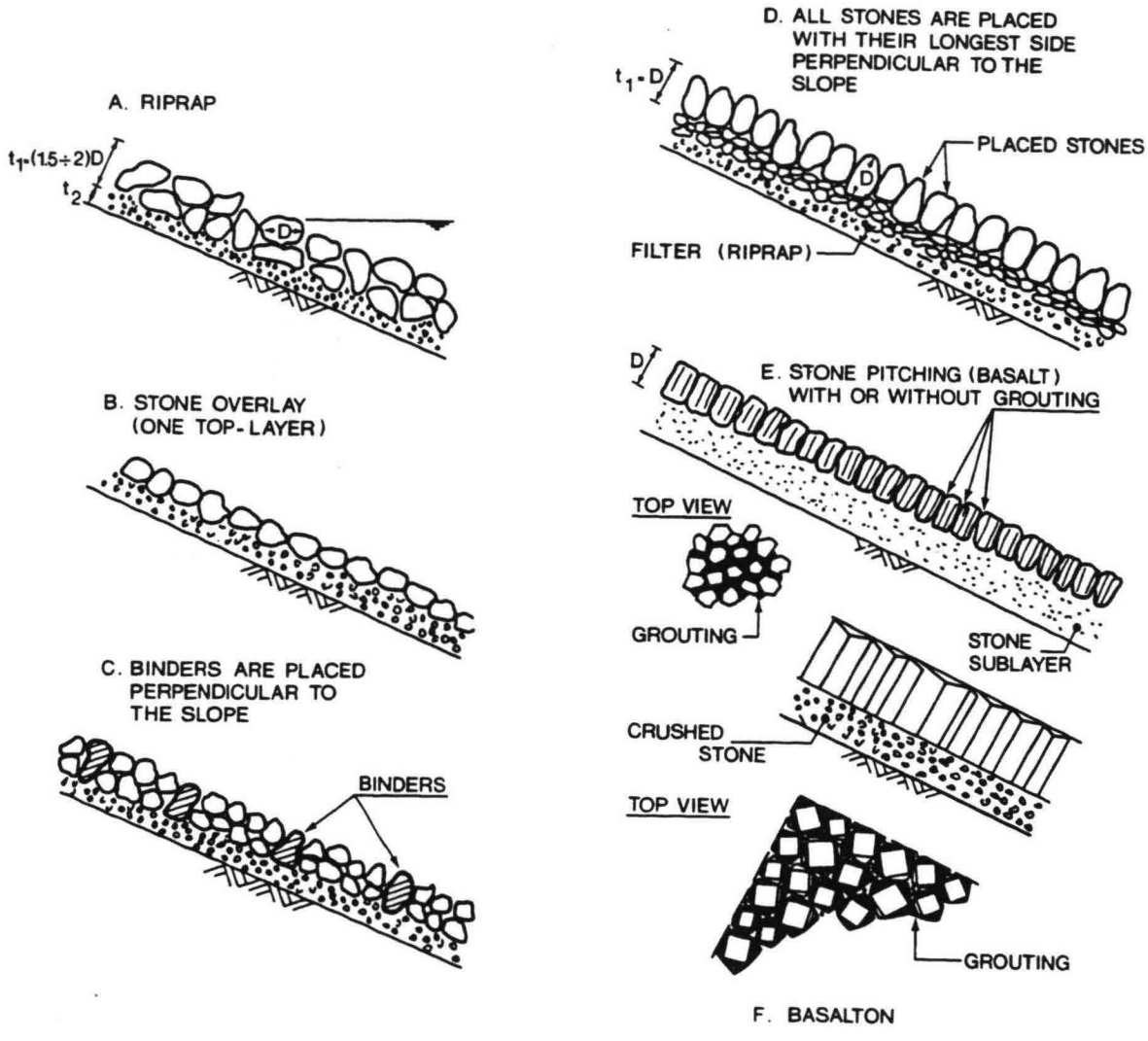


Fig. 12.4. Riprap design and improving measures.

There has been an increasing need for reliable information on the stability of riprap exposed to wave action. A riprap (also called "open filter") revetment consists of several layers of filter material of different size covered by one or more layers of rock or rubble. The weight of the individual rock or rubble forming the top layer has to be in accordance with the anticipated wave forces. However, special measures may be taken in order to increase riprap stability:

- one-layer stone overlay greatly improves riprap stability
- placing some of the largest stones as binders perpendicular to the slope
- stone pitching; a very traditional type (especially in the Netherlands) is the natural basalt prism revetment placed on a filter construction. Because of the high cost and shortage of natural basalt, an artificial concrete basalt-block (patented as 'Basalton') has been developed in the Netherlands.
- grouting; the best way substantially to increase the strength is to grout the prisms with graded crushed stones or copper slag as usually done in the Netherlands.

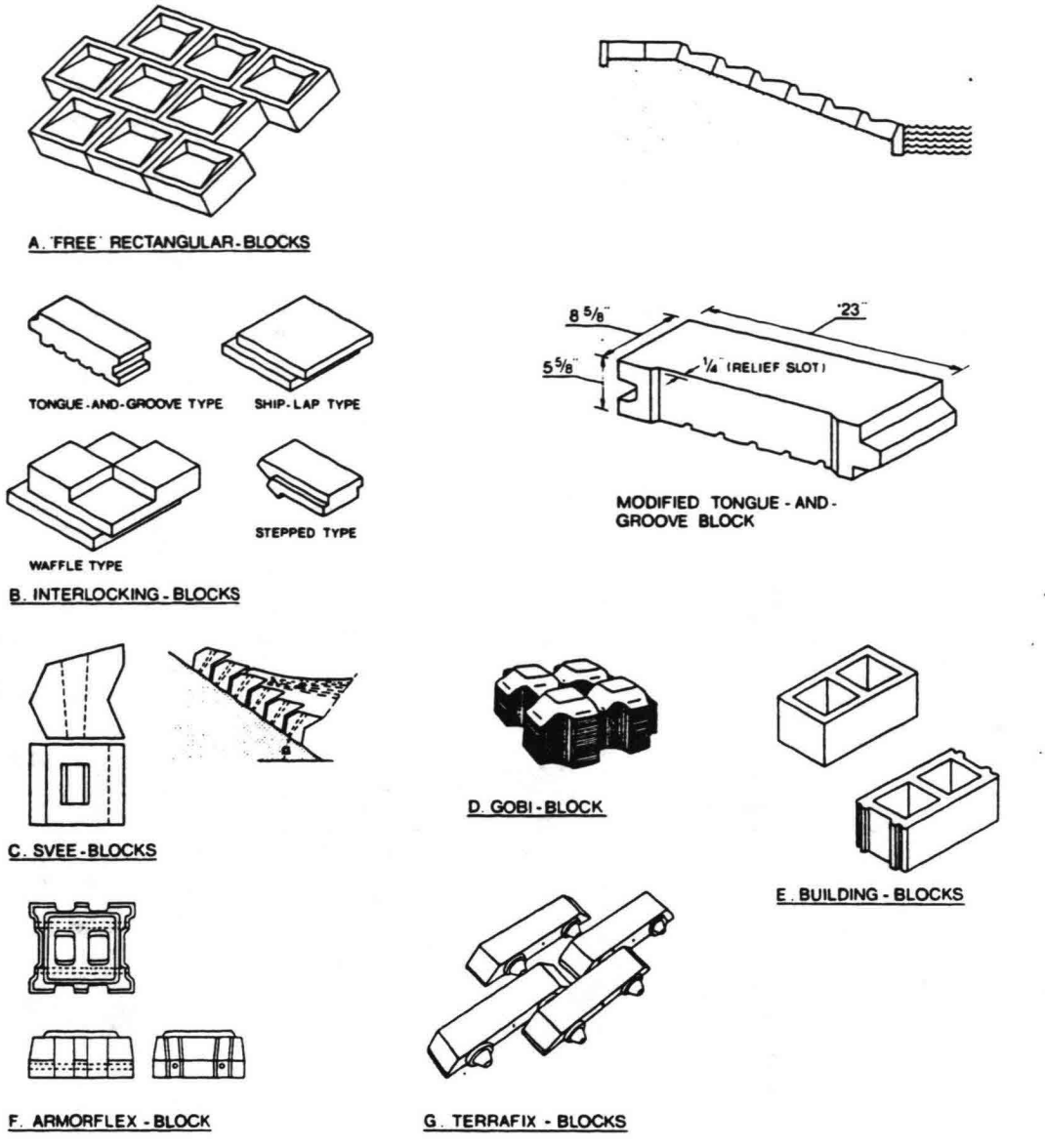
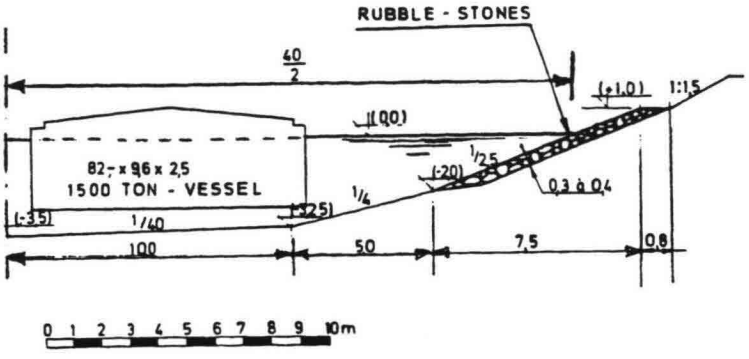


Fig. 12.5. Examples of revetment blocks

Continued demand for relatively low-cost shore protection in estuaries and along the coasts has stimulated investigations into this subject. The reason has been the increasing problems with respect to the defence of the coasts and banks and the high cost of natural materials. This demand has resulted, inter alia, in the rapid development of artificial block revetments. The quality of concrete blocks has improved and the cost has diminished as the result of mechanical placing.

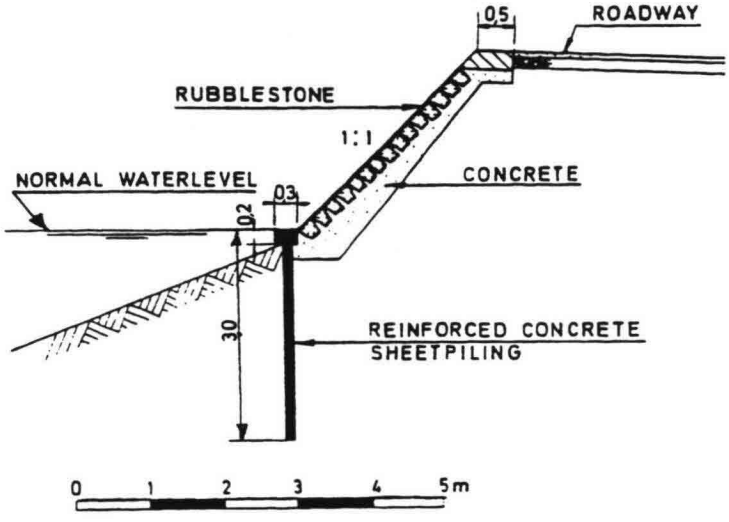
a. Free block revetments - rectangular blocks of different design (concrete, copper-slag)
b. Flexible interlocking blocks of different design
c. Svee blocks
d. Gobi blocks are small, cellular, unreinforced concrete blocks, mostly 200 x 200 mm. and 100 mm. high. Two types: one with straight and one with bevelled sides. Straight-sided blocks are placed individually on a filter cloth at a desired site; the bevel-side blocks are usually glued to a plastic filter at the factory and shipped to the site as a mat.
e. Cellular concrete building blocks.
f. Armorflex block slope protection mats; should be seen as the next step at the basis of the Gobi mat. The blocks are threaded with steel or nylon cables and bound together, thus forming a flexible and structurally integrated mat system.



Bank protection of inland navigation canals (1920 - 1960)

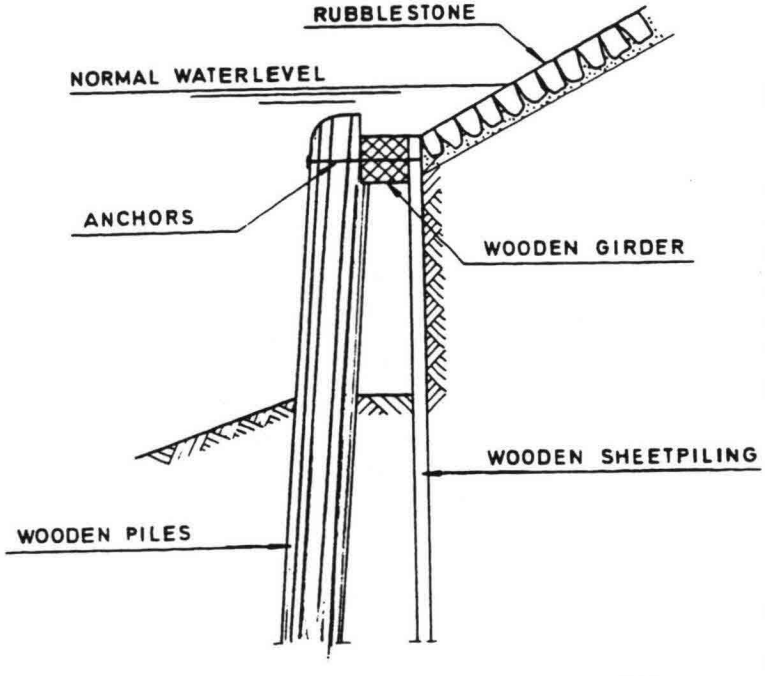
Dortmunt - Ems Canal (Germany)

Early quarry-stone protection of a polygonal cross-section of the canal. The inclination in the vicinity of the water level should not exceed 1:2.5, while the quarry stone layer should be built up gradually in order to act as filter.



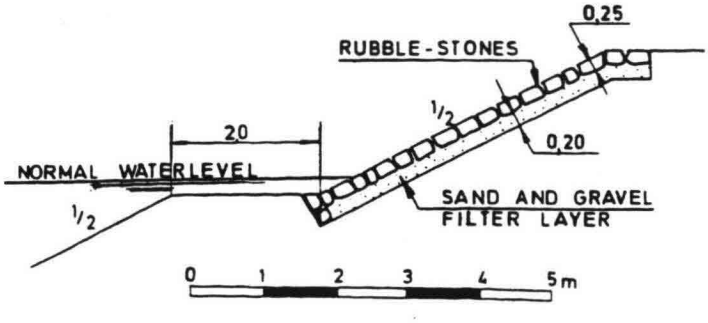
Albert Canal (Belgium)

Early bank protection type; the slope has an inclination of 1:2.5 below the normal water level; the upper part is steeper (1:1 in urban areas and 1:1.5 elsewhere), and protected with rubble-stones placed in a concrete layer. This revetment is supported by a continuous reinforced concrete capping which rests on a vertical sheet piling. Prefabricated as well as cast in place reinforced concrete elements were used.



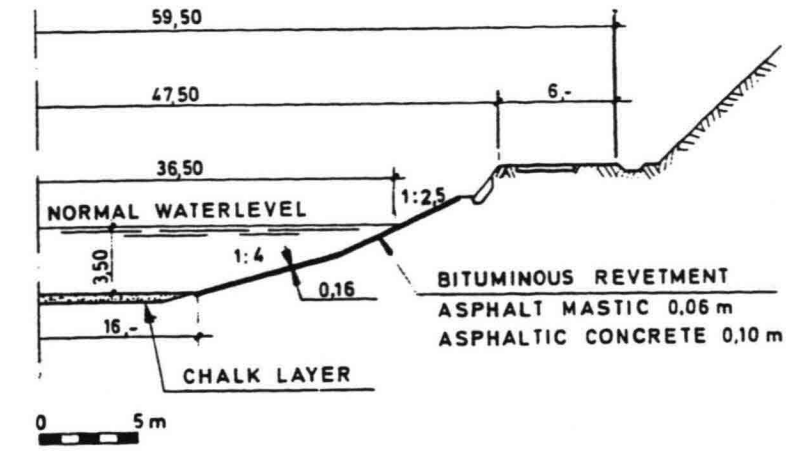
Merwede Canal (The Netherlands)

In The Netherlands vertical anchored walls were still commonly used. The figure shows wooden sheetpiles; instead of anchors inclined piles have been chosen to retain the vertical wall. The construction is kept entirely beneath the water surface with due attention being given to the design life of the wood.



River Soraksarer (Danube, Hungary)

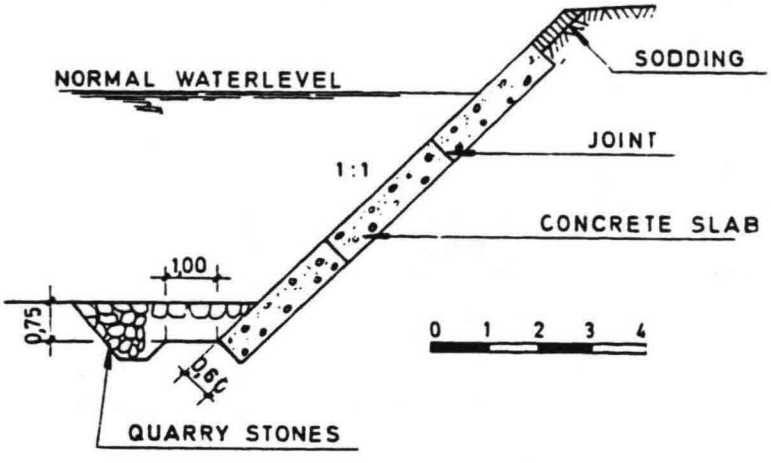
The slope is divided into two parts, an unprotected bottom slope beneath the water surface and a protected slope there above. Both parts are separated by a horizontal berm of 2 m width.



Bank protection of inland navigation canals (after 1960)

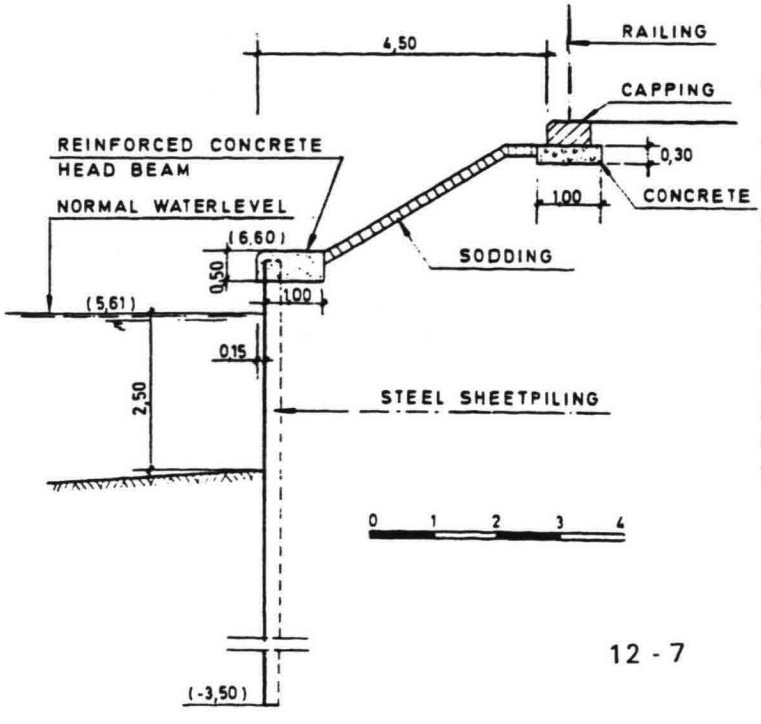
Nimy-Blaton (Belgium)

Example of the application of bitumen in revetment-construction following the successful application of bitumen in road construction. The slope of this example is entirely covered with a double bituminous revetment. The early asphaltic-revetment types were impermeable and they had the advantage of being sufficiently elastic.



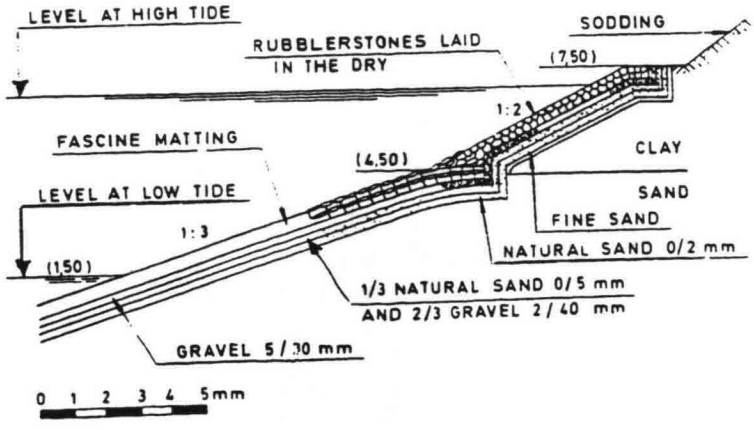
River Sambre (Belgium)

Revetment with cast-in-place concrete. In contrast with the bituminous revetment, the concrete layer was rather thick (0.60 m) and needed joints due to the fact that it was unelastic. In this case a toe-protection has been foreseen which should protect the lower part of the revetment from scouring, and as a consequence, from excessive settling.



Ghent-Bruges Canal (Belgium)

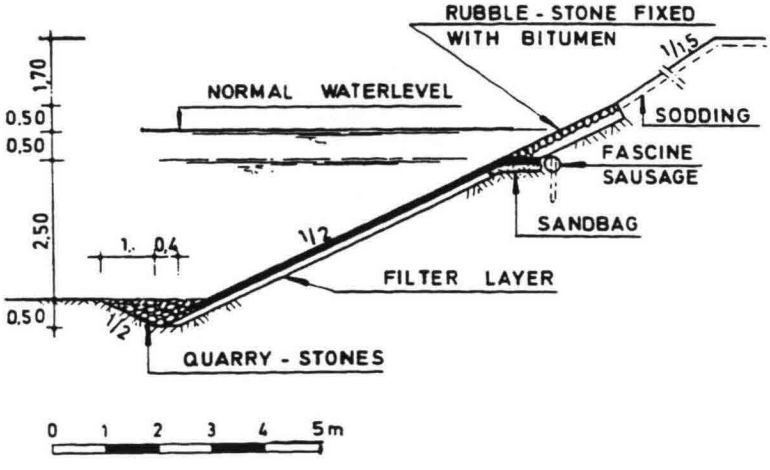
The sheetpiling dimensions increased according to the required water depth and steel sheetpilings became customary. Steel sheetpilings can be driven in the soil with economical pile-driving techniques. The separate sheets are interlocking and give good ground tightness. In general, the top of the sheet piling was covered with a cast in situ reinforced concrete beam; against which ships could be moored.



Bank protection of inland navigation canals (after 1960)

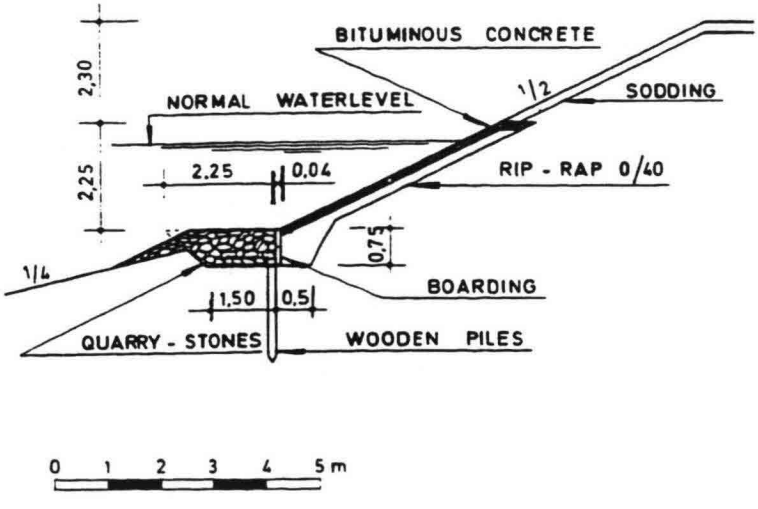
Ringvaart at Ghent (Belgium)

A filter of sand and gravel layers has been foreseen in order to overcome uplift-water pressures. The revetment itself has been built with fascine mattings on the lower part (1:3) and with hand placed rubble-stones on the main slope (1:2). Construction done in "the dry".



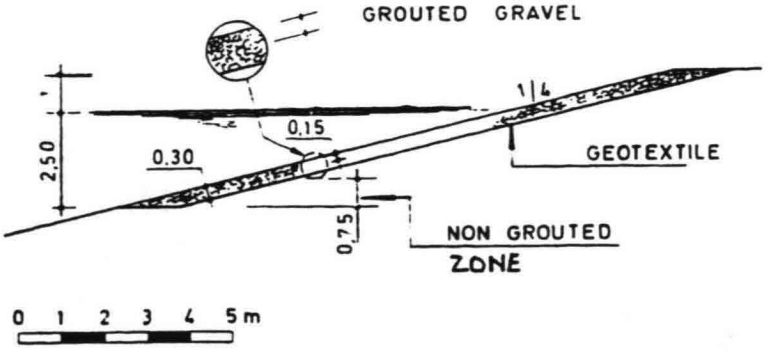
Dortmunt-Ems Canal (Germany)

Essentially composed of an asphalt matting on a slope with inclination 1:2. The matting is properly protected and anchored at the toe and top. Until then, it was quite difficult, even impossible to use hot asphalt mixes more than 0.50 m. below the water surface. Prefabricated asphalt mattings can easily be placed beneath the water surface. Special care has to be taken in order to avoid erosion at the overlapping zone between adjacent elements.



Connection of Bourbourg Canal with the Port of Dunkirk (France)

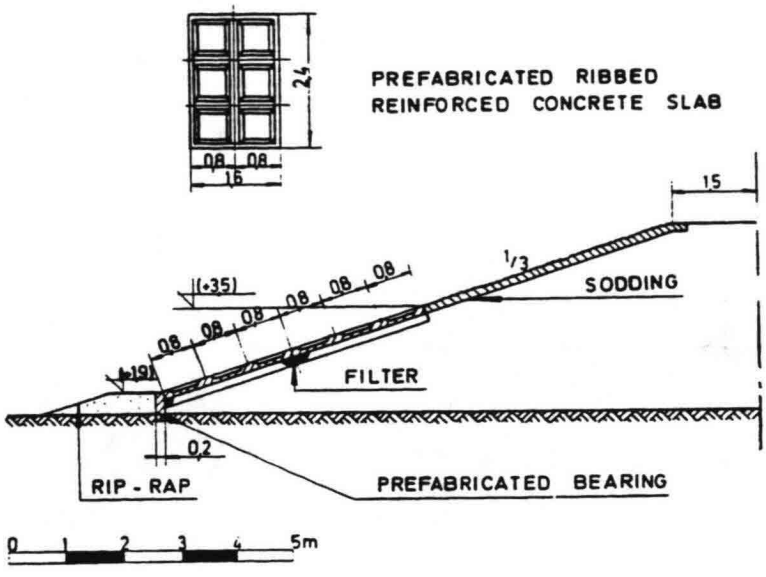
Sophisticated application of asphaltic concrete; the revetment is supported by a filter layer of rip-rap and is retained at the toe by vertical piles with boardings and a quarry-stone protection. The revetment needs no special maintenance: while on the contrary, construction costs are rather high.



Bank protection of inland navigation canals

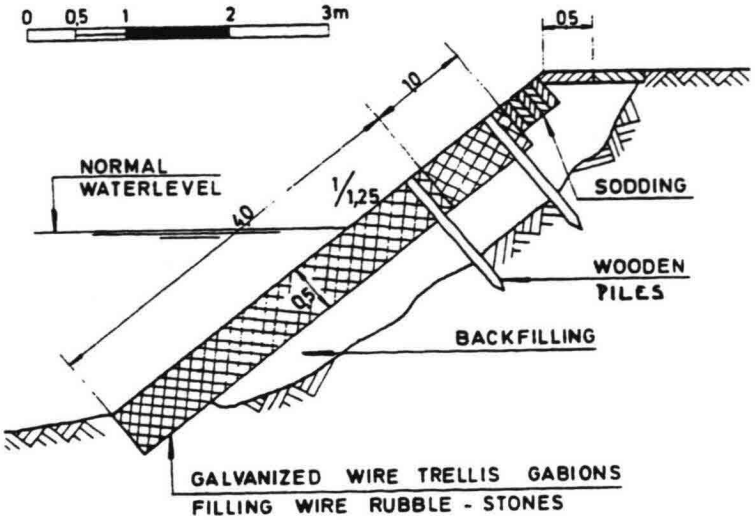
Bank protection in The Netherlands built up with gravel and grouted with cement mortar

On the slope, a nylon filter cloth is used beneath the revetment in order to prevent loss of soil particles. The gravel layer is spread in a thickness of 0.30 to 0.40 m and then grouted with a cement mortar. Its consistency is such that the mixture penetrates to a depth of about 0.15 m.



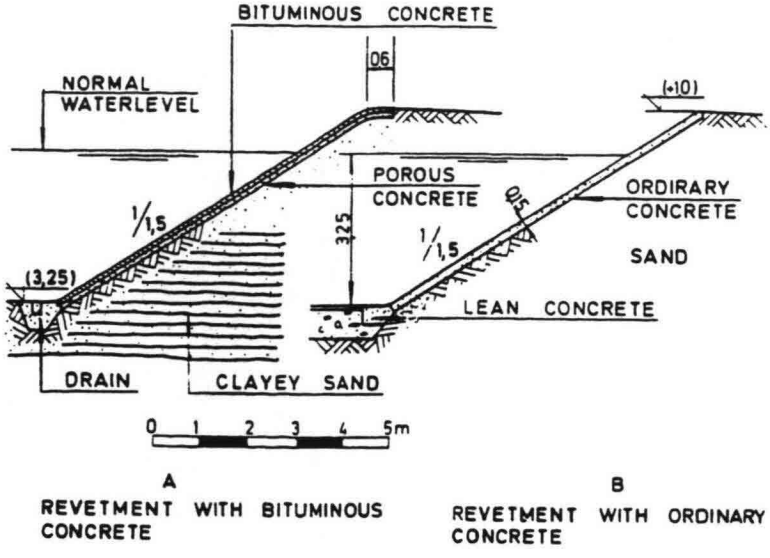
Lake Zegrne (Poland)

Prefabricated reinforced concrete slabs of small dimensions, placed on a filter layer of gravel. The slabs are only 0.04 m thick and area ribbed along the top in order to dissipate the energy of the waves.



Ghent - Bruges Canal (Belgium)

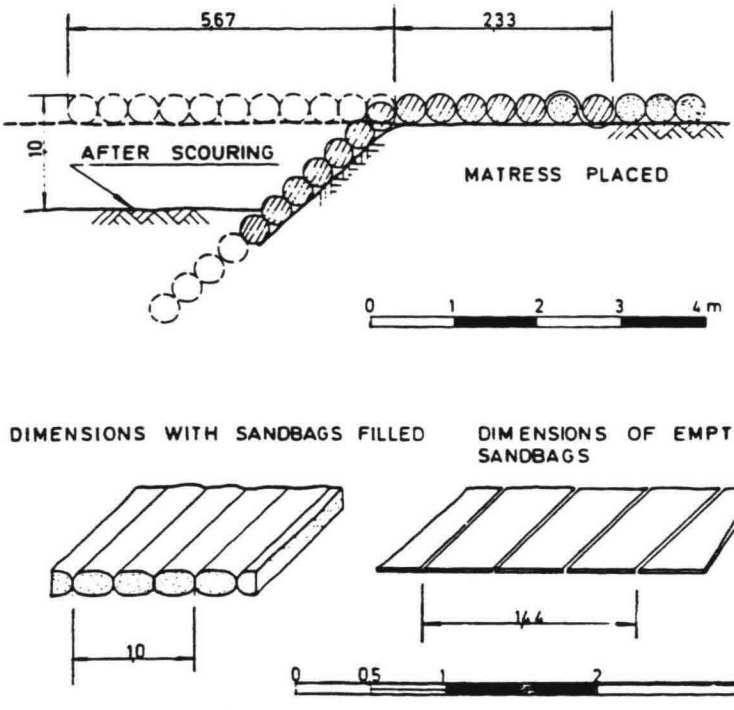
In order to improve rubble stability on banks, the rubble can be enclosed in galvanized wire mesh gabions. The mesh is rectangular (0.10 - 0.12 m). Cages are formed of 0.5 by 1.0 m. Rubble size: 0.15 - 0.30 m. Unusual steep slopes are possible - but cost price per m^2 rather high. First applications in Belgium in 1961



Bank protection of inland navigation canals

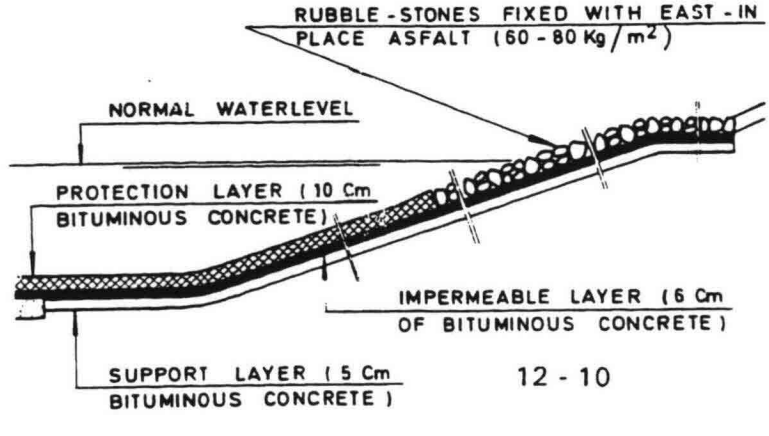
Northern Canal (France)

Impermeable revetment. In cases where deformation of the dike is expected, a ductile revetment type should be applied. Fig. A: bituminous concrete layer laid upon a porous concrete support layer. Top layer has a minimum thickness of 0.06 m. Fig. B: ordinary concrete (water tight), of 0.15 m thick and casted in 6 m width sections.



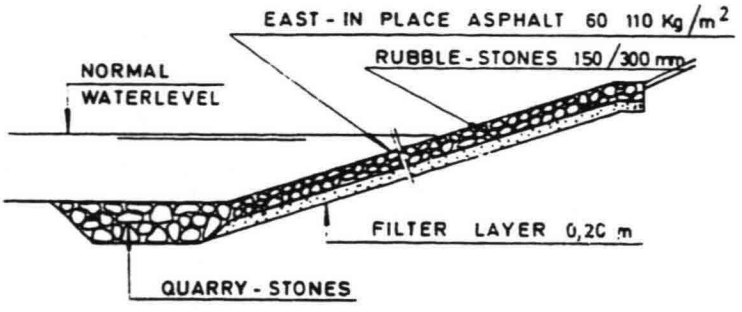
Sandbag mattresses (Japan)

This method is invented and widely used in Japan. The sacks are fabricated with nylon thread and woven to units of suitable dimensions. Special sandfilling equipment is used. The mattress is resistant against corrosion and ultraviolet rays. Highly flexible. Very light product - economic transport and handling but sand needs to be available.



Impermeable revetment (Germany)

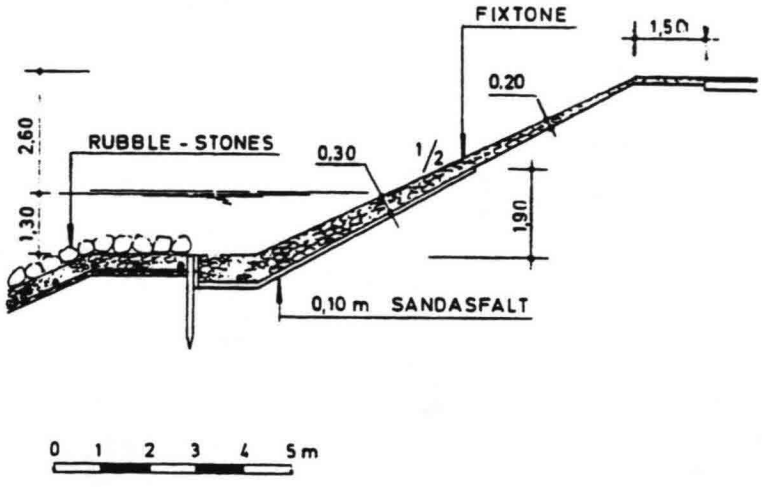
Applied in locations where the water level in the canal or the river is above the phreatic level in order to prevent the loss of water by infiltration and the formation of weak zones on the bank site.



Bank protection of inland navigation canals

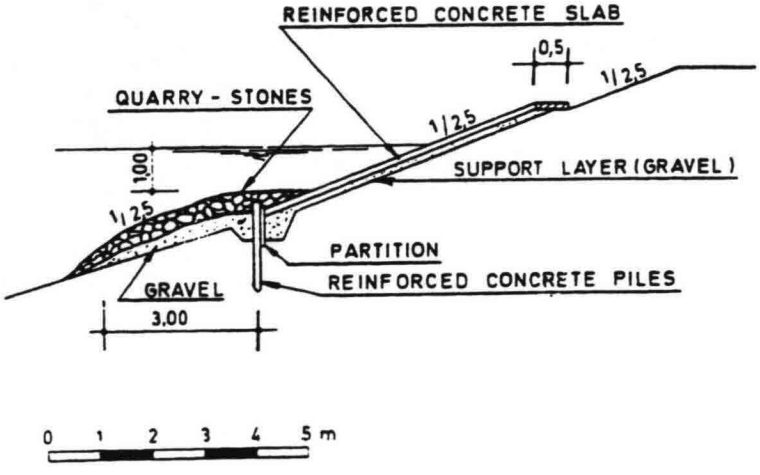
Permeable revetment (Germany)

Applied where the aquifer feeds the waterway. In order to prevent erosion of the soil beneath it, it is indispensable to construct a suitable filter layer (filter cloth + gravel).



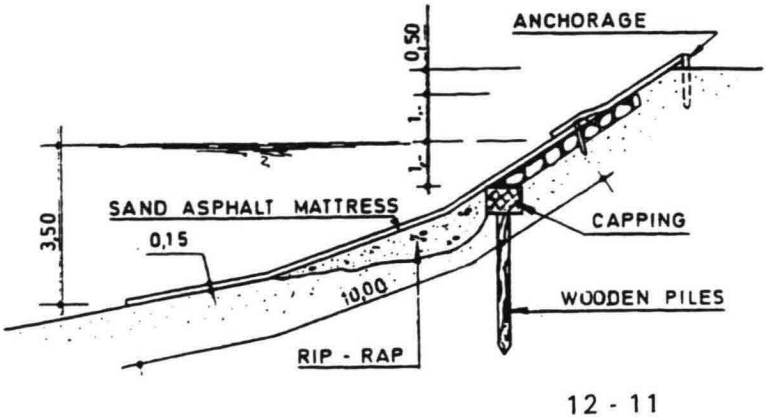
Ocean Harbour Rotterdam (The Netherlands)

Upper part is a fixtone-layer on top of a sand-bitumen filter. Lower part of the slope is protected by means of a classical fascine mattress and rubble-stone covering. The transition is formed by means of a pile row with boarding. Fixtone is a permeable asphalt mixture, consisting of gravel and a bitumen coating which is a premixed asphalt mortar.



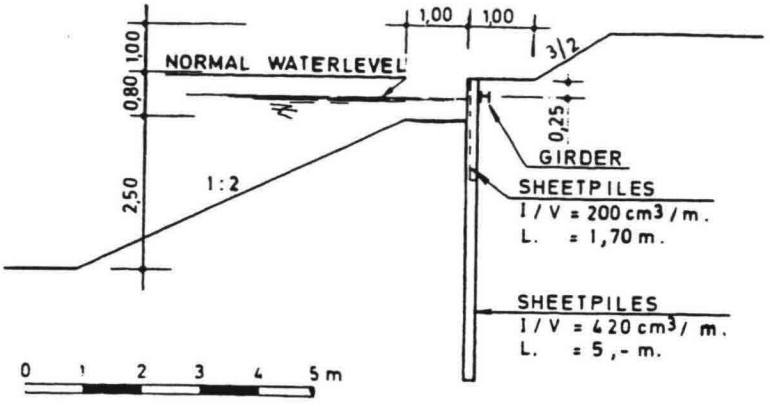
Moskva Canal (USSR)

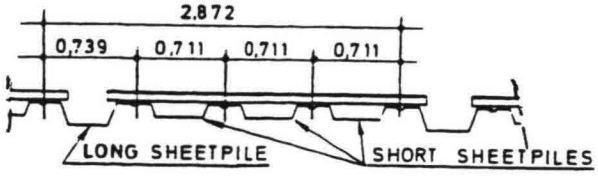
Precast reinforced concrete slabs of 6 x 2 m size and 0.15 thick-ness, placed over a 0.3 m thick pad (gravel) and inter-welded by a metal strip (60 x 5 mm). On the lower part (-1.5 m below water level) there is a cut-off in the form of vertical reinforced concrete plates.



Ferrarese waterway (Italy)

Restoration of old bank protections, the original slope was rebuilt by filling with stone rubble and refacing it with mastic grouted mattresses launched from a pontoon. "Reno" mattress, is a rectangular cage of steel wire mesh divided into several cells and filled with quarry stone.

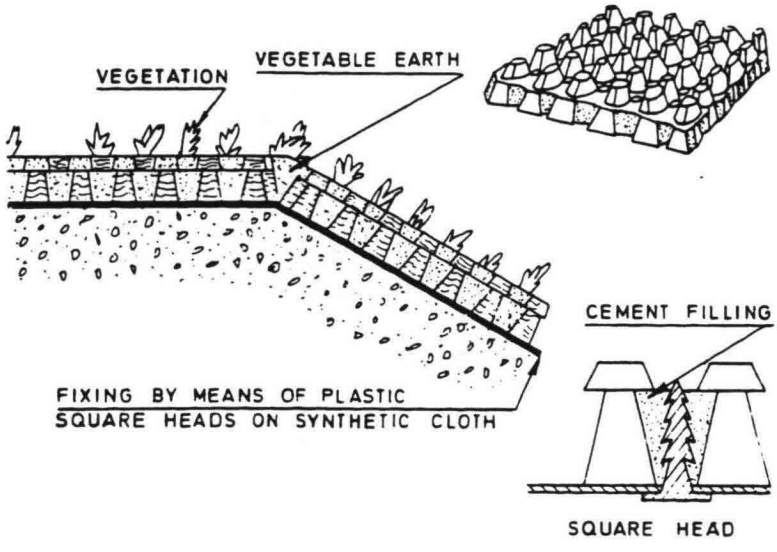




Bank protection of inland navigation canals

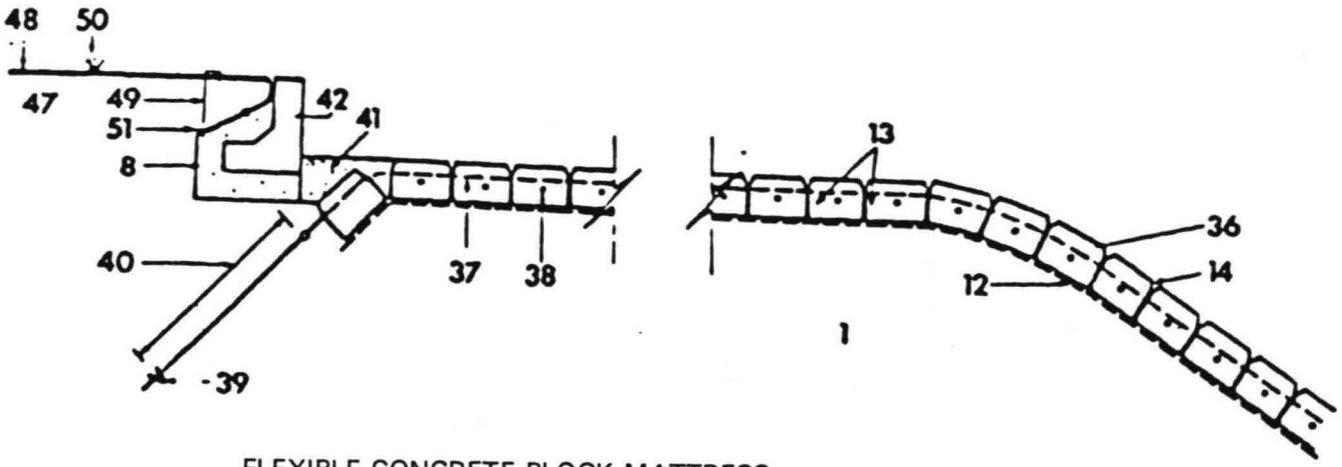
Alternate steel sheetpiling (France)

In France the use of vertical steel sheetpiles has been further optimalized. Cost have been reduced by decreasing thickness and steel area. The thin sheets are cold formed which leads to a higher yield strength. Alternated sheet piles are used in the example.

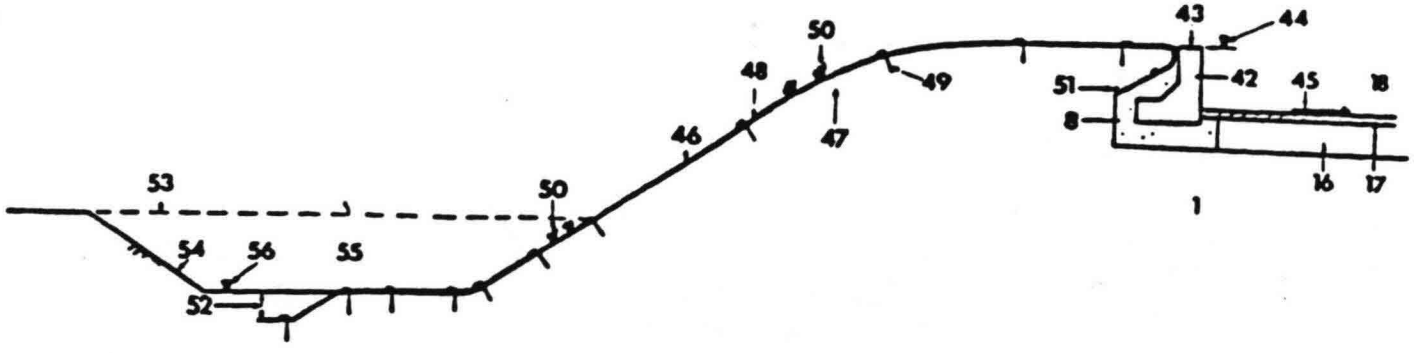


Bank protection with cellulated tiles (France)

Where banks can be designed with slope-inclinations of 1:2 or 1:3 the normal revetment with quarry stones and filter layer has been replaced by artificial revetment. These new revetment-types are formed with concrete blocks fixed on a plastic sheet.
The blocks are cellulated which allows one to fix them to the plastic sheet by means of square heads and a cement mortar filling. The plastic sheet acts as a filter: it is permeable and retains the soil particles.



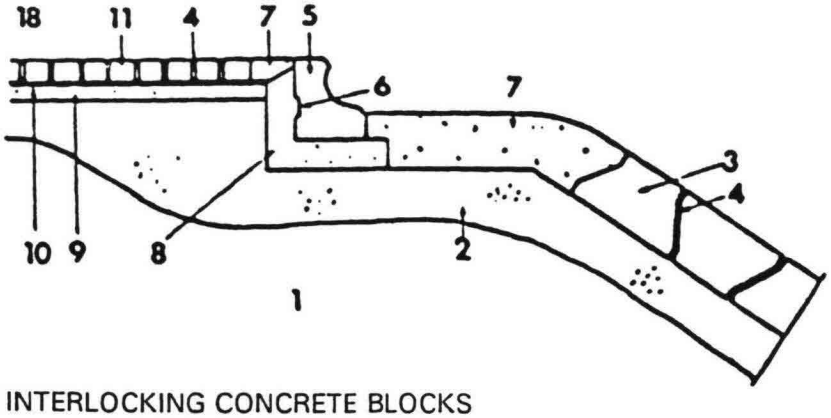
FLEXIBLE CONCRETE BLOCK MATTRESS



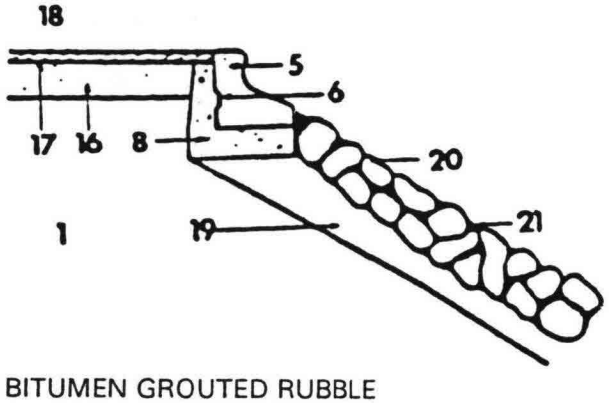
REAR FACE DETAILS

Fig. 12.6 Flexible concrete block mattress.

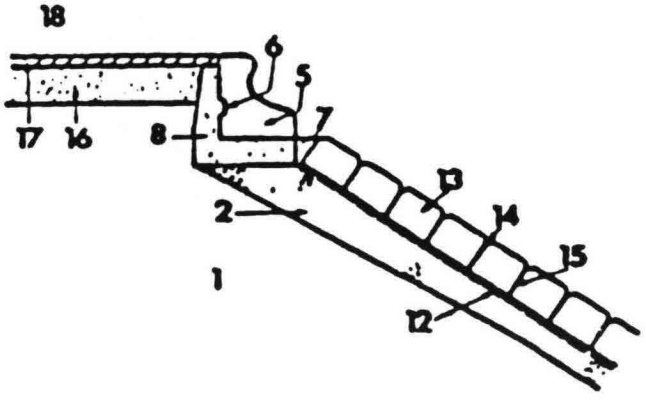
1. impermeable bank
8. insitu concrete bed and haunch
12. filter fabric (geotextile)
14. chamfered edges
16. crushed rock base
17. asphaltic concrete
18. road
36. flexible concrete block mattress
37. longitudinal connecting cablefall
38. transverse connecting cable
39. embedded anchor
40. embedded anchor
41. gravel infilling
42. precast concrete kerb
43. crest
45. fall
46. rear face
47. top soil
48. geotextile
49. steel peg ancher
50. grass
51. nail
52. burried depth
53. original ground level
54. excavated slope
55. drainage channel
56. channel invert level



INTERLOCKING CONCRETE BLOCKS



BITUMEN GROUTED RUBBLE



BITUMEN GROUTED BLOCKWORK

Fig. 12.7.

1. impermeable bank
2. gravel bed
3. interlocking concrete blocks
4. joint between blocks
5. precast concrete wave wall
6. key
7. insitu concrete infilling
8. insitu concrete bed and hound
9. lean mix concrete
10. granular bedding
11. concrete block paving
12. filter fabric (geotextile)
13. precast concrete block
14. chamfered edges
15. bitumen joint
16. crushed rock base
17. asphaltic concrete
18. road
19. sand bitumen
20. rubble
21. bitumen grout.

Bottom protection

For bottom protection three types area available:
- filter construction
- mattresses with rip-rap (conventional, sole piece)
- mastic asphalt

Conventional mattresses made from brushwood have been used for centuries in The Netherlands. Willow mattresses are manufactured on tidal flats and when completed floated to the required location and there sunk by placing stones from barges on them; this is an age-old Dutch proceeding to protect the soft shores against erosion.

The making of the conventional willow mattresses is very labour intensive. Furthermore the mattresses are subject to the action of the pile-worm.

One of the conditions that bottom protection must satisfy is that of sand-tightness. In some cases conventional mattresses proved insufficiently sand-tight, for which reason a "sole piece" was developed in which the bottom layer of the mattresses was replaced with a sand-tight fabric. The sole used was of the "coconut matting" type, namely a polypropylene monophyletic fabric which is sufficiently sand-tight while also being completely permeable. Fascine strings of willow brushwood are produced mechanically and then lashed with lanyard to the "sole" of polypropylene fabric.

Mattresses sinking in tidal currents. The mattress is transported well before the turn of the tide to the place where it will be sunk. Here it is anchored. Actual sinking to the bottom is effected by mooring a number of barges loaded with 10/60 kg stone around the mattress, the stones then being transferred to the mattress by hand. This method is used for mattresses on slopes. The last decades (from 1967 onwards) a different method is employed, whereby the stones are no longer transferred to the mattress by hand, but by means of special stone-dumping vessels known as stone dumpers, which release their load onto the mattress mechanically. In order to enable a stone dumper to transfer its load to a mattress, the latter has first of all to be pressed down in some manner. This is done by attaching a weight to one of the narrow ends of the mattress (the head), so that the edge of the mattress is carried down to the bottom.

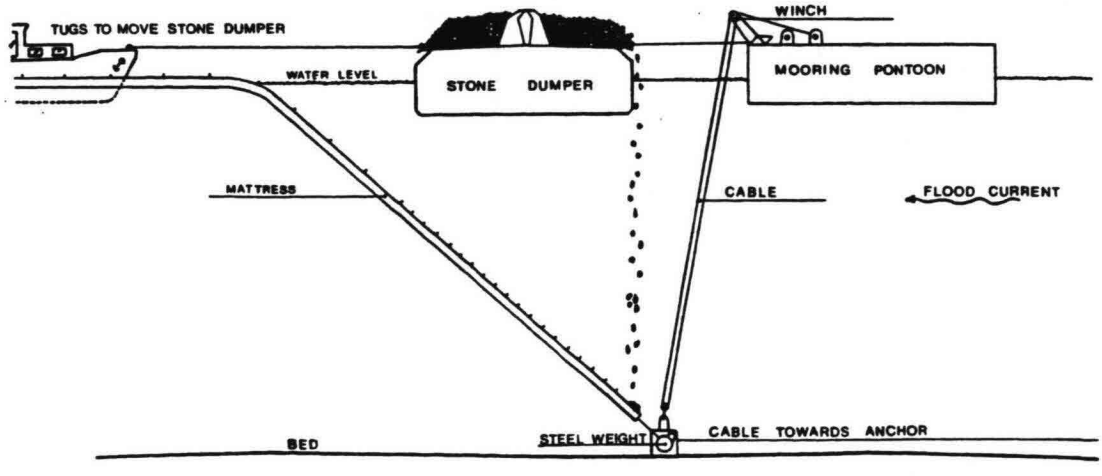
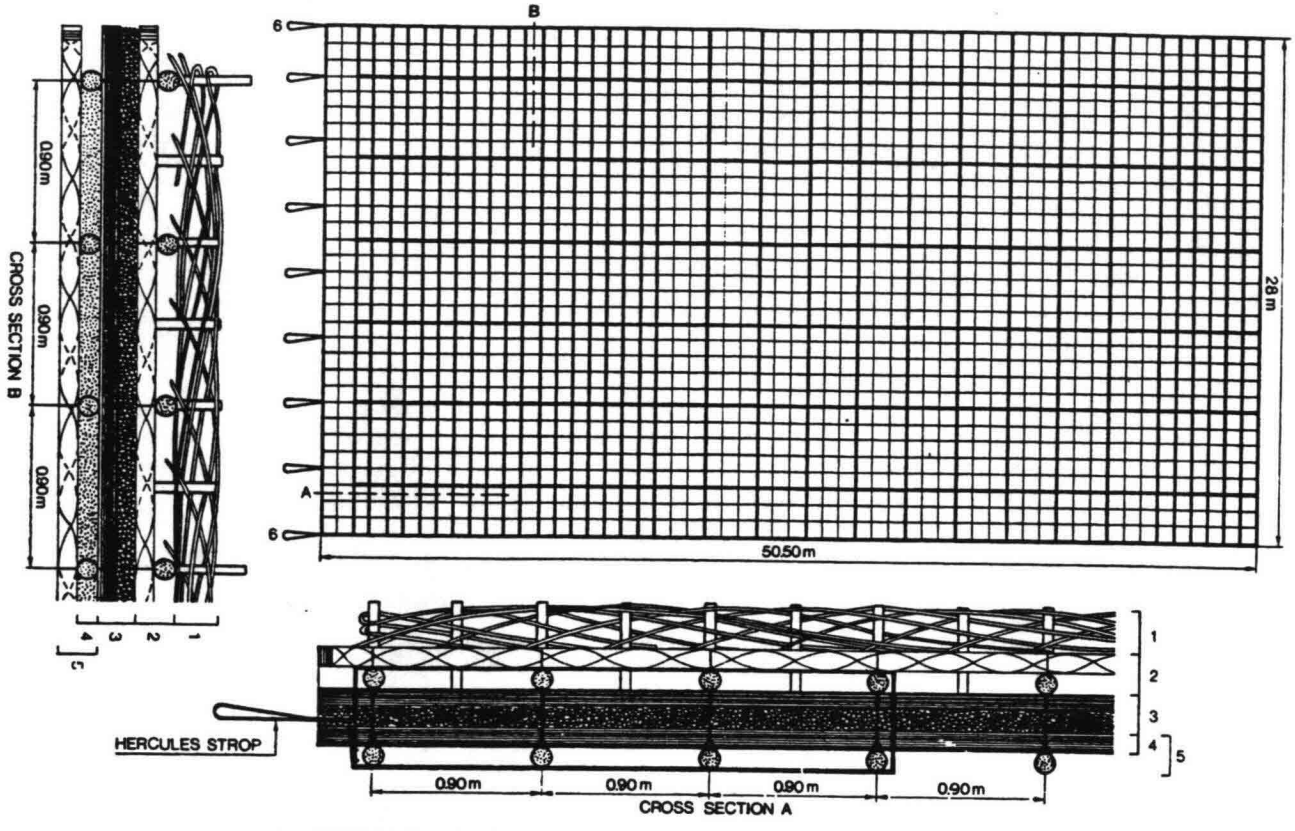


Fig. 12.8. Lowering a mattress by stone dumping.



Construction of a conventional mattress.

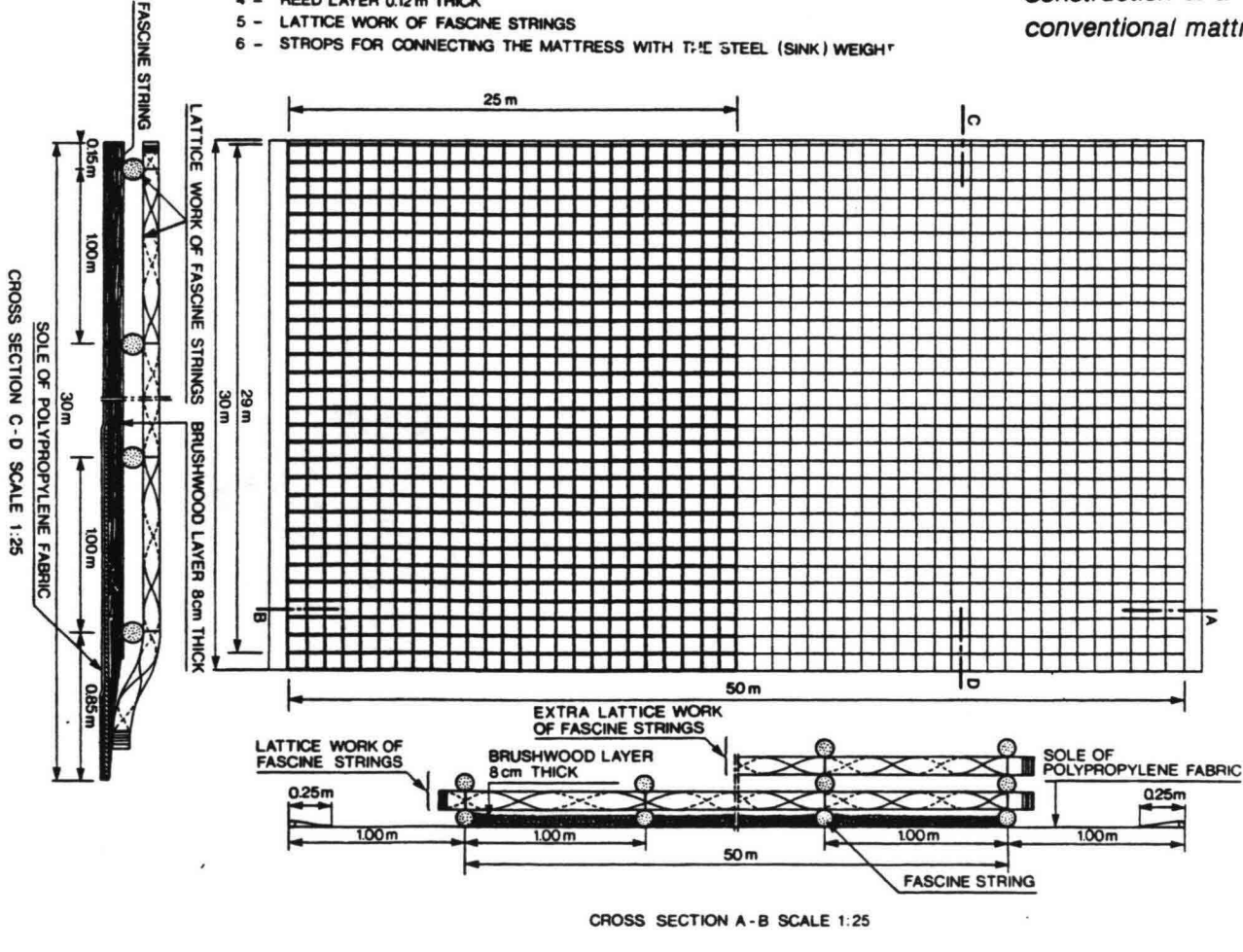
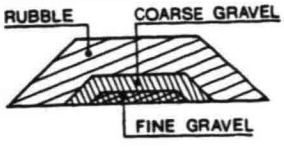
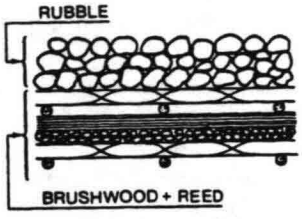
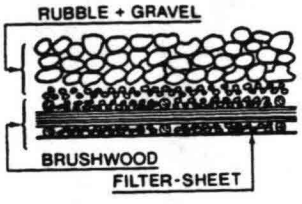
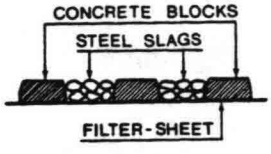
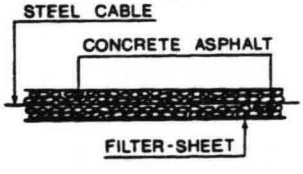
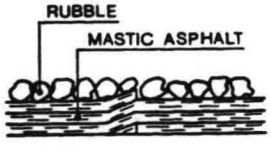
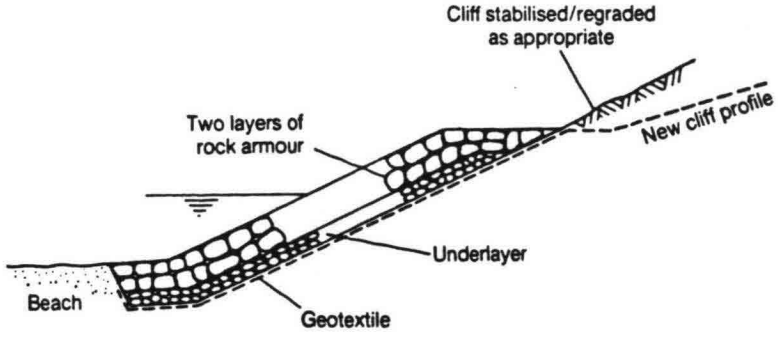


Fig. 12. 9. Construction of a mattress with a "sole" of polypropylene fabric.

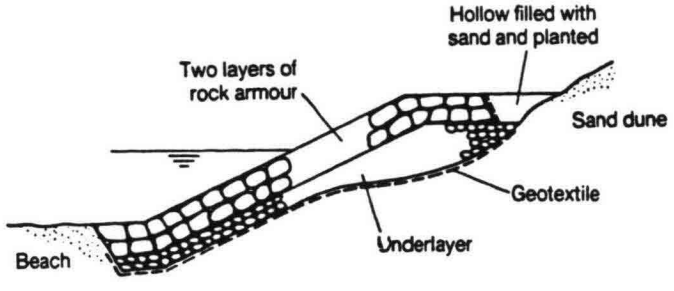
No	Type of bottom protection	construction	Principal components material	quantity	Quantity used in Brouwersdam
1	Filter (loose grained)		- rubble - coarse gravel - fine gravel	1.5 m layer 0.75 m layer min. thickness 0.50 m layer min. thickness	Over a length of 907 m with a 35 m crest in the n. gap
2	conventional mattress		- rubble - brushwood - reed	800 kg/m² 4 bundles/m² 0,5 bundles/m²	**a.** at right angles to current 155,000 m²* **b.** in direction of flow 290,000 m²*
3	sole piece		- rubble - gravel - brushwood - filter-sheet	600 kg/m² 200 kg/m² 2 bundles/m² 1.05 m²/m²	in direction of flow 140,000 m²*
4	block mat		- steel-slag - conc blocks - filter-sheet	200 kg/m² 200 kg/m² 1.05 m²/m²	not used
5	concrete asphalt mats		- concrete asphalt - filter-sheet	1.05 m²/m²	not used
6	mastic asphalt		rubble mastic asphalt	s.gap:350 kg/m² n.gap:200 kg/m² in 3 layers up to total thicknesses of 30 and 24 cm	110.000 ton

* These quantities are the sum of the combined surface areas of the sunken mattresses

Table 12.1. Survey of types of bottom protection for the Brouwersdam-The Netherlands



(a) At base of existing cliff



(b) At sand dune

Fig. 12.10. Coast-protection revetment

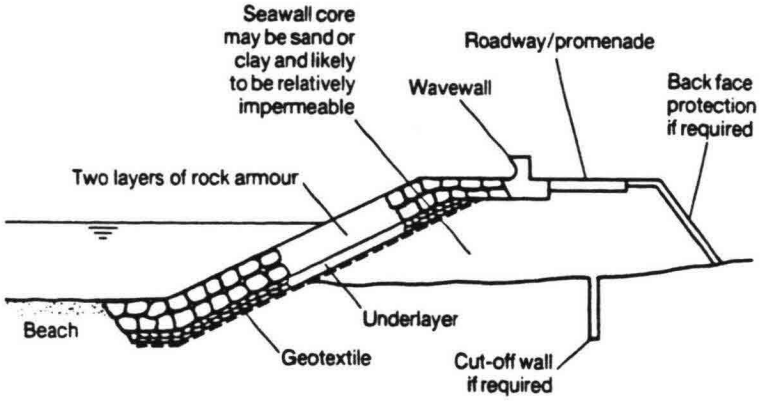


Fig. 12.11. Sea-defence revetment

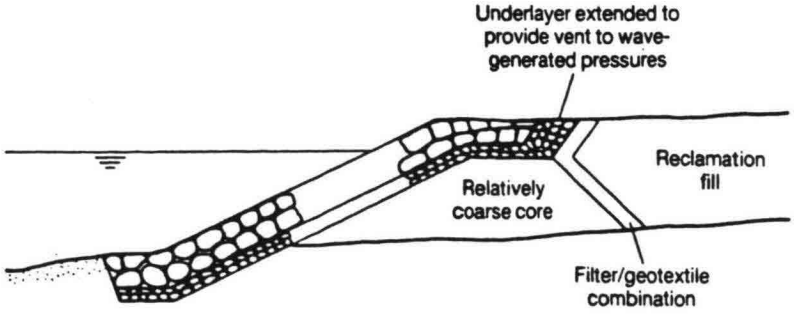


Fig. 12.12. Land reclamation revetment

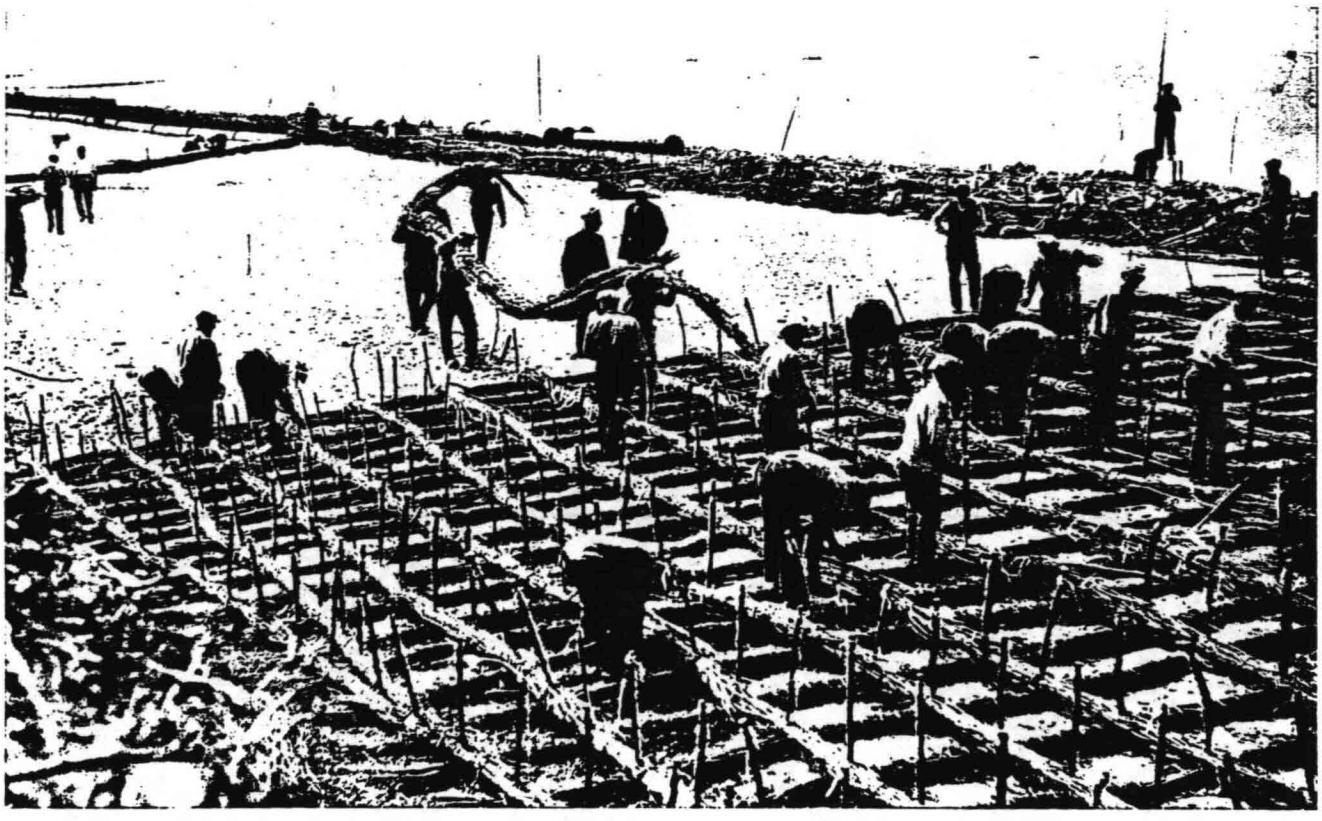


Fig. 12.15. Willow mattress. In this picture the mattress is made in-situ at low water to cover a sand dam against the scouring of the flood. There is a 'lower grid' -seen here- then two or three layers of willow, and upon those an 'upper' grid of the same material. The mattresses are weighted with stones.

13. Canals

Canals are man-made waterways. Some carry water away from marshland, making it into dry land, while others carry water into deserts so that they become fertile. Some of the longest, widest and deepest canals have been built for shipping; they link river to river, join up inland lakes or connect one ocean with another.

Water is vital for people and animals and for the growth of crops, yet one of the paradoxes of the world is that people in arid lands have to face a daily struggle against a shortage of water, while others who live among swamps or on marshland have to drain the waterlogged ground in order to grow food. Canals help to solve both these problems.

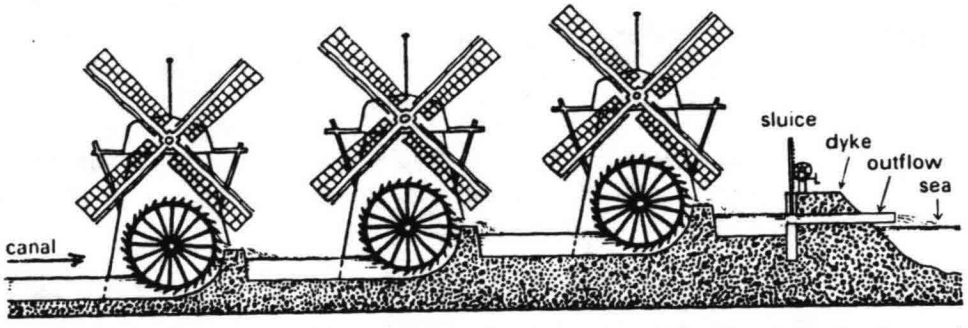


Fig. 13. 1. Year after year, windmills in Holland turn bucket-wheels that raise water from the marshland and pour it into a drainage canal.

Coastal lands, once washed over by the sea and now reclaimed, are useless for farming at first as alternative flooding and evaporation leaves a high concentration of salts in the soil, which is poisonous to most plants. Such lands have to be soaked with fresh, non-saline water for a long time until the salt is dissolved and drained away.

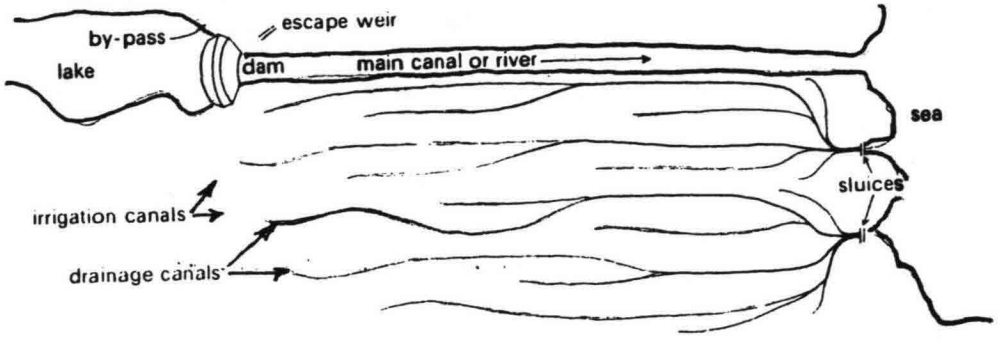


Fig. 13. 2. Irrigation and drainage canals often run side by side but at different levels. They are like the arteries and veins in a living body, bearing nourishment to the land and carrying away the poisonous waste.

Shipping canals provide another kind of nourishment. They allow bulk cargoes of food and raw materials to be transported to where they are needed. Since the development of rail networks during the last century, some shipping canals have been neglected (in particular in the U.K.).
In recent years, air-lanes and motorways have become jammed with traffic, and railway costs have risen steeply. Today, interest is turning back to the canals because they allow transport at lower cost, although at a slower pace.

13.1. Navigation Canals

Historical Review

1900 BC	Egypt	Construction of **"Ta Tenat"** (the cutting) - ancient canal link between the Nile and the Red Sea. One of the branches of the Nile (near the modern port of Port Said) was linked eastward along a linear depression in the desert with Lake Timsah. From here the **Pharaoh's route** went south through the Bitter Lakes to the Red Sea.The construction of the 'Ta Tenat' was ordered by Pharaoh Sesastris I (1971-1928 BC) when the Middle Kingdom was at the peak of its power but the work was only concluded under his successor, Pharaoh Amenehet II.
600 BC	Egypt	Another **attempt** to dig a canal to provide an all- water route was undertaken under King Necho II of the 26th dynasty; because of technical problems, however, he was forced to abandon the project.
500 BC	Egypt	The canal, which was the predecessor of the Suez Canal, was completed by the Persian conqueror **Darius I**. Part of the canal must have been executed in the wet.
285 BC	Egypt	**Ptolemy Philadelphus** rebuilds the Egyptian canal to the Red Sea from the Nile Delta.
1st century AD		The Netherlands. The Roman consul Marcus Livius **Drusus**, stepson of the Roman Emperor Augustus, organized the digging of a navigable canal link from the River Rhine above Arnhem to the IJssel and thus to the Zuiderzee to relieve the Rhine of surplus water. The Roman general **Corbulo** linked the Rhine and Meuse with a canal 38 km long to avoid the stormy North Sea passage from Germany to the coast. This canal, probably opened in the year 51 and later called the **Lek**, developed into the principal mouth of the River **Rhine**.
43-410 AD	U.K.	During the Roman occupation of Britain the Roman marshes were reclaimed. The **Fosse Dike** (ditch) of the canal from the great marsh near Lincoln to the river Trent was also constructed to serve as a drain and for the use of navigable vessels.
98 AD	Egypt	Roman occupation - **Trajan** reopened the canal and it was in use again by Arab rulers around AD 700.
600 AD	China	Chinese **Grand Canal**(Imperial Canal)- 1000 km - joins the capital Chang An to Hanchow in the south. The drainage canals in the Yangste area were unified into a single system (known as the Grand Canal) under the Sui Emperors (589-617 AD). The origin probably dates back to a canal link said to have been established at the time of the Wu Principality in 485 BC between the Yangste and Hwai Ho rivers.
793 AD	Germany	**Fossa Carolina**; attempt by Emperor Charlemagne to join Main and Donau.
1197-1257	Italy	Construction of **Naviglio Grande** from Milan to the Ticino River, a distance of 50 km. On this canal the water level was maintained by dams and transfer of barges from one level to another was achieved by pulling them over inclined planes by winches.

1280-93 AD China Chinese and Mongol engineers complete the **modern Grand Canal** from Hangchow to Peking, the first summit canal. This 1150 km northern branch of the Grand Canal was built under the Mongol conqueror Kublai Khan.

1400 Low Countries First European canals with locks.

1485 Italy **Leonardo da Vinci's** mitre gate on canal locks near Milan, superseding overhead gates, allows tall ships to pass.

1600- 1700 Europe Many small navigations dug in **England** and in **Continental Europe.**

1642 France **Briare canal** completed linking the Loire and Seine. It rose 39 m to a plateau with a summit level 6.2 km long and then dropped 81 m. It included 40 locks, of which a unique feature was a staircase of six locks to cope with a fall of 20 m.

1666-81 France Construction of the **Languedoc canal (Canal du Midi)** close to the Spanish border. This canal, 241 km long with 65 locks, provided a direct link by water between the two major French seaports of Bordeaux (Atlantic) and Marseilles (Mediterranean). It is 44 m wide and 1.8 m deep. It was built by Riquet and has locks 31 m long and 6 m wide. The Canal du Midi rose 61 m in 54 km from the Garonne at Toulouse to the summit through 26 locks, and then descended 185 m through 74 locks for 190 km. It ran through very rugged terrain; three major aqueducts carried it over rivers, and numerous streams were diverted beneath it in culverts. The most notable technical achievement was a complex summit water supply that included unique diversion of flows and storage provision.

1760 Russia Russian canals are well developed; 3000 80-ton barges a year travel by canal between the rivers **Neva** and **Volga**.

1762 U.K. Brindley completes the **Duke of Bridgwater's canal**, starting the 'canal fever'.

1784-90 France **Canal du Centre** linking the Loire and the Saone (Rhone).

1794 France Completion of **Charolais canal** connecting the Mediterranean with the English Channel.

1804 U.K. Telford begins the **Caledonian Canal**.

1807 U.S.A. Fulton's steamship Clermont, the first commercially successful passenger, regularly plies on the Hudson River.

1810-32 Sweden **Stockholm - Göteborg** canal, through the central lakes, 575 km long, of which only 87 km had to be dug but largely in rock, with 58 locks, under Telford's direction.

1817-24 U.S.A. **Erie Canal** links the Great Lakes with New York; 580 km long.

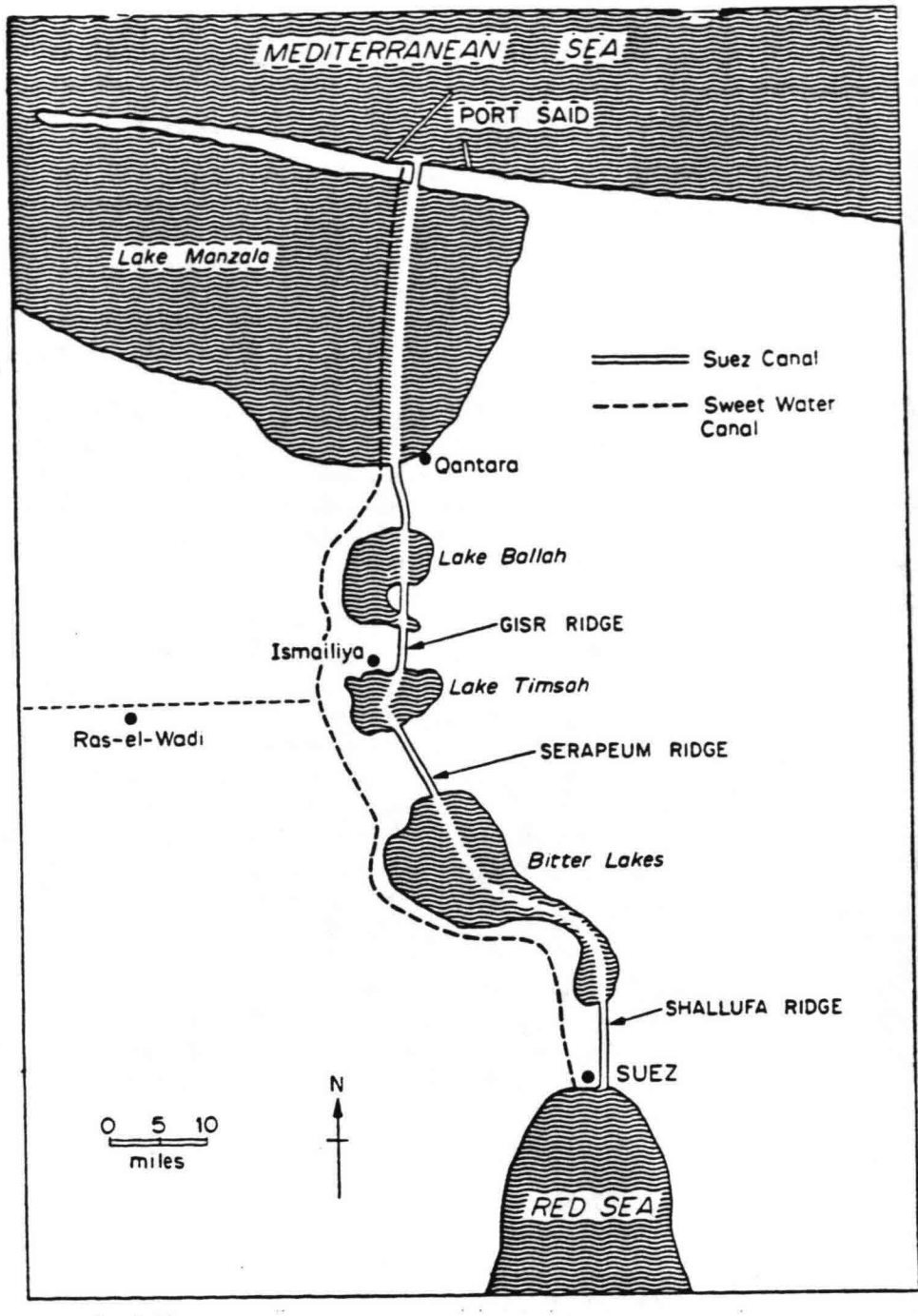
1850 Russia Russian navigable waterways amount to **80,000 km.**

1827 Belgium **Charleroi-Brussels canal; Ghent Ship Canal**, cut through to Terneuzen, giving a shorter route to the sea.

1869	Egypt	**Suez Canal** opened, designed by De Lesseps, 169 km.
1893	Greece	Opening of the **Corinth Ship Canal**. Cutting of a 6.3 km long, 25 m wide and 8 m deep canal through the Isthmus of Corinth to connect the Aegian and Ionian seas. The canal runs 86 m below almost vertical rock cliffs. The Roman emperor **Nero** had first attempted this linking in the 1st century AD.
1894	Germany	**Kiel Canal**, 95 km.
1894	U.K.	**Manchester ship Canal**, 60 km long, opened to give oceangoing vessels access from the Mersey estuary to Manchester.
1914	Panama	**Panama Canal** opened, 85 km long.
1932	U.S.A.	Opening **Welland Canal**, joining Lake Ontario with Lake Erie. Locks of 240 m long, 24 m wide and 9 m over the sill, similar to St. Lawrence Seaway.
1959	Canada/USA	Opening **St. Lawrence Seaway**.

Suez Canal

Fig. 13.3.Map of the Suez Canal.



By digging a connection between the **Mediterranean Sea** and the **Red Sea**, the sea route from North West Europe to the Far East could be shortened by 6400 km (Amsterdam--Bombay around the Cape is 16.000 km). Therefore this project had enormous economic effects. From an engineering point of view it was not difficult. It was only **64 km of ditch** to be excavated mostly through sand linking **96 km of natural depressions**, now lakes, and the highest land was only 11 m above sea level.

Napoleon sent his surveyor Lepère to the Isthmus who reported that the Red Sea was approx. 10 m higher than the Mediterranean. Actually there is virtually no difference in the levels, as a team of British engineers reported in 1833, but Lepère's error killed the idea for Napoleon.

In 1833 a group of French philosophers commissioned a survey; with the French vice-consul in Alexandria, **Ferdinand de Lesseps** acting as guide. He became interested in the canal but had to wait until 1854 and the succession of Mohammed Said as Viceroy of Egypt, for the canal to be considered seriously. Five years of convincing people followed and on 25 April 1859 he ceremonially turned the first sod on the strip of land that separated the Mediterranean from Lake Manzala.

The construction plan was essentially that drawn up thirteen years before by **Linant de Bellefonds**, a French engineer working for the Egyptian Government. It had three sections: the **Sweet Water Canal**, **Port Said**, and the **Maritime Canal** itself. The Isthmus of Suez was pure desert and 130 km east of the Nile; there was not a drop of water anywhere. The **Sweet Water Canal** was to provide this; it was 8 m wide and 2.4 m deep. It was also a transport link: for men and materials. The Egyptian Government built the 90 km from the Nile to Ras-el-Wadi, and the Canal Company built the remaining 32 km to Lake Timsah which was reached by February 1862. (1,100,000 m^3 dry excavation). The canal then turned south and Suez received fresh water in December 1863. At this time Egypt was

essentially a feudal country and 80,000 forced labourers working by hand were employed in the Sweet Water Canal's construction. The northern branch, not completed until 1869, was in the meanwhile supplied with water by pipeline.

The actual works on the **maritime canal** had to start with the creation of **Port Said**, so that all construction plant and materials could be received. The building of the artificial Port Said was the hardest task and not made easier by its inaccessible site. A narrow strip of land, 64 km long, separated the Mediterranean from **Lake Manzala** and it was on this strip that Port Said was created by building two breakwaters and dredging this 1.820 million m^2 harbour to -5 m. Over 1.6 km in length, the **breakwaters** were built of stone quarried at Alexandria and from 30,000 concrete blocks of 22 tons cast on site. In all, 118,000 m^2 of land were reclaimed.

The actual **maritime canal** was a relatively simple job; it started by a **service channel** ("rigole de service") southwards to **Lake Timsah** in the dry by manual labour. The excavation of 4,000,000 m^3 sand was completed by 15,000-25,000 forced labourers in November 1862. Now the waters of the Mediterranean could flood the 6.4 km long depression of Lake Timsah. Moving south from Port Said, 42 km through the **shallow lagoon Lake Manzala** to Qantara, where the water was only 1.5 m deep. Therefore it was necessary to dig a channel and to build retaining banks. A pilot channel, 3.7 m wide, was made by serfs scooping out the mud in baskets and laying it out in lumps.

Then **dredgers** were brought in and they reached down below the mud to stiff clay which, when sun-baked in layers, made strong banks 1.8 m above the water level. The work was delayed and undone by occasional severe storms. The canal, 100 m wide, had passed through the lake by 1866. Behind **Qantara** there was an easy 5 km of low-lying dry sand to cut through to reach **Lake Ballah** which was then a swampy depression. The spoil excavated in the lake was gypsum and unsuitable for the banks because it cracked, so material had to be brought in, presumably via the canal from Lake Manzala. The canal goes for 13 km across Lake Ballah and was virtually finished this far by 1867.

Next, lying across the path of the canal were the sandhills of the **Gisr Ridge**, a major obstacle 14 km long and rising to 11 m above sea level. The method, as with all the dry land work, was to excavate a pilot channel by hand until there was enough water to float the dredgers. The Gisr Ridge was not finished until 1866 but it was passable in 1862 so that the waters of the Mediterranean could flood the depression of Lake Timsah. The service channel was then dredged to form the Maritime Canal with a minimum depth of 7.8 m and a minimum bottom width of 22 m. Up to the end of 1864 about 15 million m^3 (including the sweet water canal) had to be excavated mainly by forced manual labour. In that year the corvée or forced labour was no longer allowed by the new Viceroy Ismael.

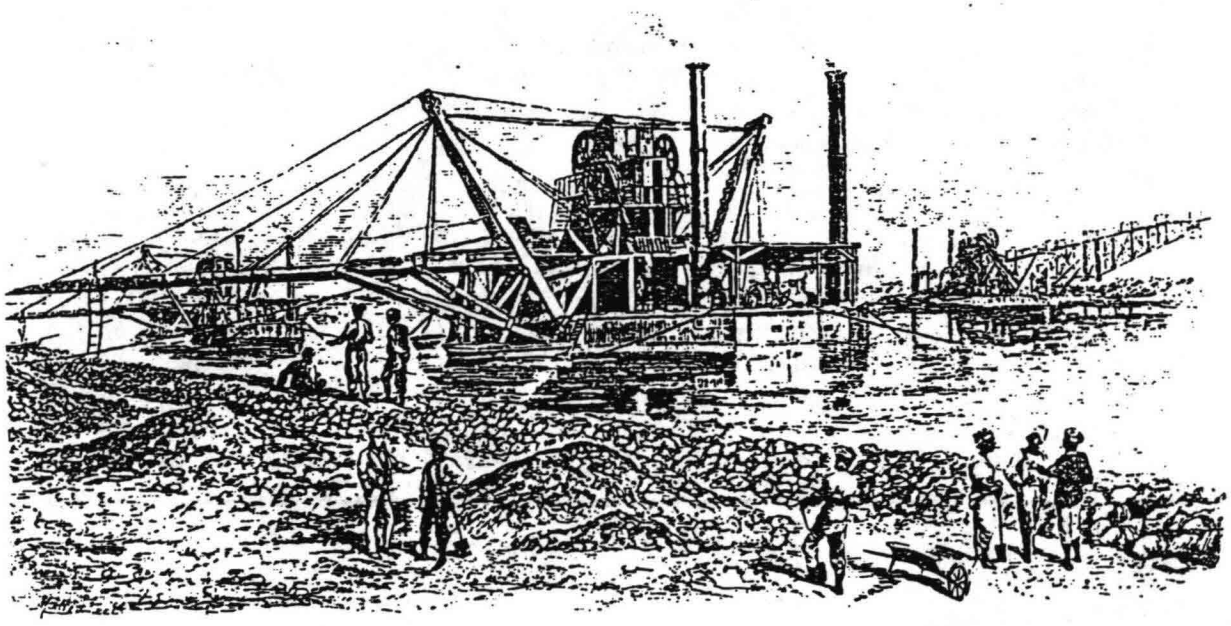
The principal contractor, the French firm **"Borel, Lavelley et Cie"**, had to design and import bigger and better dredgers to replace the manual labour. These machines were in full operation in 1865, and in 4.7 year (up to the completion in 1869) these machines dredged 60 million m^3. South of Lake Timsah came the **Serapeum Ridge** which ran for 16 km and up to 9 m above sea level. It had to be pierced before the canal could reach the Bitter Lakes. A little **rock** was encountered and it was not possible to get the manual labour to take the pilot cutting all the way to water level. Therefore the mechanical dredgers were adapted to dry excavation. Having been brought from Port Said into Lake Timsah by the 'rigole de service', the dredgers were taken 5 m above sea level into the Sweet Water Canal through the locks at Ismailia between the Maritime and Sweet Water Canals. Cross-cuttings were then made from the Sweet Water Canal to near the line of the Maritime Canal opposite the points where the ridge had to be excavated. Dams were maintained between the cross-cuttings and the Maritime Canal until the Maritime Canal had been excavated down to sea-level; then the cross-cuttings were dammed off from the Sweet Water Canal and the dams opened between the cross-cuttings and the Maritime Canal, thus enabling the dredgers to float into the Maritime Canal and revert to their normal task of underwater excavation.

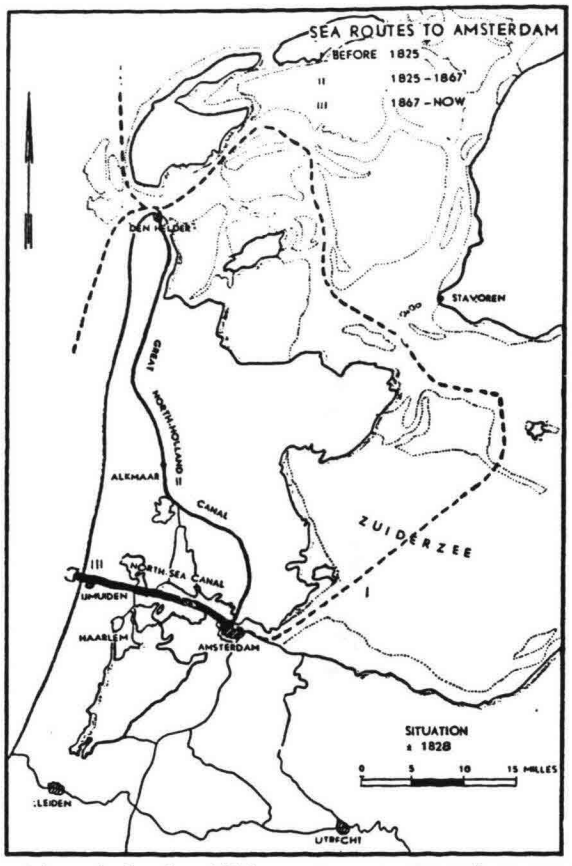
The **Bitter Lakes** were flooded in March, 1869 and formed a 40 km stretch of water. The Company was fortunate in that nearly all the material to be excavated was sand. The only place where any **rock** was found was in the **Shallufa ridge**, where 52,000 m^3 of rock had to be removed by blasting, before the waters of the Mediterranean and Red Seas could mix in the Bitter Lakes on 15 August 1869.

Port Suez was already a small port for the overland route to Cairo but had to be enlarged by building a 777 m **breakwater** out to sea, **dredging a channel** and reclaiming land.

Mechanical devices of great originality and power were employed on the Suez Canal project. The dredgers varied in size from 15 hp to 75 hp. The largest machines of 75 hp, had a length of 33 m, a beam of 8.1 m and had their drums 14.4 m above the water line. The greater part of the dredged spoil near Port Said was dumped by large sea-going barges some four to five miles out to sea in deeper water. These barges were used in the ports and in some lakes but a larger proportion of the dredged quantity was dredged from the canals by **steamdriven bucket chain dredgers** and deposited on the canal banks by using either a **"long couloir"** or an **"elevateur"**. The "long couloir" was a chute used for depositing dredged material on the banks when the dredger was working at or not far below the level of the land on each side of the canal. The chute, which was anything up to 30 m long, was carried on a barge with an iron framework superstructure which was moored at right angles to the dredger between the dredger and the bank.

A long couloir dredger at work





North-Sea Canal

Fig. 13.5. Sea routes to Amsterdam. After the formation of the Zuiderzee about 1300 Amsterdam developed in a major port for the newly reclaimed peat marches of central Holland.
Route I- had a depth of 3.3-3.6 m and was used until 1824.
Route II: the **'Great North-Holland Canal'** to the deep harbour of Den Helder was used from 1824 - 1876.
Route III: the **'North-Sea Canal'** was made for the biggest sea-going ships. At IJmuiden a lock has been made that ships of 150.000 tons and 15 m draught can pass (opened in 1876.)

Already in the **17th century** proposals were made to cut a canal through the dunes from the 'Wijkerlake' to the North Sea to obtain an outlet for the draining of the low country behind the dunes (Jan Pzn Dou - 1634). In 1772 the famous engineer and the first Inspector General of the Rijkswaterstaat Christiaan **Brunings** (1736-1805) wanted this canal through the dunes for the double purpose of giving Amsterdam a better sea entrance to the North Sea direct, and at the same time drainage of the low areas. Brunings became famous by making a deep and nice harbour at Den Helder in 1782 by letting the currents do the work, after making a simple dam on the sandflats.

Part of the North-Sea Canal plan was a closure dam in the eastern part of the IJ to combat siltation. However the idea was rejected by Amsterdam. King William I considered a new shipping canal straight through the dunes west of Amsterdam. He was advised that to cut the broad ridge of high sand dunes might be possible, but to construct a harbour on the sandy shore, where no depth at all was available, was not feasible for the time being. When in 1824 the king made his choice of the **Canal to Den Helder**, a well-known "waterstaat" engineer proposed again the direct route to the sea. In 1852 **Willem Anthonie Froger** drew a plan to dike the IJ-lake and to connect it with the North Sea; this plan was modified by **Jager** and a concession on this plan was granted in 1861 (fig.13.).

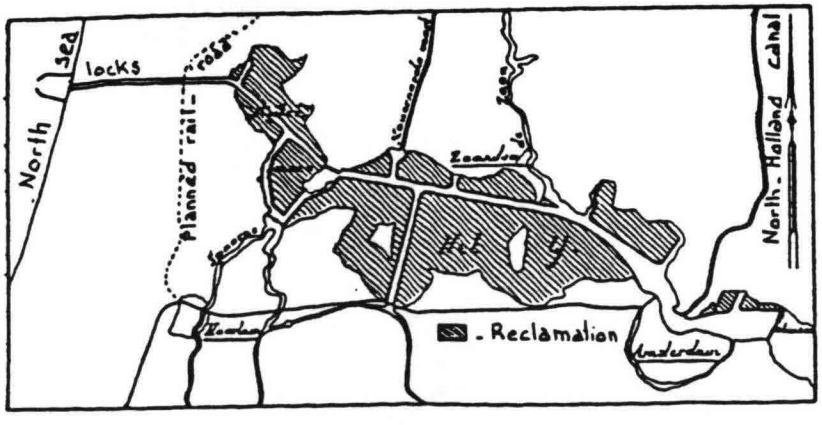


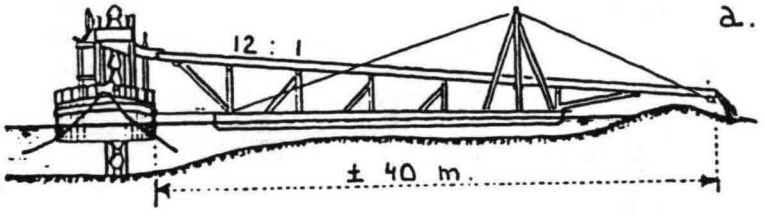
Fig.13.6. Plan, as modified in 1865 and executed.

The famous Dutch prime minister, Thorbecke, said in parliament in 1862 that both Amsterdam and Rotterdam had to be made accessible for the biggest seagoing vessels, and the plan was approved in the parliament in February 1863.

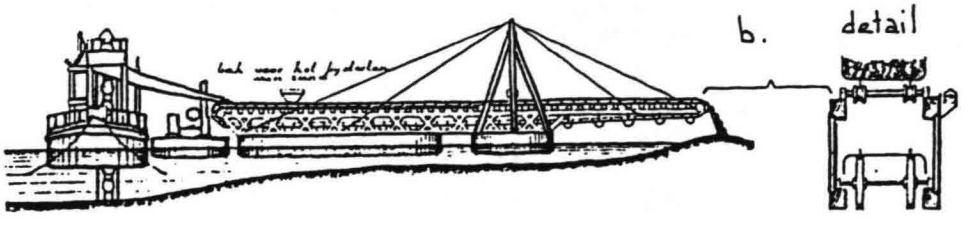
In 1863 the **'Amsterdam Canal Company'** was founded to raise the necessarily funds and on 8 March 1865 a start was made with the 'digging through Holland at its narrowest'. A contract was concluded with the British firm **Mac Cormick & Son,** which in 1865 was taken over by another British firm **Henry Lee & Son** and in March 1865 about a hundred workmen under the direction of three British and four Dutch engineers and supervisors were at work. The dune sand was excavated in the 'dry'. Wheel barrows were loaded and took the sand to the railroad wagons. Horses pulled the loaded wagons either to the IJ-lake where the sand was used to construct the dikes along the canal or to Amsterdam for the building of a railway yard. By the end of 1865 some 500,000 m^3 were excavated and hauled away by the sand-trains. Dry excavations works continued to October 1872.

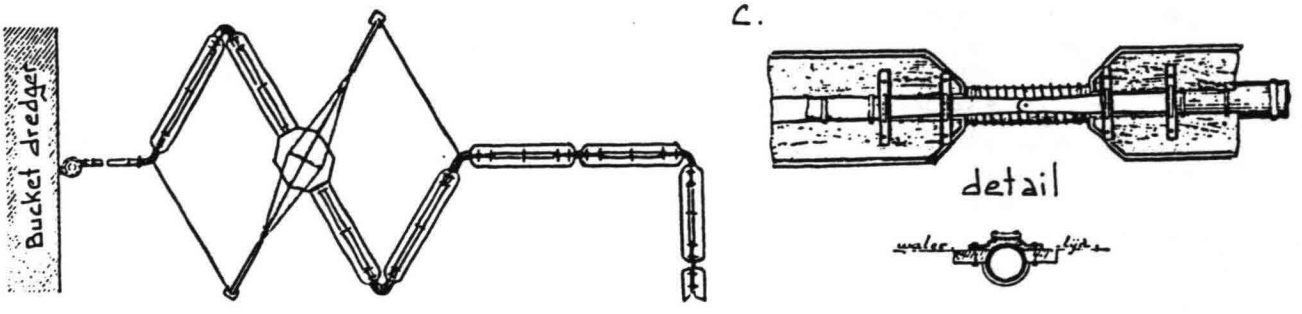
For the dredging of the canal through the **Wijkerlake** (1869) **bucket dredgers** were used; the dredged sand was deposited by chute or conveyor belt onto the excavation side for use on the dikes under construction.

The **locks** at the beginning of the canal were constructed in an open building pit and at the end of 1872 the bunds were dredged away by bucket dredgers and water was released into the canal. In October 1872 the works were taken over by the Dutch government who awarded the remainder of the works to Dutch contractors. In 1873 the first reclaimed polderland was sold.



Means of soil transport for the construction of dikes.
a. with open gutter
b. with conveyor belt
c. discharge pipe





Originally Rotterdam could be reached by the sea inlet north of **Brielle**. However this inlet was silted up in 1740 and thereafter one of the **waterways I, II or III**, allowing uncertain depths of about 4 to 5 m, were used. There was a lot of shifting sand in the delta under the influence of wind, waves and tides, so that ships had to reach Rotterdam by constantly changing routes. In 1829 the **canal through Voorne** was dug, which soon afterwards became too small.

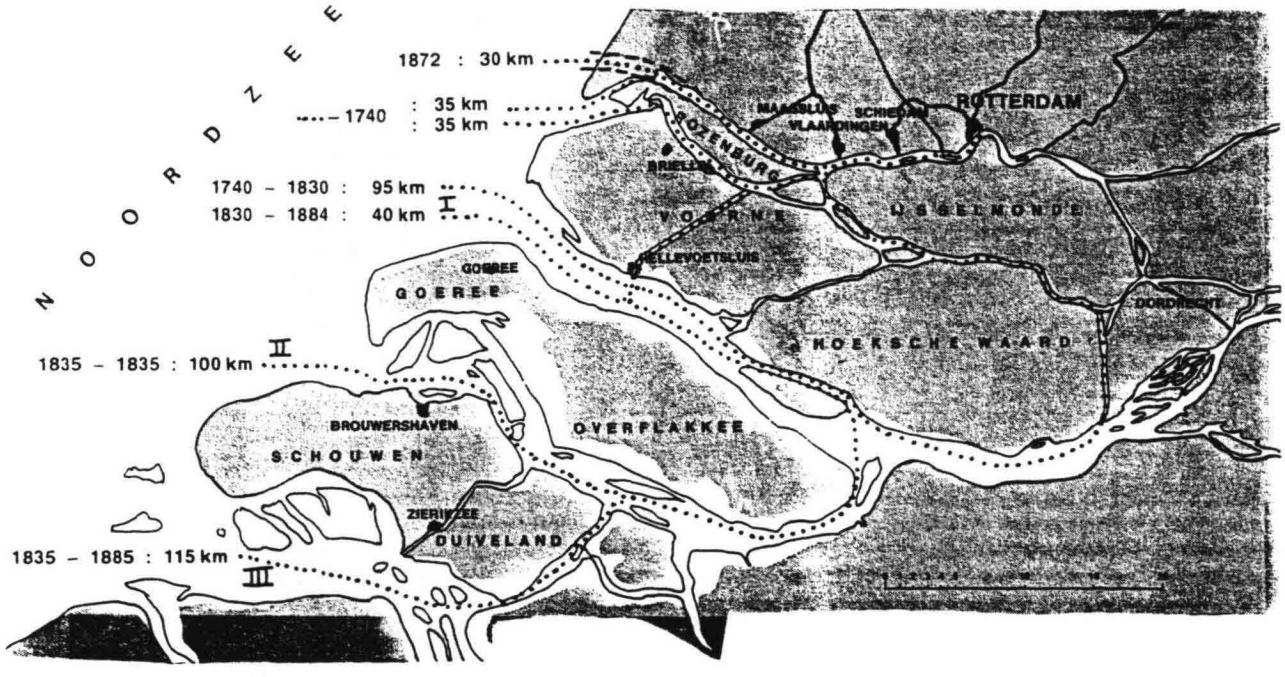


*Fig. 13.7. Sea routes to Rotterdam.*In 1866 a beginning was made with the cutting of the New Waterway through the Hook of Holland dunes, which was to give Rotterdam a reliable link with the North Sea. This was badly needed as the route via Brielle was after 1740 not longer navigable, due to siltation of the inlet. This required other routes to reach Rotterdam.

A plan of a cutting through the dunes at Hook of Holland was launched by a young engineer, **Pieter Caland**, who had made a brilliant scientific study on tidal currents in the mouth of big rivers. Earlier Caland, acting on the suggestion of a minister, had gone to Scotland and France to study how the Clyde and Garonne were being kept at a proper depth. Caland's idea was to make the currents do the main work; first a small initial cut through the dune was to be made, second the existing wide but shallow mouth of the old river was to be closed off and then, it was hoped, the currents would widen the initial trench and make a new deep river.

Caland's idea was risky. He wanted to induce **Nature** to make a new river mouth, deep enough for the sea-going ships. In 1862 Thorbecke defended the plan in the House of Parliament with the words: "Our coast must be made accessible for the big shipping of our times. If we remain what we are now we shall he outdone and shall be lost". So the risky work was started on October 31, 1866 with an initial ditch excavated 2 m below low water and a width at the bottom of no more than 9 m (completed In 1868). Then the existing river mouth (Scheur) was dammed after which the scouring of the new tidal river could begin. Although it did scour slightly, the new waterway became clearly wider and wider but hardly any deeper. When, in 1872, the "Richard Young", entered the waterway as first ship its depth was still no more than 3,5 m. Caland's theory had evidently failed.

Only eighty years later did Dr J. van Veen establish the cause. The salt wedge flowing in over the bottom whilst the fresh water was still flowing out, brought in new sediment. In particular, in the mouth between the two new breakwaters, the sedimentation was very bad. Therefore it was decided to use **steam bucket dredgers**, the recently commissioned powerful new tools to carry out the required dredging. The dredging at the **mouth** was however, due to the sea conditions, very difficult and so only a few weeks per year could be worked at that location. The lengthening of the groynes in 1872-73 did not help, as the sandbar shifted seawards as well.

Up to 1875 only a few steam bucket dredgers of the firm Volker of Sliedrecht were at work. It became evident that larger capacities were required, so a joint venture between the firms Volker and Bos (also Sliedrecht/Gorinchem), who had been undertaken the removal of soil for the contract sum of Dfl. 3.328.000, brought in 15 bucket dredgers. But the weather delay due to swell was the reason that the cleaning of the mouth could still not be done. It was a suction dredger that was needed. The first **suction dredger**, the Adam I, had its pump in the middle of the ship, and discharged into barges laying alongside, and was a combination of the Hutton dredger and the Bazin dredger. This dredger had an even higher delay for swell conditions (1878). Then the idea of loading the spoil in the hold of the same dredger occurred to someone and the **self-propelled seaworthy hopper suction dredger** was built (also called hoppers). They anchored in the entrance to the New Waterway, pumped their hoppers full of sand, went to sea and dumped the spoil by means of bottomdoors in the hoppers. Without the invention of these hopper suction dredgers the excavation of the New Waterway at Hook of Holland would have been impossible. The first suction dredger of this type was named "Maasmond I", followed by the "Adam II". In 1881 the type was modified by shifting the suction pipe from the middle of the ship to the side. Between 1878 and 1880 no less than 18 of these hoppers were built, with hopper capacities of 150-300 m3 In the years 1878-1883 the successful employment of 19 hoppers resulted in the completion of the works. In 1884 the navigation channel had a width of 100 m and a depth of 5 m at low water. Total costs were 25 million guilders. In about 1850 the Netherlands Government decided, owing to the high wages demanded by the dredger operators, to buy a number of steam dredgers from Belgium, man them with a mechanic and make them available to the contractors executing dredging works. This was probably an incentive for the contractors to consider purchasing their own equipment. In 1880 the Netherlands authorities disposed of their last dredger; the government equipment had served its purpose, and during that period the basis was laid for the present dredging industry.

The massive employment of dredgers, 15 bucket dredgers, 19 hopper dredgers (invented for this work) was needed (1878 - 1883) to complete the work successfully. In 1884 the navigation channel had a width of 100 m and a depth of 5 m at low water.

The entrance to Rotterdam was continuously deepened to keep pace with the increased volume of shipping (dredging in 1885, 1895, 1905 and 1914 - total 165 million m^3). The. depth of the new waterway was increased from -5 m in 1884 to -10 m in 1914.

Panama Canal - first attempt 1879-1888

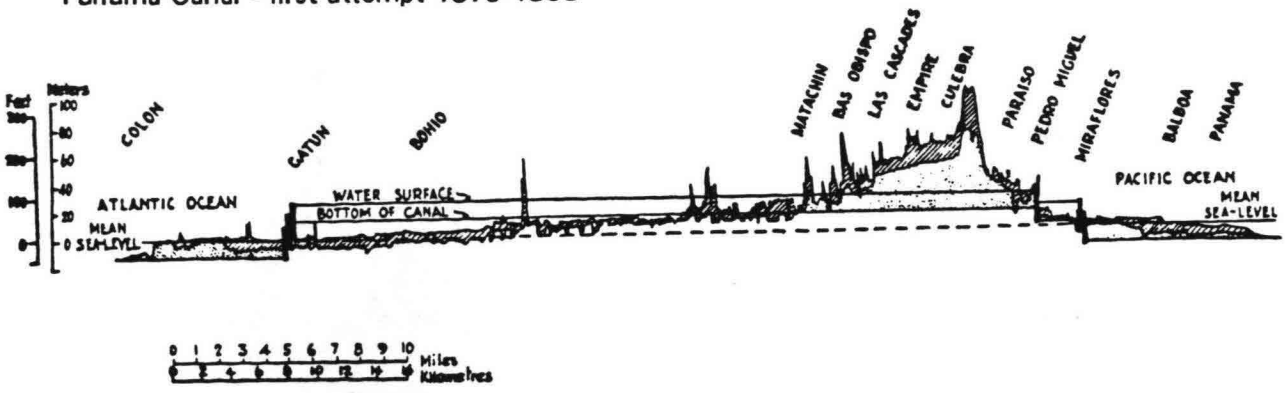


Fig. 13.8. *Longitudinal section of the Panama canal, 'Not so much a canal as a bridge of water.*

Ferdinand de Lesseps, known as 'le grand Français' was the main promoter of the French attempt to make a sea-level canal without locks through Panama (from Colon to Panama). He became president of Le Societé de Geographie in 1878 when he was seventy-four years old. In 1879 his **'Compagnie Universelle du Canal Interoceanique'** was launched to finance the works. The contractor **'Couvreux and Hersent'**, with their experience at Suez, amassed large amounts of equipment in 1881/82, a.o. 14 dredgers. One of the reasons the French failed in their attempt to build a canal, at least in part, was that their equipment was too diversified. Even after five years of experiment they were still excavating and disposing of the soil by a great variety of methods: by bucket-dredger, suction-dredger, shovel, elevator, cableway and even by hand; they then loaded this soil onto one of eleven different types of flat-car running on one of six different gauges of track.

This was largely due to the first fact that after 1882 (when Couvreux and Hersent withdrew) the work of construction was split up among a series of sub-contractors, each of whom had its own ideas on how to cope with his own particular section of the canal. Only in the field of dredging could the French equipment be described as ideally suited to the work is was expected to. Dredgers work as individual units and it was in this highly individual field that the French achieved the best results. The dredging works started in 1882 on a number of sites.

Equipment was bought by the Company and rented out at a remarkable low rate to the various contractors. This enabled even the smaller firms to use top-class machinery, but it lead, however, to administrative difficulties. The total quantity of excavation was in 1880 estimated by the Technical Commission to be 71 million m^3. This was revised by **Dingler,** the director- general as from 1883 (after the withdrawal of Couvreux and Hersent) and the man who came very close to success, to 120 million m^3. The main reason for this 70 % increase was the reduction of the slopes of the cutting.

Three huge dredgers, the 'Comte de Lesseps', the 'Prosper Huerne' and the 'Nathan Appleton', started dredging the 'Dingler's canal' at Limon Bay (Atlantic side) in april 1883. These formidable machines belonged to the only contractor who worked continuously on the isthmus throughout the whole regime of the Compagnie Universelle - **Huerne, Slaven and Co.** of San Francisco. The channel they excavated run through mud flats, mangrove swamps, coastal plains and finally up through the gentle gradient of the Mindi Hills. In the mud flats daily productions of 3.000 m^3 were exceeded. Altogether Huerne, Slaven and Co (later known as the **American Contracting and Dredging Company**) excavated about 17 million m^3. They were the only contractor, among more than two hundred, that completed their allotted work on time.

Part of the Panama canal at the **East side** was planned in the lower reaches of the Chagres river. The bulk of the excavation was done by more than twenty bucket dredgers. When

they were working in a lake discharge was via a floating pipeline, but when they working beneath high banks and discharge was far away hydraulic hopper were used. The conditions were very trying, not only because of the heat but because the flood-plains of the valley was a breeding ground for mosquitoes and malaria and yellow fever took a heavy toll (1800 death in the period 1881-'84).

The cut through the mountains near Culebra turned out to be one of the most difficult parts of the whole canal; de Lesseps had allowed for slopes of 1:3, Dingler had altered it in 1: 4 but in fact the slopes had, in places, to be as gentle as 1:10. This was due to the peculiar geological formation of the divide which caused many landslides.

At the **Pacific Ocean side** the Canal consisted of three clearly defined sections: the **Culebra Cut, the valley of the Rio Grande** and the channel **through Panama Bay.** Of the three sections the 5 km long channel between the mouth of the Rio Grande and Perico Island was the easiest.

The work of excavating this channel was carried out by A-I and A-3; probably the most successful of the first-class dredgers used by the French. They were marine-type, self-propelled dredgers, built by Lobnitz and Company of Renfrew, Scotland.
The channel in the valley of the Rio Grande was excavated by lighter dredgers.

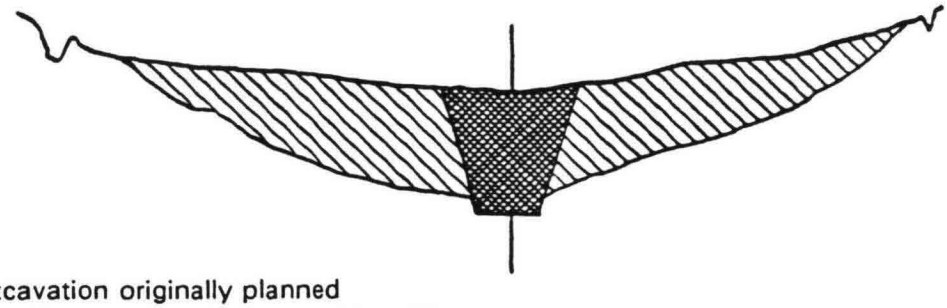
For the excavation of the Culebra Cut: the mile long saddle across the highest points of the hills some 90 m above sea level, a contract was signed in 1884 with **Cutbill, de Longo, Watson and van Hattum** (usually referred to as the **Anglo-Dutch Company)** (total quantity: 10 million m^3). The rocks consisted of volcanic breccia overlaid with a soft red clay. The company amassed an impressive array of equipment and a working force of 2.000 men. The principal machines used were excavators mounted on trucks. The method of soil disposal became the basic problem; in the rainy season the spoil banks built above and parallel to the Cut became saturated and slid back in the excavated channel.

Although it was not recognised at that time the basic problem in excavating the Culebra Cut was one of transportation. In 18 month only 670.000 m^3 (out of 10 million m^3) was excavated and the contract was annulled. The year 1885 was a disastrous year for the French; the excavation was pegged back by floods, revolution, sickness and slides. In 1884 about 7.500.000 m^3 was excavated, in 1885 7.200.000 m^3.

By 1886 it was clear that although progress in the coastal plains was not unsatisfactory, work in the central section among the hills was falling seriously behind schedule. To complete his canal de Lesseps (more than 80 years old) needed more money. Although a consulting committee advised to change the sea-level canal into a canal with locks, this was not acceptable to de Lesseps. In Dec. 1888 the compaigne Universelle went into liquidation and the French abandoned the Panama Canal. The profit of some contractors were enormous, up to 60 % of the contract sum. For performance see table.

Table 13.2 Performance main contractors:

Name	Contract quantity	Actual removed	Contract sum (million francs)
American Contracting and Dredging Company	18.000.000	16.990.000	70.8
Jacob Company	not specified	2.325.000	16.5
Artigue, Sonderegger and Company	20.000.000	2.250.000	32.6
Vignaud, Barbaud, Blanleuil and Company	20.000.000	3.640.000	30.0
Le Societe des Travaux Publics et Contructions	29.000.000	3.420.000	76.2
Baratoux, Letellier and Company	8.000.000	7.000.000	37.6



Excavation originally planned
Additional excavation caused by slides

Fig. 13.9. Additional excavation necessitated by the flat slopes of the Culebra Cut.

Panama Canal: second attempt 1904-1915.

In 1904 the works on the 40 % complete Panama Canal were restarted by the **United States**. It took them about 12 years to complete a lock-canal with much improved excavation machinery. **Steam shovels** did ninety percent of the dry excavation. They broke the back of the continental divide and without them the canal would never have been completed. More than 100, including enormous 105-ton monsters, were ordered with **Marion and Bucyrus**. The architect of new canal was **John Stevens**, who was for two years (1905-1907) in charge of the greatest construction project the world has ever seen. His successor was **Goethals**, who divided the construction works into three regions: **the Atlantic division, the Central division** and the Pacific division.

The **Atlantic division** had two tasks: to dredge a new approach channel through **Limon Bay** and its immediate surroundings and to construct a **dam and locks at Gatun**. The new American canal was sited in the centre of the shallow, unprotected Limon Bay making breakwaters essential. The approach channel was basically dredging of a channel 12.5 m deep, 150 m wide and almost 11 km long. It was a complex armada of vessels which carried out the excavation. The mud-cum-silt in the bay was almost single-handed excavated by one suction dredger. Between 1907 and 1912 this dredger removed 10 million m^3 silt and dumped it in open sea (km 0 - km 5.600). This sturdy 3.000 ton dredger made a major contribution to the excavation.

The second section of the channel, the mangrove swamps (km 5.600 - km 7.400), was more complex and was carried out by a great variety of vessels. The ground consisted of silt, coral, soft blue rock and clay and was excavated by pipeline and dipperdredgers.

In the last section (km 7.400 - km 10.700) the soil was much harder - blue rock, heavy clay, gravel and boulders had to be blasted by drills before it could be excavated by bucket ladder dredger, suctiondredger or steamshovel. Many of the former French dredgers, already 25 years old, were used here. They were built extremely well and did excellent service.
In the summer of 1912 the Atlantic approach channel had been completed:

wet excavation (dredging)	: 23.000.000 m^3 at $ 0.31/m^3
dry excavation (steam-shovel)	: 4.650.000 m^3 at S 0.88/m^3.

The **Gatun Dam**, designed to create a vast lake and to tame the violent **Chagres-river**, was partly constructed by dredging. Suction dredgers pumped clay from the bed of the old French canal into the centre of dam between rock bunds. In 1910 four suction dredgers were discharging 450.000 m^3 per month as hydraulic fill into the dam. The core thus created was built up gradually to a height of 29 meter (520 m long). Slopes 1: 8 and 1: 12.

The **Central Division was** in charge to make 52 km of canal. It consisted of two clearly defined areas: 36 km of the Chagres valley (which had to be flooded) and 16 km of the **continental divide** (which had to be breached). Most of the excavation was done by steam shovels and transported by spoil trains. The Cucaracha Slide close to the centre of the Cut was dredged by four dredgers.

The **Pacific Division**, was responsible to make 18 km of canal. The excavation of the Pacific Approach-Channel was threatened by silting up by the littoral drift. For that reason a 5 km long breakwater was built parallel to the channel. This **Naos-breakwater** was completed in 1914 by which time more than 1.700.000 m^3 rock and soil had been tipped into the unstable mud of Panama Bay; more than ten times the amount originally estimated.

The excavation was carried out in three ways:

- dry excavation by steam-shovel	: 1.900.000 m^3 at S 1.05/ m^3
- hydraulic jet	: 1.900.000 m^3 at S 0.94/ m^3
- dredging	: 27.000.000 m^3 at S 0.33/ m^3

Excavation by hydraulic jet was an innovation. The earth was first dislodged by means of powerful jets of water, then carried (water-borne) to a series of centrifugal suction pumps, which pumped it to adjacent disposal areas. There were three of these pumps, each mounted on a concrete barge of 19.5 m x 7.3 m and 1.73 m deep. The pumps were electrical driven by Westing-house engines, the power supplied by the **Miraflores central station** via armoured submarine cables.

When the first ship slipped silently through the blue green waters of the Panama Canal on August 15, 1914 a dream dating back to explorations of Columbus was fulfilled. Between 1881 and 1914 a total quantity of approx. 200 million m3 of earth and rock had been excavated. A little known fact, however, is that more material has been removed from the Panama Canal since its completion then was removed during its creation. Between 1915 and 1981 approx. 297 million m^3 of spoil were dredged for maintenance or improvement of the waterway. Dredging is a year-round operation in the canal.

The Panama canal measured from shore to shore is 22 miles long, but the total length if measured from deep water to deep water is 40 miles long. The total channel improvement scheme from 1918 to 1970 cost approximately *S* 95 million, of which S 71 million was spent on the widening of the Gaillard Cut (the new name for the Culebra Cut, named after the Army Engineer in charge from 1907 to 1933), from a width of 90 m to 150 m.

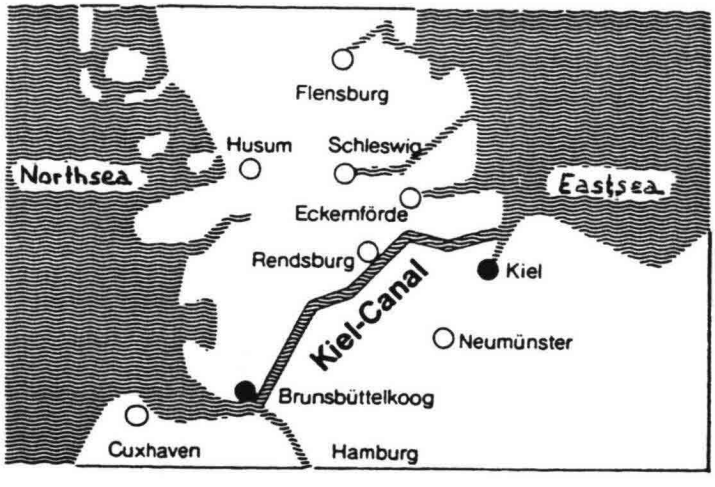
Kiel - Canal (Nord-Ostsee kanal)

At first named Kaiser Wilhelm Kanal, now internationally known as the Kiel Canal - the Nord Ostsee Kanal links the North Sea with the Baltic. Opened in 1895 it passes through fairly level country, but the different tidal ranges of the North Sea (about 3.50 m) and the Baltic (30-40 cm) made it necessary to create an independent water level between locks. Drainage canals and many small streams supply the water for the canal. Sometimes when an on-shore wind has raised the outer sea level, extra summit water flows into the canal through one of the locks.
It is a very busy waterway with a constant traffic of ships of many sizes. Very small ships pass each other in especially widened sections but large ships sometimes have to stop there. Traffic centres regulate the flow of ships by means of traffic lights displayed on tall masts along the canal; in addition, each canal pilot carries with him a small radio transceiver. Through this, he is informed of the speeds and positions of other ships in his neighbourhood and can be told when and where to stop his own vessel should the need arise.

Sometimes canal pilots come aboard long before the ship reaches the canal area. They direct the helmsman on the bridge and are responsible for the safe passage of the ship.
A shipping canal shortens a sea route but at the same time cuts through existing land routes. So road and rail traffic has to cross over high bridges, allowing the ships to pass underneath; other roads cross via underpasses (tunnels) and the Kiel Canal has six such crossings. Traffic from minor roads is carried across by fast ferries which operate a regular service. At Rendsburg there is a high level railway bridge with a transporter bridge hanging on wires under it. The Canal Administration as far as traffic control, shipping police, river police, construction, maintenance and operation is concerned is the responsibility of the "Wasser-und Schiffahrts direction Nord" (authority for shipping and water).

Length of canal	99 km
Width at water level	162 m
(partly still	102.5 m)
Width at bottom	90 m
(partly still 44 m)	
Depth	11 m
Draught allowed	9.5 m
Clearance above water	40 m
Maximum speed allowed	15 km/h

Locks:
4 side by side at Brunsbüttel
4 side by side at Kiel-Holtenau
Inside dimensions 310 m by 40 m
(a third middle gate can be used
to halve the lock capacity)



Originally the Kiel Canal was built for naval strategic reasons; after seven years construction time the canal opened for ocean-going ships on June 21, 1895. Only ten years after its inauguration the Kiel Canal became too small to meet the requirements of the Imperial Navy. The resulting enlargement, carried out between 1907 and 1914 doubled the cross section area. Until the 1950's the canal fulfilled all expectations. Due to a rapid increase in both the number and size of ships, the erosion of the underwater slopes became a serious problem. Extensive measures to prevent the slopes from collapsing resulted in another enlargement of the cross section over an 80 km length. This protection programme by widening by some 60 m at water level until km 80 begun in 1965. (estimated costs 1988 : Dm 840 million)
Dredging : 6.5 million m^3 per annum fluid mud from the brackish water area Brunsbüttel km

Water management : total catchment area of the Kiel Canal is 1580 km², of which 250 km² lie below water level and are drained by 19 pumping stations - average water ingress from this catchment area 20 m³/s (min. 4 m³/s, max. 190 m³/s). Surplus water is discharged mainly into the Elbe river via special openings in the Old Lock upper gates Brunsbüttel.

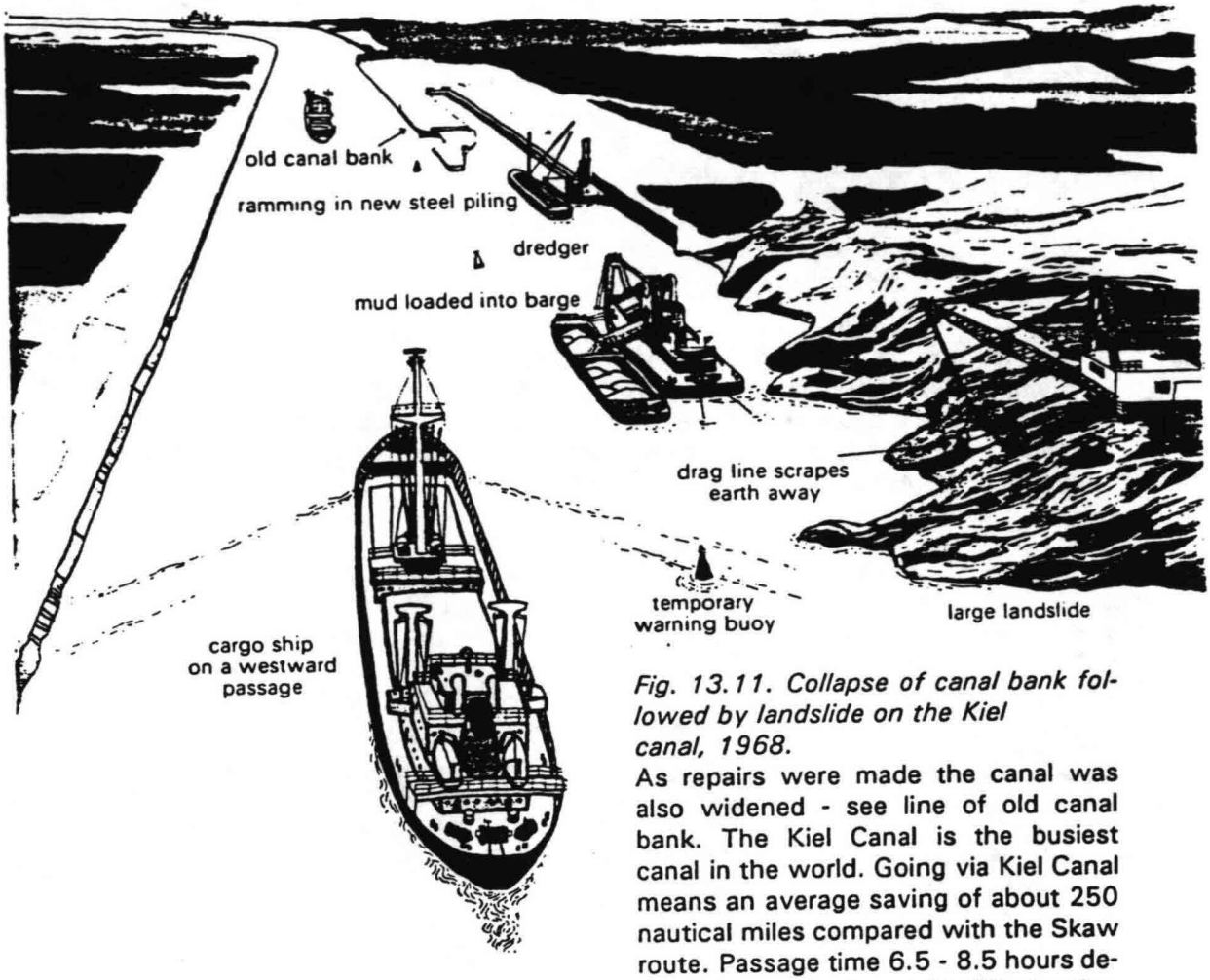


Fig. 13.11. Collapse of canal bank followed by landslide on the Kiel canal, 1968.
As repairs were made the canal was also widened - see line of old canal bank. The Kiel Canal is the busiest canal in the world. Going via Kiel Canal means an average saving of about 250 nautical miles compared with the Skaw route. Passage time 6.5 - 8.5 hours depending on ship size and traffic density.

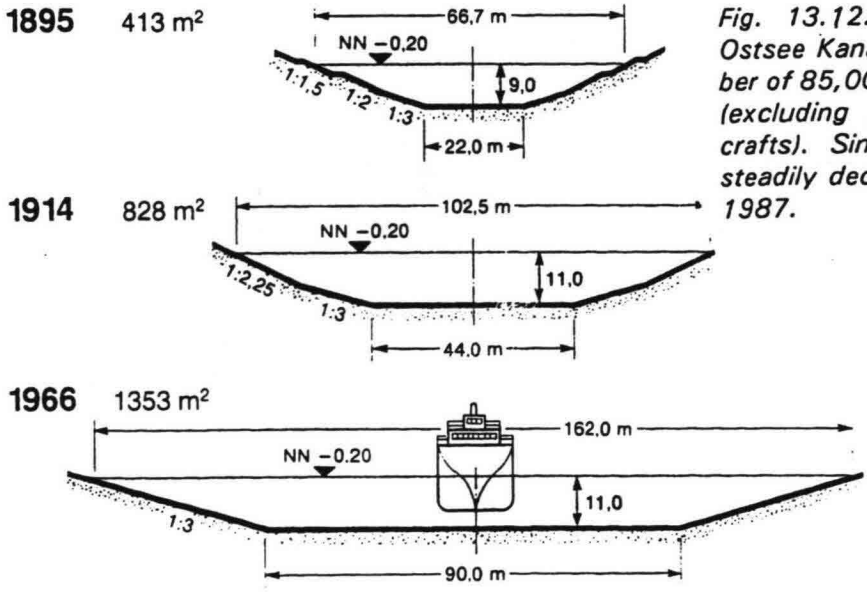


Fig. 13.12. Widening of the "Nord Ostsee Kanal" In 1965 a record number of 85,000 ships used the Kiel Canal (excluding pleasure and other small crafts). Since then this number has steadily decreased to about 45,000 in 1987.

This is 124 ships per day. It still is by far the busiest canal for larger ships in the world compared to Suez Canal with 20,000 and Panama Canal with 10,000 passages. Although the number of ships has gone down, the volume of cargo remained roughly constant (80 million BRT, 58 million ton). This means that the

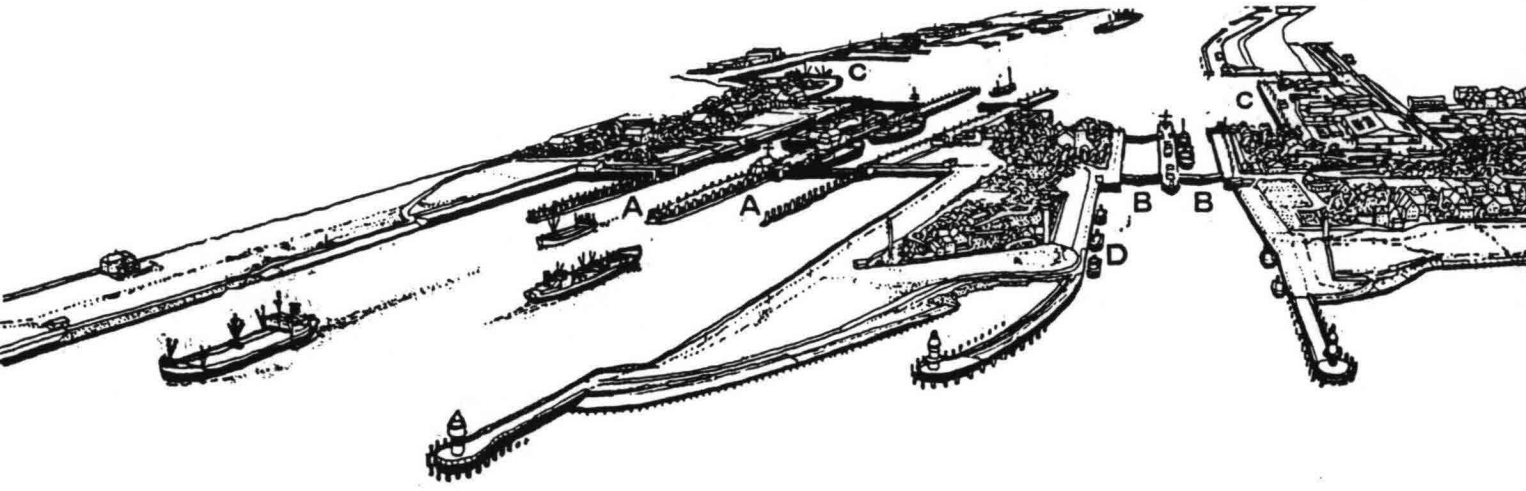
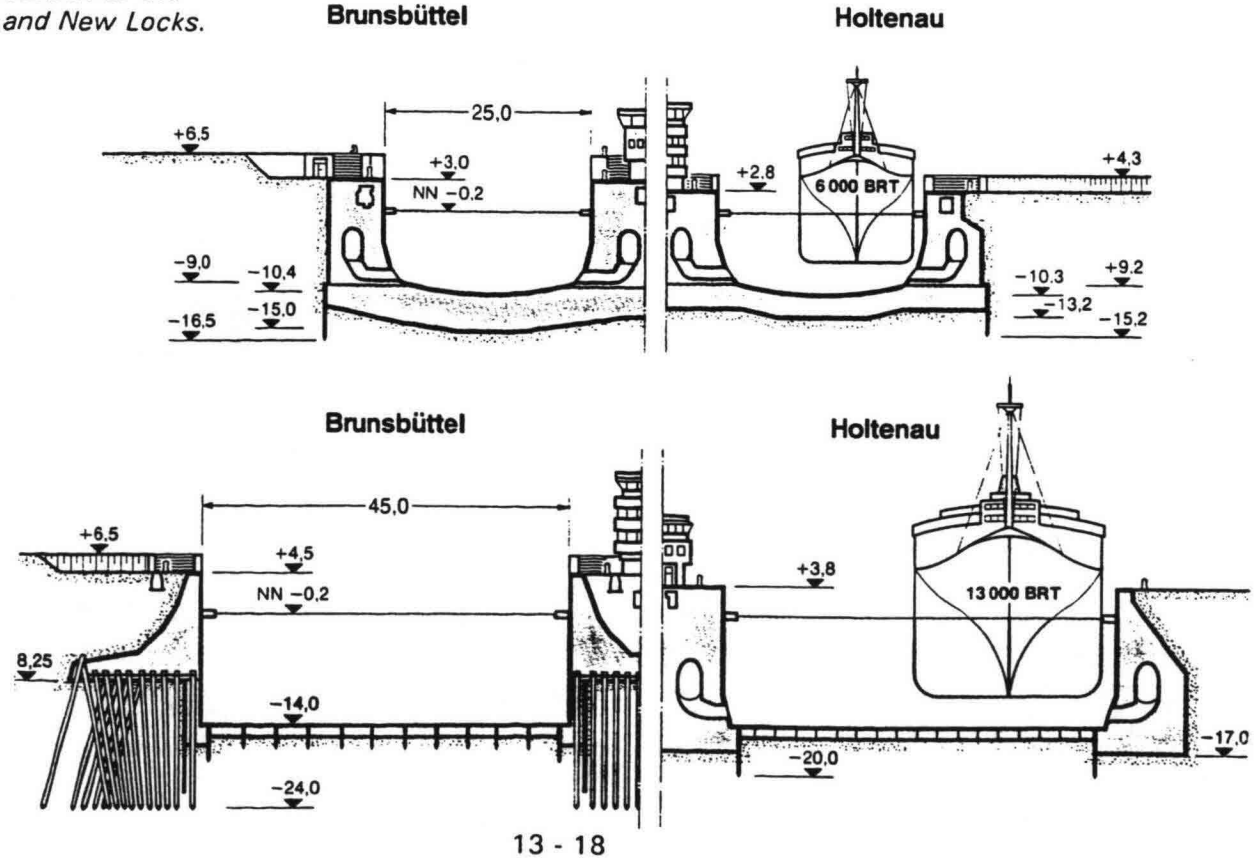


Fig. 13.13. Locks at the entrance from the Elbe to the Kiel Canal (Brunsbüttel).
A. New locks for large vessels, B. Older locks for smaller vessels, C. Harbours and jetties, D. Waiting pilot boats. Four locks at each end of Kiel Canal prevent the tides of the Elbe estuary and Baltic Sea level fluctuation from affecting the canal water level. The locks at each end consist of a pair of small locks dating from 1895 (the Old Locks) and a pair of large locks completed in 1914 (the New Locks).

Old Locks : effective length : 125 m, width : 22 m, depth of sill Brunsbüttel - 10.2 m, mitre gates, each lock chamber with two pairs of ebb gates and two pairs of flood gates, filling and draining of lock be means of two lateral canals with 12 culverts each. Average locking time : 30 minutes.

New locks : effective length : 310 m, width : 42 m, depth of sill - 14.0 m, sliding gates, each lock chamber with 3 gates (the middle gates, also used as spare gates allow reduction of used lock length in order to reduce locking time); filling and draining Brunsbüttel by means of bypasses, average locking time : 45 minutes.

Fig. 13.14. Cross section of Old and New Locks.



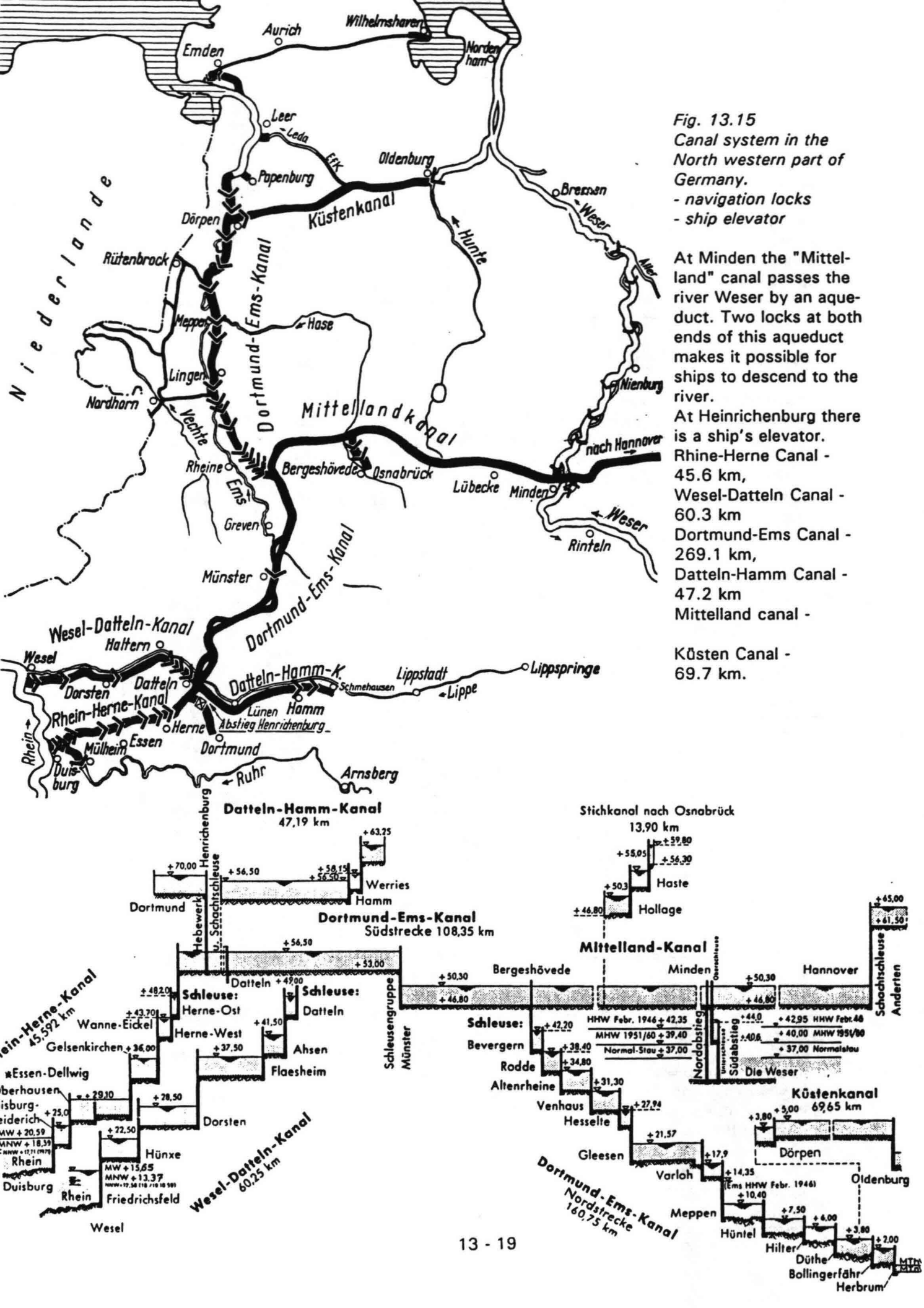


Fig. 13.15
Canal system in the North western part of Germany.
- navigation locks
- ship elevator

At Minden the "Mittelland" canal passes the river Weser by an aqueduct. Two locks at both ends of this aqueduct makes it possible for ships to descend to the river.
At Heinrichenburg there is a ship's elevator.
Rhine-Herne Canal - 45.6 km,
Wesel-Datteln Canal - 60.3 km
Dortmund-Ems Canal - 269.1 km,
Datteln-Hamm Canal - 47.2 km
Mittelland canal -

Küsten Canal - 69.7 km.

Saint Lawrence Seaway

The Saint Lawrence River, one of the major streams in North America, is the outlet of the Great Lakes. The Saint Lawrence River and Saint Lawrence Gulf are part of a continuous waterway (the Great Lakes - St. Lawrence system) from Duluth, Minn. to the Atlantic. The St. Lawrence was utilised as an Indian water 'highway' and it was known as the 'River without end'. Since 1842, through treaty arrangements, the river has been opened to ships of the United States. The St. Lawrence Great Lakes system, some 3,800 km in length, has its source in the St. Louis River, which enters Lake Superior at Duluth, Minn. From this point, the waterway follows the Great Lakes to the northeastern end of Lake Ontario, where the St. Lawrence River itself begins. The river flows northeast until it enters the Gulf of St.Lawrence and, eventually, the Atlantic Ocean.

One of the most important canal-building projects of recent times was the **St Lawrence Seaway**, constructed jointly by the United States and Canada. This great system of rivers, canals and lakes makes the entire distance from the Atlantic to **Duluth**, Minnesota, on Lake Superior, navigable for ocean-going ships. The St. Lawrence Seaway connects with the **Mississippi- Missouri- Ohio River system** by means of the **Chicago Sanitary and Ship Canal**; indeed, it opens up the entire centre of the North American continent to shipping.

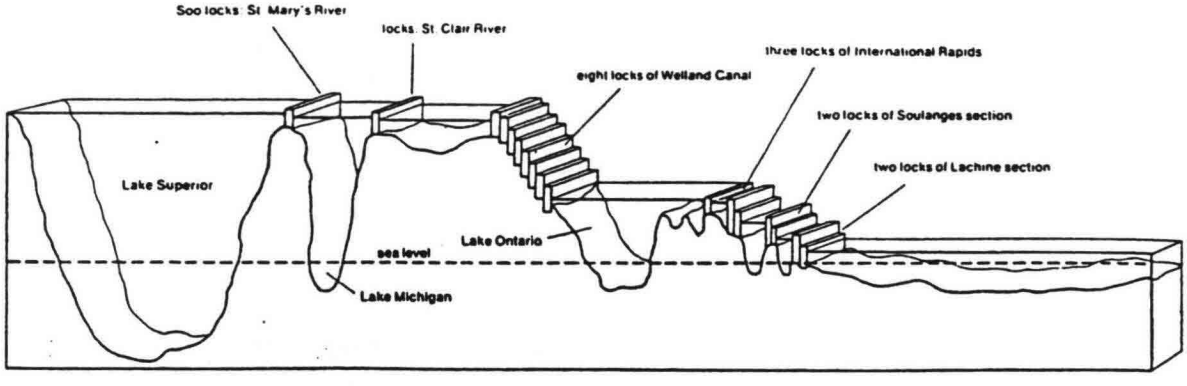


Fig.13. Saint Lawrence Seaway.

In about 1826 a Canadian, Hamilton Meritt, started work on a canal with 40 wooden locks, which was completed in 1829. From the early 18th century, the Canadians conducted a program of channel dredging and canal construction that by 1903 provided a 4.25 meter waterway from the Atlantic to **Lake Erie**. Beginning 1823, the United States carried out improvements in the connecting channels of the Great Lakes which, by 1914, provided 6.4 meter navigation for upbound and 7.6 meter for downbound traffic, from the eastern end of **Lake Erie** to the western end of **Lake Superior**.
The **Erie Canal** was made in the nineteenth century, connecting Albany on the Hudson River with the Great Lakes across the length of northern New York State. New York City in large part owes its commercial preeminence to the Erie Canal.

By the 1930's the Canadian-built 45 km long **Welland Ship Canal**, bypassing Niagara Falls, made it possible for vessels to navigate between **Lake Ontario** and **Lake Erie**. Nearly 20 years in the building (1913-1932), the Welland Canal overcame the 99 meter difference in water levels of the two lakes by means of eight huge locks with minimum depths of 9 meters over the sills.

The improvement of the **St. Lawrence river** is one of the most important engineering works in **Canada**. The first improvement was started in 1844. The machinery first used was built in Glasgow, Scotland, but since then nearly all the dredgers have been built in Canada. The work done from 1844-'50 was in **Lake St. Peter** to deepen some shallow places in order to allow 500-ton vessels to reach Montreal. In the period 1851-1888 about 15 million m^3 of material was dredged at a total cost of 26 cents/m^3 between Montreal and Quebec (160 miles distance). The work from the start has been done with dredgers and machines owned by the channel authorities. In 1889-1899 another 2,7 million m^3 was dredged in widening and cleaning the 8.3 m deep channel.

In 1899 work started on a new project namely a 9 m channel from **Montreal** to the **Traverse**, the distance to be actually dredged being 70 miles. The minimum width of this channel is 120 m in the straight portions and from 150 to 225 m on the curves or turns. The total quantity dredged was approx. 52 million m^3. The material dredged was dumped into the river at places where it did not interfere with navigation.

In 1911 the federal governments of Canada and the United States created the **International Joint Commission** to deal with boundary water problems, to conduct studies and make recommendations with respect to the improvements of St. Lawrence navigation between Montreal and Lake Ontario. The commission concluded that the waterway should be cooperatively improved and suggested that this could be accomplished most economically in the International Rapids section through a combined navigation-power project. In the 1920's the government of Canada was reluctant to enter into treaty negotiations, but after a change of government negotiations were started, culminating in the **St. Lawrence Deep Waterway Treaty** of July 19, 1932.

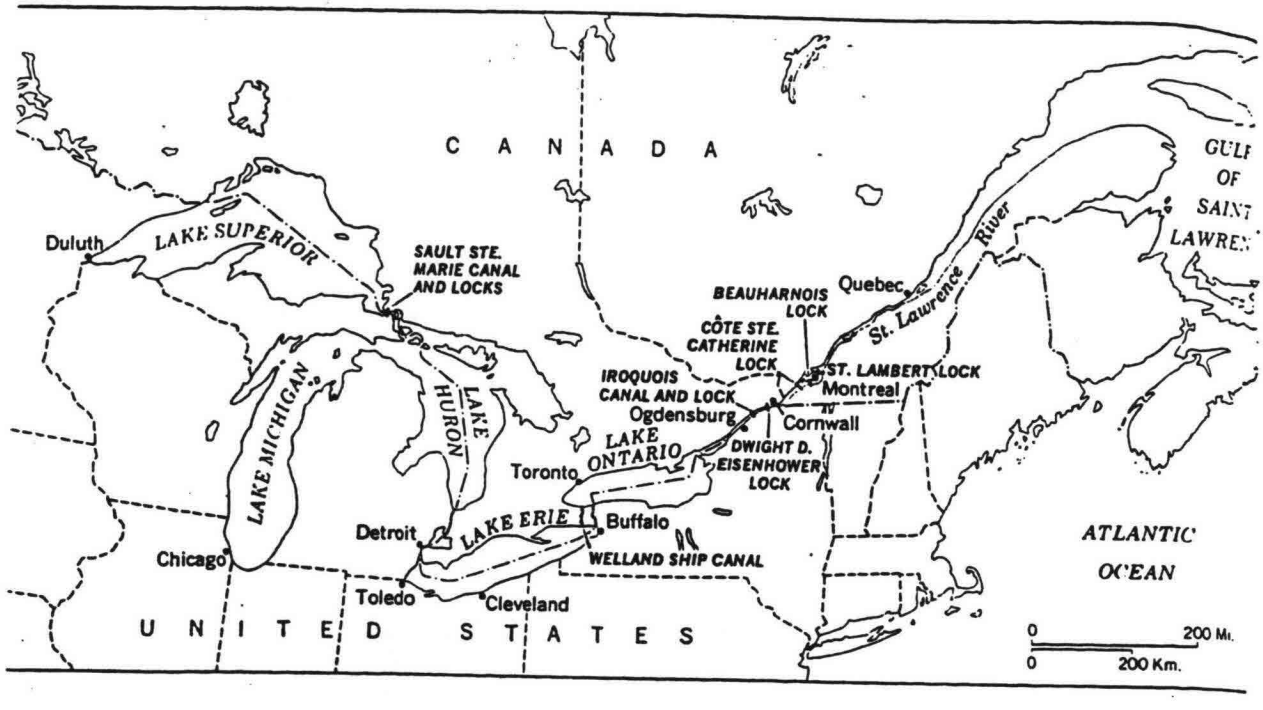
The treaty provided for the cooperative construction of a 8.2 m deep waterway from Lake Superior to the Atlantic. It also provided for the development of the potential electric power of the International Rapids section, the power and the costs to be shared equally by the two countries. The U.S. Senate failed in 1934 to give the treaty the two-thirds majority approval necessary for ratification. In 1948 the separation of the power phase from the navigational phase was proposed. In 1954 the **St. Lawrence Development** was created after both governments agreed of developing a 8.2 m channel from Lake Erie to the Atlantic Ocean. This ended the purely political phase of the project and at last cleared the way for the beginning of construction in late 1954.

At that time most of the seaway was deep enough for large-scale navigation. From the open ocean to Montreal, the river had a depth of over 10 meters and could accommodate all but the largest oceangoing vessels. From Ogdensburg (New York State) to the head of the lakes, the waterway had a minimum depth of 7.6 m, easily navigable by specially built lake steamers carrying up to 25,000 tons. But in between these two long stretches of open navigation, from Montreal to Ogdensburg, a distance of less than 195 km, river navigation was impeded by dangerous shoals and rapids. Trough this section all traffic was obliged to pass through a series of 4 meters canals, containing a total of 22 locks, built by Canada before 1903. Only small canallers of less than 3,000 tons were able to squeeze through these restricted channels. It was the removal of this "bottle-neck" that was the major objective of the construction programme carried out in 1954-1959.
The related hydroelectric project was completed in 1958.

Part of the work was the digging of the **Wiley-Dondero** ship canal to bypass the power-project spillway dam. This canal, which is 16 km long and 8 m deep, has two locks, the Dwight D. Eisenhower Lock at the upstream end and the Bertrand H. Snell Lock at the downstream end. Each is 260 m long and 24 m wide. Canada constructed a short canal and the Iroquois lock to bypass the Iroquois Control Dam. It also built a canal and two locks (Upper and Lower Beauharnois) between lakes St. Francis and St. Louis, and farther downstreams, another canal and two more locks to bypass the Lachine Rapids near Montreal. All the locks were comparable in size.

After Montreal Harbour the first lock is the **St. Lambert**, which rises 4.5 m to the laprairie Basin and proceeds 14 km to the **second Côte Ste. Catherina Lock**, which rises 9 m to Lake St.Louis and bypasses the Lachine Rapids. Thereafter, the channel runs to the lower **Beauharnois Lock**, which rises 12 m to the level of Lake St. Francis via a 21 km canal. Fifty km farther, the seaway crosses the international boundary to the **Bertrand H. Snell Lock**, with its lift of 14 m to the **Wiley-Dondero Canal**; it then lifts another 12 m by the **Dwight D. Eisenhower Lock** into Lake St. Lawrence. Leaving the western end of the lake, the seaway bypasses the **Iroquois Control Dam** and proceeds through the Thousands Islands to Lake Ontario. Eight locks raise the water 98 m over 45 km from Lake Ontario to Lake Erie. The **St. Marys Falls Canal**, with a lift of about 6 m, carries the waterway to Lake Superior, where the seaway terminates. The Saint Lawrence Seaway was officially opened as a deep waterway on June 26, 1959.

As anticipated, seaway traffic increased steadily after the deep waterway was opened. Cargo tonnage through the Montreal-Lake Ontario rose from an annual average of about 10 million tons in the mid-50's to more than 50 million tons in the early 1970's. The traffic rise was soon causing congestion, especially in the older, **Welland Canal Section** of the seaway. In 1967 a six-year project was launched to build a new 13 km channel bypassing the city of Welland, Ontario, where drawbridges had impeded rapid movement. Other improvements included the introduction of a modern traffic control system.



Rhine- Main- Danube Canal (Germany)

A first attempt to link the two great river systems of Europe - the Rhine and the Danube (Donau) - was made by Emperor Charlemagne in A.D. 793. A canal of running only five kilometres between the Rivers Altmuhl and Schwabische Rezat would connect the Donau with the Main. This project, the **Fossa Carolina**, was to be 91 m wide. An army of men was set to work digging a 2.4 m deep trench just outside the present-day village of Graben (the name means "trench" in German). The Fossa Carolina was never deep enough to accommodate the difference in elevation between the two small rivers it was intended to connect. The work was abandoned after just two month as the trench repeatedly filled with water under heavy rains and the banks turned into an unstable ooze.

In 1837, King Ludwig I of Bavaria put a corps of labourers to work digging a navigable trench between Bamberg and Kelheim that took in much of the route of today's waterway. The Ludwig Danube- Main Canal was completed in 1845 and operated for a century in a mostly desultory and loss-making fashion, unable to compete effectively with the new railways and left idle for long periods by low water in the Main. With locks only 4.8 m wide, the canal was plagued by the problem of having to transfer goods from river barges to narrow canal boats.

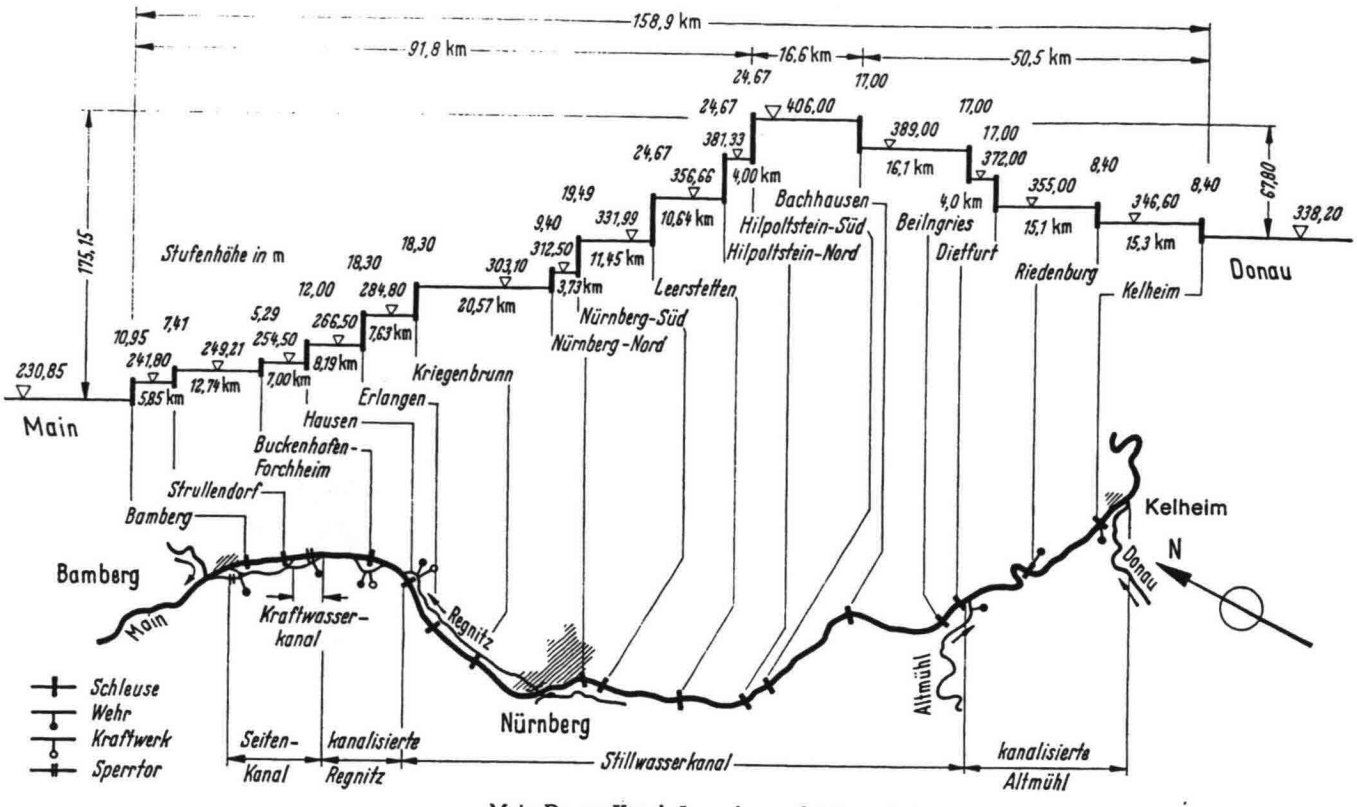
In 1921, under the auspices of the federal and Bavarian state governments, a company was formed to build a replacement canal on a far grander scale. Apart from a hiatus after the Second World War, the firm of Rhein-Main-Donau AG (RMD) has spent the past 70 years building the waterway piece by piece, first improving the channels of both the Main and the Danube. It has also erected 55 hydroelectric power stations (the number will eventually rise to 60). Profits from the power stations - 55 million marks a year - underwrite much of the expense of construction.

The Main-Danube canal was built mostly in the past 30 years. The waterway, opened in September 1992, can accommodate huge Euro-barges carrying up to 2,425 tons of bulk cargo. It runs for 171 km between Bamberg and Kelheim and climbing and dropping a total of 240 m as it crosses the Frankische Alb; the highest point at 406 m (thereby the highest point of any commercial waterway in Europe).

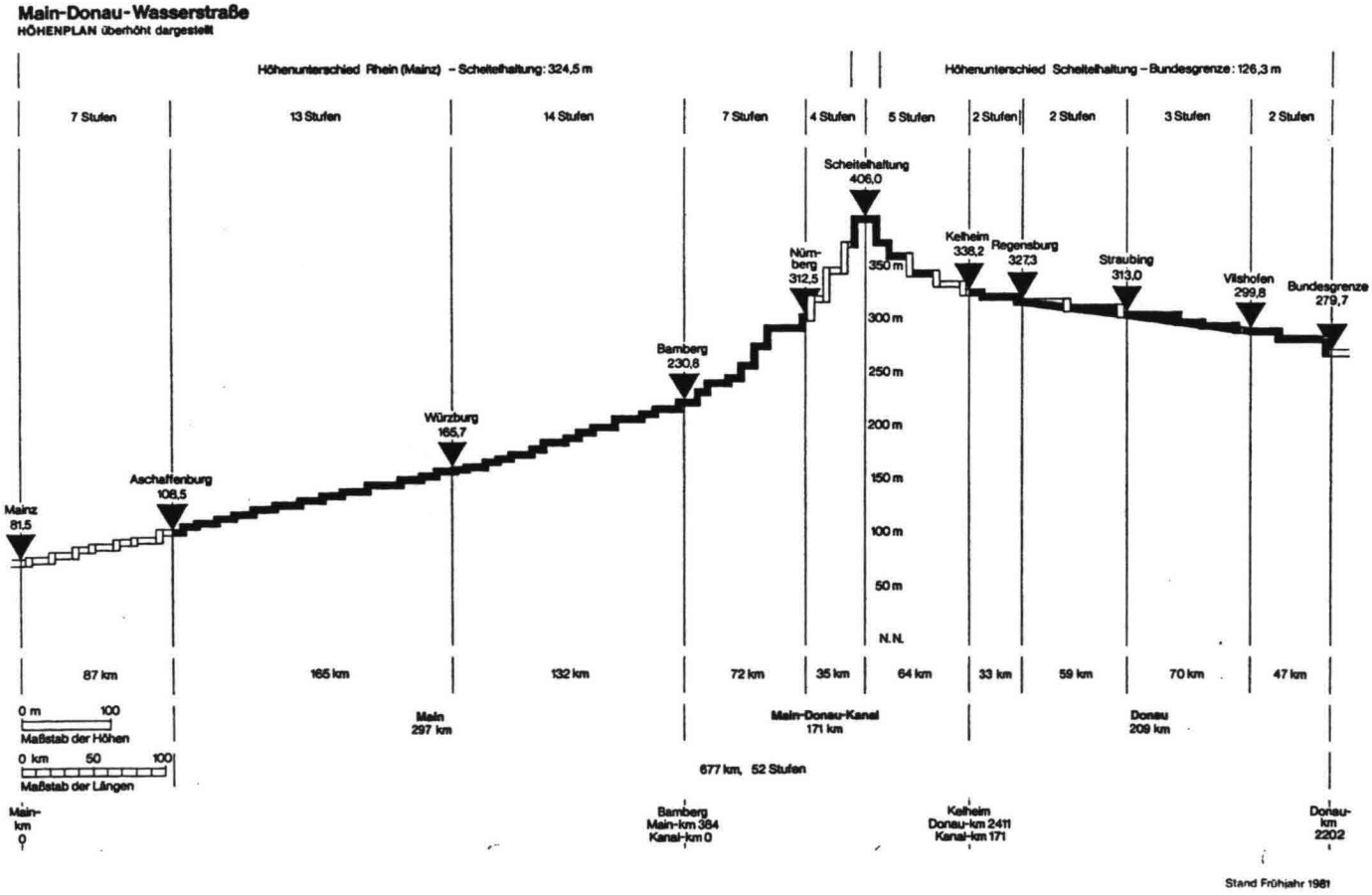
Under pressure of environmentalists the part of the canal descending into the Altmuhl valley does not look like a straight, functional, businesslike - impressive but charmless canal but like a river - stately, winding, varied in width, its sloping banks crowded with foliage, its backwaters teeming with birds. For the first time landscape architecture was applied on a massive scale building a canal. Some 280 million mark was spent on landscaping alone. Along this stretch every bridge is different to avoid monotony. Architects were employed to design bridges that are beautiful as well as functional; with a novel wooden footbridge in Europe (190 m) across the canal built in the shape of a two lazy waves. Therefor not just a canal was built but the whole valley was transformed. A total of 100 million m^3 of earth has been removed to make the canal.

Each lock chamber on the Main-Danube Canal is 190 m long, 12 m wide and up to 30 m deep. The vast chambers can be filled in just 20 minutes. Barges on the waterway have to negotiate many locks - more than 50 between Frankfurt and Passau alone.

Fig. 13. Stairsteps locks on the Main River lift barge to Bamberg, northern entry point to the canal. From there 11 locks raise ships to the highest point on any commercial waterway in Europe. Five more locks then lower vessels to Kelheim, the southern terminus of the canal.

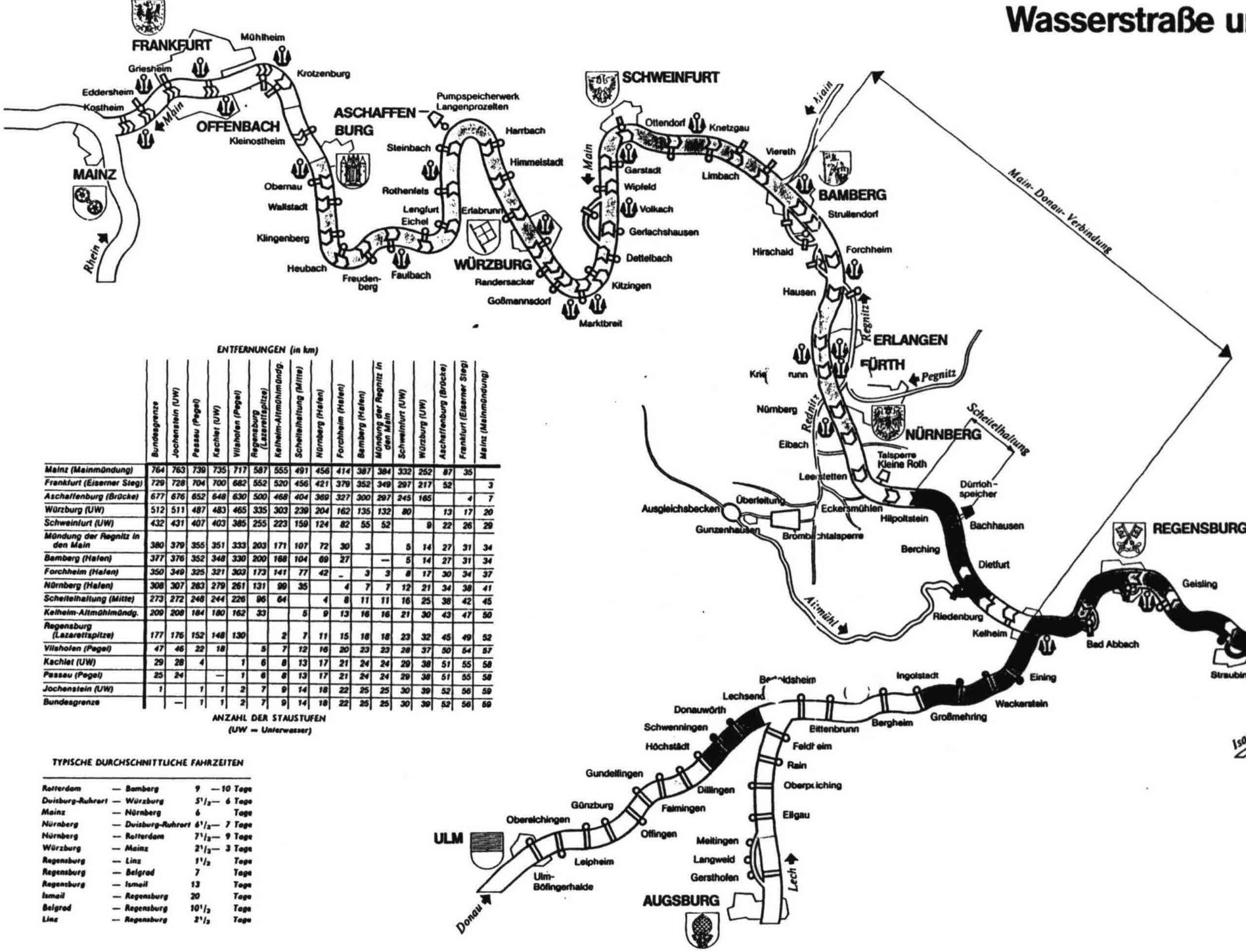


Main-Donau-Kanal, Lageplan und Längsschnitt



Wasserstraße und Wasserkraftwerke

LAGEPLAN



ENTFERNUNGEN (in km)

	Bundesgrenze	Jochenstein (UW)	Passau (Pegel)	Kachlet (UW)	Vilshofen (Pegel)	Regensburg (Lazarettspitze)	Kelheim-Altmühlmündg.	Scheitelhaltung (Mitte)	Nürnberg (Hafen)	Forchheim (Hafen)	Bamberg (Hafen)	Mündung der Regnitz in den Main	Schweinfurt (UW)	Würzburg (UW)	Aschaffenburg (Brücke)	Frankfurt (Eiserner Steg)	Mainz (Mainmündung)
Mainz (Mainmündung)	764	763	739	735	717	587	555	491	456	414	387	384	332	252	87	35	
Frankfurt (Eiserner Steg)	729	728	704	700	682	552	520	456	421	379	352	349	297	217	52		3
Aschaffenburg (Brücke)	677	676	652	648	630	500	468	404	369	327	300	297	245	165		4	7
Würzburg (UW)	512	511	487	483	465	335	303	239	204	162	135	132	80		13	17	20
Schweinfurt (UW)	432	431	407	403	385	255	223	159	124	82	55	52		9	22	26	29
Mündung der Regnitz in den Main	380	379	355	351	333	203	171	107	72	30	3		5	14	27	31	34
Bamberg (Hafen)	377	376	352	348	330	200	168	104	69	27		—	5	14	27	31	34
Forchheim (Hafen)	350	349	325	321	303	173	141	77	42	–	3	3	8	17	30	34	37
Nürnberg (Hafen)	308	307	283	279	261	131	99	35		4	7	7	12	21	34	38	41
Scheitelhaltung (Mitte)	273	272	248	244	226	96	64		4	8	11	11	16	25	38	42	45
Kelheim-Altmühlmündg.	209	208	184	180	162	33		5	9	13	16	16	21	30	43	47	50
Regensburg (Lazarettspitze)	177	176	152	148	130		2	7	11	15	18	18	23	32	45	49	52
Vilshofen (Pegel)	47	46	22	18		5	7	12	16	20	23	23	28	37	50	54	57
Kachlet (UW)	29	28	4		1	6	8	13	17	21	24	24	29	38	51	55	58
Passau (Pegel)	25	24		—	1	6	8	13	17	21	24	24	29	38	51	55	58
Jochenstein (UW)	1		1	1	2	7	9	14	18	22	25	25	30	39	52	56	59
Bundesgrenze		—	1	1	2	7	9	14	18	22	25	25	30	39	52	56	59

ANZAHL DER STAUSTUFEN
(UW = Unterwasser)

TYPISCHE DURCHSCHNITTLICHE FAHRZEITEN

Rotterdam	— Bamberg	9	— 10 Tage
Duisburg-Ruhrort	— Würzburg	5½	— 6 Tage
Mainz	— Nürnberg	6	Tage
Nürnberg	— Duisburg-Ruhrort	6½	— 7 Tage
Nürnberg	— Rotterdam	7½	— 9 Tage
Würzburg	— Mainz	2½	— 3 Tage
Regensburg	— Linz	1½	Tage
Regensburg	— Belgrad	7	Tage
Regensburg	— Ismail	13	Tage
Ismail	— Regensburg	20	Tage
Belgrad	— Regensburg	10½	Tage
Linz	— Regensburg	2½	Tage

fertig | im Bau | geplant | Wasserkraftausbau fertig | Wasserkraftausbau geplant | Wehr und Schleuse | Schleuse | Doppelschleuse | Wehr | Kraftwerk | Talsperre oder Speicher | Hafen oder große Umschlagstelle

0 10 20 30 40 50 km

Rhein-Main-Donau AG, Leopoldstraße 28, 8000 München 40

Stand Frühjahr 1981

13.2. Canals for water conveyance

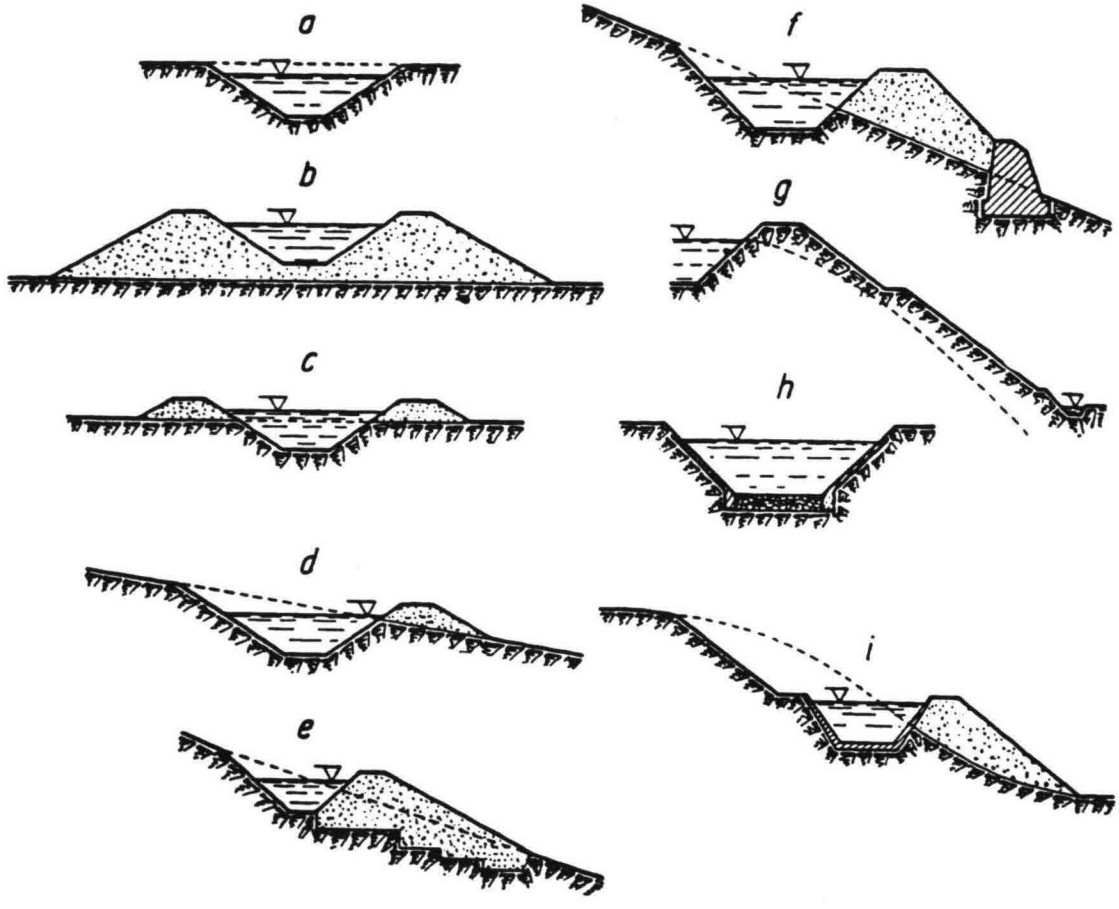


Fig. 13.18. Earth canals.

a. cut into the ground
b. elevated above the terrain
c. partly elevated, in level ground
d. with cut- and- fill cross-section in gently sloping ground
e. with cut- and- fill cross-section in steep ground
f. with cut- and- fill cross-section in steep ground, with the bank supported by a retaining wall
g. in very high fill
h. the side slopes lined with concrete, the impermeable bottom protected by a gravel layer
i. concrete-lined canal with cut- and- fill cross-section.

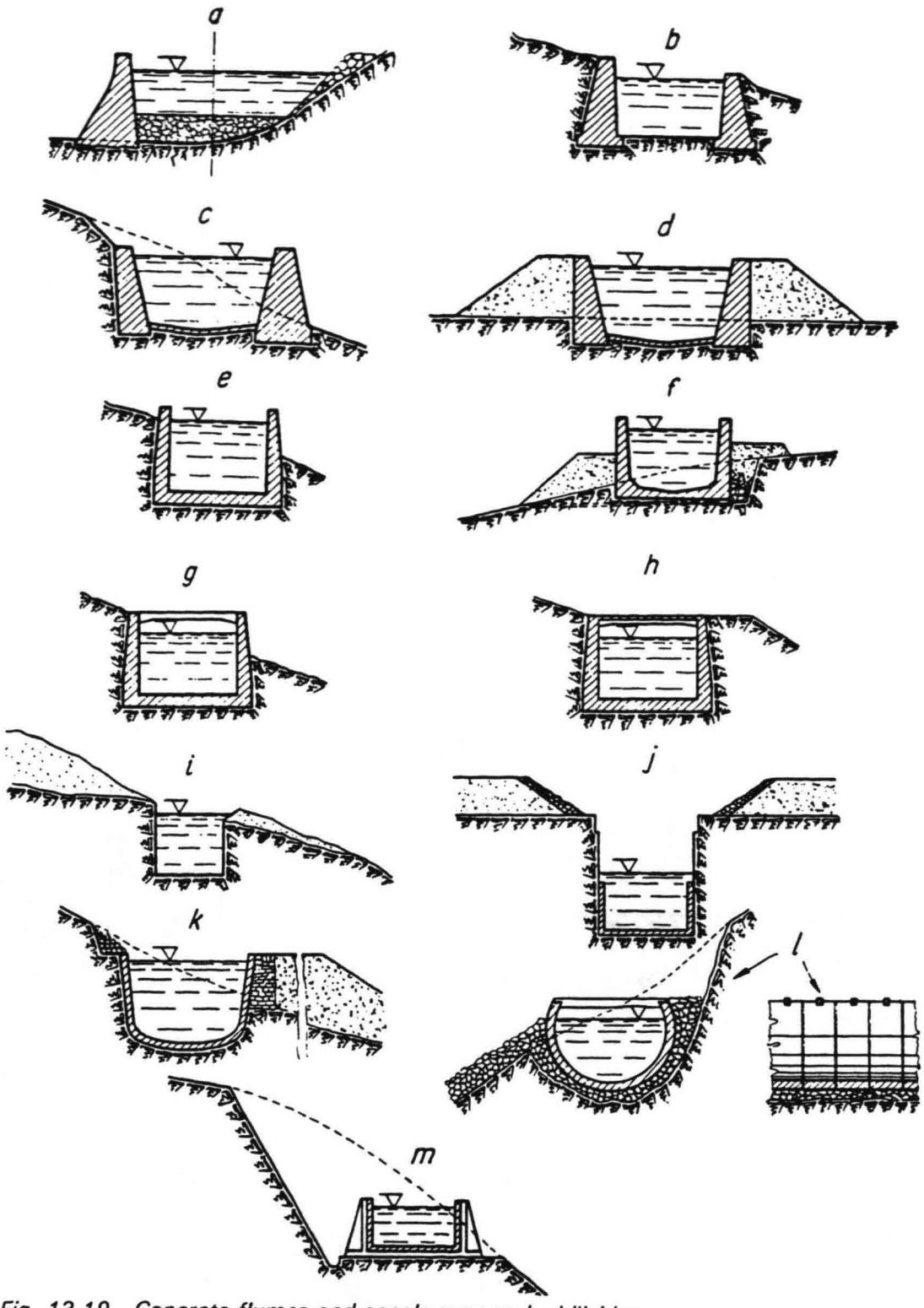


Fig. 13.19. Concrete flumes and canals over rocky hillsides.

a. canal with downhill retaining wall
b. canal with retaining wall at both sides
c. canal with retaining wall at both sides, and with lined bottom
d. canal with retaining wall at both sides, supported by embankments, and with lined bottom
e. open reinforced-concrete flume recessed into the ground
f. open reinforced-concrete flume supported by embankments
g. braced reinforced-concrete flume
h. covered reinforced-concrete flume recessed into the ground
i. unlined canal cut in rock
j. partly lined canal cut in rock
k. canal with cut- and- fill cross-section, supporting embankments, cut in rock
l. reinforced-concrete flume in trench excavated in rock
m. reinforced-concrete flume set on beach cut in rock.

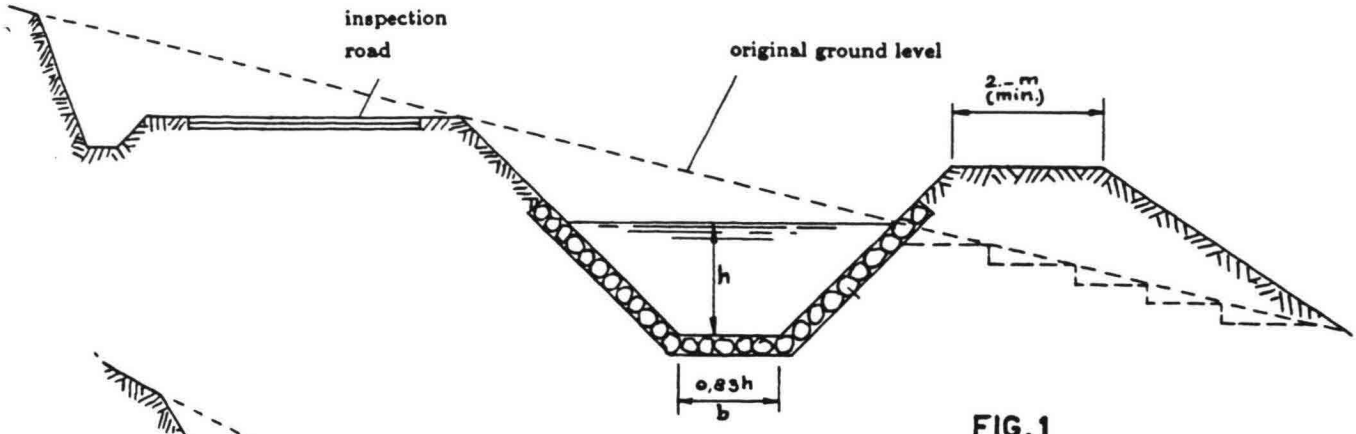


FIG. 1

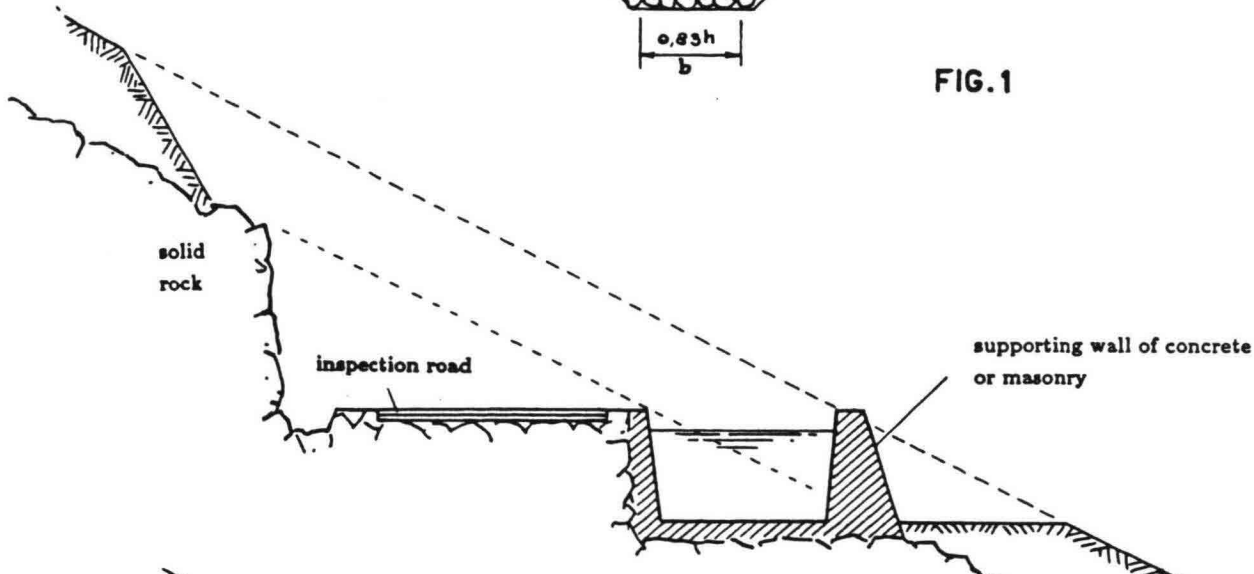


FIG. 2

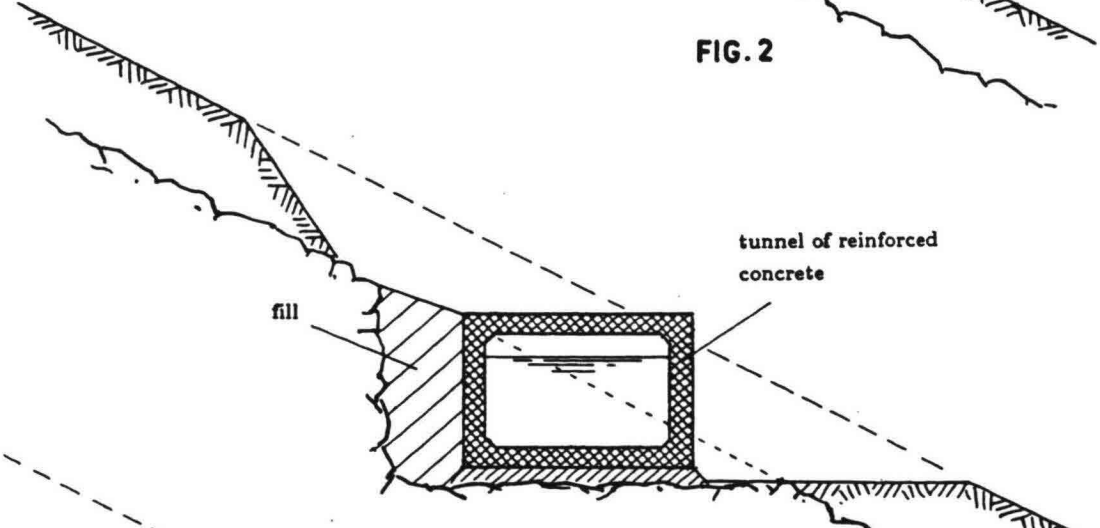


FIG. 3

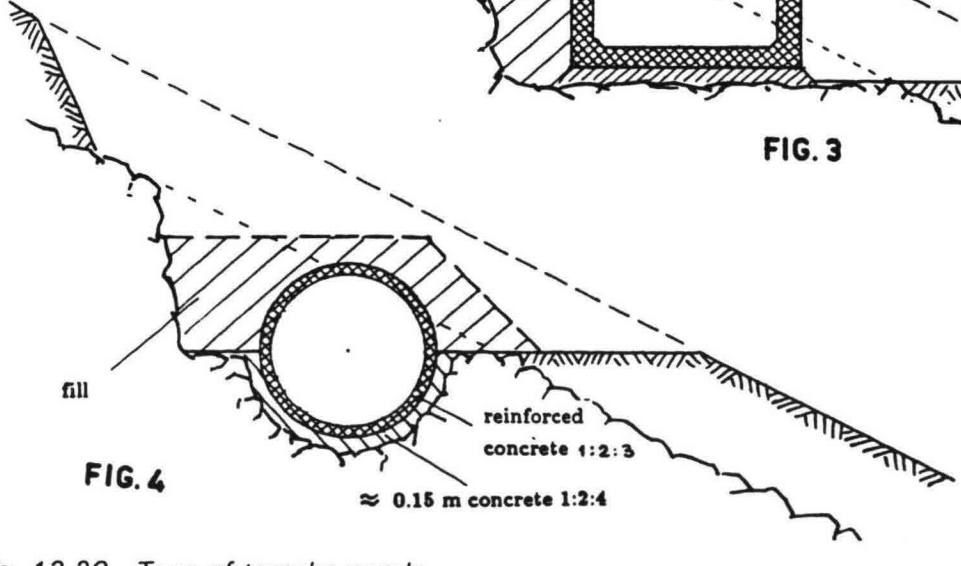


FIG. 4

Fig. 13.20. Type of tranche-canals.

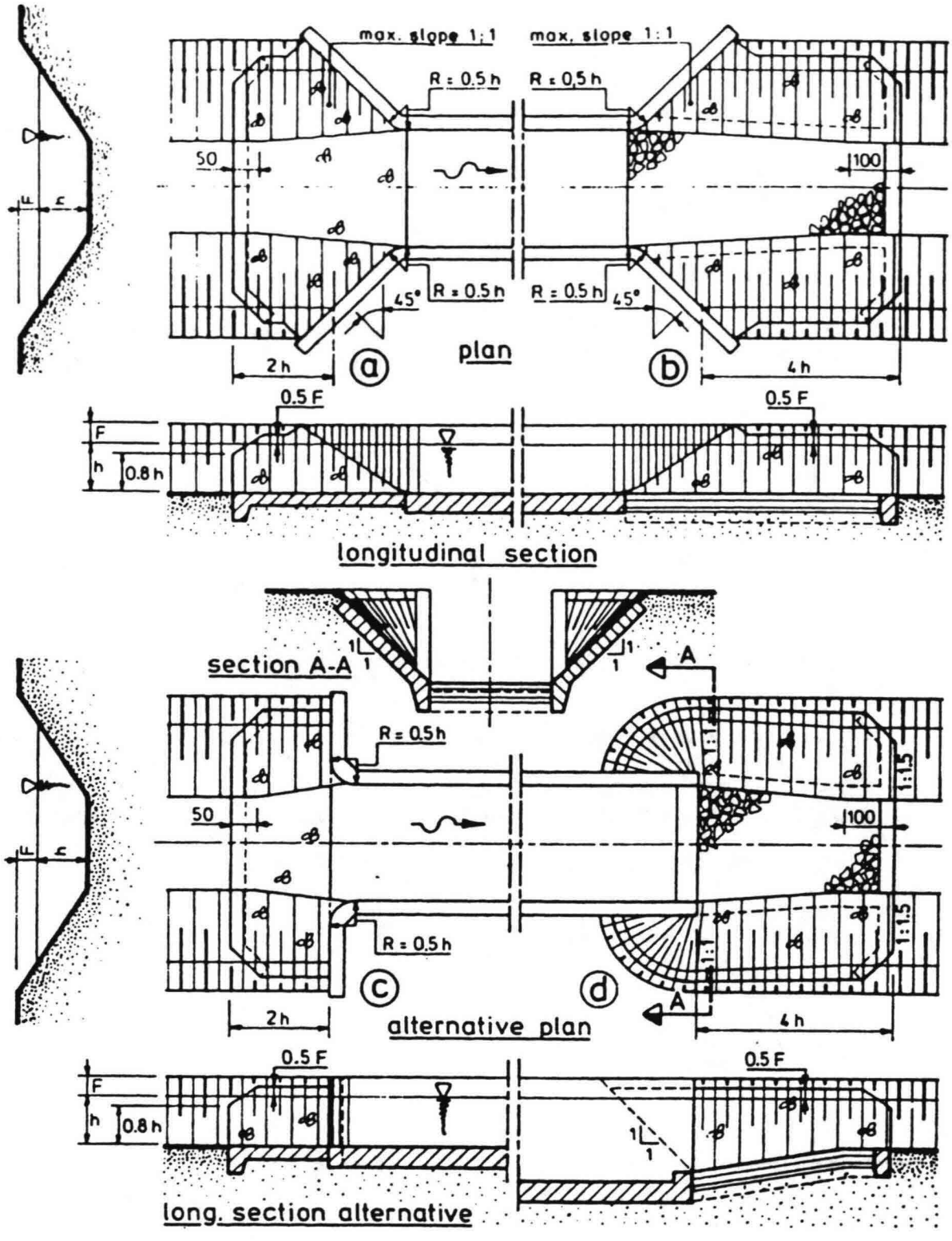


Fig. 13.21. Standard canal transitions.

Lining

Canals may be lined for one of the following reasons:

- reduction of roughness coefficient (the discharge conveyed in a lined canal is greater than in a similar earth canal for the same slope)
- increase of permissible velocity : lining protects the canal against erosion at a flow of great velocity
- reduction and/or prevention of seepage (water losses)
- increase of steepness of the slopes of the banks (less area occupied by the canal)
- partial lining around the water surface for protection against wave action (see chapter 12)

Types of lining

1. Stone paving, without cementing materials (against erosion
   - 1.1. rip-rap
   - 1.2. simple stone paving
   - 1.3. pavements upon a gravel or crushed stone draining layer
   - 1.4. rock fill

2. Stone and brick lining laid up in cement mortar (to reduced seepage)

3. Concrete lining
   Concrete lining are most extensively used; they reduce roughness, protect against erosion and prevent/reduce seepage
   - 3.1. concrete lining poured insitu
   - 3.2. lining of prefabricated slabs

4. Reinforced-concrete lining

5. Linings made of lean mixes and stone pavings sprayed with cement mortar (gunite process) (to reduce seepage)

6. Bitumen and asphalt linings

7. Bentonite linings

Bitumen and asphalt linings have been applied mainly in irrigation canals. They are more watertight and elastic than concrete linings. Thickness: 30-60 mm. Much experience has been gained with bituminous linings in the USA, especially in irrigation canals.

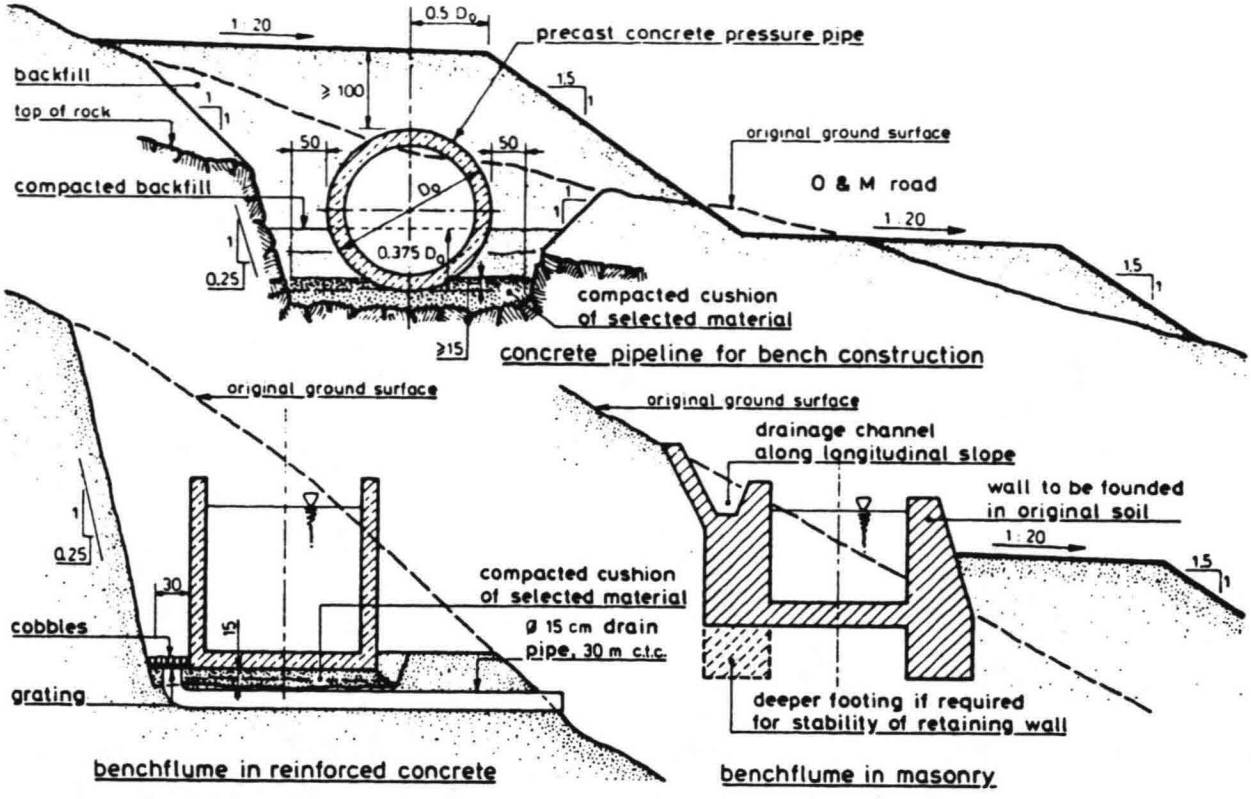


Fig. 13.22. Example of bench flumes

14. Dams and appurtenant works

Dams are of large size in order to create reservoirs for water storage. The most appropriate place for a dam is a narrow valley with an extended area upstream. In that case the dam is small and the volume of the reservoir large.

Dams up to 300 m high have been built. The high water level difference might be used in a water-power plant for generating electricity. The dam regulates also the river discharge : instead of extreme discharges with danger for flooding, and periods of low discharges with problems for navigation, there is a (more or less) uniform discharge over the year. In some cases it is possible to use the reservoir for irrigation purpose. It is obvious to try for a multi-purpose use of a dam project.

Several purposes:
- irrigation
- flood control
- water power
- storage of drinking water
- recreation

A dam project has several structures
- dam itself
- spillway
- intake structures

and sometimes:
- bottom outlet
- powerhouse
- diversion tunnel (during construction)

Table Large dams: World Register statistics (ICOLD 1984).

Group	Type	ICOLD code	Number	%
embankment dams	earthfill rockfill	TE ER	28 845	82.9
concrete dams (incl. masonry dams)	gravity	PG	3 953	11.3
	arch	VA	1 527	4.4
	buttress	CB	337	1.0
	multiple arch	MV	136	0.4
total large dams (to end 1982)			34 798	

Table Highest dams (after Mermel 1988).

Dam	Country	Type	Completed	Height (m)
Rogun	USSR	TE/ER	1989	335
Nurek	USSR	TE	1980	300
Grand Dixence	Switzerland	PG	1962	285
Inguri	USSR	VA	1980	272
Vaiont	Italy	VA	1961	262
50 dams greater than 200 m in height				

Table Largest-volume dams (after Mermel 1988).

Dam	Country	Type	Height (m)	Completed	Fill volume ($\times 10^6$ m^3)
Chapetón	Argentina	TE/PG	35	(1996)	296.2
Pati	Argentina	TE/PG	36	(1990)	238.2
Tarbela	Pakistan	TE/ER	143	1976	105.9
Fort Peck	USA	TE	76	1937	96.1
Lower Usuma	Nigeria	TE	49	(1990)	93.0
23 dams of volume greater than 50×10^6 m^3					

Table Dams with largest-capacity reservoirs (after Mermel 1988).

Dam	Country	Type	Height (m)	Completed	Reservoir capacity ($\times 10^9$ m^3)
Owen Falls	Uganda	PG	31	1954	2700.0[a]
Kariba	Zimbabwe/Zambia	VA	128	1959	180.6
Bratsk	USSR	TE/PG	125	1964	169.3
Aswan (High)	Egypt	TE/ER	111	1970	168.9
Akosombo	Ghana	TE/ER	134	1965	148.0
20 reservoirs with a storage capacity greater than 50×10^9 m^3					

[a]The major part is the natural capacity of the lake.

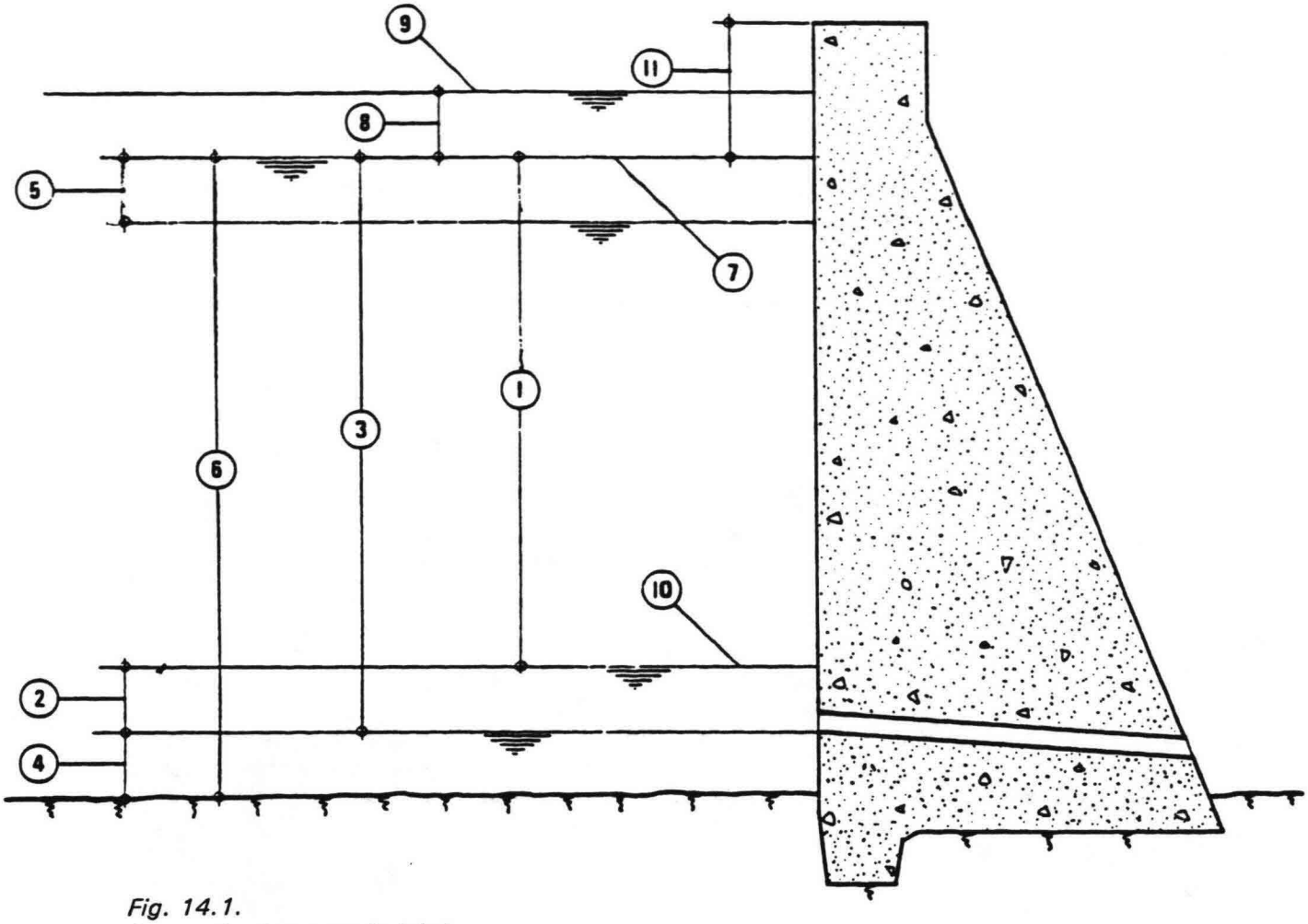


Fig. 14.1.
Reservoirs (man-made lake)

1. Active storage
2. Inactive storage
3. Live storage
4. Dead storage
5. Flood storage
6. Reservoir capacity; gross capacity of reservoir; gross storage; storage capacity
7. Retention water level; top water level; normal top water level; full supply level; normal water level (Am.)
8. Flood surcharge; surcharge
9. Maximum water level; top of joint use (Am)
   The term "joint use" (Am.) means that part of reservoir capacity including both surcharge and the empty part of active storage, assigned to flood control or conservation depending on the time of the year.
10. Minimum operating level; top of inactive storage (Am).
11. Freeboard

14.1. Dam body

The two main requirements of a dam are:
1. provide **stability** against external actions
2. provide **water tightness** of the structure

The stability of a gravity dam is achieved by the weights of the dam structure.
Sometimes the shape of the dam is arc-shaped: in that case a considerable part of the water pressure will be transferred to the flancs of the valley.

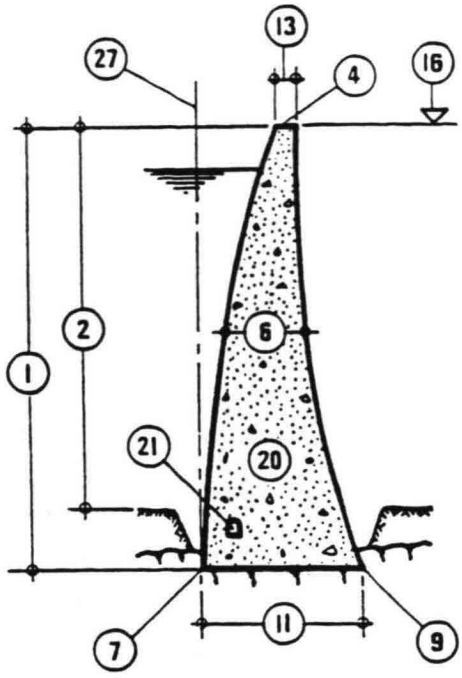
Water tightness: there is no dam anywhere in the world that is absolutely watertight. Always seepage; this has to stay within limits, then no danger.

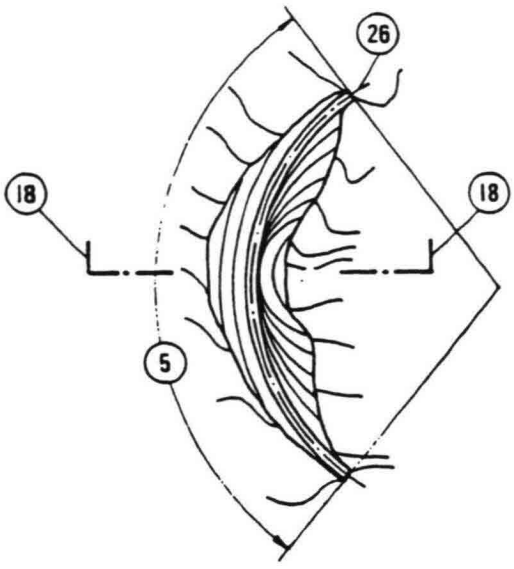
Three technical possibilities:
1. Material is watertight such as **concrete;**
2. **Earthdam** from sand, silt or clay - steady flow through the dam (=seepage water) make the slopes flat enough;
3. **Rockfill dams**: The slope can be remarkably steeper than an earth dam, as the shear strength of rockfill is much higher.
   An impervious element has to be added, otherwise there will be a very high and unacceptable seepage.
   Two possibilities
   a. surface blanket at the waterside;
   b. impervious core (asphalt, concrete, plates, plastic, etc).

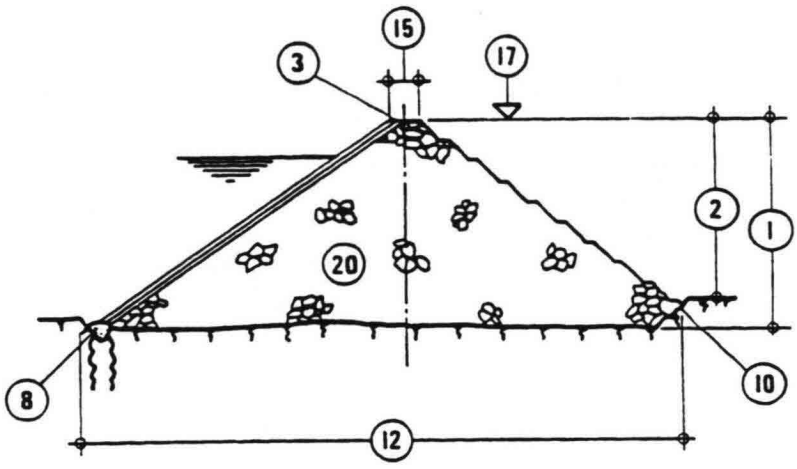
A **rockfill dam** is often combined with an earthdam. The low permeability of the fine grained material is combined with the high shear strength (=steep slopes) of the rockfill. This is called a **zoned dam**, that is a rockfill dam with an earthen core (inner part).

Concrete or masonry dam
High concrete dams must be founded on rock, while for fill type dams the foundation can be a compacted soil or rock.

Type of concrete dams:
1. Gravity dam (oldest type)
2. Gravity buttress dam
3. Buttress dam
4. Conventional arch dam (thick)
5. Thin arch dam (with vertical culviture)
6. Arch gravity dam (curved gravity dam)







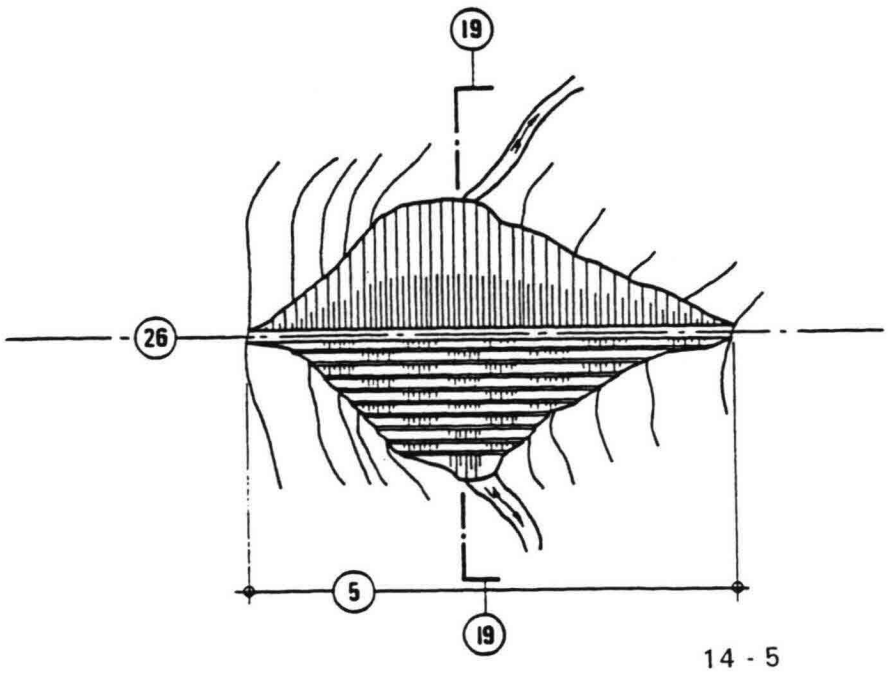


Fig. 14.2.
Dam terminology.

1. Height above lowest foundation of dam
2. Height above ground level
3. Top of dam
4. Crest of dam
5. Crest length
6. Thickness of dam (generally for gravity or arch dam)
7. Heel of dam (concrete dams)
8. Upstream toe of dam (others)
9. Toe of dam (concrete dams)
10. Downstream toe of dam (others)
11. Base thickness (generally for gravity or arch dam)
12. Base width (generally for other dams)
13. Top thickness (generally for gravity or arch dams)
15. Top width
16. Elevation of top of dam
17. -- do --
18. Cross section at crown (arch dam)
19. Maximum cross-section of dam
20. Body of dam; mass of dam (Am.)
21. Drainage gallery
26. Axis of dam
27. Setting out line

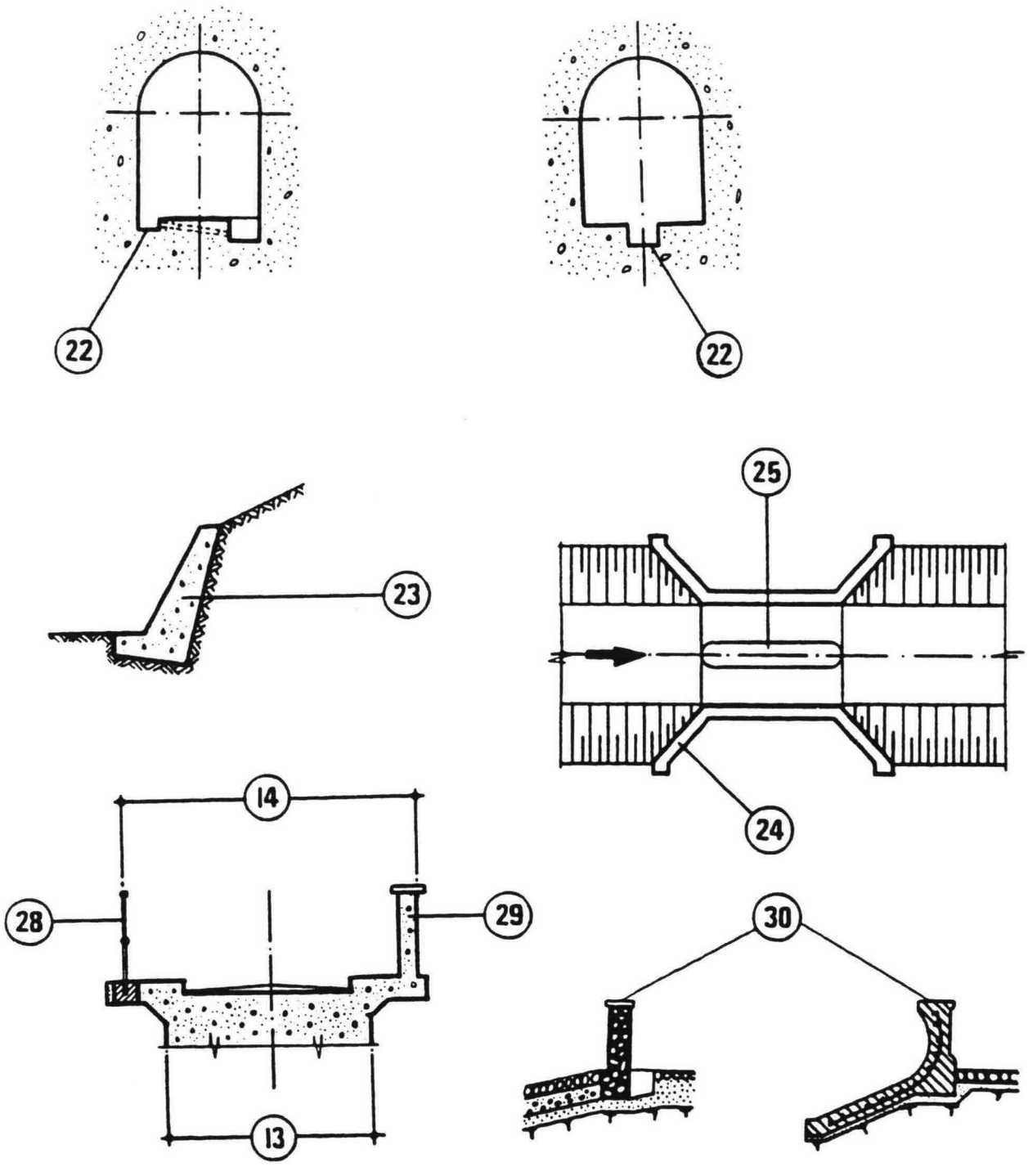


Fig. 14.3.
Dam terminology.

13. Top thickness (generally for gravity or arch dam)
14. Top width
22. Gutter
23. Retaining wall
24. Wing wall
25. Training wall
28. Guard rail
29. Parapet wall, parapet
30. Wave wall

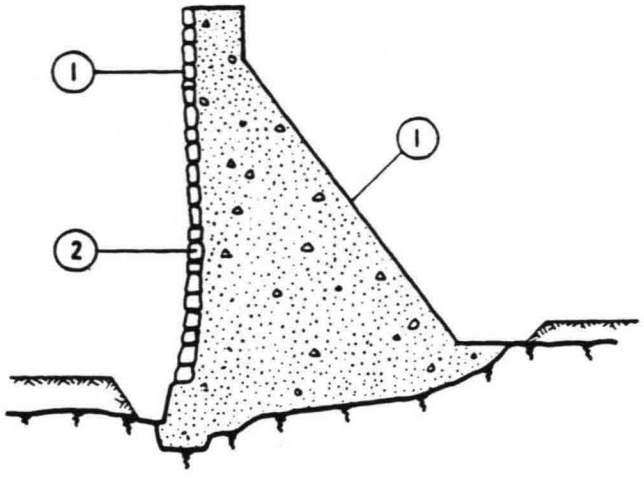
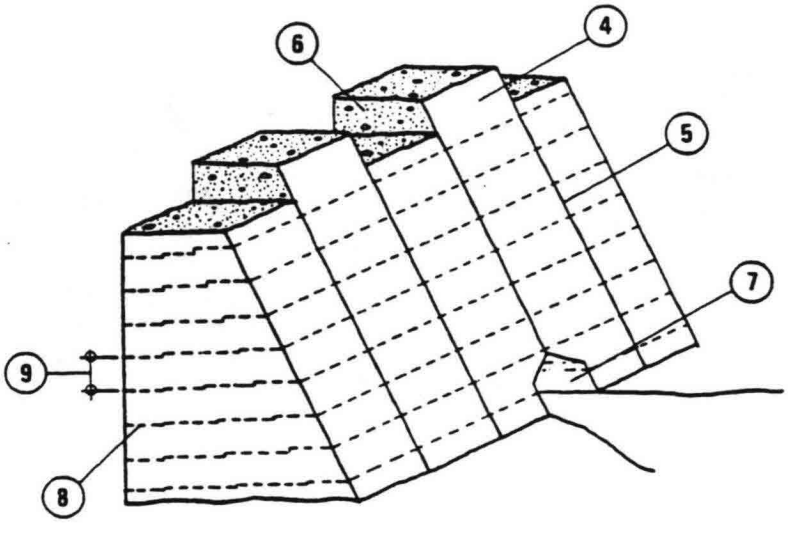
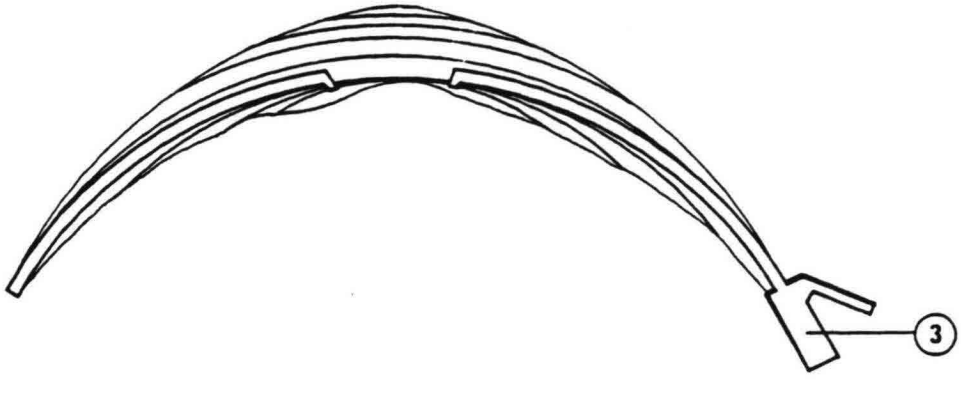
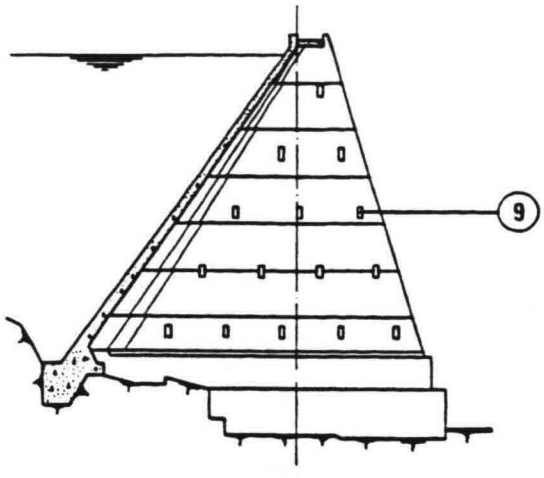


Fig. 14.4
Concrete dams.
1. Face
2. Facing
3. Artificial abutment; abutment block
4. Block
5. Joint; construction joint
6. Joint face
7. Temporary hole located at joint between blocks for passage of river flow (no English word)
8. Construction joint
9. Concrete lift; placement lift (Am).







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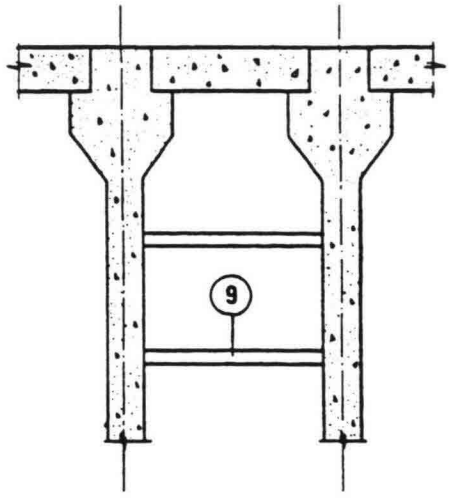

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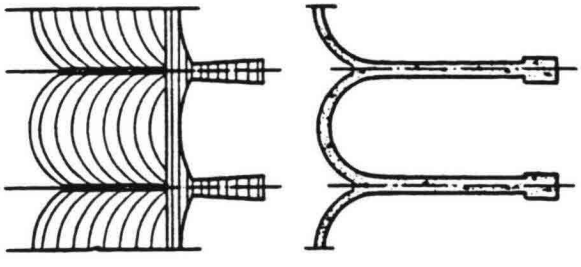

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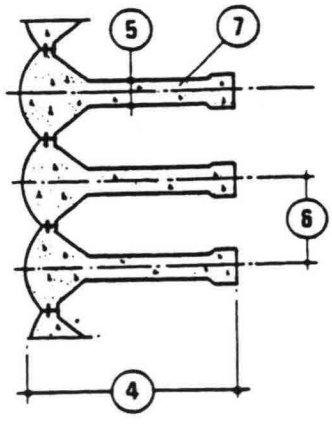

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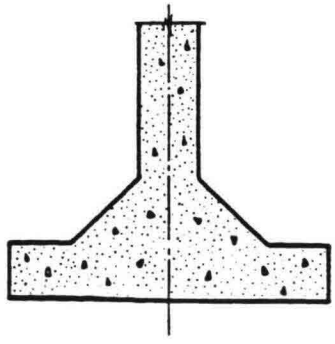


8

Fig. 14.5.
Buttress dam.

1. Flat slab dam; Ambursen dam; deck dam (Am.)
2. Multiple arch dam
3. Round head buttress dam
4. Length of buttress
5. Thickness of buttress
6. Buttress spacing; distance between buttress centres
7. Buttress web
8. Splayed footing
9. Buttress strut

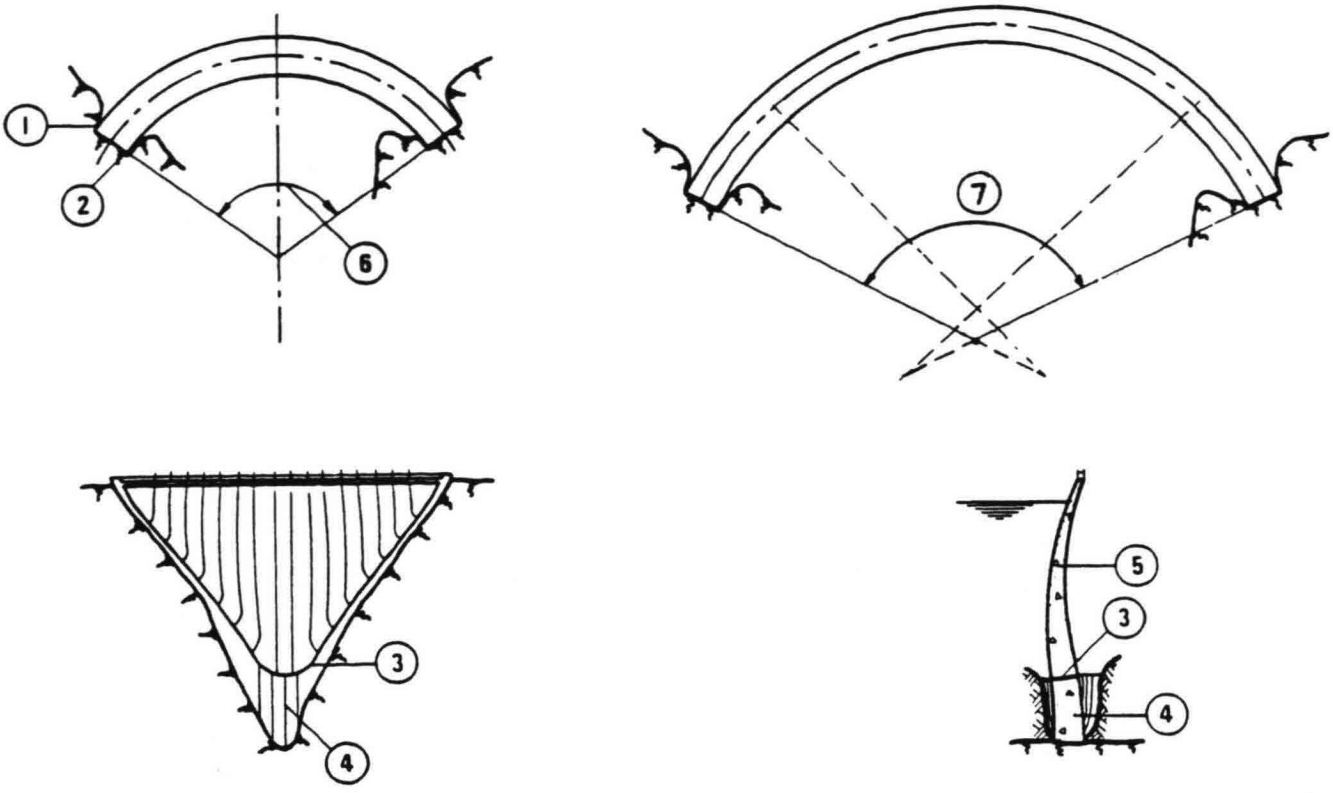


Fig. 14.6.
Arch dams.

1. Springing of extrados
2. Springing of intrados
3. Peripheral joint
4. Pulvino (an element between an arch and its support)
5. Cantilever; cantilever element
6. Central angle (angle subtended at the centre of a circular arc)
7. Subtended angle (angle of intersection of lines normal to tangent at the extremities of a non -circular arc.
8. Articulated arch; hinged arch.

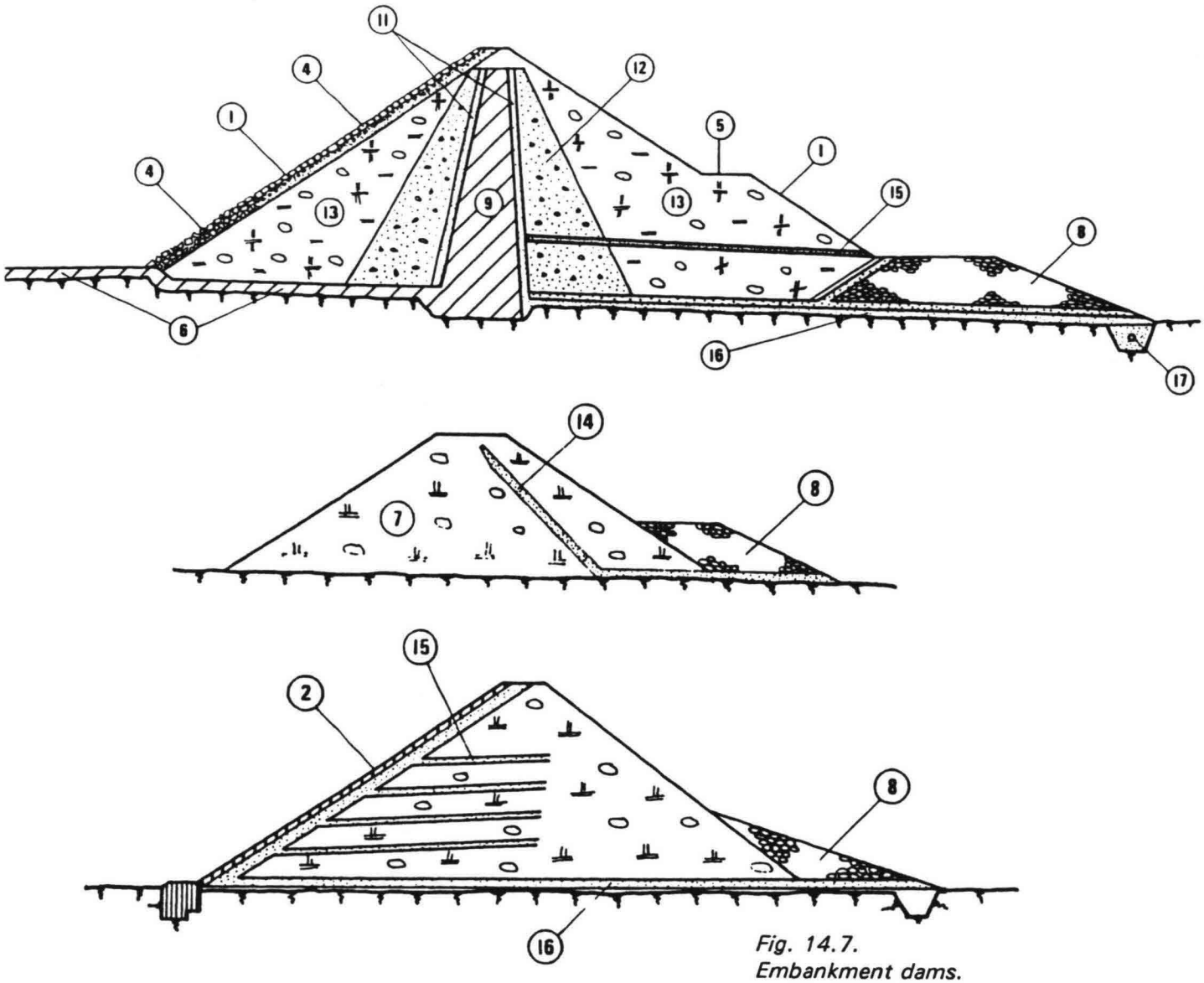
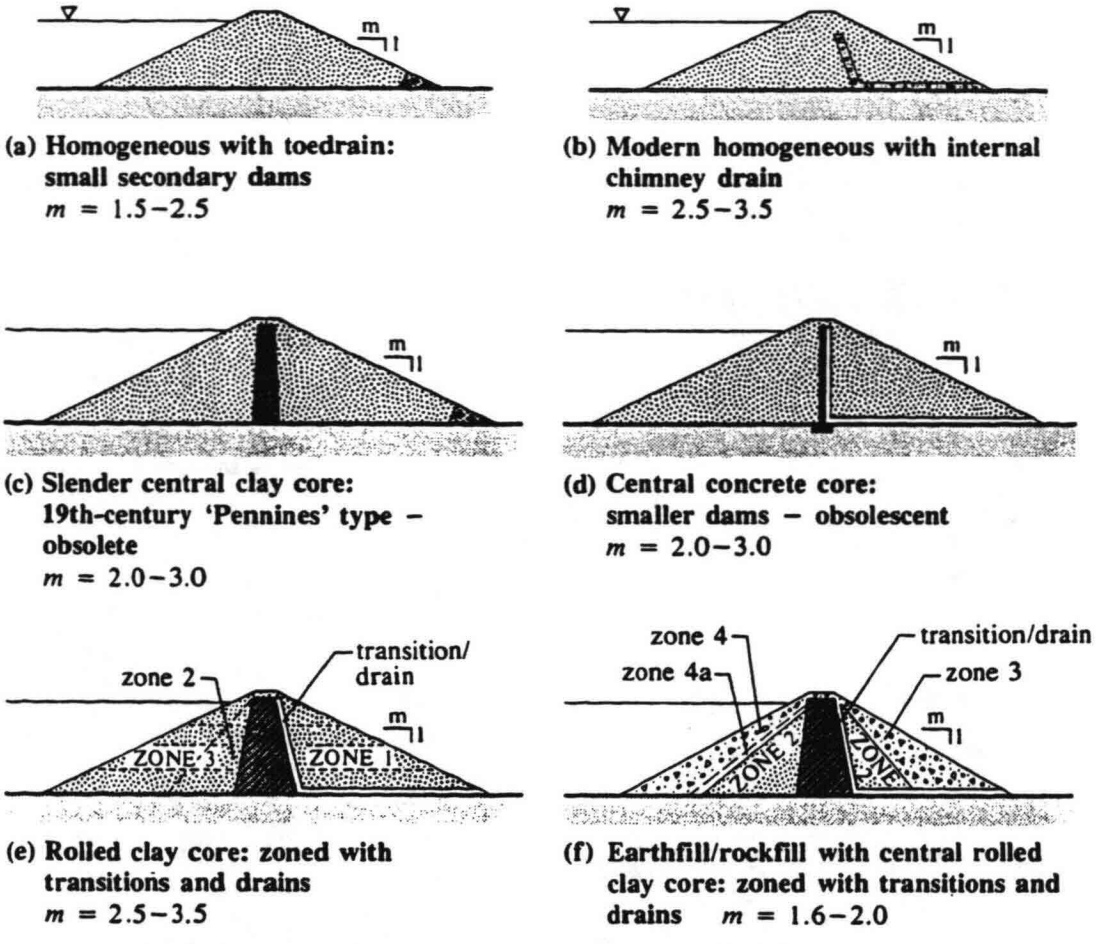


Fig. 14.7.
Embankment dams.

1. Slope
2. Facing, upstream membrane, upstream diaphragm
3. Diaphragm wall, diaphragm
4. Slope protection, revetment
5. Berm
6. Upstream blanket
7. Shoulder; shell (Am.)
8. Toe weight
9. Core; impervious core; impervious (inclined vertical)
10. Core wall
11. Filter, filter zone
12. Transition zone, semi-pervious zone
13. Pervious zone
14. Interception drain chim ney drain (Am.)
15. Drainage layer
16. Drainage blanket



(a) Homogeneous with toedrain: small secondary dams $m = 1.5-2.5$

(b) Modern homogeneous with internal chimney drain $m = 2.5-3.5$

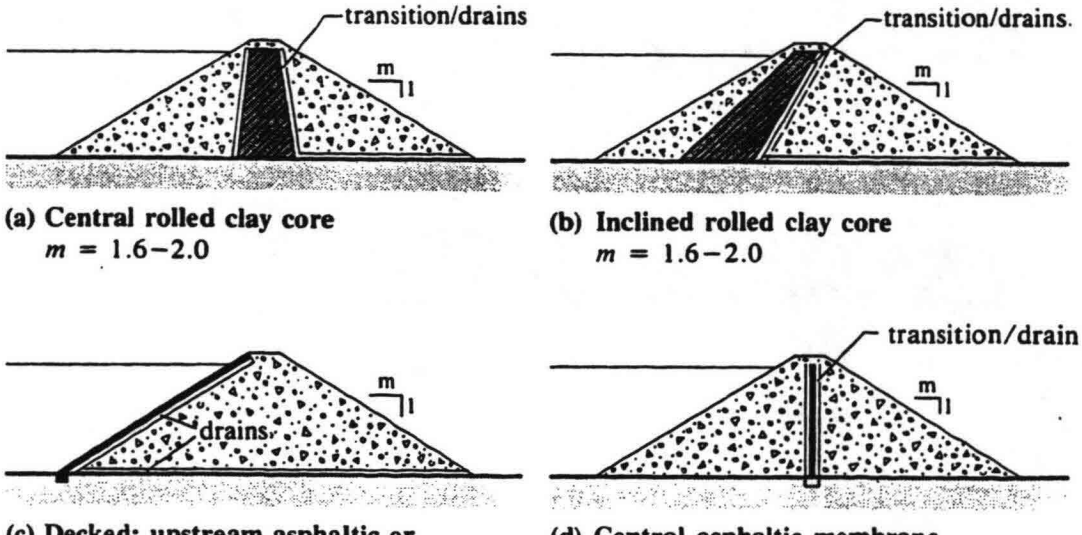
(c) Slender central clay core: 19th-century 'Pennines' type – obsolete $m = 2.0-3.0$

(d) Central concrete core: smaller dams – obsolescent $m = 2.0-3.0$

(e) Rolled clay core: zoned with transitions and drains $m = 2.5-3.5$

(f) Earthfill/rockfill with central rolled clay core: zoned with transitions and drains $m = 1.6-2.0$

Fig. 14.8. Principal variants of eartfill and eartfill/rockfill embankment dams.



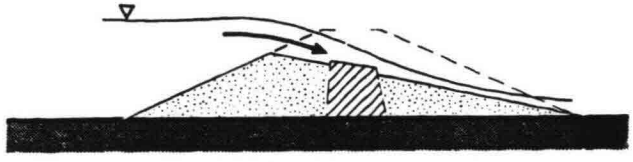
(a) Central rolled clay core $m = 1.6-2.0$

(b) Inclined rolled clay core $m = 1.6-2.0$

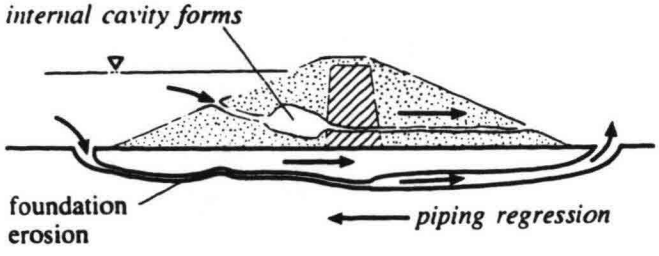
(c) Decked: upstream asphaltic or concrete membrane $m = 1.6-2.0$

(d) Central asphaltic membrane $m = 1.6-2.0$

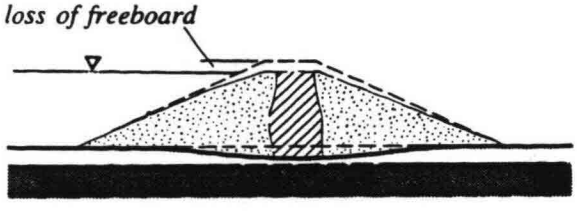
Fig. 14.9. Principal variants of rockfill embankment dams.



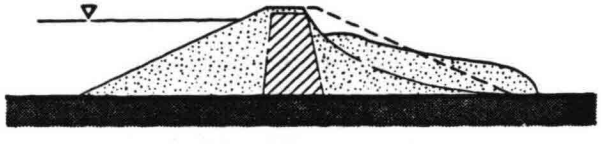
(a) Overtopping leading to washout; less cohesive silts, sands, etc. at greatest short-term risk



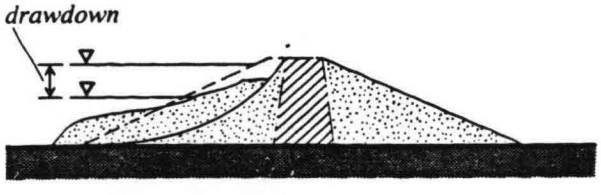
(b) Internal erosion and piping with migration of fines from core etc. (note regression of 'pipe' and formation of internal cavities; may initiate by formation of internal crack or by seepage along culvert perimeter etc.)



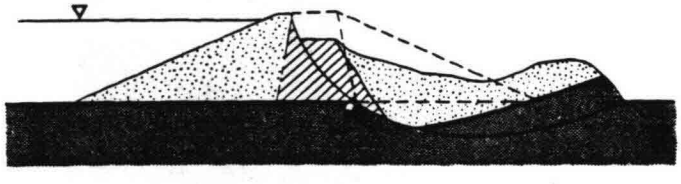
(c) Embankment and foundation settlement (deformation and internal cracking); note also cross-valley deformation modes:



(d) Instability (1): downstream slope too high and/or too steep in relation to shear strength of the shoulder material

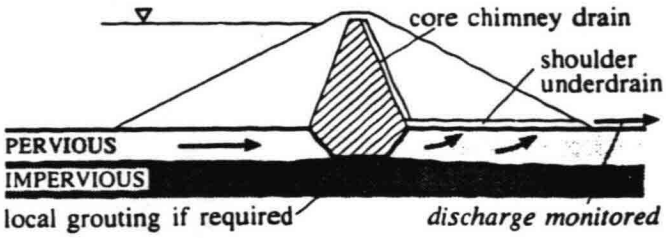


(e) Instability (2): upstream slope slips following rapid drawdown of water level

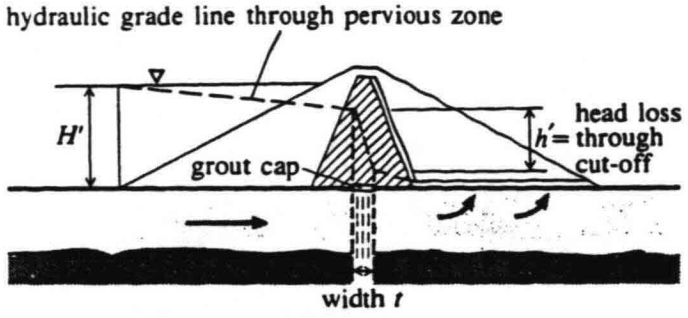


(f) Instability (3): failure of downstream foundation due to overstress of soft horizons

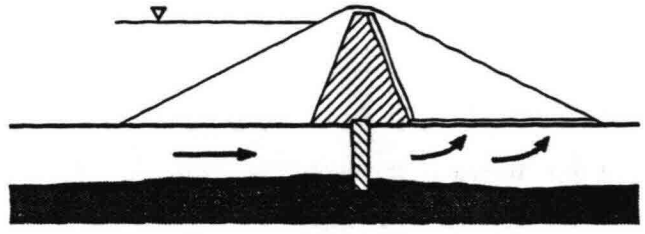
Fig. 14.10. Embankment defect mechanisms and failure modes.



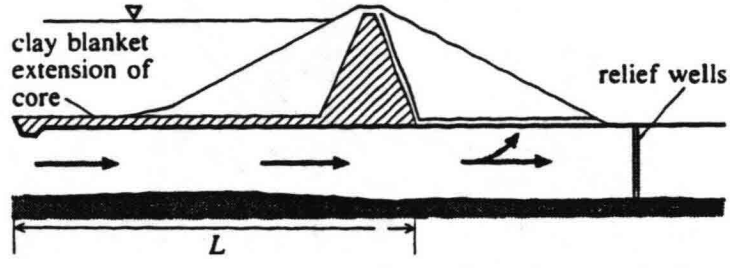
(a) Open trench cut-off



(b) Grouted cut-off (need not penetrate impervious horizons)

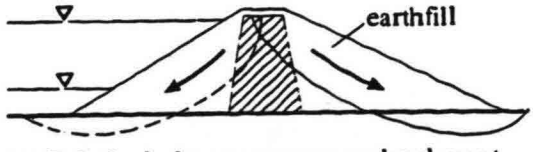


(c) Diaphragm cut-off (need not penetrate impervious horizons)

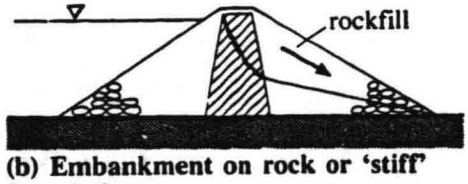


(d) Upstream blanket (may employ underdrain with relief wells)

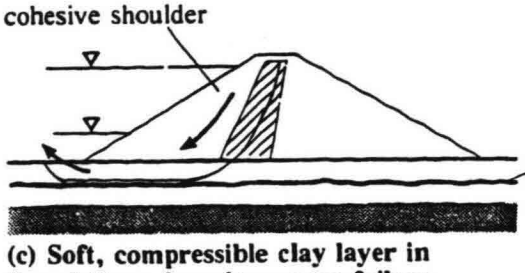
Cut-offs and control of underseepage.



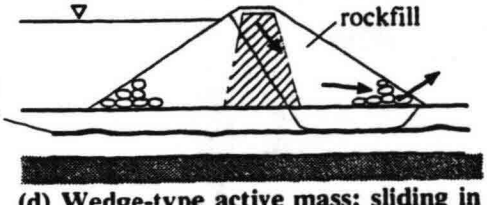
(a) Relatively homogeneous embankment and soil foundation



(b) Embankment on rock or 'stiff' foundation



(c) Soft, compressible clay layer in foundation; drawdown-type failure



(d) Wedge-type active mass; sliding in part on soft horizon

Fig. 14.11. Stability analysis; failure surface schematics.

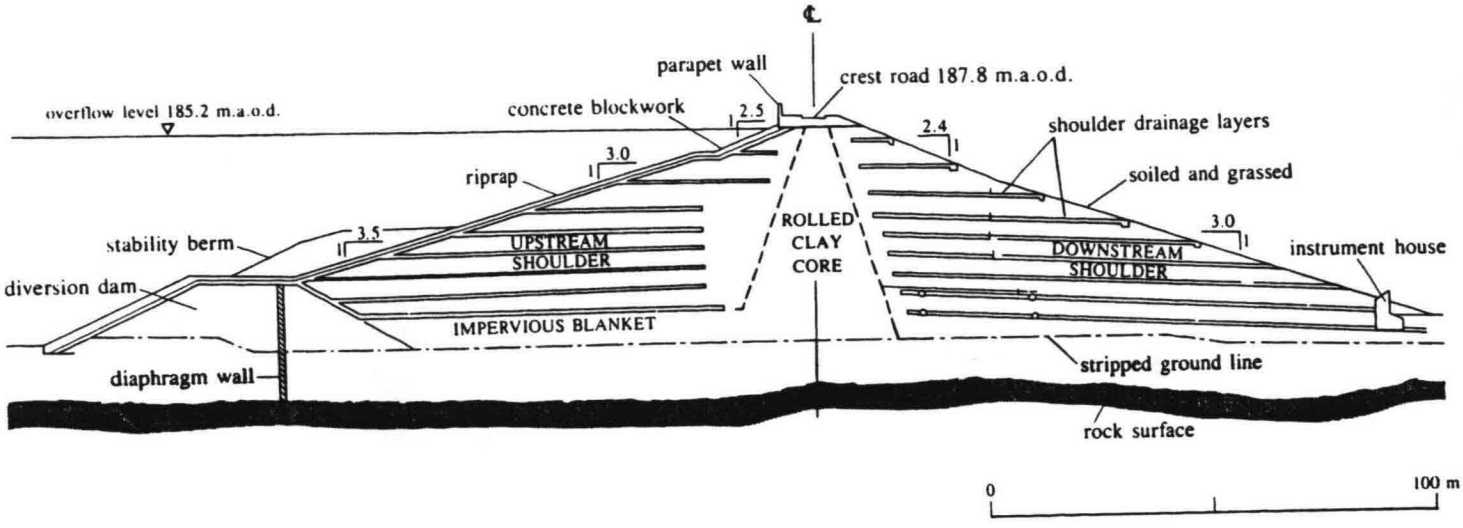


Fig. 14.13. Kielder earthfill embankment dam, England.

embankment : core and impervious blanket in selected glacial till; upstream and downstream shoulders in glacial till

foundations : up to 30 m glacial till overlying sandstones and mudstones

section at river: typical flank section to south of river has no diaphragm wall into drift and has underdrain.

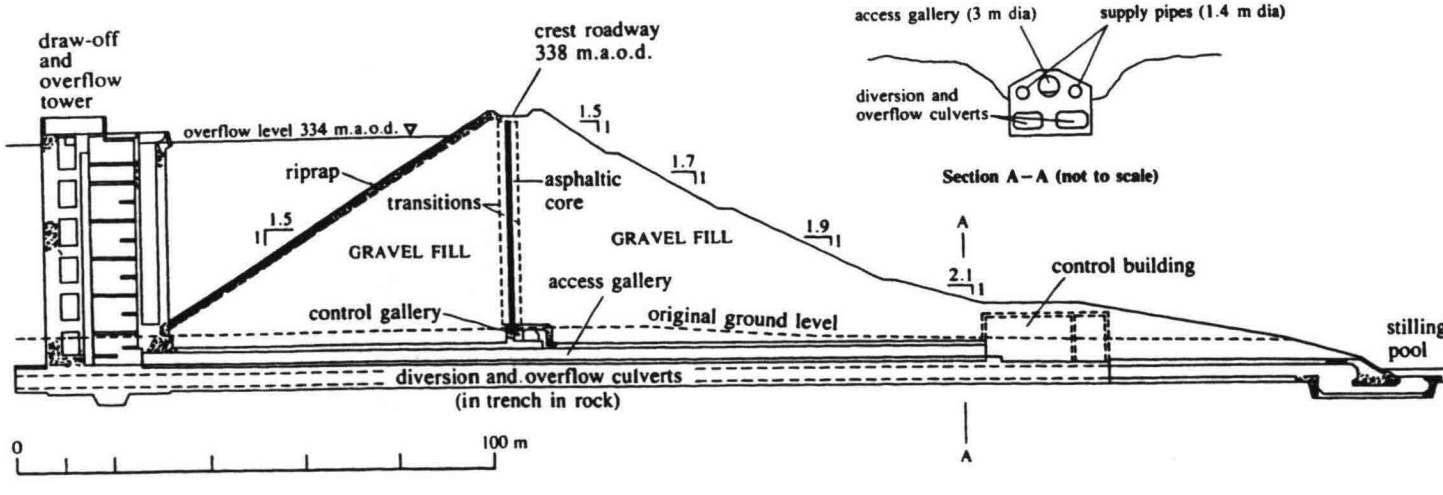


Fig. 14.14. Megget asphaltic core rockfill embankment dam, Scotland.

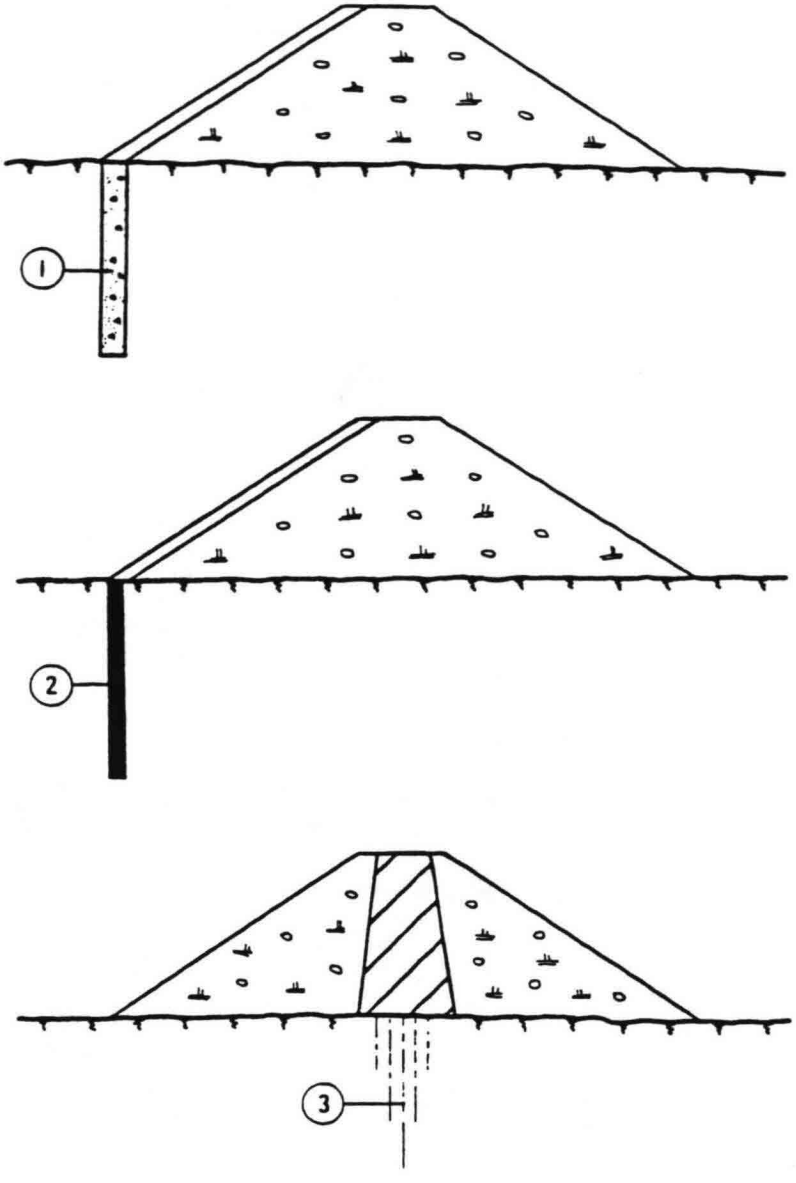


Fig. 14.15.
Cut-off
1. Cut-off wall
2. Diaphragm wall
3. Grout cut-off; grout curtain.

Grouting:
- grout; grout mix (Am.), cement grout, bitumen grout, chemical grout
- gel
- stable grout
- parent slurry
- pressure grouting (for water tightness)
- cavity grouting; backfill grouting
- consolidation grouting
- grout blanket

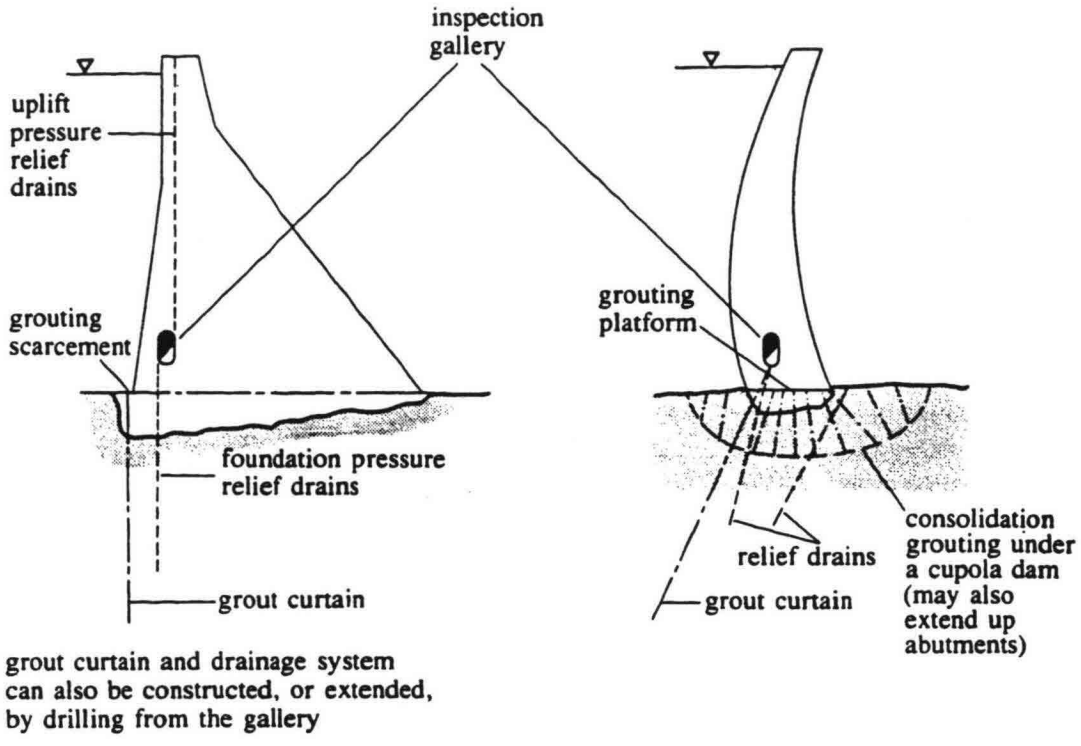


Fig. 14.16. Grouting and pressure relief drain system.

Roller Compacted Concrete Dams

Roller compacted concrete is no-slump concrete placed by earthfill methods. Because the concrete is brought to the dam site in trucks instead of buckets and because compaction is by rollers instead of by immersion vibrators the rate of placement is much faster and the cost of placement is much less than the cost of conventional concrete. The cost may be one-half and possibly one-third the cost of conventional concrete. Since the first roller compacted dams, **Shimajiga, Japan,** in 1980 and **Willow Creek, USA,** in 1982 were constructed, there has been a steady increase in the number of roller compacted concrete gravity dams. They are competitive in cost to embankment dams and arch dams and have the advantage over embankment dams that they can be overtopped with minimal, if any, damage during construction should the construction design flood be exceeded. Roller compacted concrete gravity dams generally require a rock foundation (like arch and other concrete dams), although some low height dams have been built on firm overburden. The principles of design developed for conventional concrete gravity dams can generally be applied to the design of roller compacted concrete dams. However special features are required to insure imperviousness and to control uplift.

The placement of concrete by earthfill methods involves three operations: **transportation** from mixer to site, **spreading** in lifts at site, and **compaction**. The first use of roller compacted concrete in a dam proper was in 1960-61 for the core of the 65 m. high cofferdam, which was incorporated in the 104 m. high **Shihmen Earthfill Dam, Taiwan**. The term "**rollcrete**" was coined at this time as an abbreviation of "roller compacted concrete". The rollcrete was batched in the same plant as used for the conventional concrete. Euclid trucks were used to haul the material from the batch plants to the cofferdam. D-8 bulldozers spread the material in 0.3 m lifts. The material was compacted by the trucks and by passes of the bulldozers. In the early 1970's several engineers proposed the use of roller compacted concrete for gravity dams. The successful use of soil cement and dry lean concrete for base courses of highways, and soil cement for slope protection on dams being the motive.

In 1974-75 352,000 cu. m. of roller compacted concrete was used to restore a portion of the right abutment of **Tarbela Dam, Pakistan,** which has been eroded away when the upstream portion of Tunnel 2 collapsed. From 1977 to 1980 920,000 cu. m. of rollcrete was used to line the sides of the Tarbela Service Spillway Plunge Pool. From 1980 to 1982 940,000 cu m of rollcrete was used to create a similar lining for the sides of the Auxiliary Spillway Plunge Pool at Tarbela Dam Project. Up to 1986 a total of 2,680,000 cu.m. of rollcrete was used for various features at this project.

In 1980 the Japanese Ministry of Construction completed the 89 m. high **Shimajigawa Dam,** the first dam to be constructed by the roller compacted dam concrete (RCD) method. The dam is located in a relatively narrow valley so that transport of concrete to the dam was by cableway utilizing a 13,500 kg. bucket. The concrete was deposited into a portable hopper set on the dam body placement surface. From there it was hauled by truck to the place of deposition, spread by bulldozer (layers of 0.5 and 0.7 m.) and then compacted by a vibratory roller. The upstream face of the dam is covered with a 3.0 m. thickness of conventional concrete. Total quantity: 170,000 cu. m. The water content was such that the vibratory rollers were able to bring the paste in the mix to the surface of a lift by the end of compaction.

Willow Creek Dam (52 m. high), completed in 1982 was the first roller compacted dam in the United States. It was the first roller compacted dam fully utilizing earthfill methods of construction. That is, scrapers for transport. Also the concrete was compacted at close to its optimum moisture content rather than so wet that paste could be brought to the surface of lifts by the vibratory rollers. Since these two dams, construction of roller compacted concrete gravity dams has accelerated and by 1987 some 19 (with a height of over 15 m) were completed. USA 8, South Africa 3, Australia 2, Japan 2, China, Brazil, France and Spain each 1.

Roller compacted concrete can be divided into three basic types:

1. The material is placed at close to its optimum moisture content for compaction; ideally the aggregate should be fairly well graded, the material resembling a mechanically stable type of base course. This type is called "**rollcrete**". Compaction of such materials, does not require a vibratory roller; 50 or 100 ton rubber tired rollers may be more effective. The material is too sticky to be vibrated into a dense state. Also called rolled dry lean concrete (**RDLC**).
2. **RCC** - vibration is essential for its densification and in that every effort is made for the material to duplicate conventional concrete (Fuller "**C**" shape gradation). The intent is to have paste fill the voids of the aggregate as completely as practical. A wetter mix than for rollcrete thus results. (layers of 0.3 m. - USA).
3. **RCD** - (Rolled-concrete dam) Japanese version of RCC; vibration is also essential; spreading is in about 0.30 m. layers, but compaction is done only after spreading two or three layers. Also the surface is cured with excess water and green cutting is carried out.

Two distinct approaches to the design of roller compacted concrete dams have developed: the traditional or concrete technology approach (type 2 and 3) and the more radical 'geotechnical' approach (type 1).

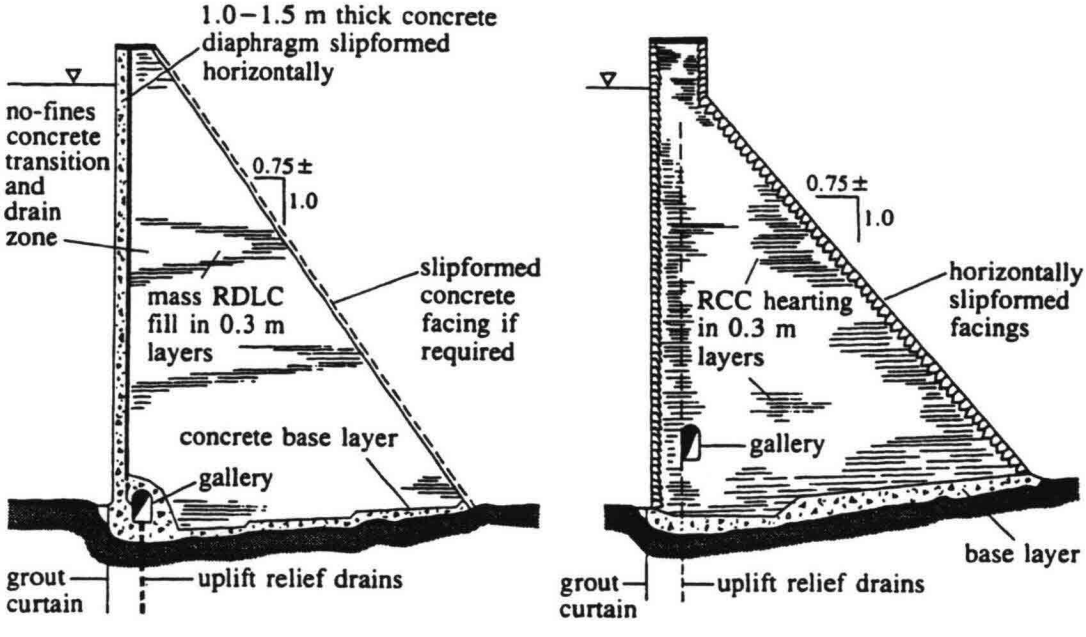


Fig. 14.17. Illustrative roller-compacted concrete gravity dam profiles.
a. Rolled dry lean concrete (type 1)
b. High-paste roller compacted concrete (type 2).

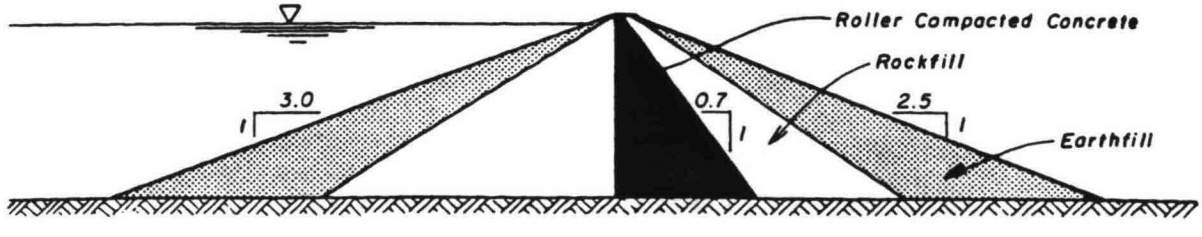


Fig. 14.18. Comparison of cross sections for various types of dam.

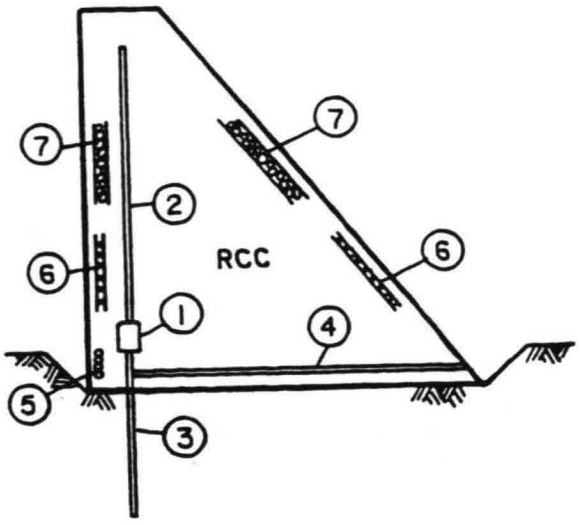


Fig. 14.19.
Method of collecting seepage.
1. drainage gallery
2. internal drain holes
3. foundation drain holes
4. foundation drain manifold
5. drain/grout tubes
6. wick drains
7. sand/porous concrete chimney drain

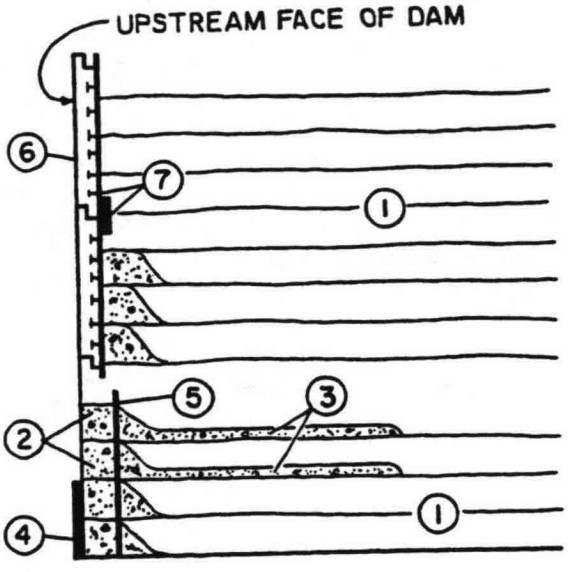


Fig. 14.20.
Method of reducing seepage.
1. RCC
2. cast-in-place concrete
3. bedding concrete
4. elastomeric liner
5. water stop
6. precast panel
7. PVC liner.

Hydraulic Fill Type Dams

A true hydraulic fill dam is built by conveying earth materials from the borrow to the embankments as a liquid mixture, and placing them in the embankment using water. This entails continuous pounding. An alternative to the true hydraulic fill is the **semi-hydraulic fill**, which has been adopted at some sites where water transportation is not favourable for material transport over longer distances. In this type of construction, transport from the borrow area is accomplished by other means.

A hydraulic fill dam is composed of silty clay in the core and usually of sands and gravels in the shell. The core material is placed as a muddy mixture which settles out after placement, but if it is excessively fine -- colloidal material, it remains in suspension for a long time. By contrast, sand and gravel shells tend to stabilize readily and gain strength quickly. In the core, very high pore pressures would be observed over a long period. Thus, hydraulic fill dams are considered, typically, to have a lower factor of safety during the construction period.One of the common rules in construction of hydraulic fills has been that the permeability of the shell should by much higher than the permeability of the core.

Good results have been obtained in the Soviet Union, China and the USA, as well as West Europe, mainly Holland, where several million m^3 earth were moved for dyke construction round polders, the bodies of which were filled up with sandy filler.
Considering Central Europe, much experience has been gained in FRG and Poland, while in Austria, Hungary and Czechoslovakia there is little experience in the technology mentioned above. However, experts from the USSR have repeatedly recommended this technology. In these recommendations, the following advantages can be seen.

1. Large volumes in performance and a great efficiency of machines and transport equipment. Time efficiency extends from 70 to 98% of the working season. The worky period, whereas fill in trucks has to stop for several days or even several weeks in autumn - if clay material is used.

2. Hydraulic fill in canal construction requires 0.1 - 0.2 man-hours per 1 m^3 of material laid out in the embankment; this is nearly half the traditional rate in dam construction.

3. Energy consumption is 1.5 - 4 kWh for sands and for sandy gravel 5 - 6 kWh, per 1 m^3 of fill. Transport by pumps reduces this still further - in the USSR 3.6 - 4.8 kWh per 1 m^3 fill.

4. In favourable geological and geographical conditions - a slow slope from the borrow site to the dam - the monetary outlay for hydraulic fill may be a half to a third of the usual fills.

On the other hand, many disadvantages have been cited for hydraulic and semi-hydraulic fills.

(1) Certain deficiencies in performance, serious sliding during construction (Calaveras dam) and after construction (Fort Peck dam).
(2) Many uncertainties in the sealing effect caused by intrusions of sand lenses into the core.
(3) A high content of fine colloidal materials would severely inhibit consolidation - high pore pressure leads to soil instability.
(4) Typical damage consists of cracks extending longitudinally along the embankment of canals.
(5) Many difficulties arise in changing the placement; the pipeline must be transported, resulting in a great deal of cost and trouble.
(6) Environmental deficiencies.
(7) A pronounced effect from seismic activity in a hydraulic fill - manifested by cracking or settlements. All these effects can be intensified by liquefaction.

Susceptibility to this is highest in saturated low-density soils with uniform gradation of silt to fine sand. Liquefaction is a potential problem in any embankment, such as a hydraulic fill, which may have continuous layers of such materials. Especially vulnerable are poorly compacted saturated layers near the top of the dam, where amplitudes of seismic vibration are often largest. This danger was very thoroughly investigated by Napetvaridze (1956) in the USSR and Seed et al. (1973).

It can be said that hydraulic fill technology is a hundred years old, having enjoyed its greatest popularity in the USA in the period of the 1930s. In the USSR, its peak was in the 1950s, and in China its popularity remains to this day. In the USA, a massive slide (more than 9 million m^3 fill) on the Fort Peck dam in 1938 ensured that the hydraulic fill concept came under a cloud of suspicion. A few years later, heavy transport machines and perfect compaction equipment brought the rolled embankment to the fore as a competitive alternative to hydraulic fill. In the Soviet Union, the same process can be observed in the decade from 1950 to 1960.

The popularity of this concept with Chinese designers and contractors can be explained by the lack of heavy machinery and mass-man power.

In the developed countries, we observe from time to time a certain interest from engineers responsible for dam design and construction, towards hydraulic fill under favourable conditions (in Holland and FRG) where appropriate soils (sands) and sufficient water make this technology favourable, and disadvantages such as earthquakes and liquefaction are rare.

To all engineers interested in hydraulic fill construction and its problems, an excellent review of the subjects from Hansen et al. (1976) can be recommended.

14. 2. Spillway

The function of a spillway is to release floodwater that cannot be contained in the storage (reservoir). The excess water must be discharged safely from the reservoir. In many cases, to allow the water simply to overtop the dam would result in a catastrophic failure of the structure. For this reason, carefully designed overflow passages - known as **'spillways'** - are incorporated as part of the dam design. The spillway capacity must be sufficient to accommodate the **'largest'** flood discharge likely to occur in the life of the dam.

Types :

- **Free overflow** or **ogee** (overflow) spillway
- The **overfall** and **'ogee'** spillways are by far the most widely adopted: they may be used on masonry or concrete dams, which have sufficient crest length to obtain the required discharge
- **Conduit spillway** through the dam
- **Side channel** or **tunnel spillway** are useful for dams sited in narrow gorges
- **Drop inlet (shaft) spillway** (morning glory) is often used on earthfill dams.
  A morning glory spillway consist of four parts :
  - a circular weir at the entry
  - a flared transition which conforms to the shape of the shaft
  - the vertical drop shaft
  - the horizontal (or gently sloping) outlet shaft.
- **Controlled weir**
- **Siphon spillway.**

There are many different solutions for the spillway, a concrete dam may have an overflow, but an overflow is not possible for a fill-type dam. In that case a separate structure, such as a **"morning glory'** is necessary.

A spillway consist of three parts :

1. **control structure**
2. **waterway** : a discharge structure that guides the spilling water. Lateral walls are needed.
3. terminal structure, a dissipation structure such as a flip bucket.

Energy dissipators

The flow discharged from the spillway outlet is usually highly supercritical. If this flow were left uncontrolled severe erosion at the toe of the dam would occur. Therefore, it is necessary to dissipate much of the energy. This is achieved by a dissipating or 'stilling' device :

a. stilling basin
b. submerged bucket
c. ski jump/ deflector bucket.

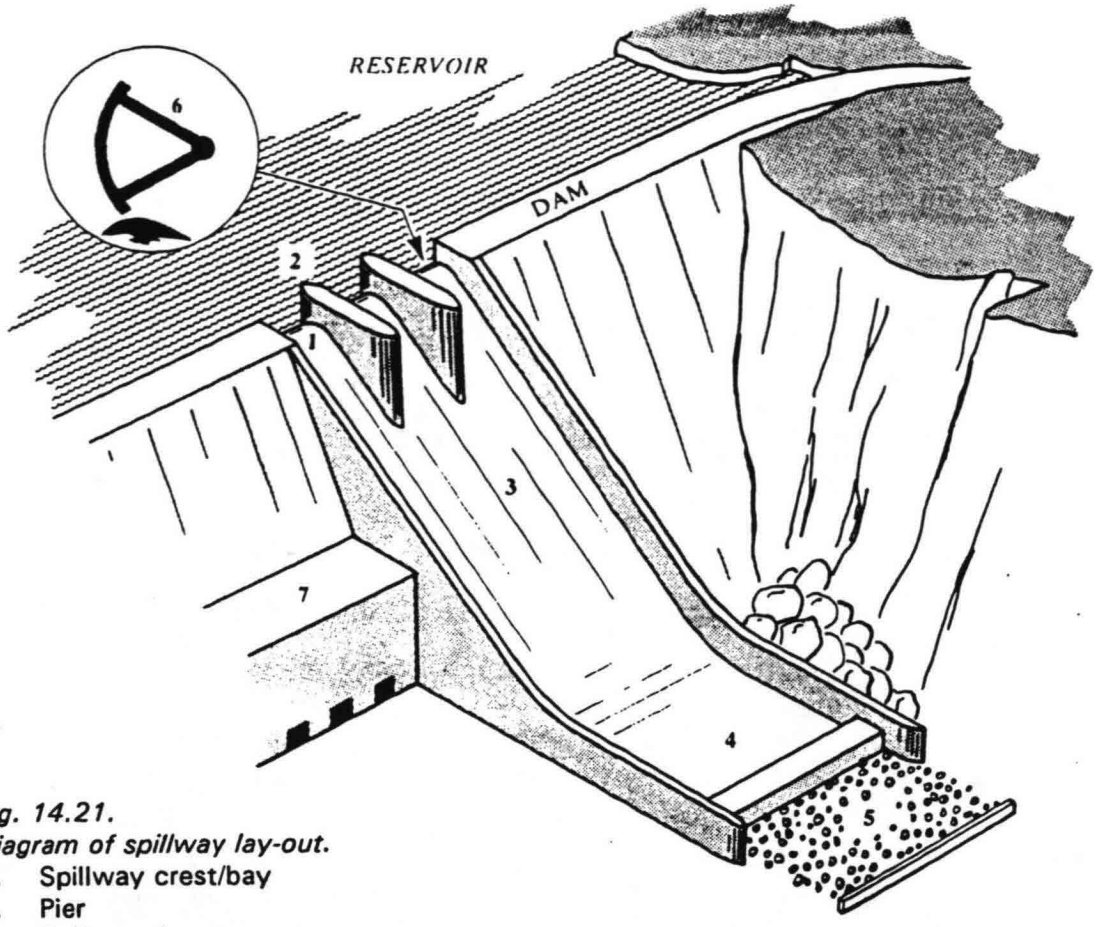


Fig. 14.21.
Diagram of spillway lay-out.

1. Spillway crest/bay
2. Pier
3. Spillway face
4. Stilling basin
5. Armoured scour prevention bed
6. Section through control gate
7. Power station.

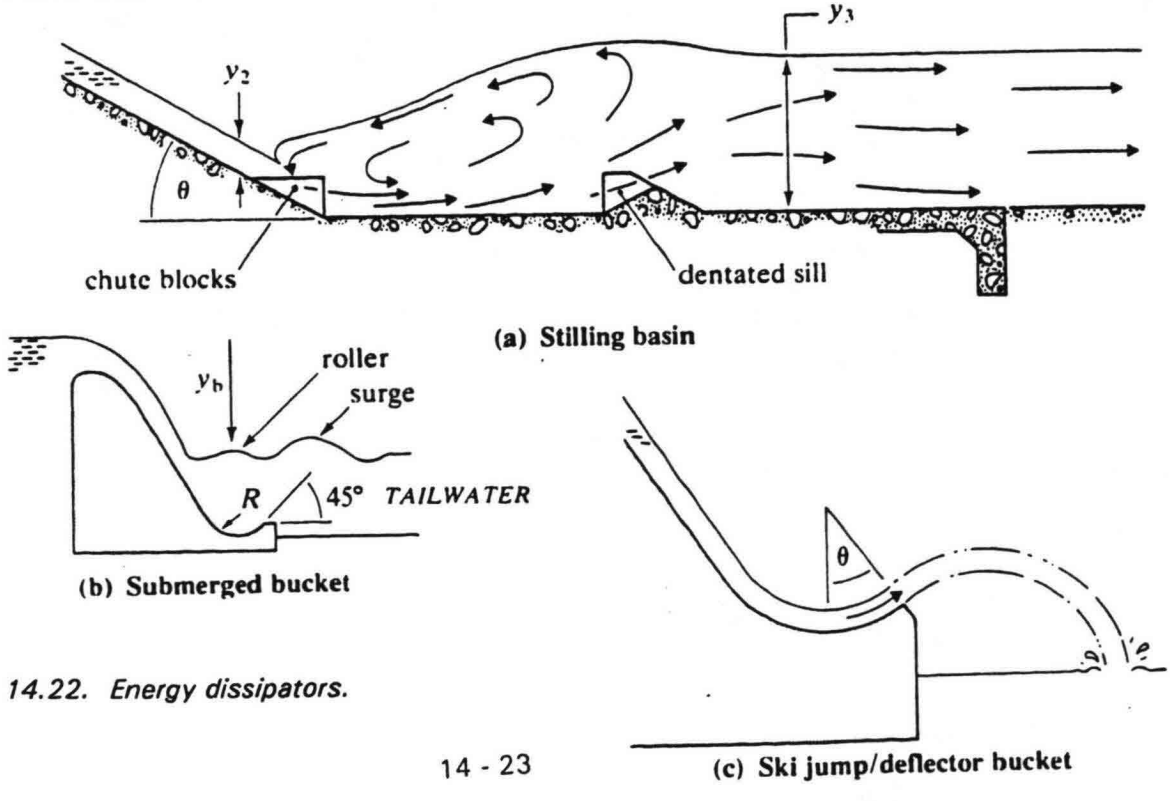


Fig. 14.22. Energy dissipators.

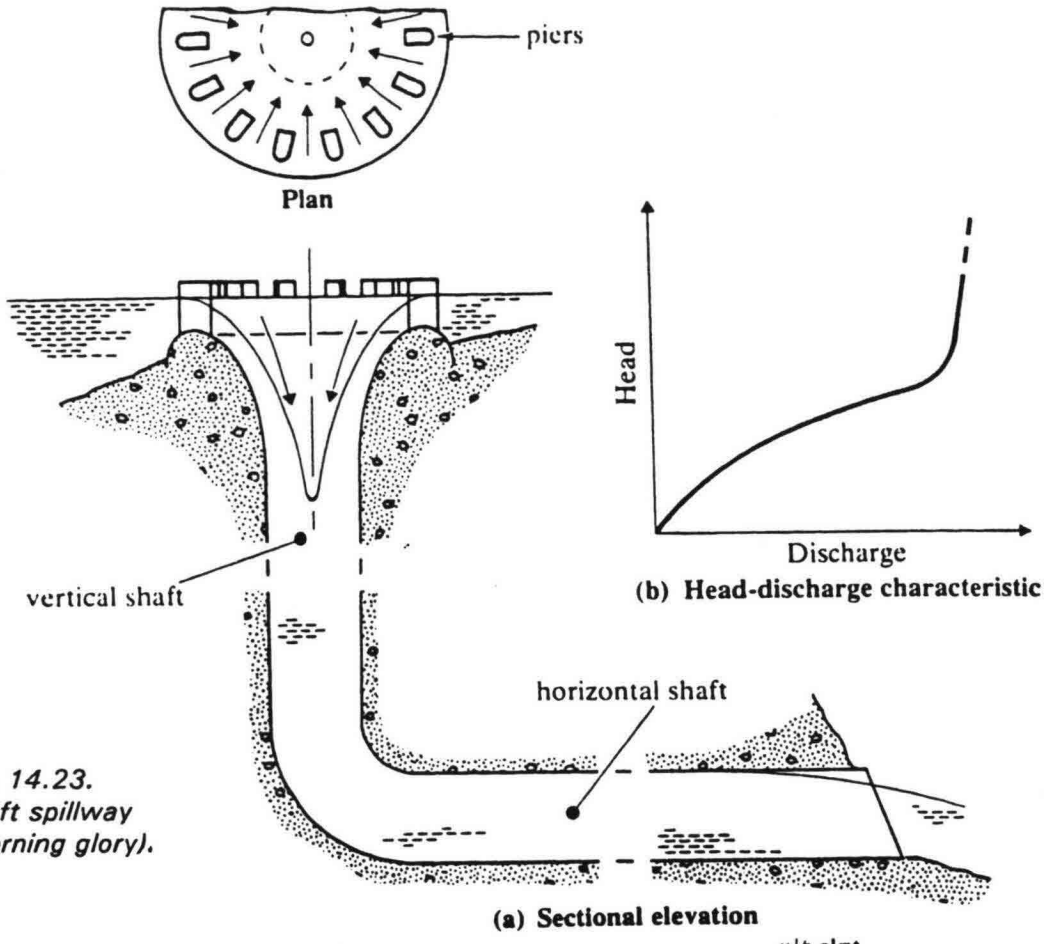


Fig. 14.23. Shaft spillway (Morning glory).

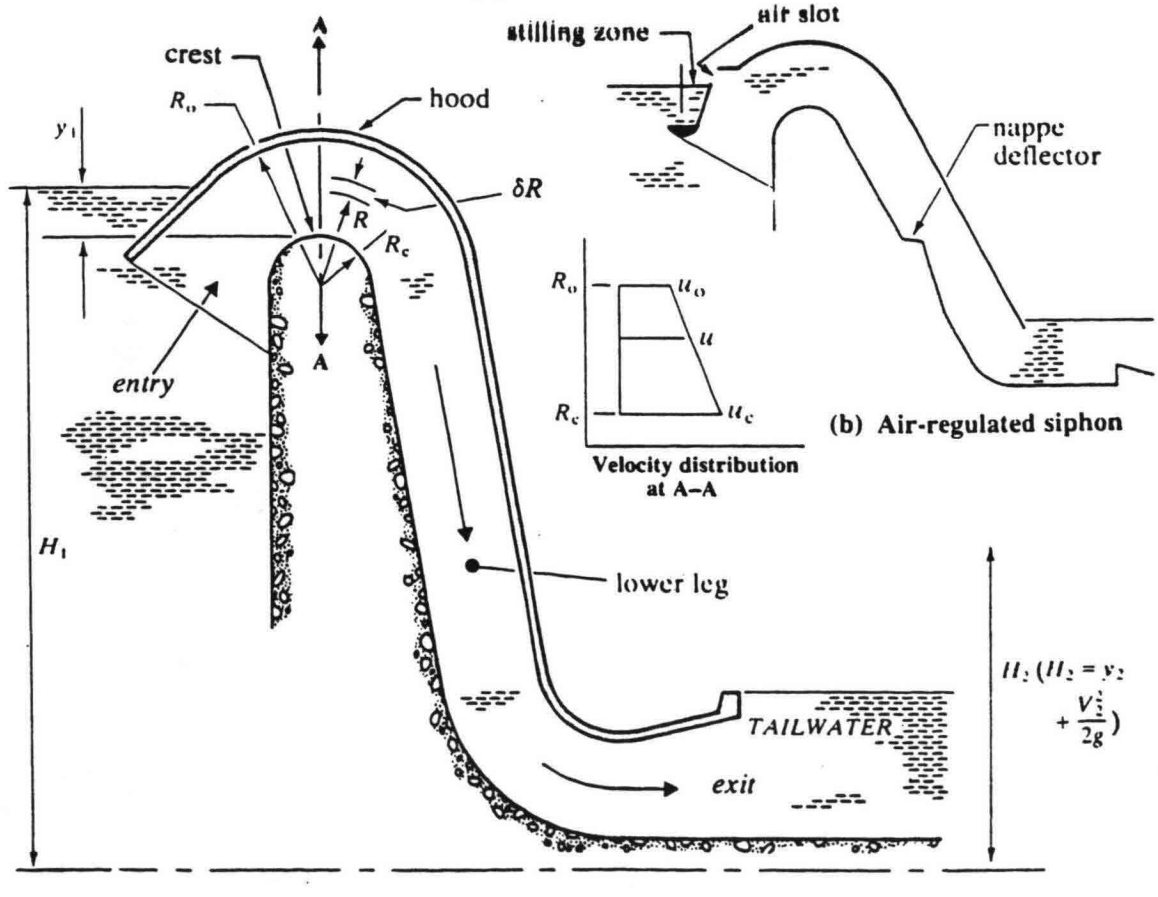


Fig. 14.24. Siphon spillway.

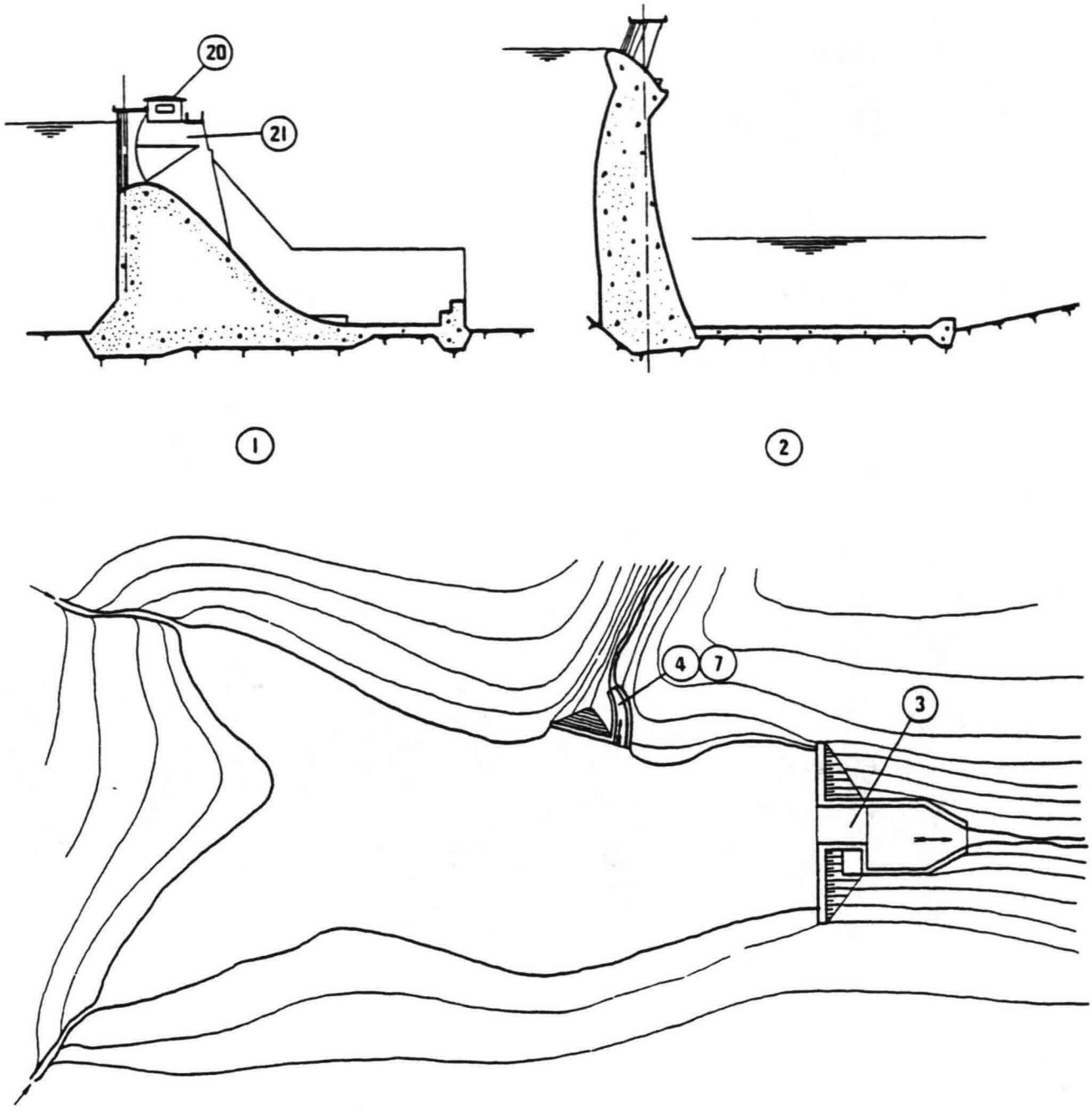
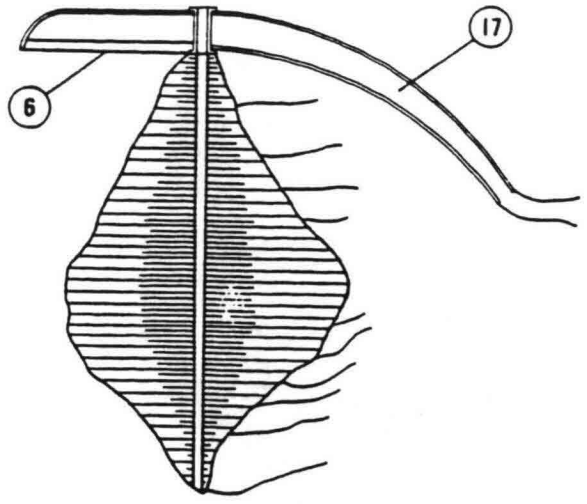
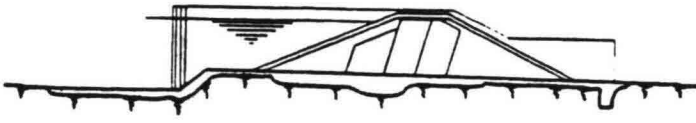


Fig. 14.25. Spillways.

1. Controlled spillway, gated spillway
2. Uncontrolled spillway
3. Main spillway
4. Auxiliary spillway
7. Saddle spillway
20. Gate control house
21. Spillway pier





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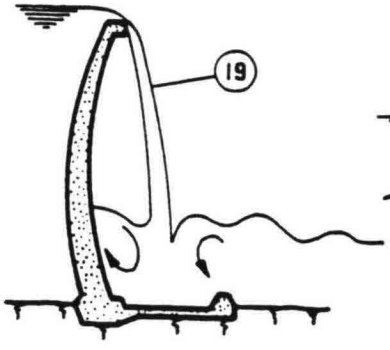

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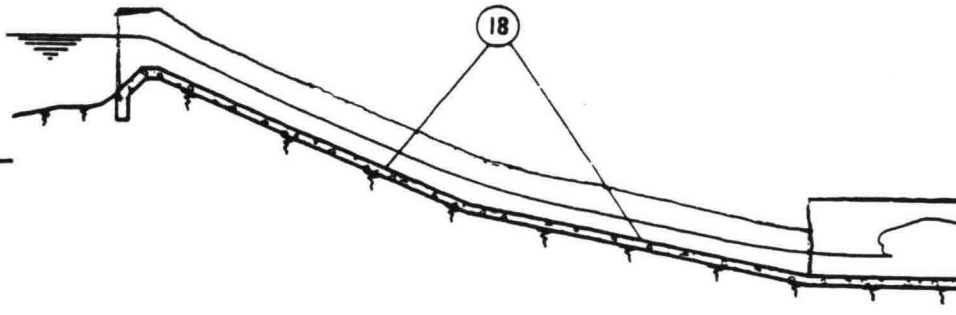

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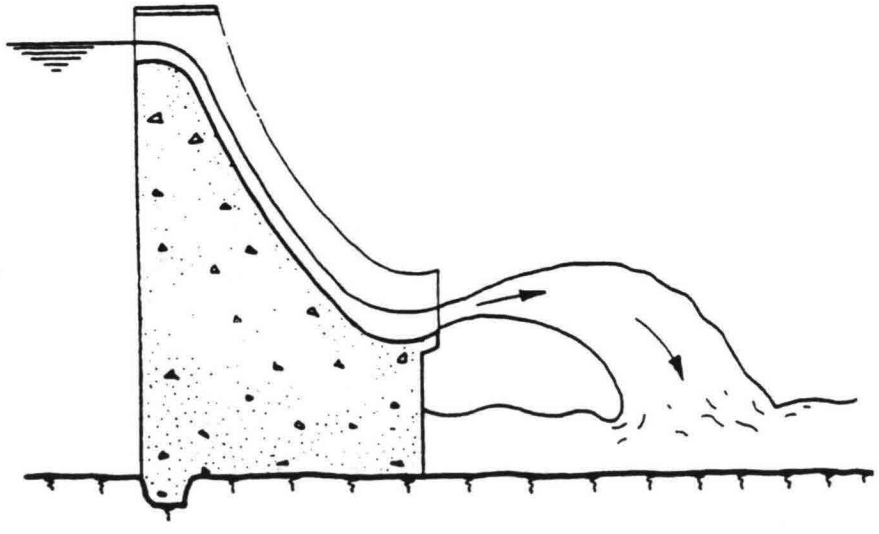

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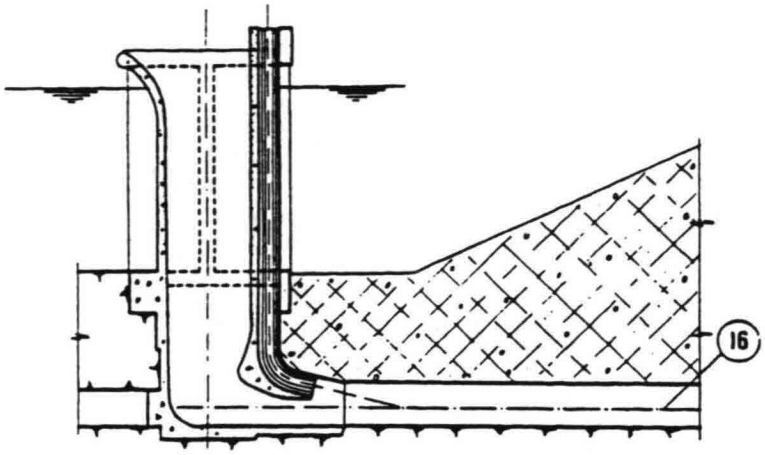
Fig. 14.26.
Spillways.
5. Fuse plug spillway
6. Side spillway
8. Overfall spillway
10. Skyjump spillway
17. Spillway channel
18. Spillway chute
19. Nappe



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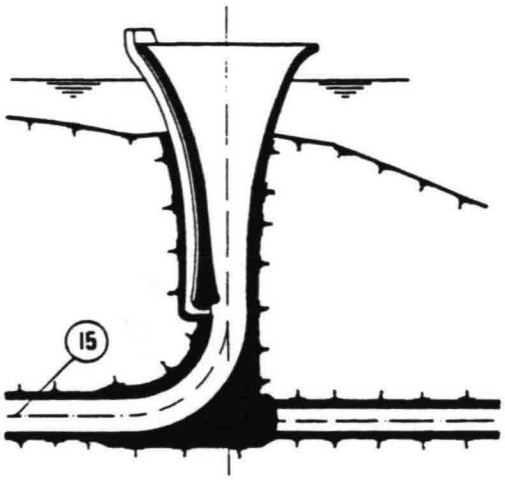

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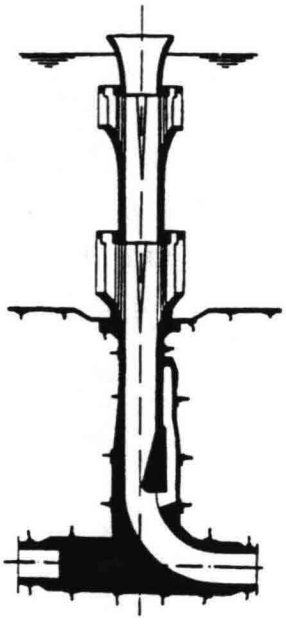

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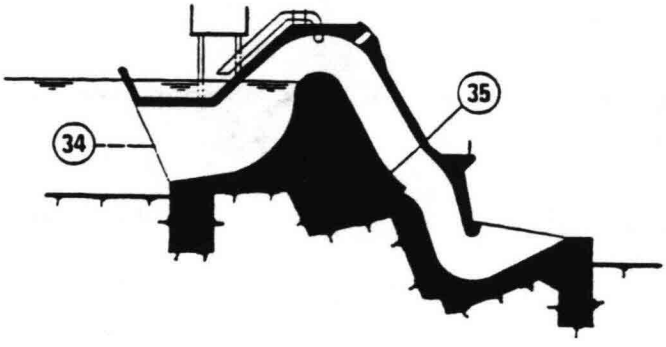

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Fig. 14.27.
Spillways.

11. Shaft spillway
12. Belmouth spillway, morning glory spillway
13. Multi-level outlet shaft spillway
14. Siphon spillway
15. Spillway tunnel
16. Spillway culvert
34. Siphon mouth
35. Deflector (for priming)

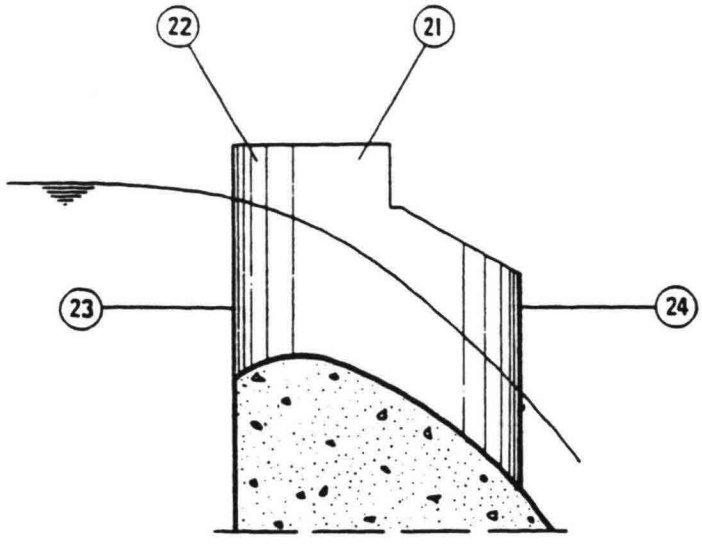
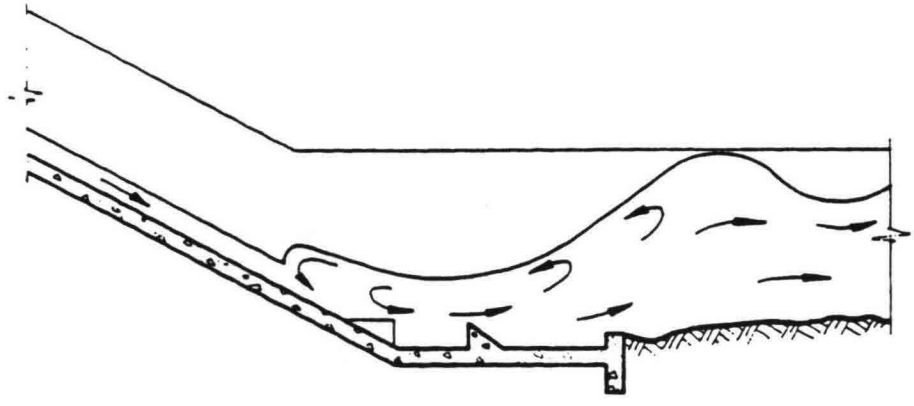


Fig. 14.28.
Spillways.
21. Spillway pier
22. Pier nose
23. Cutwater; upstream fairing (of pier)
24. Downstream fairing (of pier)
25. Stilling basin; stilling pool
26. Plunge basin; plunge pool
27. Hydraulic jump basin.



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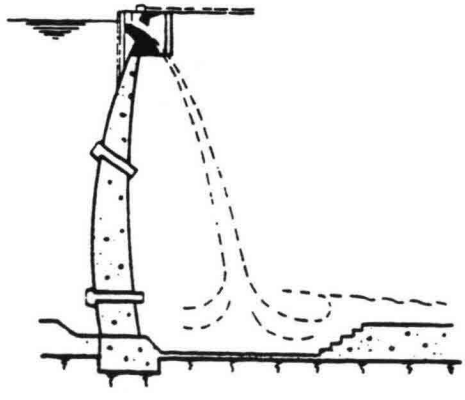

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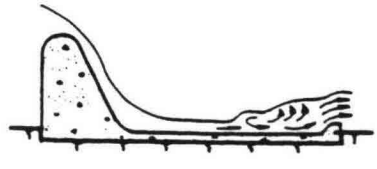

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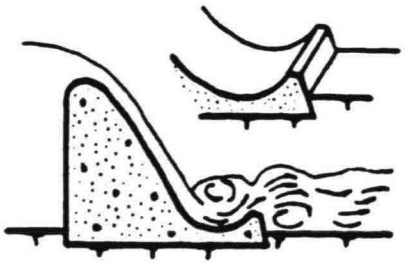

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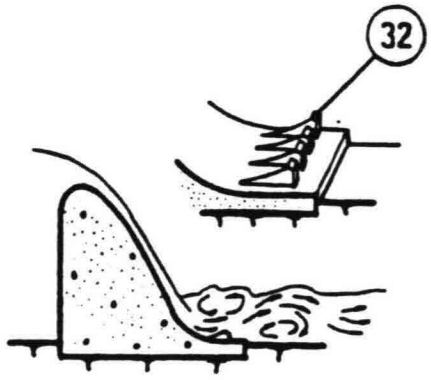

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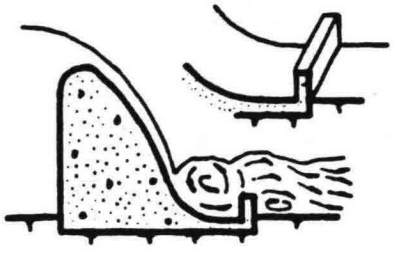

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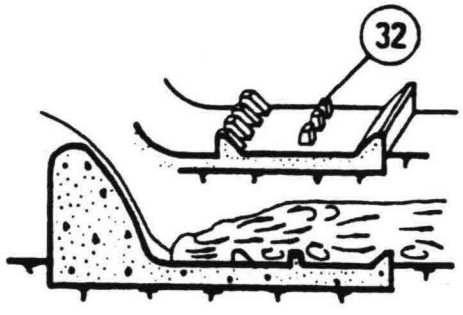

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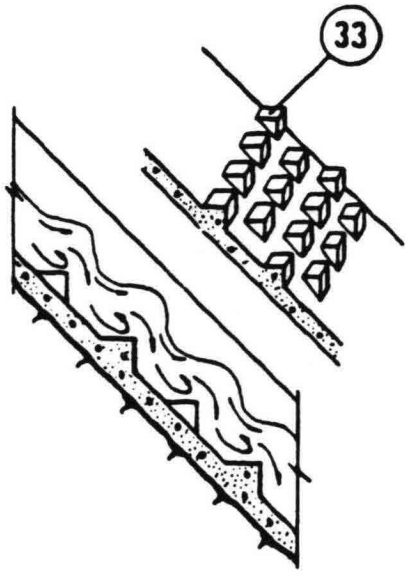

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Fig. 14.29.
Spillways.
28. Solid bucket basin; roller bucket basin
29. Slotted bucket basin
30. Bucket basin with sill
31. Hydraulic jump basin with impact blocks
32. Baffle block; impact block
33. Chute block.

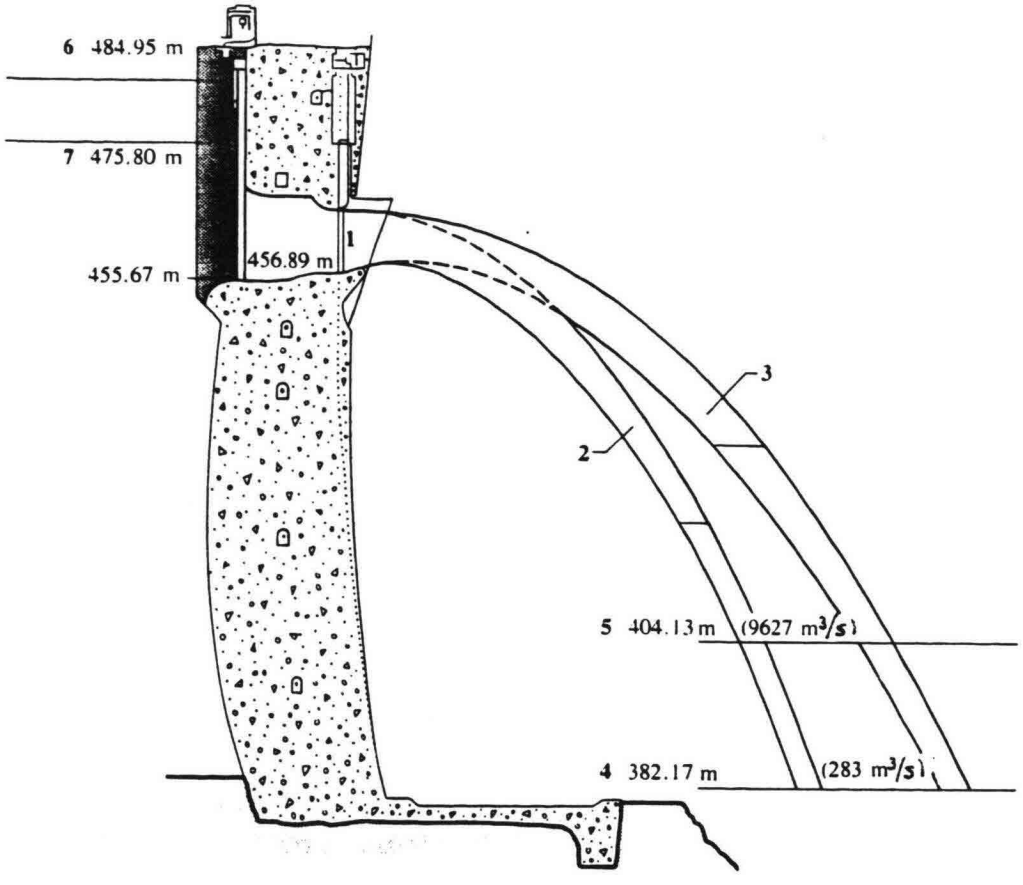


Fig. 14.30. Kariba dam spillway, Zimbabwe (after ICOLD 1986).

1. Six orifices, each 8.5 x 9.1 m^2, controlled by fixed-wheel flat gates
2. Jet profile for reservoir at 475.80 m
3. Jet profile for reservoir at 494.90 m
4. Minimum tailwater level 382.17 m (283 m^3/s)
5. Maximum tailwater level 404.13 m (9627 m^3/s)
6. Normal retention level
7. Minimal drawdown level.

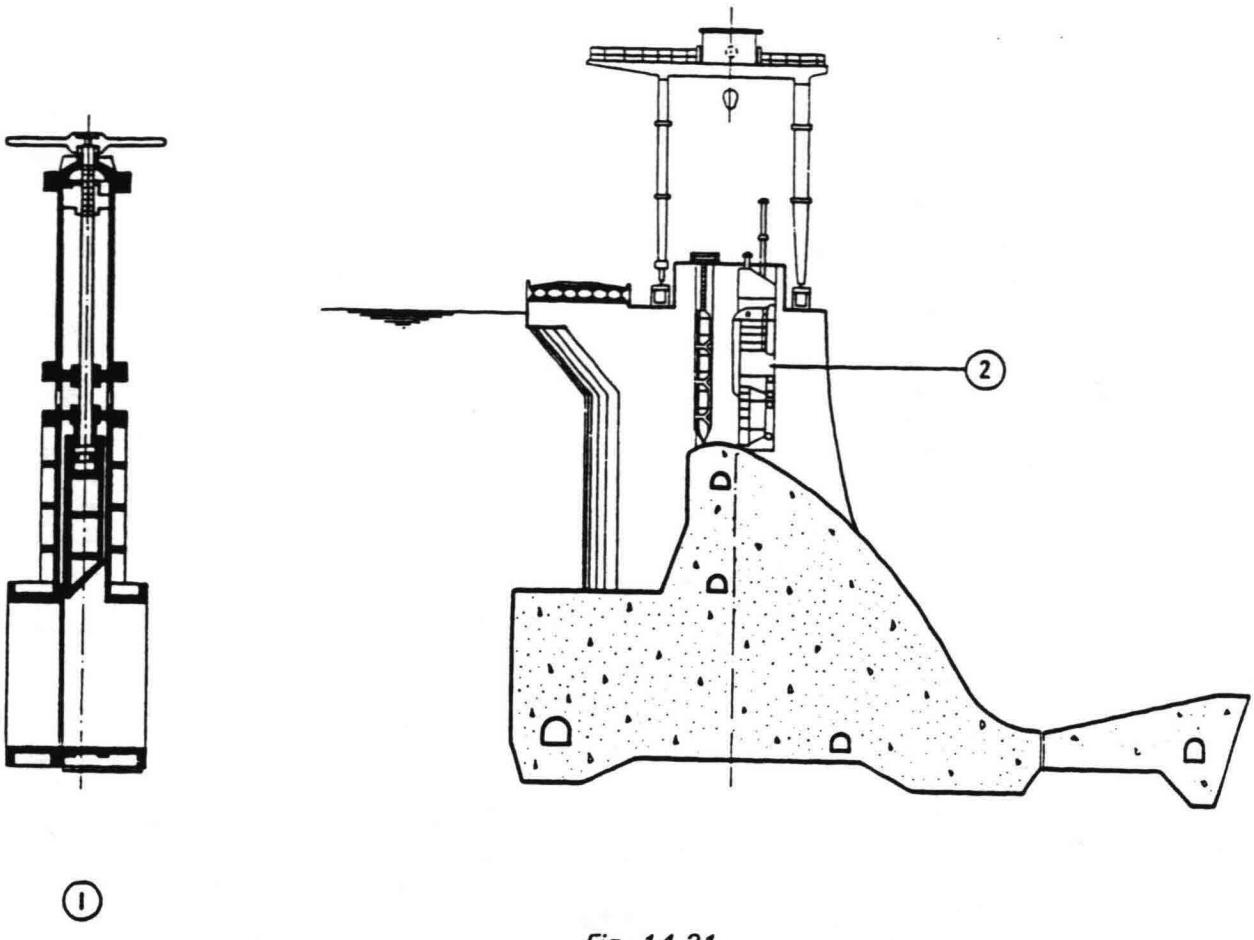


Fig. 14.31.
Gates and valves.

1. Wedge gate valve, sluice valve
2. Spillway gate, flood gate
3. Sluice gate; penstock(U.K.); slide gate (Am.).

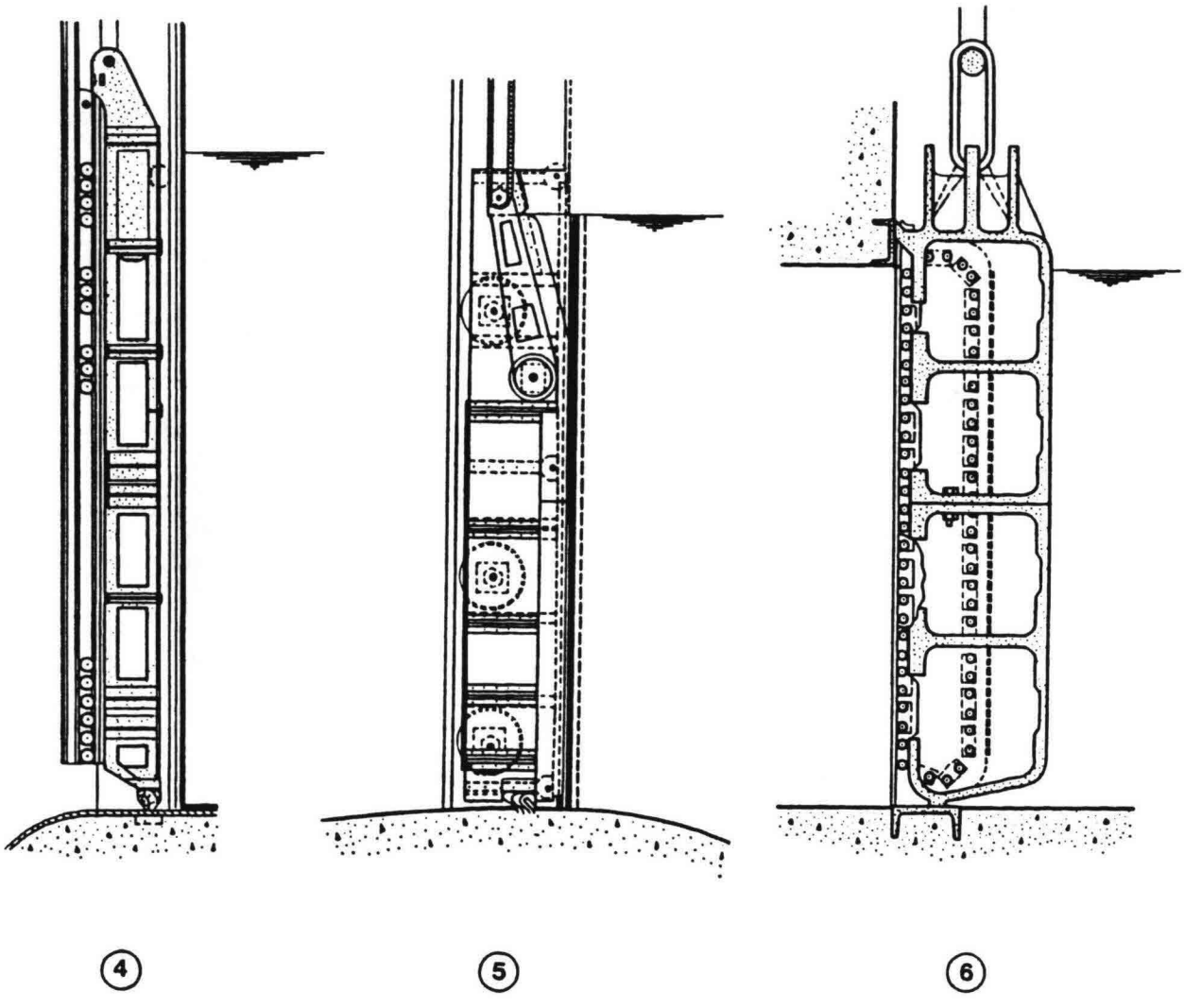


Fig. 14.32. Gates and valves.
4. Roller gate; stoney gate
5. Fixed wheel gate, fixed roller gate, fixed axle gate (Am.)
6. Caterpillar gate.

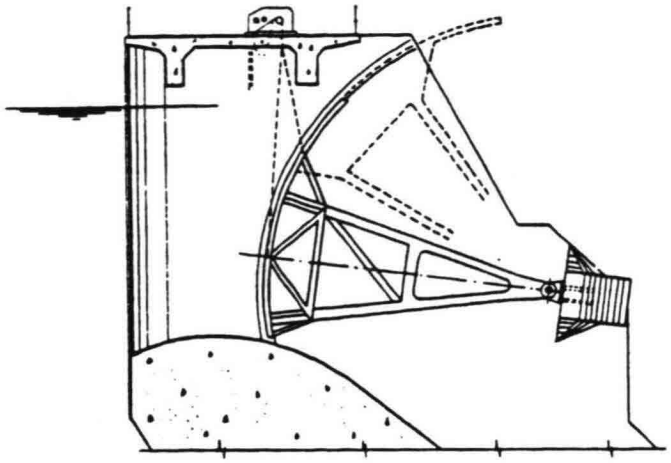
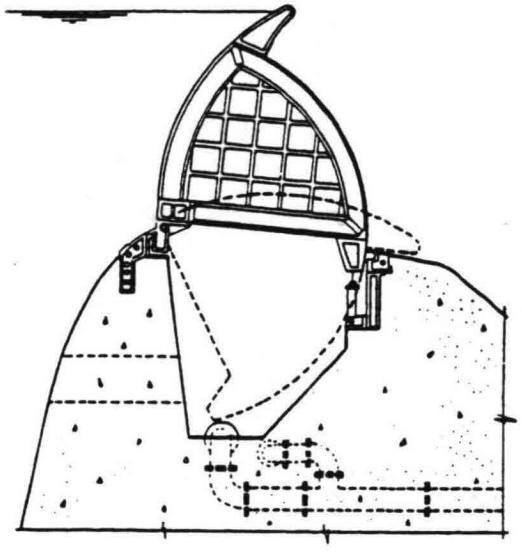


Fig. 14.33.
Gates and valves.
7. Radial gate; tainter gate
8. Drum gate; sector gate
9. Roller drum gate; rolling gate.

(7)



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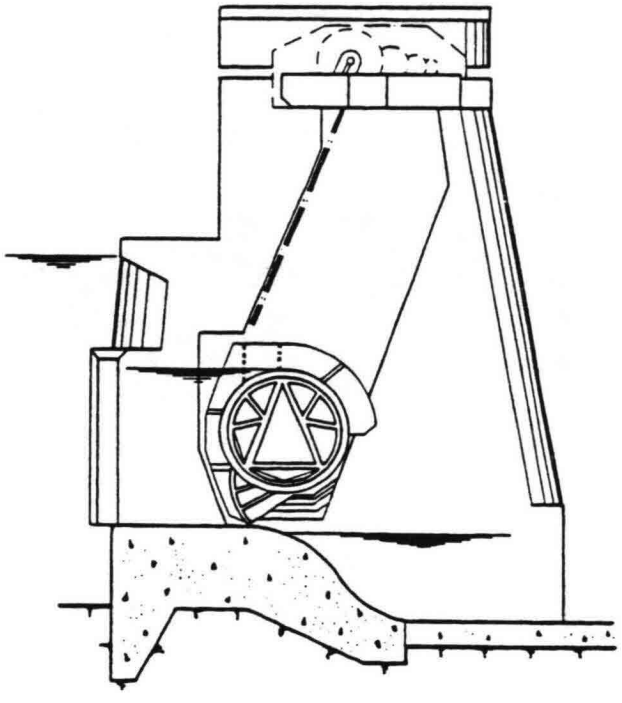

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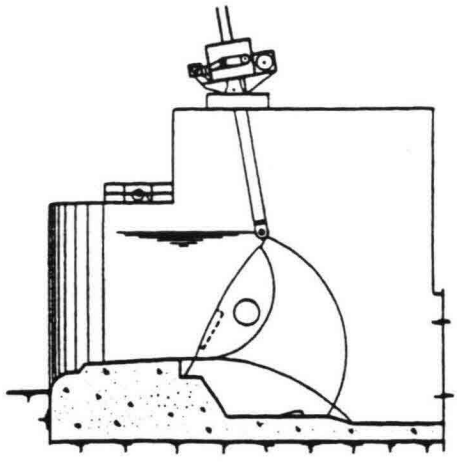
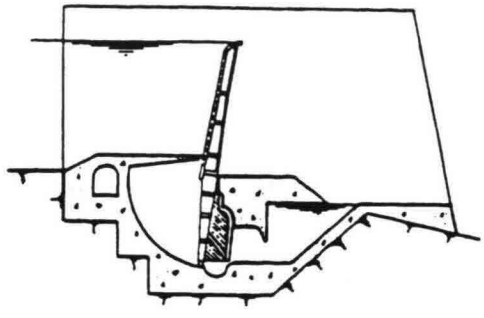
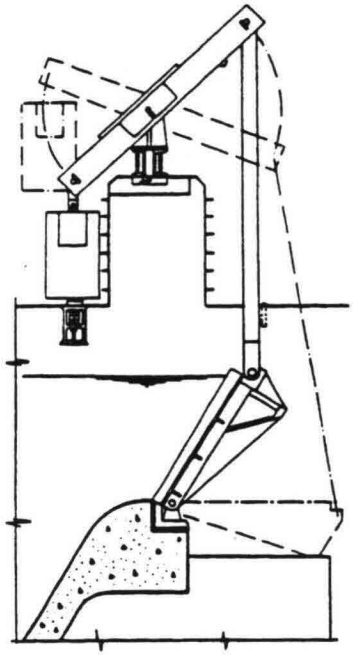
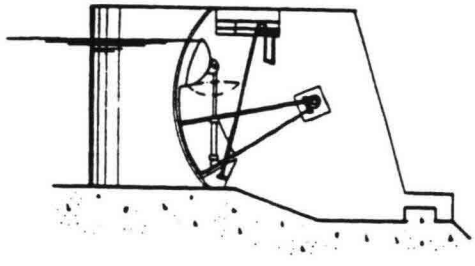
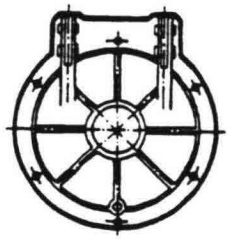










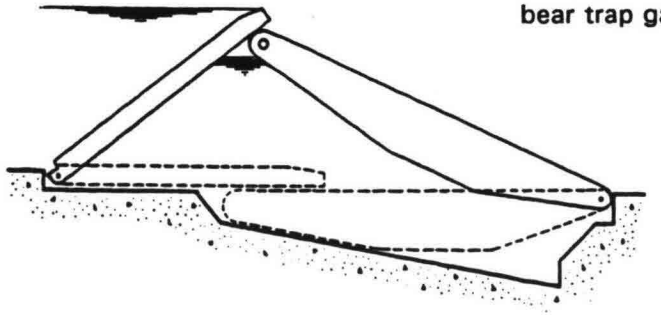
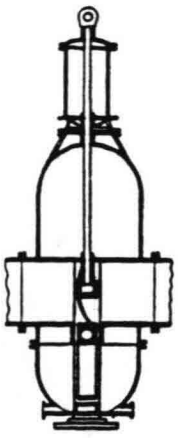


Fig. 14.34.
Gates and valves.

10. Tilting gate oscillating flashboard
11. Automatic tilting gate
12. Flap (on a radial gate)
13. Flap gate
14. Mushroom valve, open chamber, needle valve
15. Roof gate; roof weir; bear trap gate (Am.).



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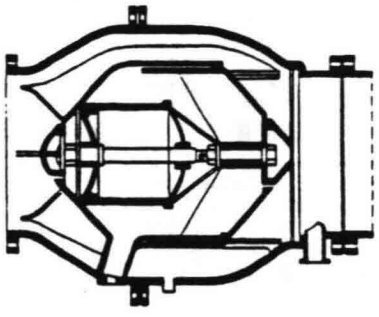

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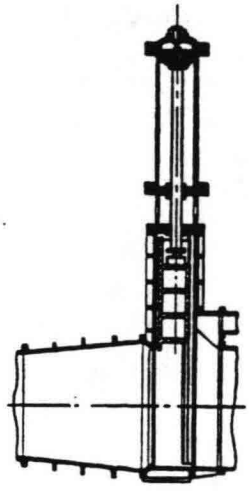

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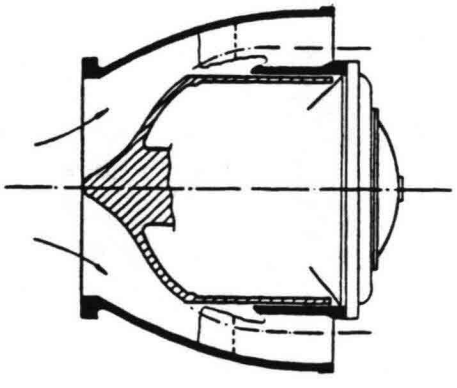

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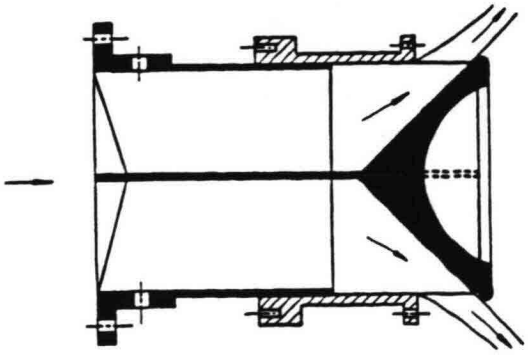

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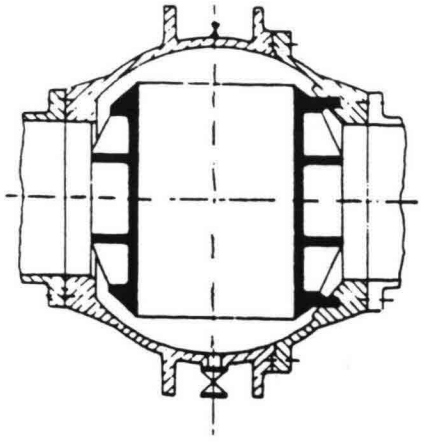
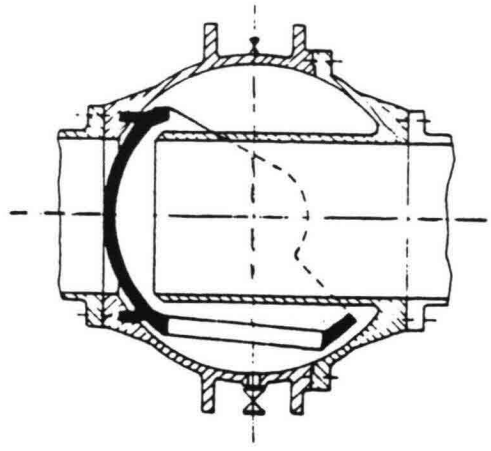
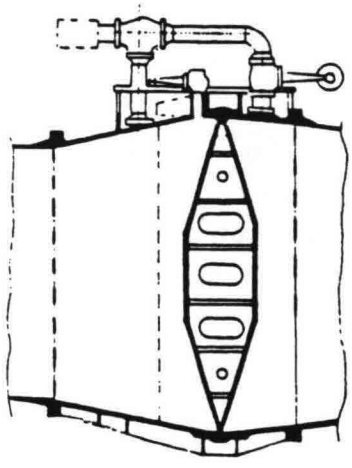
Fig. 14.35.
Gates and valves.

16. Ring-follower gate; ring-seal gate; paradox gate (Am.)
17. Needle valve
18. Jet flow gate
19. Hollow jet valve hollow core valve(Am.)
20. Howell-Bunger valve.







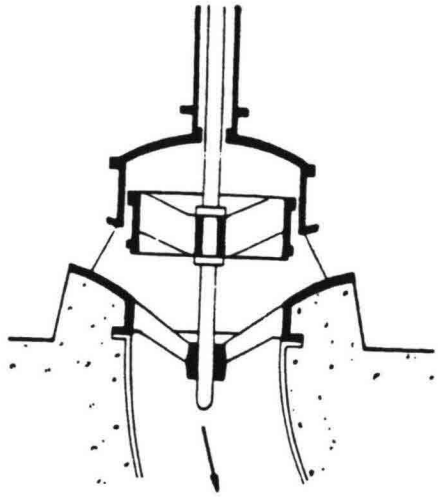


Fig. 4.36. Gates and valves.
21. Butterfly valve
22. Rotary valve; sphere valve
23. Cylinder valve.

14.3. Bottom outlets and intake structures

Bottom outlets are facilities devised to empty the reservoir flush sediments and maintain flow in the downstream channel. They may integrated in the dam structure or located in the abutment zones of a dam. These low level outlets are always gate controlled and may be of culvert type or tunnel type. Flow control equipment are plane gates or radial gates.

Intake structures have the function of allowing the withdrawal of water from the reservoir for various purposes such as hydropower, irrigation etc. These structures may be an integral part of the dam itself or located in the abutments or even an independent structure.

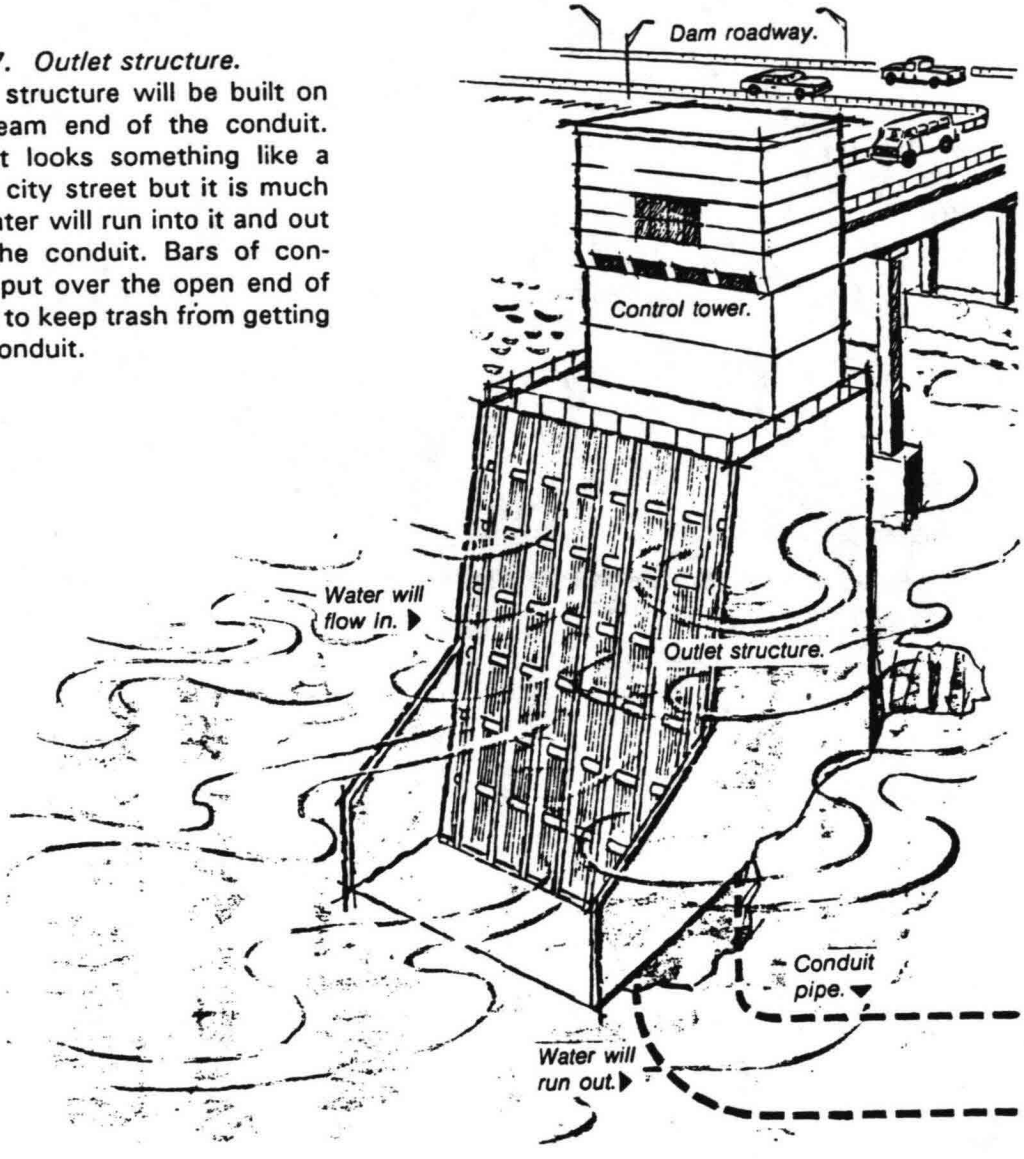


Fig. 14.37. Outlet structure.
An outlet structure will be built on the upstream end of the conduit. The outlet looks something like a drain in a city street but it is much larger. Water will run into it and out through the conduit. Bars of concrete are put over the open end of the outlet to keep trash from getting into the conduit.

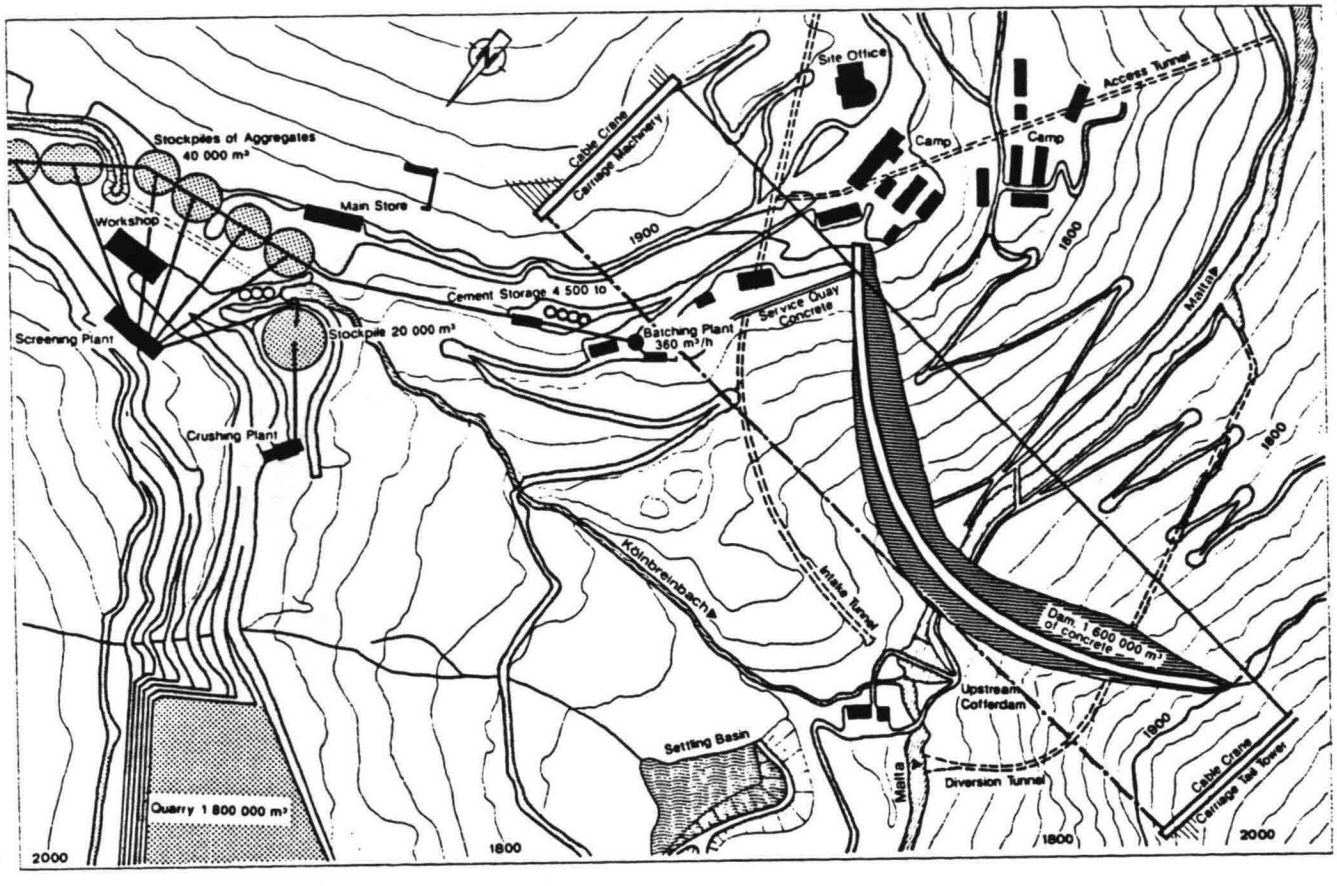


Fig. 14.38. Location plan of site installations for Kölnbrein dam project, Austria.

Hoover (Boulder) dam

In 1931 when no dam in the world was higher than 122 m, work began on the Boulder Dam. It was to be 222 m tall. The Colorado was an unpredictable and uncontrollable river. It could turn from a gentle stream to a current in a few hours and it was a very rare year when its floods did no damage. This state of affairs could not go, so the United Sates Bureau of Reclamation planned a series of dams to control and use the river for hydro-electricity, irrigation and town water supply, the most significant being the Boulder Dam. In its journey from the Rocky Mountains to the Gulf of California, the Colorado River has carved itself 455 km of gorges; some are 1,830 m deep. Most of these were examined in looking for a likely site or the new dam. Some were only accessible from narrow paths; other could only be surveyed from boats. Two showed themselves to be most suitable: Boulder Canyon and Black Canyon. Test holes were drilled up to 61 m to sample the rock and Black Canyon was chosen, but the dam is called the Boulder Dam. The site is 160 km downstream from the Grand Canyon.

The first job was to build four diversion tunnels, two in each side of the canyon. They are of horseshoe cross section, 17 m wide and 12.8 m high, and their lengths vary from 1,097 to 1,280 m. The rock was hard and needed no shoring, and there was no trouble with underground water. Driving the tunnels became a routine: a carriage was put up to the face, thirty holes were pneumatically drilled and charged, the carriage was withdrawn, the charges fired, and electric shovels moved up to clear the broken rock. The cycle took about ten hours to perform and the tunnel advanced at least 4.5 m each time. They were finished in less than two years, including a 1-m thick concrete lining. Sloping shafts into the outer pair of tunnels were made at the same time to be used later for the overflows when the lake behind the dam was full: Lake Mead was to be 178 m deep at the dam and stretch back for 185 km.

The engineers built coffer dam upstream and downstream of the dam site to keep it dry and divert the river into the tunnels. The base of the dam has a maximum width of 201 m and goes down at its deepest 42 m below the river bed. Laying the 5,000 million kg of concrete in the restricted space presented some difficulties. An unprecedented problem to be solved was the fact that concrete gives off heat as it sets. Normally this heat is easily lost to the air, but in such a large volume as this the heat would not be able to get away, so the concrete would heat up and expand, and then crack as it cooled. If the concrete were laid layer by layer in the normal manner it would have taken 200 years to construct the Boulder Dam. The answer was to use an immense refrigerator to cool water to near freezing, which was pumped through more than 917 km of pipes embedded in the concrete as it was poured. The concreting took two years.

There are four intake towers on the valley floor upstream, slightly higher than the dam. They are 9.1 m diameter vertical pipes with openings, supported by a concrete grid, and they take water to the turbines in the power station at the foot of the dam. Fifteen turbo-generators were installed, each giving 115,000 hp. The dam is arched upstream and the road along its crest is 400 m long. It was all finished in 1936 when the diversion tunnels were closed by steel shutters.

Hoover Dam, one of the world's largest.

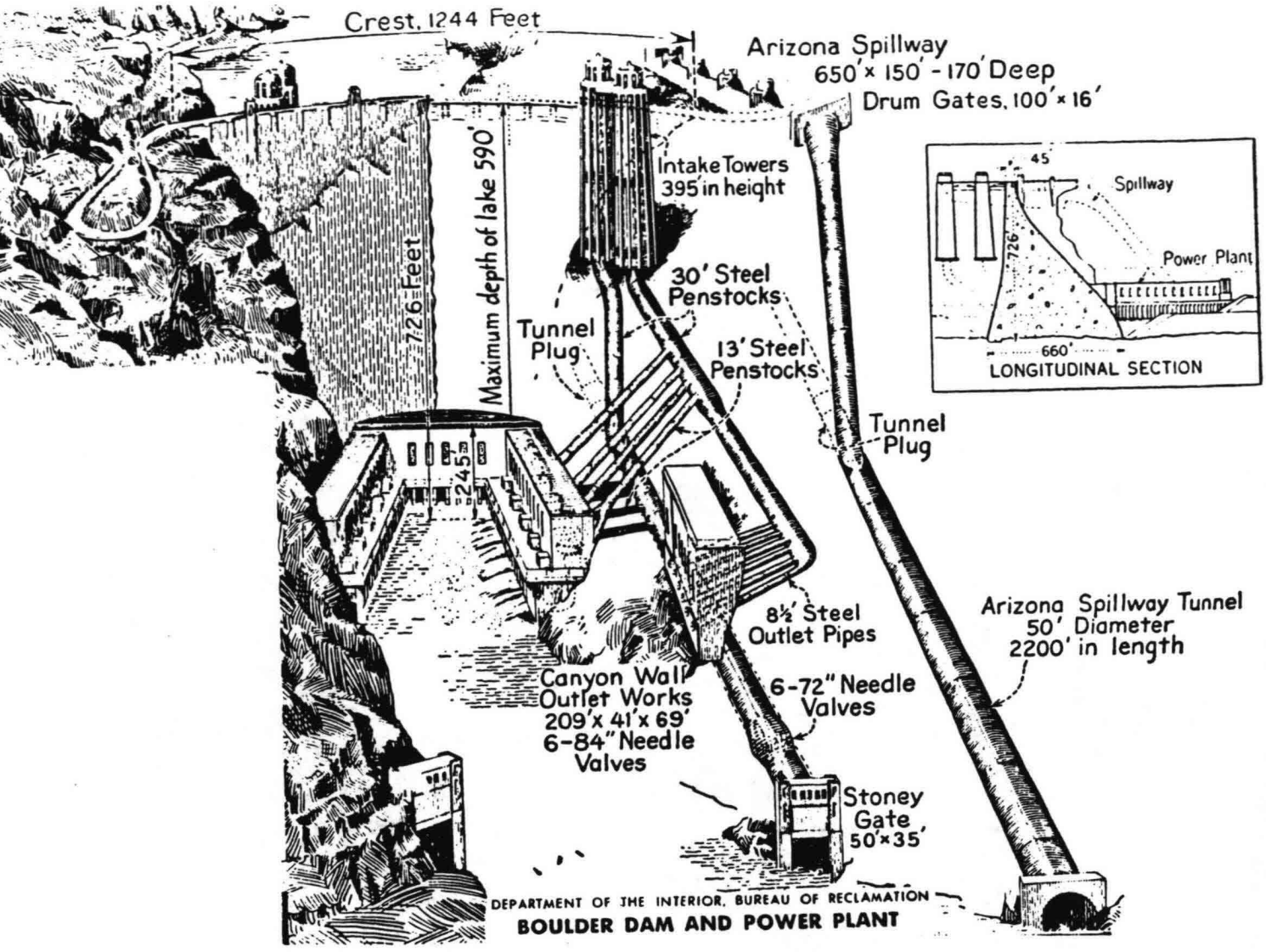


Fig. 14.39. Hoover (Boulder) Dam.
Construction started on May 16, 1931 and was completed and accepted by the U.S. Bureau of Reclamation on March 1, 1936 more than two years ahead of schedule. The dam was dedicated on September 30, 1935 by President Roosevelt. The name "Boulder Dam" was changed in 1947 by the Congress to Hoover dam, after Herbert Hoover chairman of the Commission charged with the work. With its 222 m height it has been the highest dam in the world for a long time.

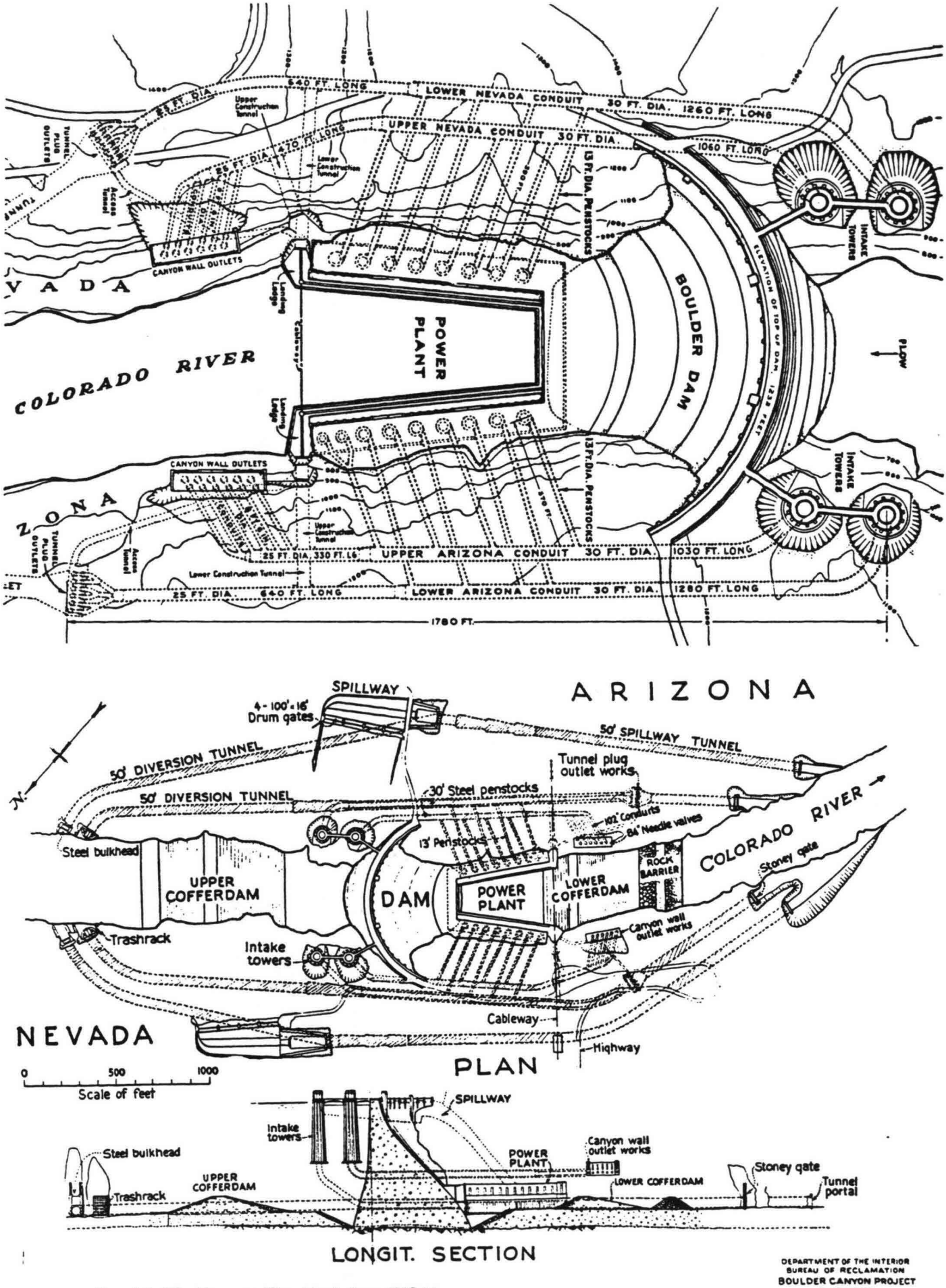
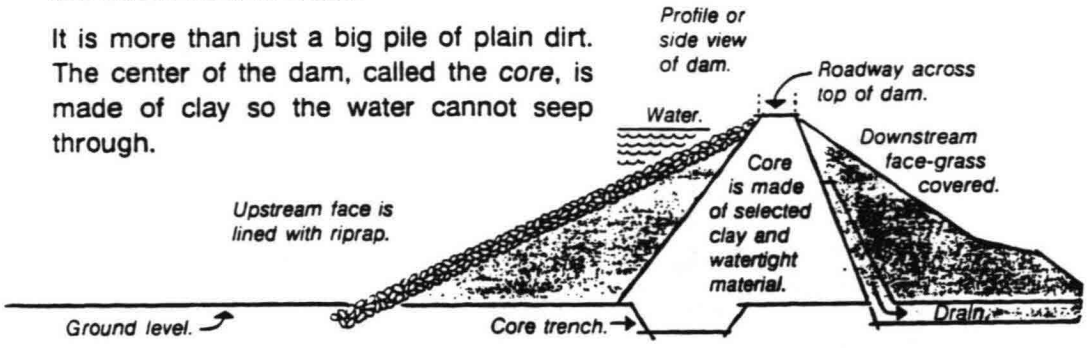


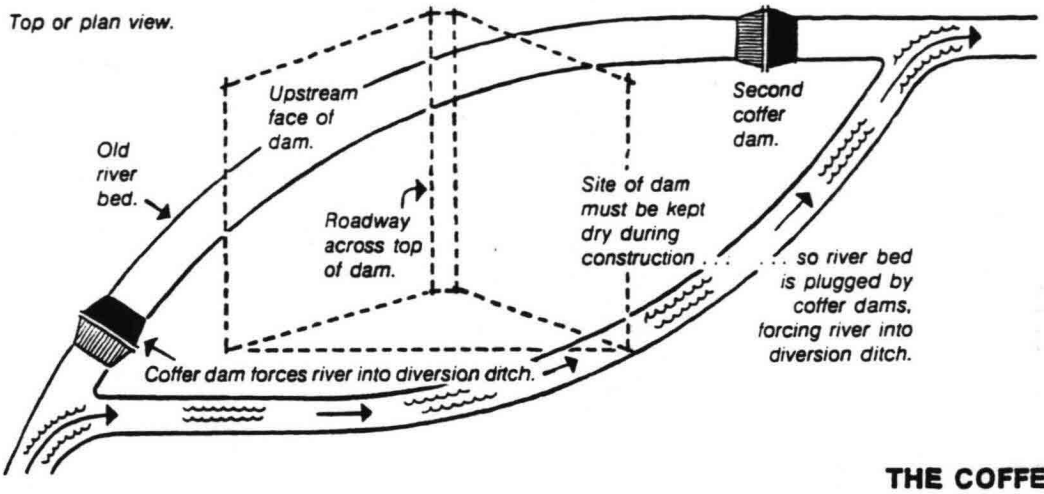
Fig. 14.40. Hoover (Boulder) dam (USA).

EARTHFILL DAMS

Many dams are made mostly of dirt. They are called *earthfill dams.*

It is more than just a big pile of plain dirt. The center of the dam, called the *core*, is made of clay so the water cannot seep through.



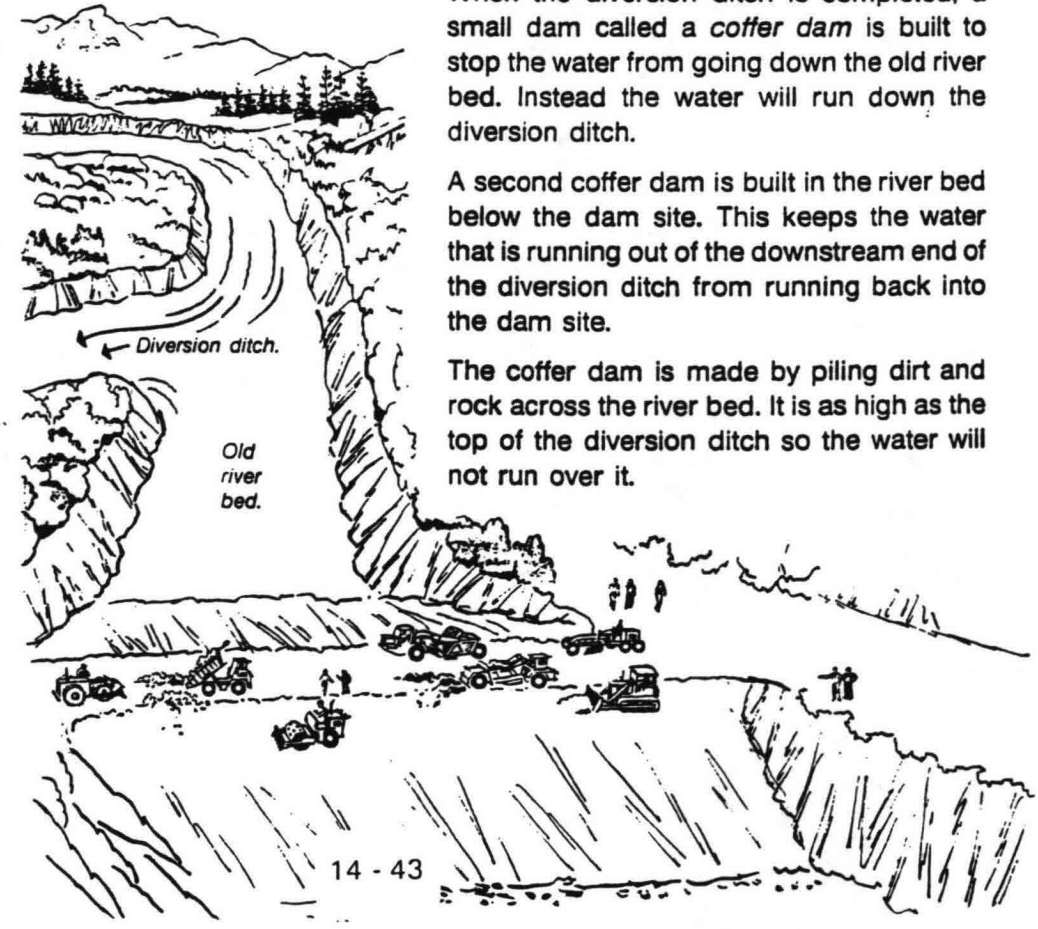


THE COFFER DAM

When the diversion ditch is completed, a small dam called a *coffer dam* is built to stop the water from going down the old river bed. Instead the water will run down the diversion ditch.

A second coffer dam is built in the river bed below the dam site. This keeps the water that is running out of the downstream end of the diversion ditch from running back into the dam site.

The coffer dam is made by piling dirt and rock across the river bed. It is as high as the top of the diversion ditch so the water will not run over it.



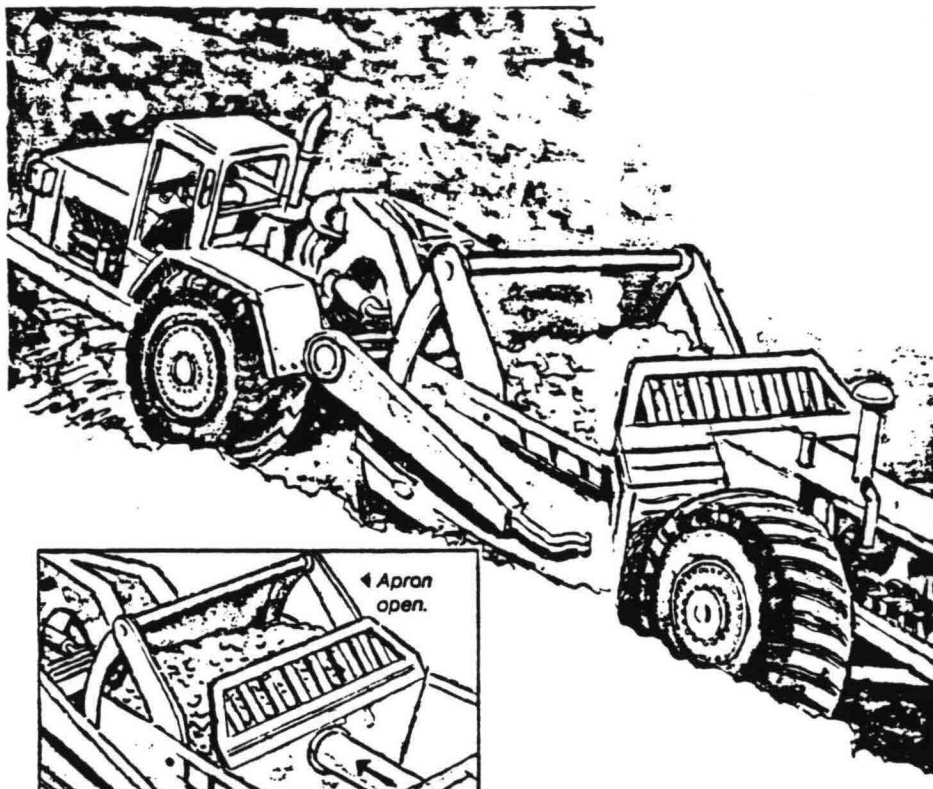
MOTOR SCRAPERS

STANDARD SCRAPER.

ELEVATING SCRAPER.

The dirt for the coffer dam is brought by *motor scrapers.* These big machines roll back and forth dumping big loads of dirt on the coffer dam. The front part of the motor scraper is a tractor. The tractor pulls a big scoop behind it. The scoop has wheels on it and it looks like a wagon. A sharp blade on the front of the scoop is lowered against the ground. The tractor grunts and roars as it pulls the wagon along. The blade scrapes the dirt up into the bowl.

When the motor scraper is in the right place to unload, the back wall of the scoop moves forward and pushes the dirt out.



When it is time to unload, back wall of scoop moves forward and pushes the dirt out.

CRAWLER LOADER.

CRAWLER DOZER.

CRAWLER TRACTORS AND BULLDOZERS

Scrapers load dirt at the borrow pit.

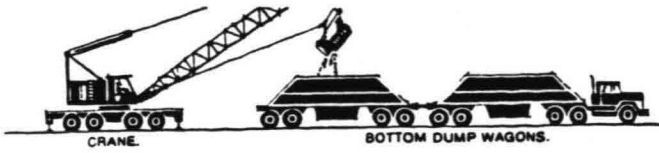
The place where the scrapers load the dirt is called the *borrow pit.*

Sometimes another kind of tractor is used to push the scrapers and help them load. It is a *crawler tractor* and moves along on steel tracks instead of rubber tires. It is not as fast as the scrapers but it is more powerful.

When a big steel blade is put on the front of a crawler tractor it is called a *bulldozer.* You will see many bulldozers at work pushing big piles of dirt around.

Large twin engine scraper.

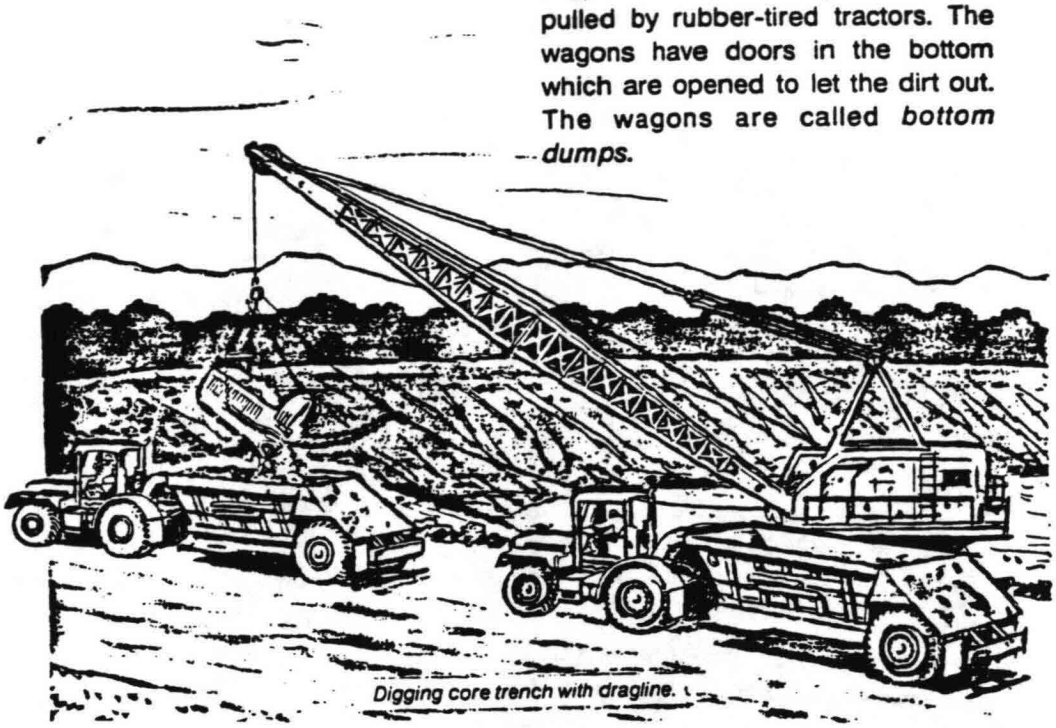
Bulldozers push the scrapers and help them to load.



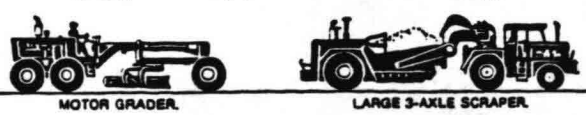
BOTTOM DUMPS

The core of the dam must go deep in the ground so that the water cannot seep under it. A *core trench* is dug first.

A dragline scoops up the dirt and drops it into steel wagons that are pulled by rubber-tired tractors. The wagons have doors in the bottom which are opened to let the dirt out. The wagons are called *bottom dumps.*



Digging core trench with dragline.

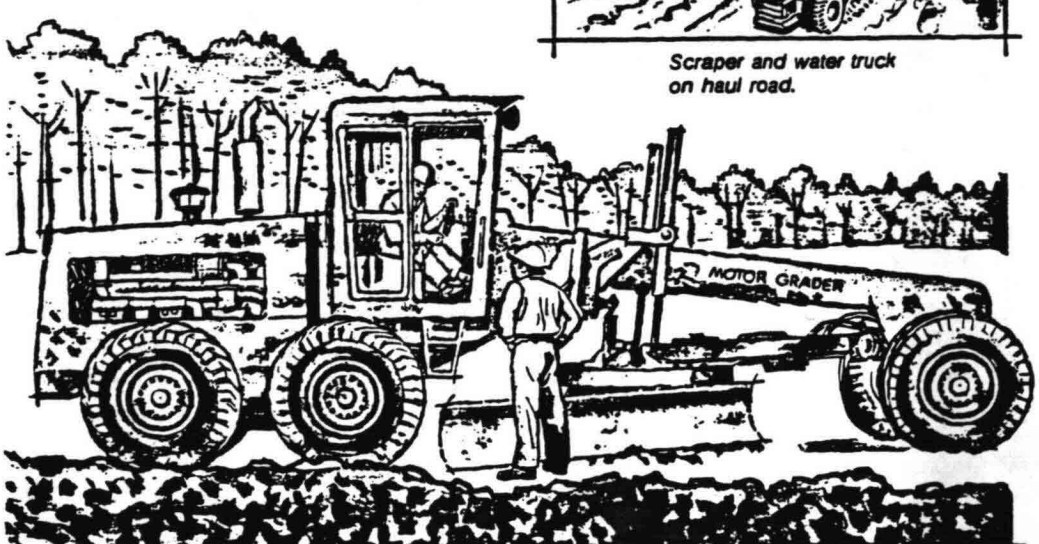


MOTOR GRADERS

The scrapers run back and forth from the dam to the borrow-pit on a *haul road.* A machine called a *motor grader* is used to make the haul road smooth. The motor grader has rubber tires and a big steel blade underneath. The blade scrapes the ground smooth.

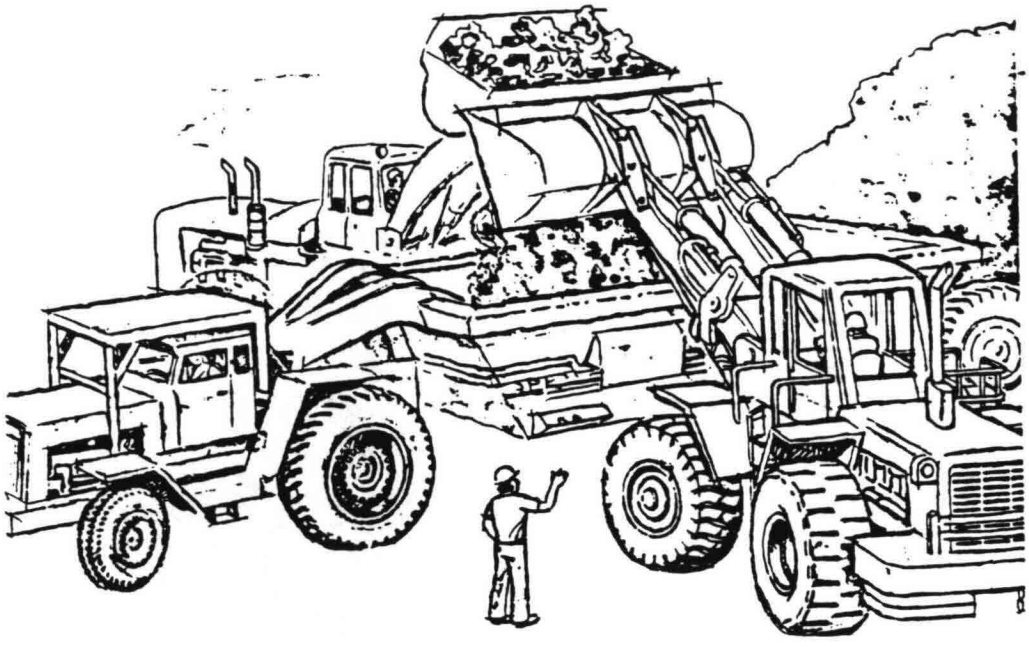
A water truck sprays water on the haul road to keep it from getting too dusty.

Scraper and water truck on haul road.



FRONT-END LOADERS

FRONT-END WHEEL LOADERS. CRAWLER LOADER.



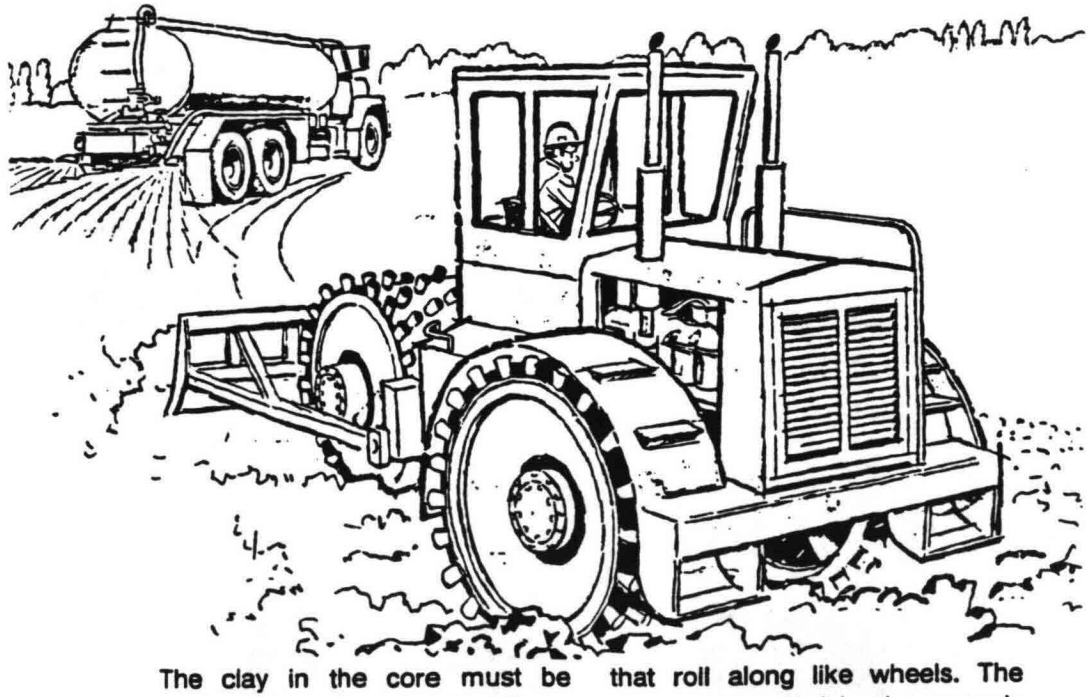
When the core trench is finished, it is ready to be filled. The bottom dumps are loaded with clay by *front-end loaders.*

The huge rubber tires on the front-end loaders are so big that you could just barely roll them through the door of a house.

A large steel scoop on the front of the machine moves up and down. It lifts the clay up and dumps it into the bottom dump.

PNEUMATIC ROLLER. SHEEPSFOOT COMPACTOR. VIBRATORY COMPACTOR.

COMPACTORS



The clay in the core must be packed down so that it will be watertight.

Compactors roll back and forth across the dirt as it is unloaded from the bottom dumps. The compactors have big steel drums that roll along like wheels. The drums have steel knobs or pads sticking out on them. The knobs push into the ground and pack it down until it is hard.

A water truck sprays water on the dam, making it easier to compact.

BUILDING UP THE EARTHFILL DAM

As the watertight core rises above the ground, regular dirt is dumped on each side of it. This dirt is also packed down by compactors. The dirt on the side of the dam where the water comes from is called the *upstream shell.* The dirt on the other side is called the *downstream shell.*

The downstream shell is built with a layer of sand at the bottom. This layer of sand runs up the side of the core and is called a *drain.* Sand is used because water will pass through it easily. The sand allows the small amount of water that might seep through the core to run out the back of the dam.

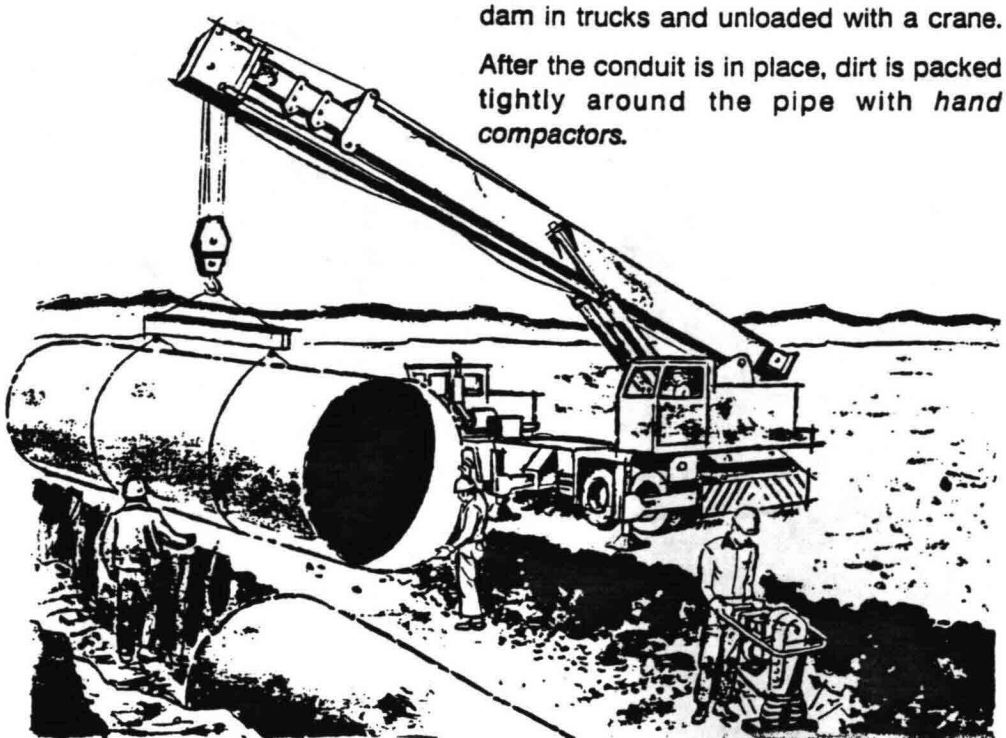
Giant dump truck speeds up the work.

THE CONDUIT

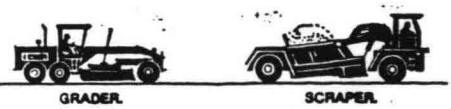
There must be a way of releasing water through the dam so there will be a supply of water downstream.

A large pipe called a *conduit* is run through the dam. The conduit pipe is carried to the dam in trucks and unloaded with a crane.

After the conduit is in place, dirt is packed tightly around the pipe with *hand compactors.*

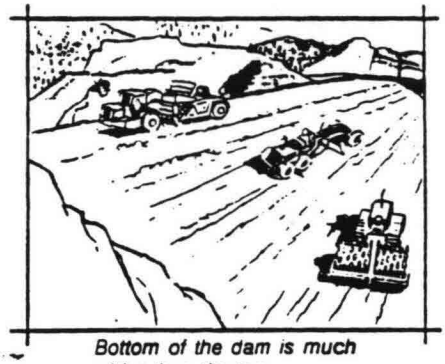


FINISHING THE EARTHFILL DAM

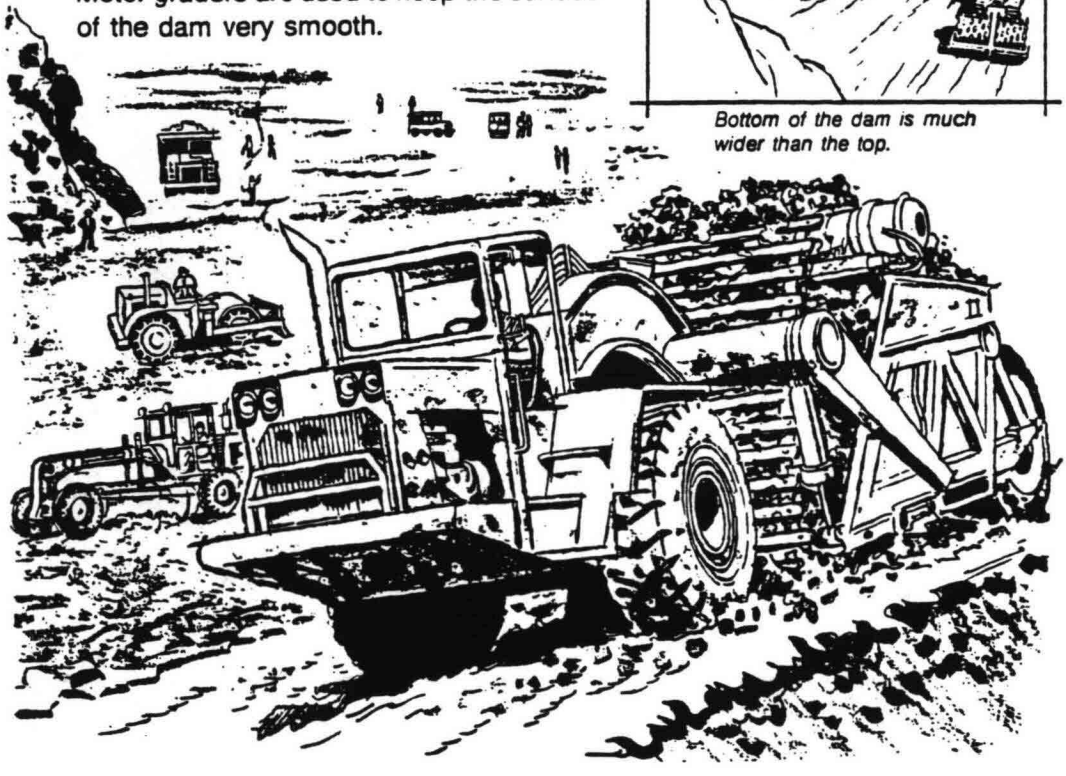


The bottom of the earthfill dam is much thicker than the top. When all of the dirt has been put in and packed down hard, it is just wide enough at the top for one scraper to unload at a time.

Motor graders are used to keep the surface of the dam very smooth.



Bottom of the dam is much wider than the top.



RIPRAP

Grass will be planted on the downstream slope to keep the rain from washing the dirt away. But something else must be done to the upstream side to keep the dirt from being washed away by the waves splashing against it.

A thick cover called stone *riprap* is placed on the upstream slope to protect it.

Off-highway trucks loaded with big stones rumble along the top of the dam. They dump the riprap stones down on the upstream slope.

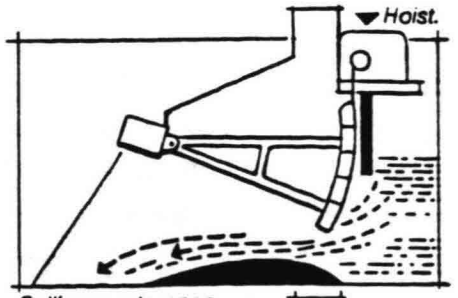
Some of the stones do not land in the right place. So they are picked up with *rock tongs* that are swung into place by a crane. The rock tongs work like the claw of a giant bird. The claw picks up the big stones and drops them in the right place.

THE SPILLWAY

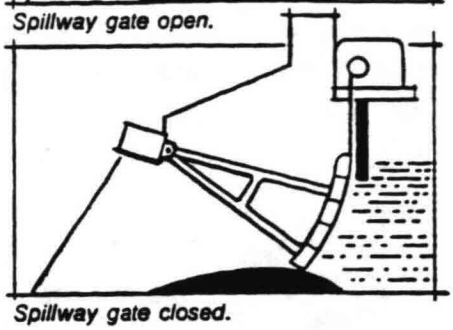
If you have ever tried to stop water from running down a ditch by piling dirt in the ditch, you probably learned something very important about building a dam.

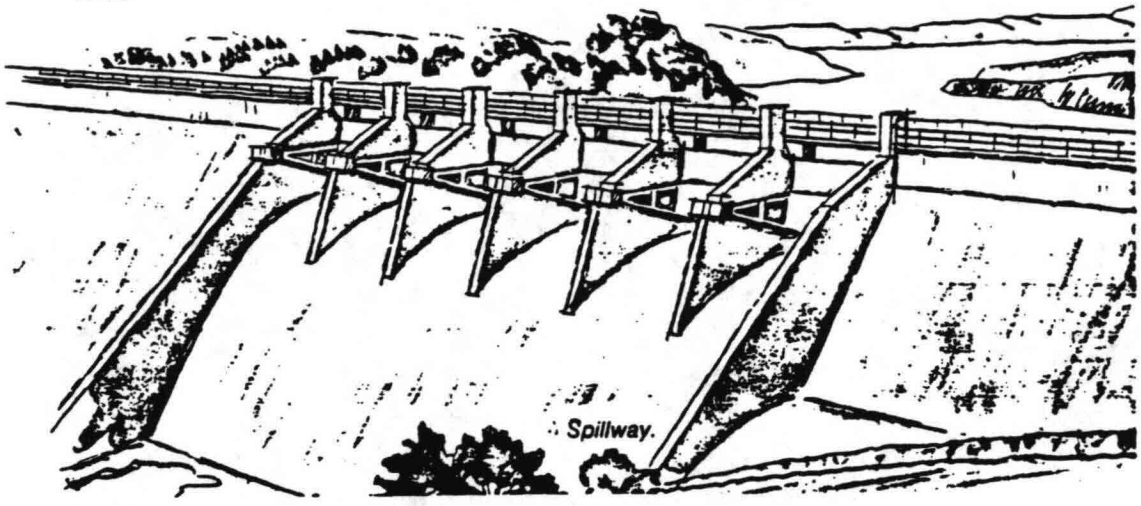
The water finally rises high enough to spill over the top of the dirt. When it does, it soon washes the dam away. The dam builders must make a place for the high water to spill over. This place is called a *spillway* and it is made of concrete so it will not wash away.

The spillway may have moveable gates that open to let the water out and close to keep it in.



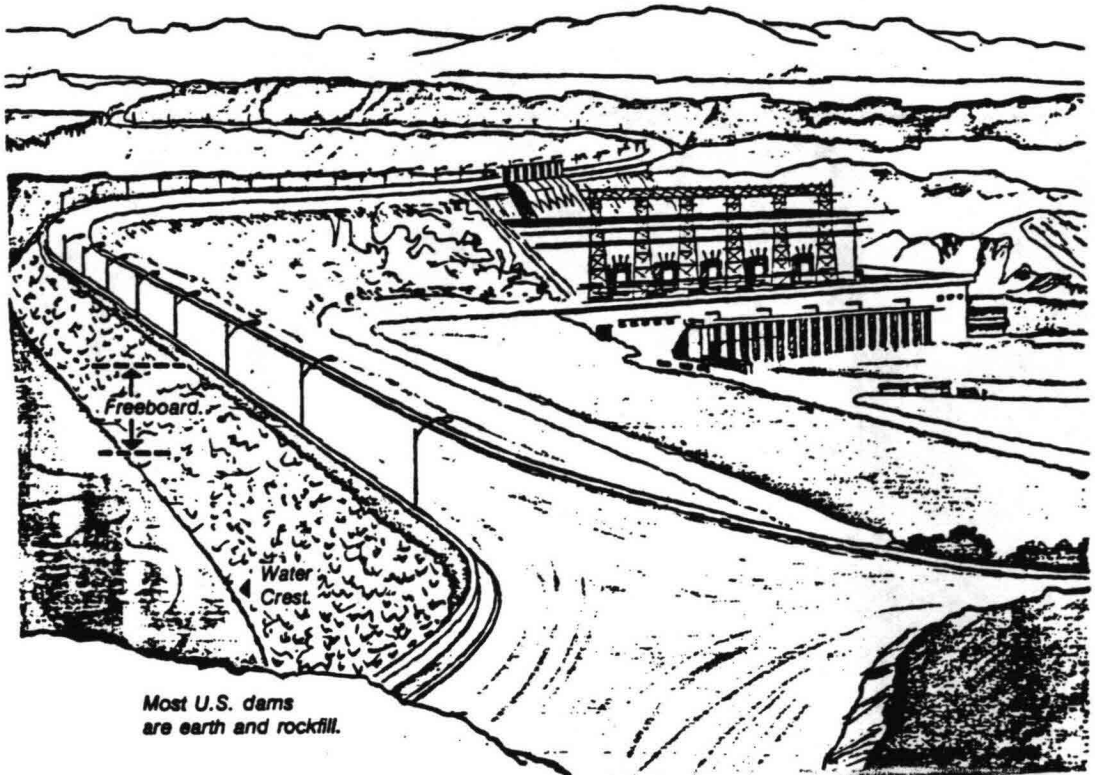
Spillway gate open.


Spillway gate closed.



The top of the dam must be higher than the spillway. This means that it will be higher than the water will ever go.

The part of the dam which sticks up above the water is called the *freeboard*. It is there to keep high waves from lapping over the top.



Most U.S. dams are earth and rockfill.

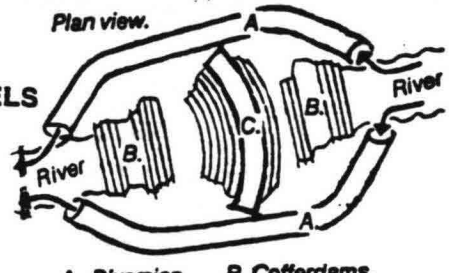
MASONRY DAMS

The most exciting dams of all are the masonry dams. These dams are usually made of stone or concrete and are very high but not as wide as earthfill dams.

Masonry dams are often built in places where it is hard to haul dirt. Often they are built in rocky and mountainous areas. Sometimes they are built in canyons where rock cliffs rise on either side.

Concrete for the dam rises from the canyon floor.

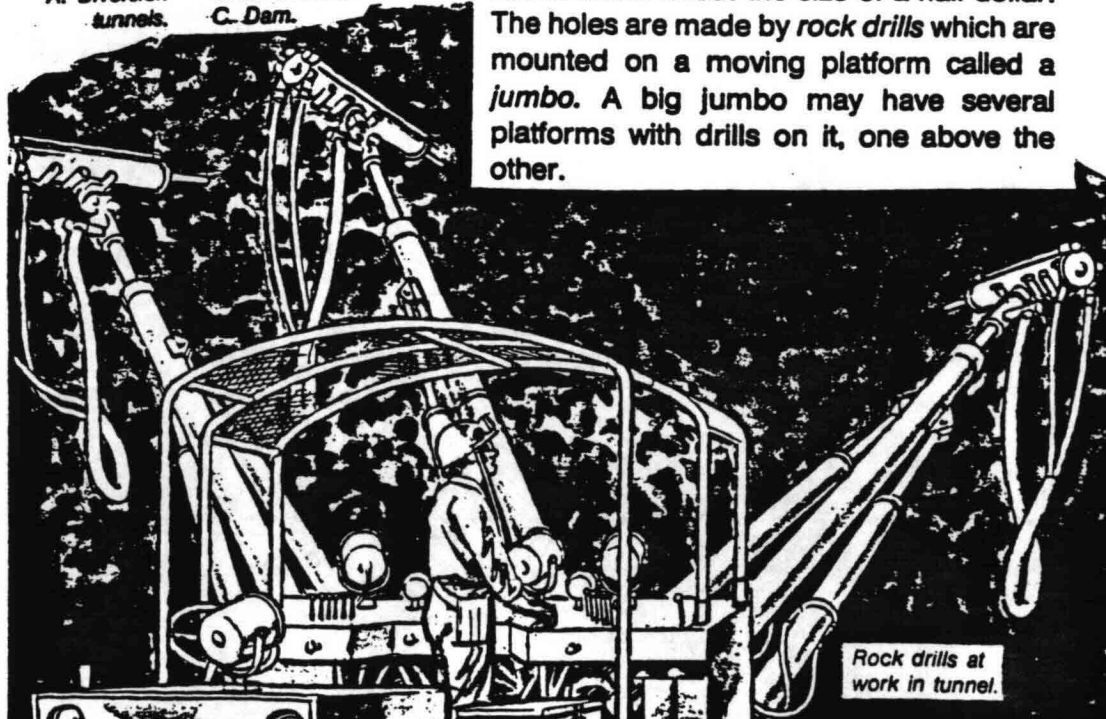
DIVERSION TUNNELS



A. Diversion tunnels. B. Cofferdams. C. Dam.

Just like the earthfill dam, coffer dams must be built first to make the water go around the dam site. This is hard to do, with big rock cliffs on both sides of the dam site.

Instead of a diversion ditch, *diversion tunnels* must be built. The tunnel is made through solid rock by first drilling small round holes about the size of a half dollar. The holes are made by *rock drills* which are mounted on a moving platform called a *jumbo*. A big jumbo may have several platforms with drills on it, one above the other.



Rock drills at work in tunnel.

BLASTING THE ROCK

DRILL. LOAD. EXPLODE.

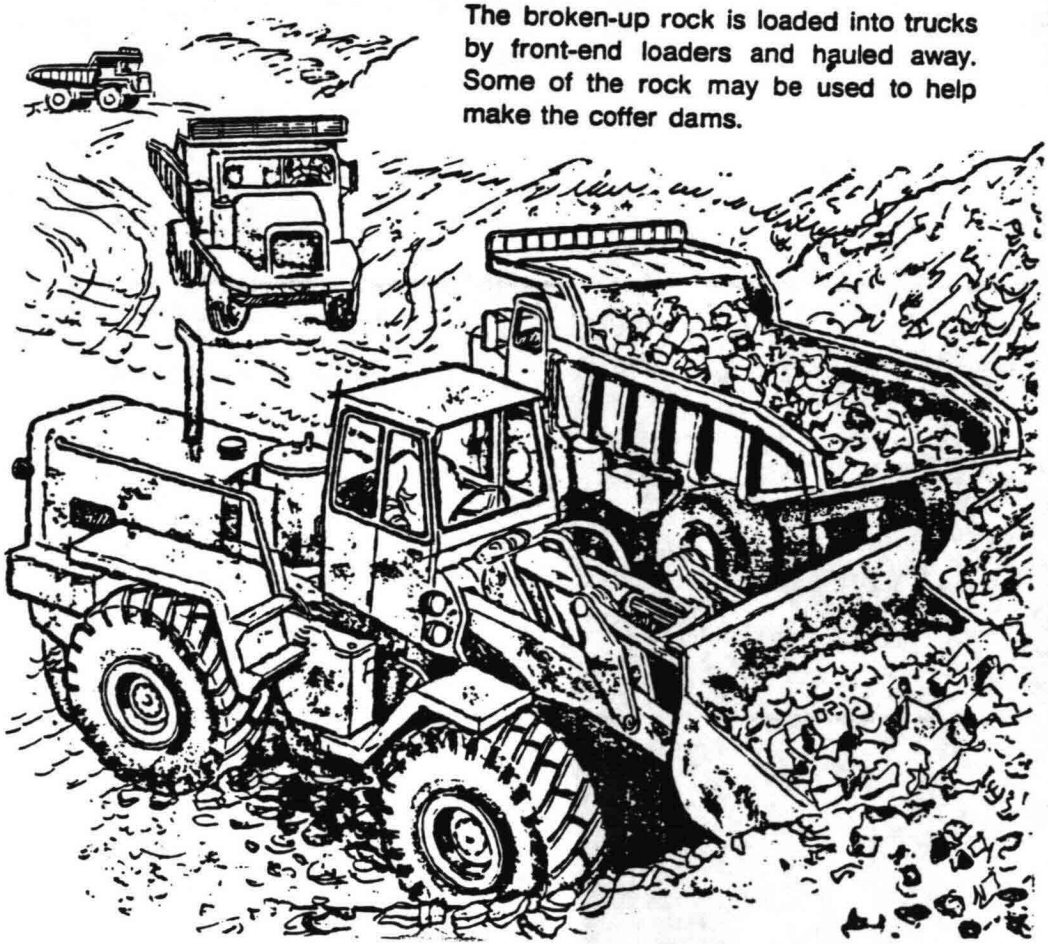
When the holes have been driven in the rock, men push dynamite deep into the holes with long wooden poles. Then dirt is packed into the holes. When everything is ready, the men yell, "Fire in the hole!" Everybody knows that this means to move a safe distance away from the danger.

When everyone has left the tunnel, one man flips a lever that sends electricity through wires attached to the dynamite. The dynamite explodes with a loud boom and breaks up the rock into small pieces.

Men push dynamite deep into the holes.

FRONT-END WHEEL LOADERS LOAD OFF-HIGHWAY TRUCKS.

The broken-up rock is loaded into trucks by front-end loaders and hauled away. Some of the rock may be used to help make the coffer dams.

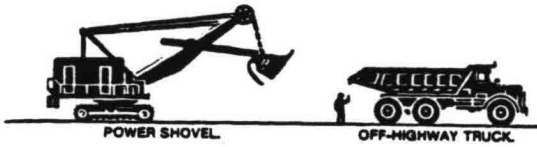


MAKING THE DAM SITE SAFE

As soon as the dam site is free of water, work can begin on the dam. Men are lowered from the top of the cliffs in *rope slings*. These slings look much like playground swings, but they are made to lower the men safely as they work.

The men use iron bars and small *jack hammers* to pry away loose rocks which might otherwise fall on the dam builders who will be working below later.

Rope slings hold men while they pry away loose rocks.

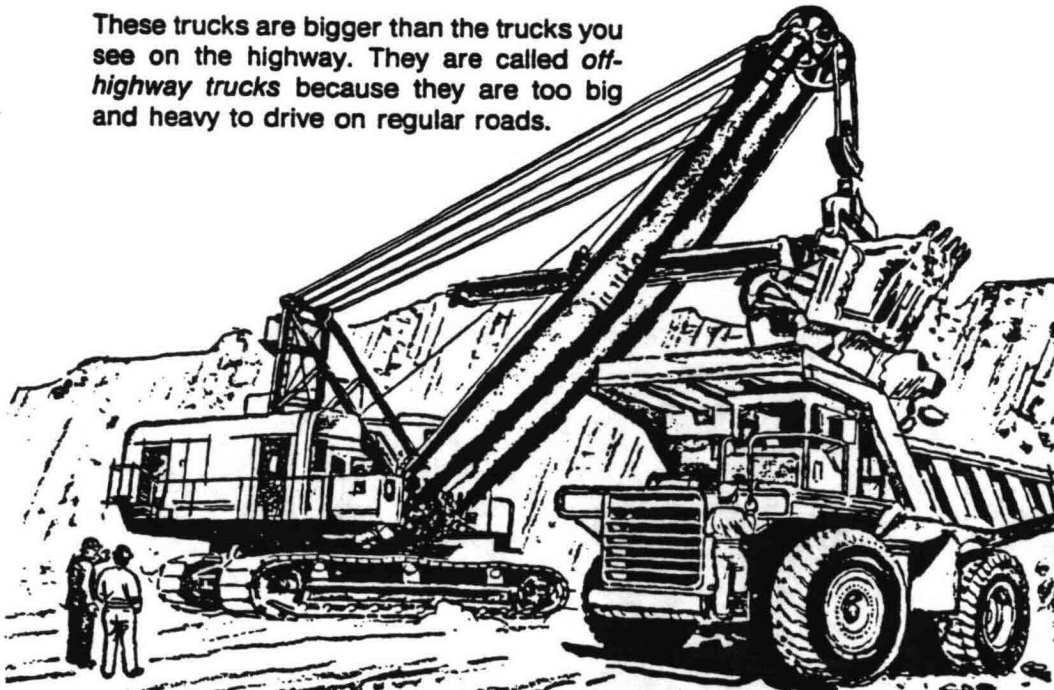


POWER SHOVELS AND TRUCKS

Big, powerful *power shovels* are next moved into the dam site. A power shovel looks like a huge machine with a giant steel arm. On the end of the arm is a large steel box with steel teeth on the edge. Dirt and rocks are scooped up in the box and dumped into trucks.

These trucks are bigger than the trucks you see on the highway. They are called *off-highway trucks* because they are too big and heavy to drive on regular roads.

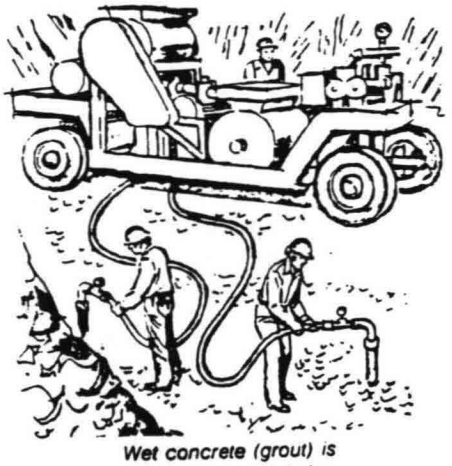
Off-highway trucks are loaded by big powerful shovels.



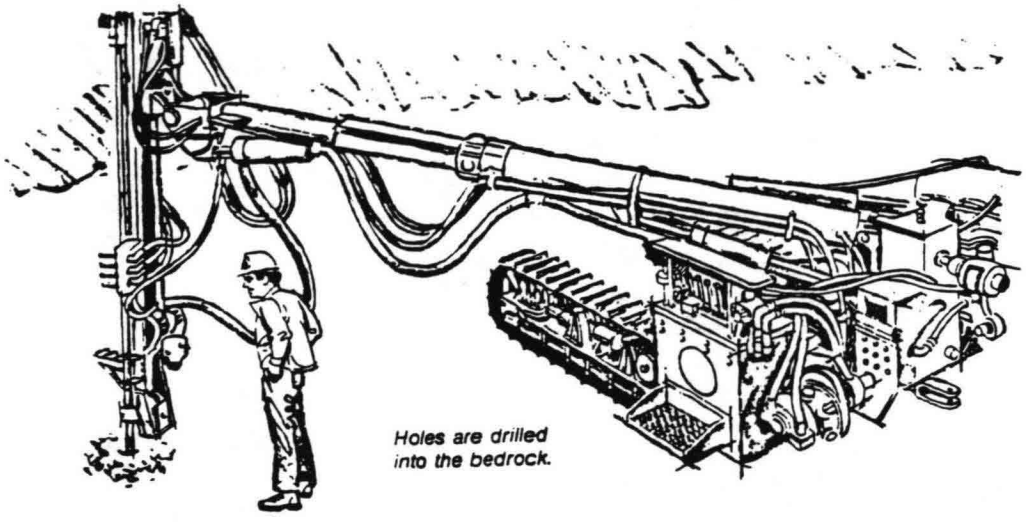
PREPARING THE FOUNDATIONS

The power shovel digs away the gravel and loose rock between the coffer dams. Finally it reaches a layer of rock called *bed rock.*

The bed rock has cracks in it which would let water escape under the dam. Holes are drilled into the bed rock. Then wet concrete is pumped into the holes. The wet concrete fills in all of the empty spaces in the rock and makes it watertight. This is called *grouting the foundation.*



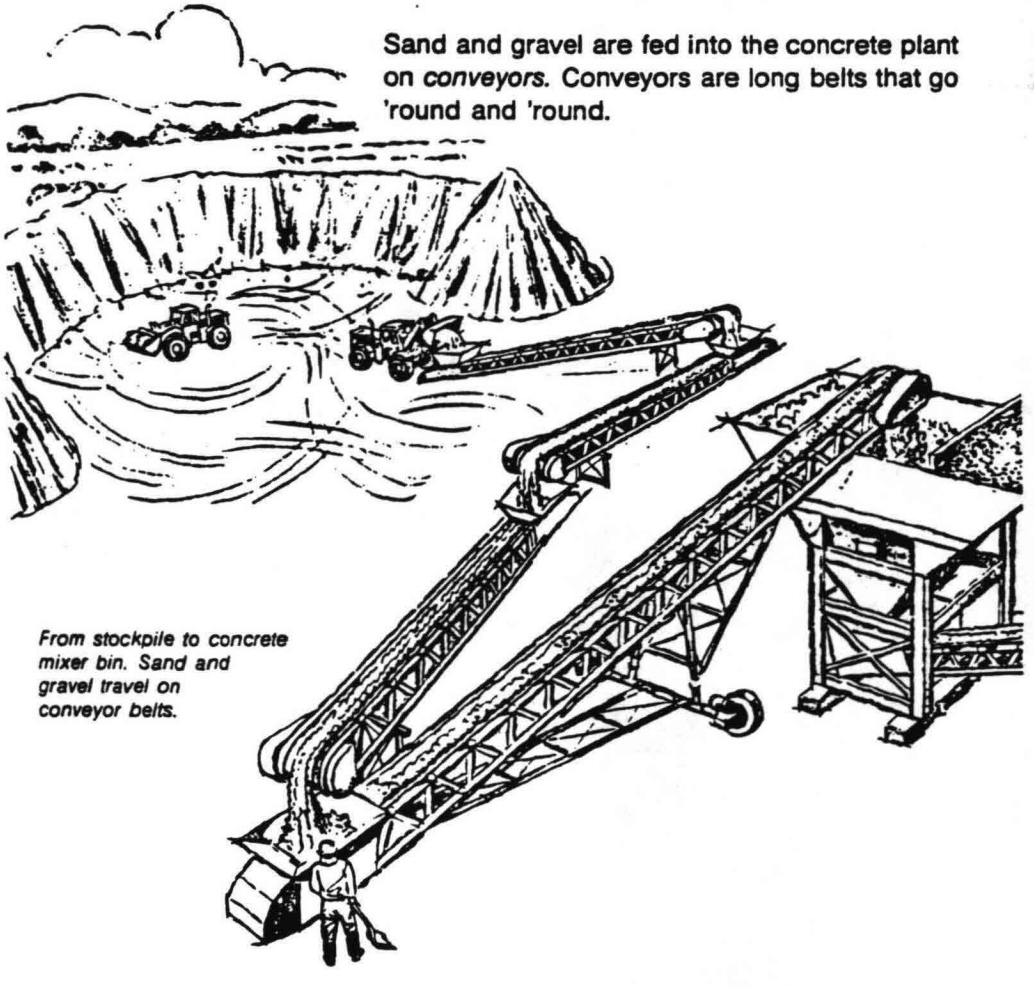
Wet concrete (grout) is pumped into the holes.



Holes are drilled into the bedrock.

MIXING THE CONCRETE

Sand and gravel are fed into the concrete plant on *conveyors.* Conveyors are long belts that go 'round and 'round.



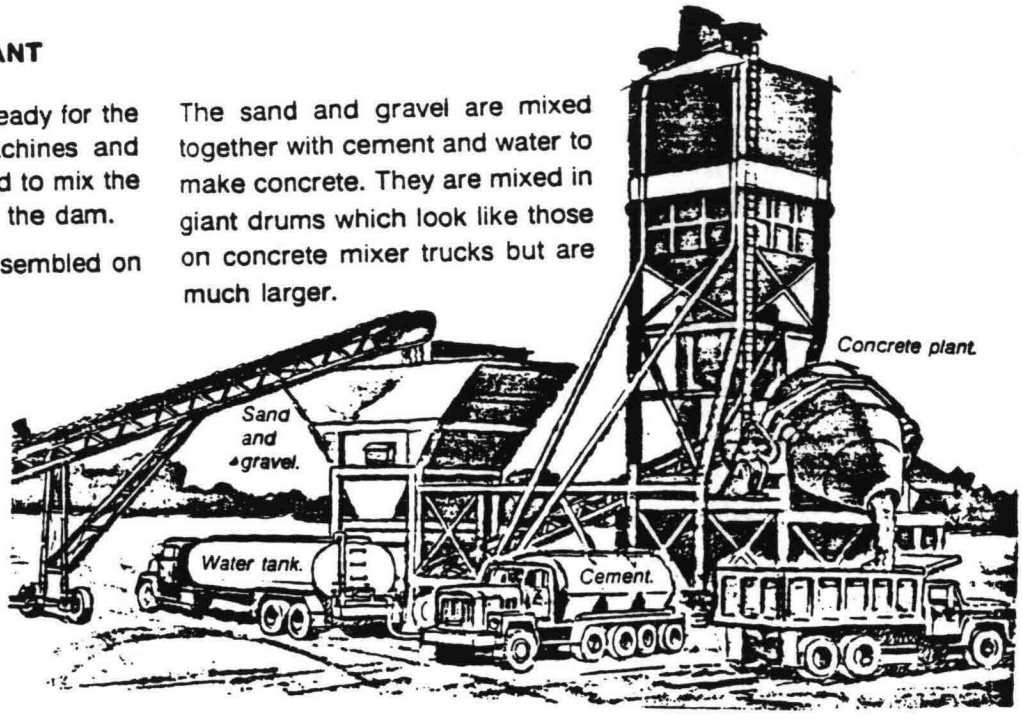
From stockpile to concrete mixer bin. Sand and gravel travel on conveyor belts.

THE CONCRETE PLANT

At last, everything is ready for the concrete. Special machines and many men will be used to mix the concrete and put it in the dam.

A concrete plant is assembled on the dam site.

The sand and gravel are mixed together with cement and water to make concrete. They are mixed in giant drums which look like those on concrete mixer trucks but are much larger.

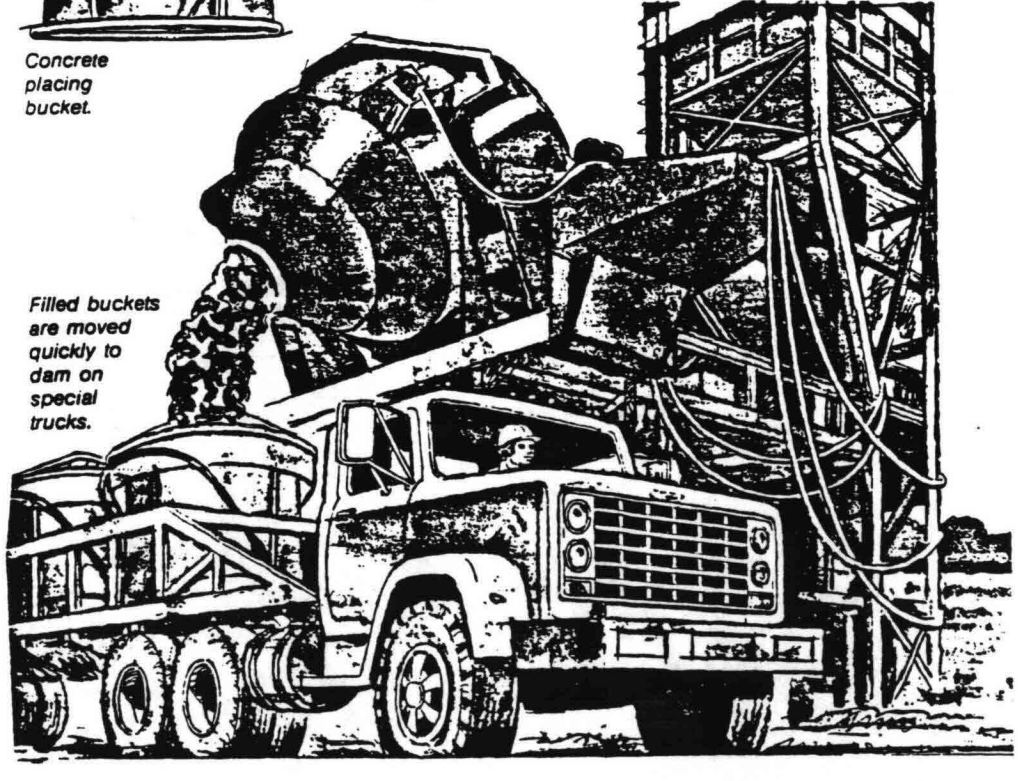


HAULING THE CONCRETE IN BUCKETS

The concrete must be quickly moved to the dam after it is mixed. One way to do this is to take concrete to the dam in *concrete buckets.*

Filled concrete buckets are hauled to the dam on special trucks.

Concrete placing bucket.



Filled buckets are moved quickly to dam on special trucks.

HAULING THE CONCRETE IN BUCKETS

At the dam the concrete buckets are fastened onto the long wire ropes of big cranes. Then they are swung to the right place on the dam. Even though they are really very heavy, they look like big balloons floating through the air.

Small doors on the bottom of the concrete buckets can be opened by pulling a big lever on the side.

HAULING THE CONCRETE BY CABLEWAYS

Sometimes the wet concrete is moved by *cableways*. These are made by stretching wire ropes across the canyon. On one side of the canyon is a steel tower that has wheels like a railroad car. The whole tower moves along on tracks that are like railroad tracks but much farther apart. This is called the *head tower*. A smaller tower on the other side of the canyon is called the *tail tower*. It moves the same way on parallel tracks.

The cable can be pulled back and forth between the towers or moved sideways by them.

Cableway head tower.

Cableway tail towers.

Concrete mixing plant

HAULING THE CONCRETE BY CABLEWAYS

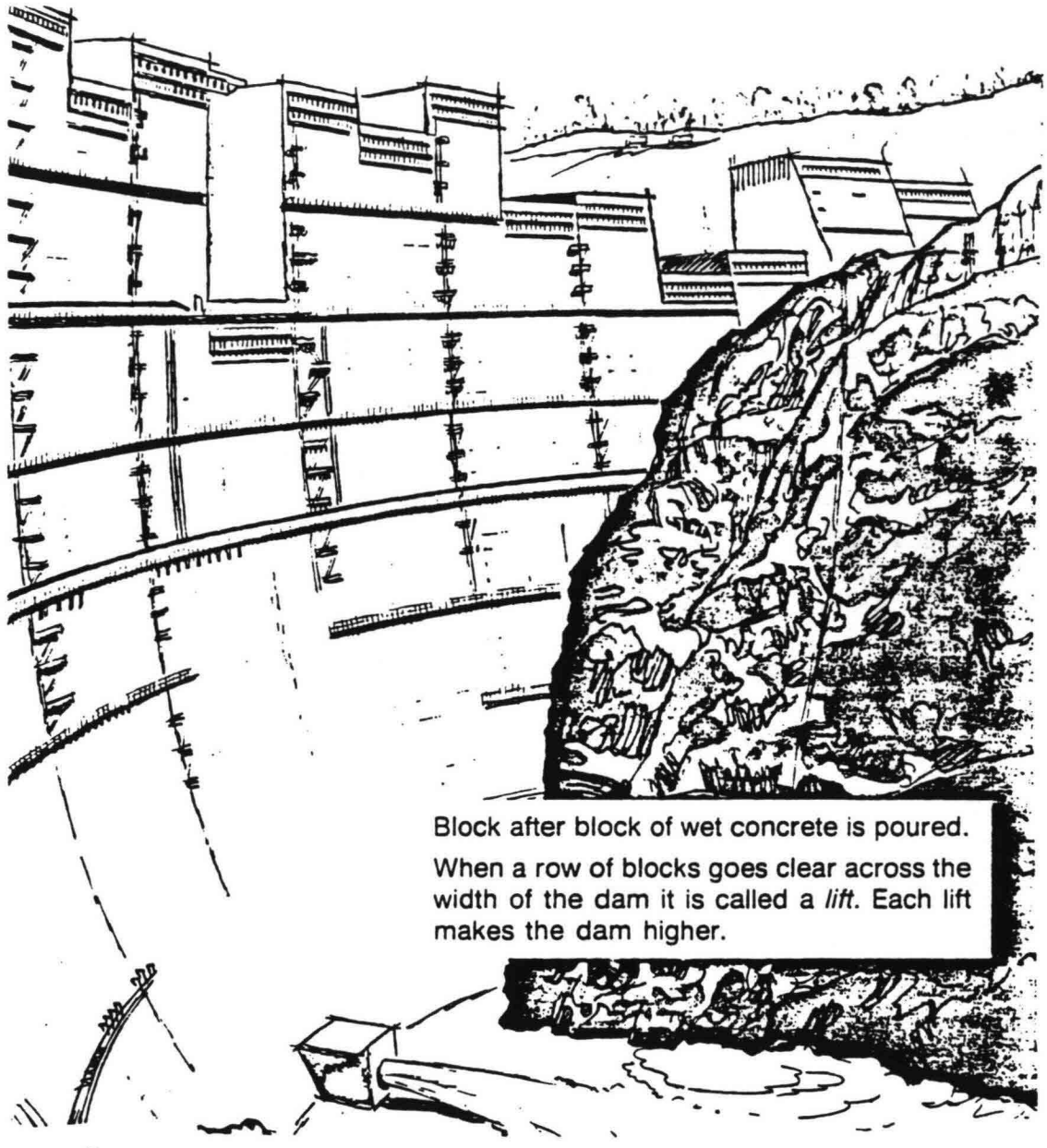
Attached to the big cable are smaller cables that hang straight down. On the end of these smaller cables is a concrete bucket which can be raised or lowered.

CONTROLLING THE CABLEWAYS

The man who operates the cableway sits at a *control panel* in the head tower. By pushing buttons and levers he can move the concrete buckets to any part of the dam.

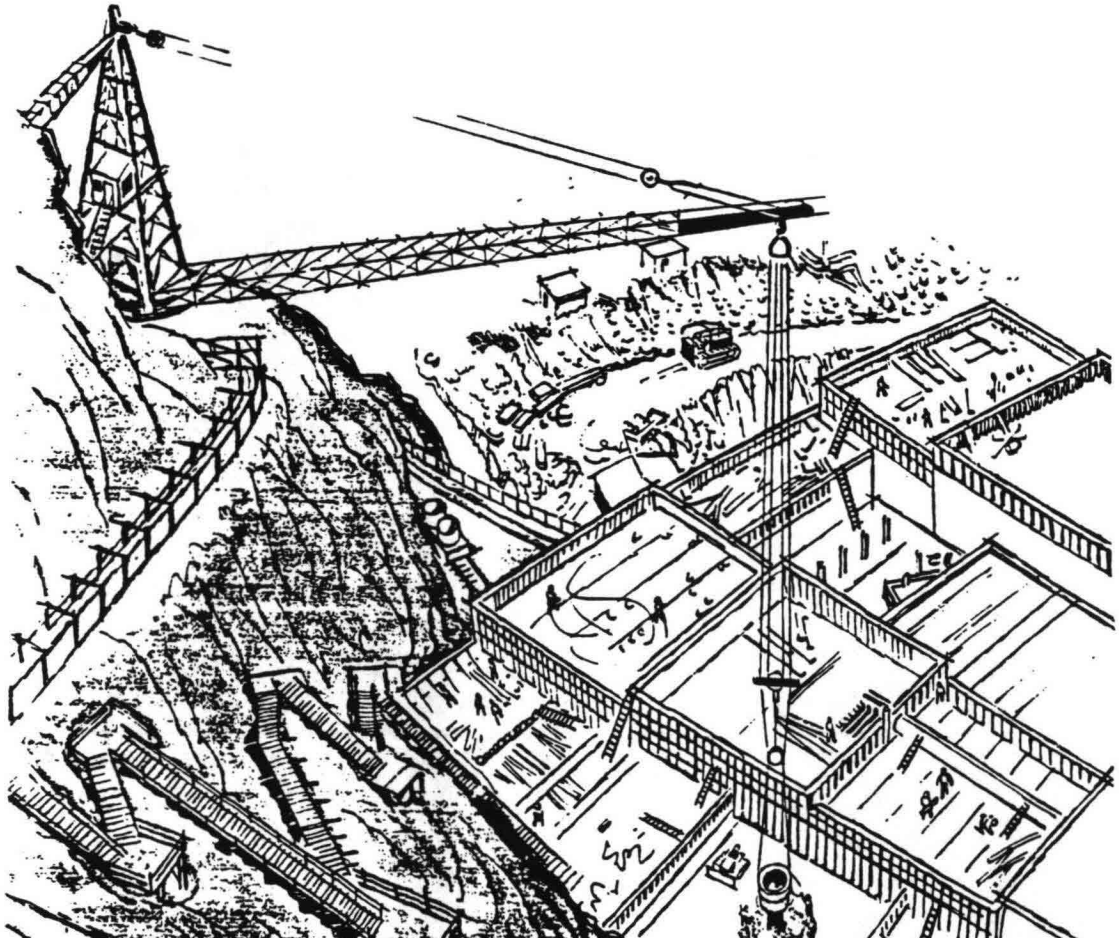
Sometimes the place where he will dump the concrete is far away. So the man may watch where he puts the concrete on a closed-circuit television set.

He can also talk to the men on the dam with a two-way radio.



Block after block of wet concrete is poured.

When a row of blocks goes clear across the width of the dam it is called a *lift*. Each lift makes the dam higher.



15. **Check dams** (erosion control and debris retaining dams)

Structural measures for protection of natural waterways against erosion fall into two types. The first type of structure is known as a grade control structure while the second type is commonly called a bank protection structure. **Grade control structures** are utilized to control the gradient of the waterway channel in a manner that will reduce the velocity of flow and thereby minimize both channel and bank erosion.

The most common grade control structure is the check dam. Check dams are short dams constructed of a wide variety of materials including logs, treated lumber, stone, concrete, and synthetic materials which flatten the slope of the stream and dissipate the downstream toe of the check dam in order to prevent undercutting of the structure. Check dams should be used with caution on streams which are susceptible to flooding, since they reduce flow rates and thus increase the chance of flooding.

Controlling debris flow
Example: volcanic debris, mud flow, etc.

- In the upper reaches (where debris flow takes place):
- check dams (either **'full'** or **'permeable'** to intercept materials - continuous maintenance is required
- embankments and bottom protection of streams.

- In the lower reaches (where sediment transport takes place):
- permeable check dams to intercept material during exceptional events

Controlling debris flow (alpine situation: sediment input can be controlled).

- In the upper reaches (where debris flow takes place):
- **'full'** check dams to decrease bottom slope (reduction of debris flow occurrence) and to stabilize valley slopes (reduction of sediment input from landslide)

- In the lower reaches (where sediment and bedload transport takes place):
- **'permeable'** check dams to intercept material during exceptional events

Usually check dam are constructed with **'windows'** in the wall. The main function is to reduce the water pressure on the structure and conveyance of small flows at low level.

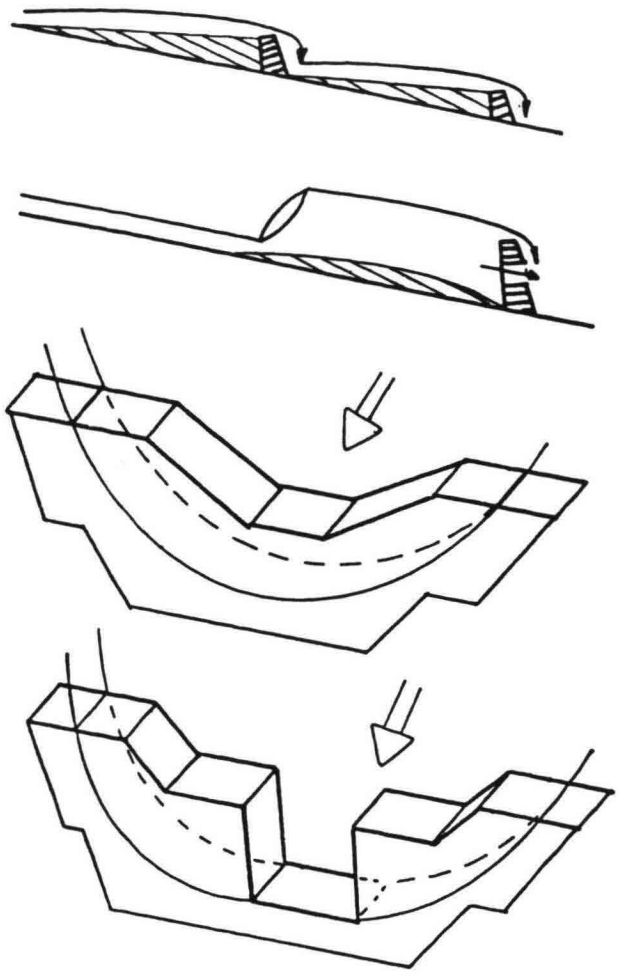


Fig. 15.1. Hydraulic behaviour of "full" and "permeable" check-dams.

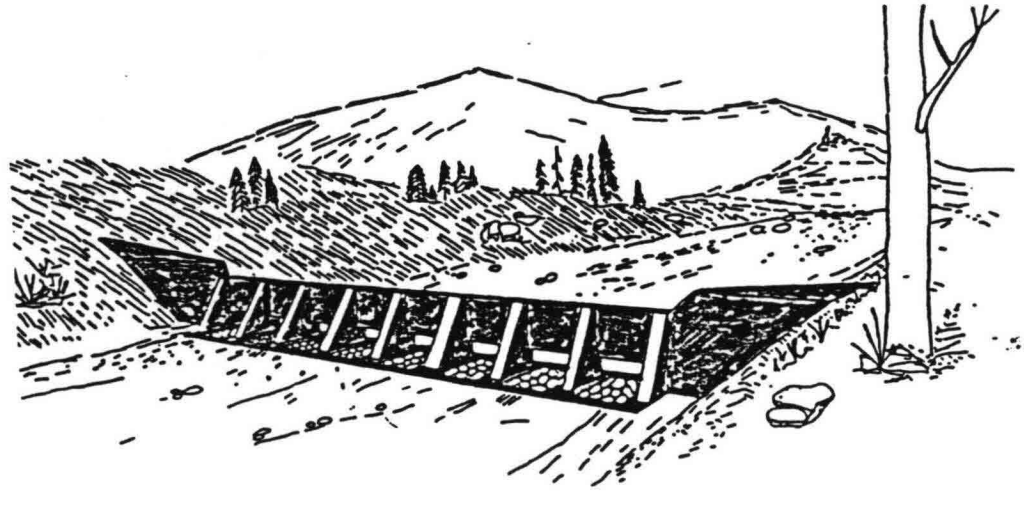


Fig. 15.2. Emergency Dam - In an emergency dam the window should be large enough in order to let the trees through (clogging up). Maintenance is required.

Transport of high concentrations of sediment

Classification:
Motion of sediment-water mixtures at very high concentrations along steep streams.

Classification according to different criteria (morphology, climate, dynamics, etc.)

Classification based on grain size
- mud flow (fine)
- sand-flow (coarser)
- debris-flow (different types of material/sediment)

Classification based on (the origin of) the material:
- volcanic mud-flow/debris flow (Lahar)
- alpine debris flow (sand, gravel, boulders)

Loose terminology
Alpine debris-flow
- Muren (German)
- Colate di fango (Italian)
- Laves torrentielles (French)

(different dialects)

A recent disaster took place in Colombia near the city of Armero. Mud-flow from the volcano the Nevada del Ruiz caused by a melting of snow (fluidization of material by an excess of water) covered the city of Armero a.o. with mud. Many hundreds of people were killed.

Another example of mud-flow of a larger grain size and with big boulders occurred in an Alpine valley in Northern Italy.

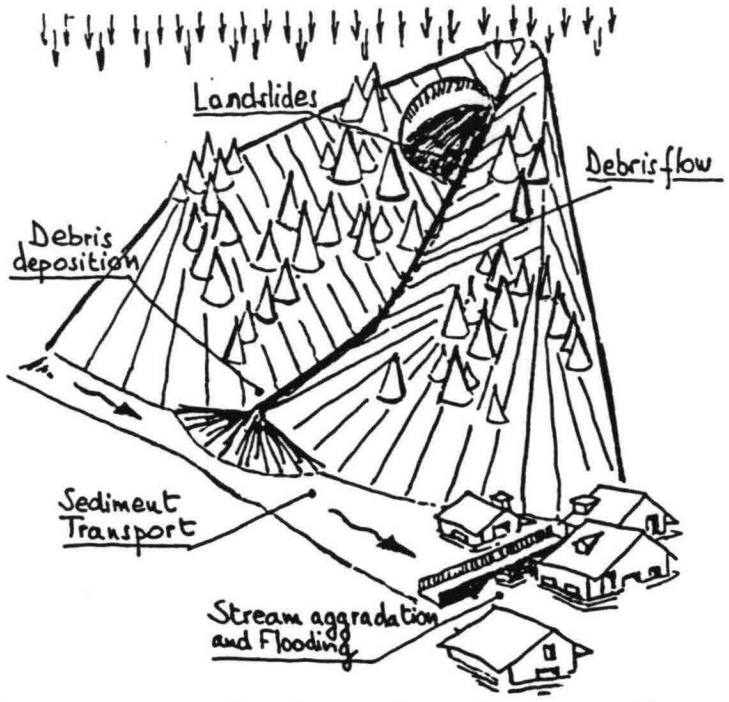
Japan has had experiences with both types of phenomena, volcanic as well as alpine. They cause a lot of problems there because of the rough morphology of the country (high steepness of rivers). Quite a large number of people have been killed as a result of disasters connected with sediment motions.

Depending on the amount of water you may move from a limit state of a dry-slide (steep slope) via a totally saturated slope, to the common bed-load sediment transport (flat slope). In between you find the phenomenon of debris flow.

Typical sequence of debris-flow related phenomena
1) Heavy rainfall (critical conditions in alpine regions : 150 mm/day, 40 mm/hour
2) Landslides in the steepest slopes and clogging of the upper streams;
3) Saturation of streambed, water emergence and debris-flow in the steepest streams
4) Debris deposition, sediment transport, stream aggradation and flooding.

At a certain place the slope is reduced and there you get debris deposition in the main stream. Typical sediment transport in a non-equilibrium situation. There is much more inflow than can be transported. Therefore : aggradation, clogging of bridges and finally flooding (of villages). This is a typical situation in a mountainous area.

Fig. 15.3.



Controlling debris flow (volcanic situation : sediment input cannot be controlled)

- In the upper reaches (where debris flow takes place):
  - check dams (either "full" or "permeable" to intercept materials-continuous maintenance is required
  - embankments and bottom protection of streams

- In the lower reaches (where sediment transport takes place):
  - permeable check-dams to intercept material during exceptional events

Controlling debris flow (alpine situation : sediment input can be controlled)

- In the upper reaches (where debris flow takes place):
  - "full" check dams to decrease bottom slope (reduction of debris flow occurrence) and to stabilize valley slopes (reduction of sediment input from landslides)

- In the lower reaches (where suspended and bedload transport takes place):
  - "permeable" check-dams to intercept material during exceptional events

Structural problems of debris-flow check dams

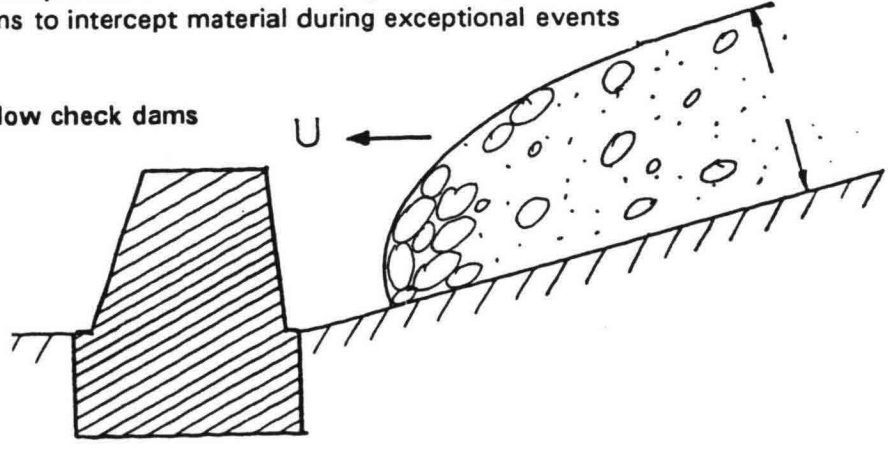


Fig. 15.4

Height : up to 10 m
Velocity : up to 20 m/s
Concentration : " 60%
Boulder size : 2 - 3 m
Local pressure : " 600 N/cm^2
(instantaneous) (6 MPa) (60 m of water)

Swiss experience: assume hydrostatic pressure distribution with a "virtual" density of debris 7 - 10 times larger than water. This Swiss rule is simple and safe but such a large dam would be very expensive.

16. Pumping stations

Pumping stations are necessary for the discharge of water from a lower level to a higher level. In cases where the natural discharge (by gravity) is too small or sometimes impossible pumping stations are used. Especially in tidal regions, the combination of sluice (natural discharge) and pumping-stations (forced discharge) are customary: during low tide the sluice is in operation, during high tide the pumping station.

A pumping station has three elements:

1. The **building**, in which the pumps and (diesel- or electro) engines are installed.
2. The **suction pipe** connected to the lower head. A trash rack is usually mounted in front of the suction pipe.
3. The **discharge pipes** connected to the upper head.
   The pipes are provided with valves to prevent the water flowing back if the pump is not working (non-return valve, flap gate).

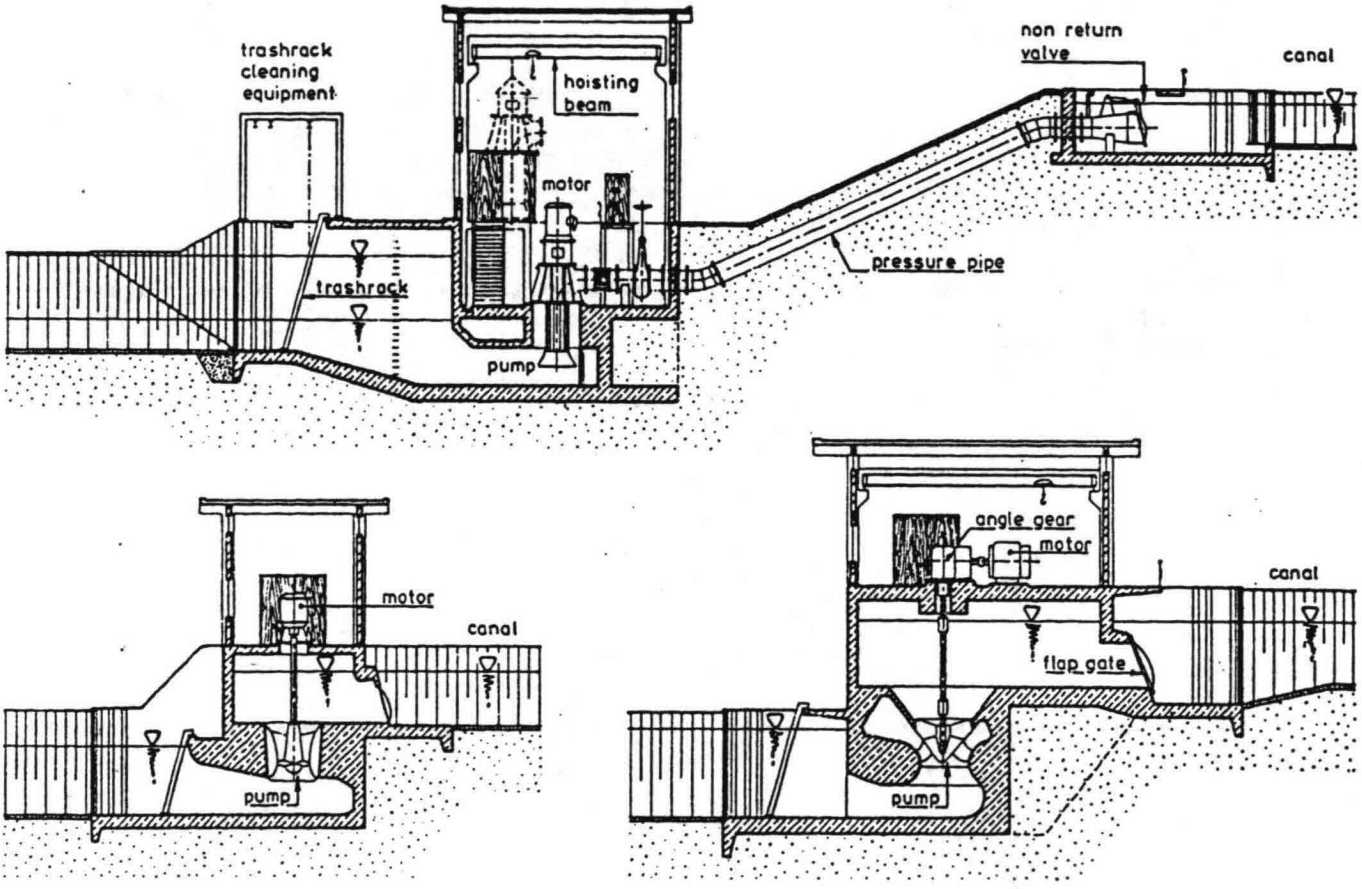


Fig. 16.1. Typical low-head pumping stations.

Windmills

Windmills in Holland served two purposes, to **drain** (**polder mills**) or to carry out some domestic or industrial task (**industrial mills** or grinding mills). The latter came first and may well have been in use by the 11C although the first record is one of the 13C; the first poldermill was recorded in 1414.

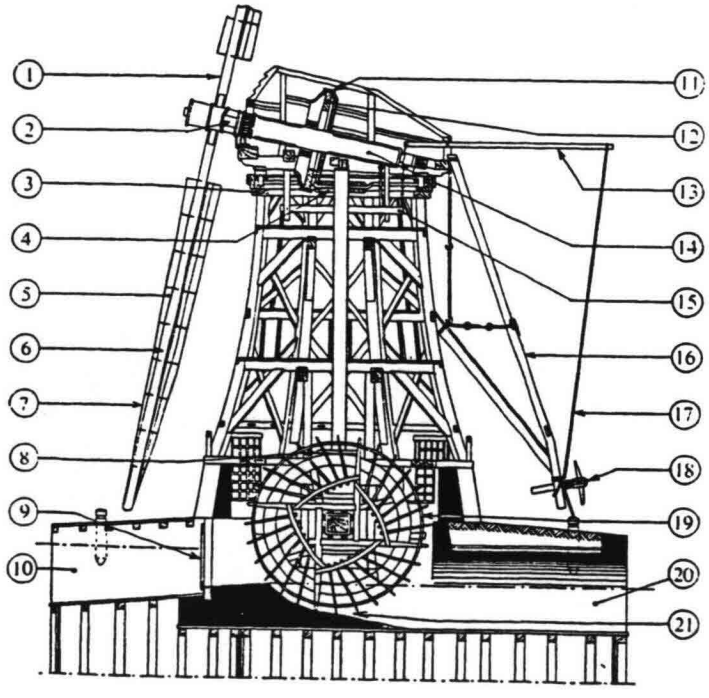
The early industrial mills turned as one body on a massive wooden post to face the wind; this type was not suitable for drainage as the **scoop-wheel** (a wheel with several buckets) had to remain in a fixed position. The solution was found in the "Hollow Post" mill which enabled the shaft to be carried down within the post.

Polder mills at first operated under the disadvantage that a scoop-wheel could not raise water more than about 2 metres. A system evolved by which mills were used **in rows** to increase pumping heights. The great disadvantage of the "Hollow Post" type was that to get the sails to face the wind the entire structure had to be turned on its post, a cumbersome process. Change came in the early 16C with the invention of the rotating cap. This change gave way to the **brick Tower type**. These, in order to catch the wind, especially in towns, growing taller and taller; an additional advantage being the accommodation and storage space now provided.

Windmills continued in wide use until the turn of the 19-20C when steam and oil began to take over. At this time there were some 9000 windmills in Holland of which around 950 still exist. These can be seen in most parts of Holland, but mostly in the west, in Friesland and in Groningen. Three places at which there are good groups are Kinderdijk, Zaanse Schans and the town of Schiedam.

For centuries the setting of the sails of windmills has been a method of conveying messages (birth, mourning, etc.) and it is also the custom on special occasions to hang the sails with traditional ornaments.

Fig. 16.2.



Haarlemmermeer (Haarlem lake)

Several centuries ago the low-lying fenlands southwest of Amsterdam contained a number of large lakes, but by the mid-16th century flooding had converted these into one lake. This lake was so large (18,000 ha) that in two storms of 1836 the waters reached the gates of Amsterdam and flooded parts of Leiden. As early as 1643 Jan Leeghwater had proposed reclamation, but only after the threat to the city of Amsterdam became obvious King William I ordered action. First a 60 kilometre long dike was built encircling the inland sea and concrete plans to drain the lake were made.

The major problem at the time was whether **windmills** should be used or the newest advance in technology, the **steam pump**. In view of the limited raising capacity of windmills (1 - 1½ metres) it would have been necessary to group these together in series of 3 mills or "stepped" so as to obtain the required lift of 5 meters from the lake bottom to the level of the Ring Canal. In all about 160 windmills would have been required to complete the job. On the other hand, a **steam-driven pumping station** could raise the water 5 metres with a single stroke, but steam power was a very new unknown phenomenon. Finally in 1838 the decision was made to drain the Haarlemmermeer by means of three steam pumping stations. These three pumping stations, the **"Leeghwater"**, the **"Lynden"** and the **"Cruquius"** (the latinized name of the surveyor Jacobus de Kruik = Jacob the Jar, born in Delft in 1678), worked non-stop for three years from 1849 to 1852 and pumped up 832 million cu. m. of water.

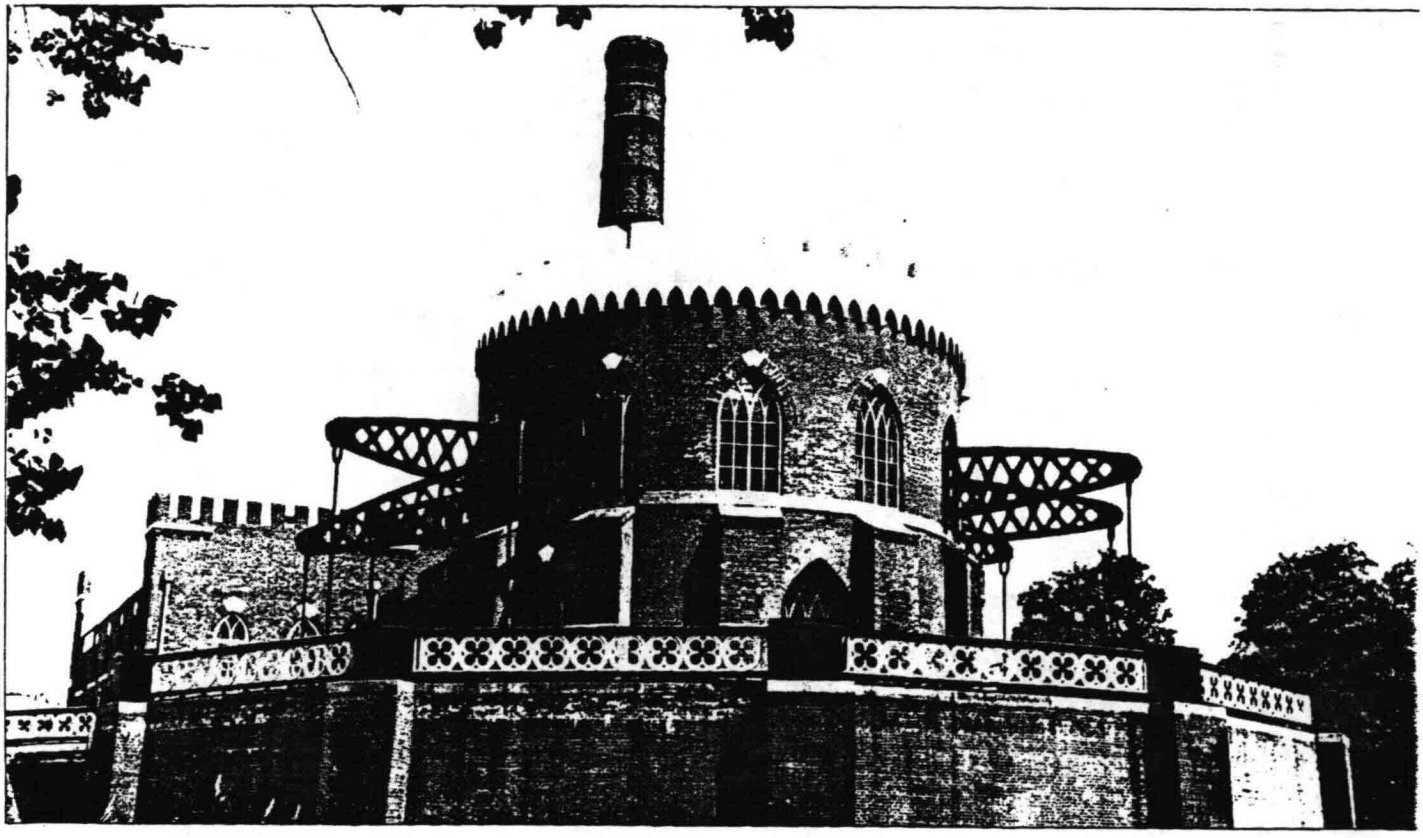


Fig. 16.3. Pumping station "The Cruquius".

The Cruquius, which has become a monument, is a particular interesting pumping station. It rests on a foundation of 1,110 piles and was in operation for a total of 84 years (1849-1933). As can still be seen today the Cruquius pumping station has eight beams each of which pumped up 8 cu. m. (or 8000 litres) of water on each stroke - an enormous pumping capacity for that time. At a rate of five strokes per minute this meant that 320 cu.m. were pumped each minute. The design of the engine was based on the Cornwall-engine as used at that time in the mines of Cornwall (U.K.).

The Cruquius has been an important step in the development of Mechanical Engineering and was on June 19, 1991 presented with the "International Historic Mechanical Engineering Landmark" of the American Society of Mechanical Engineers (ASME).

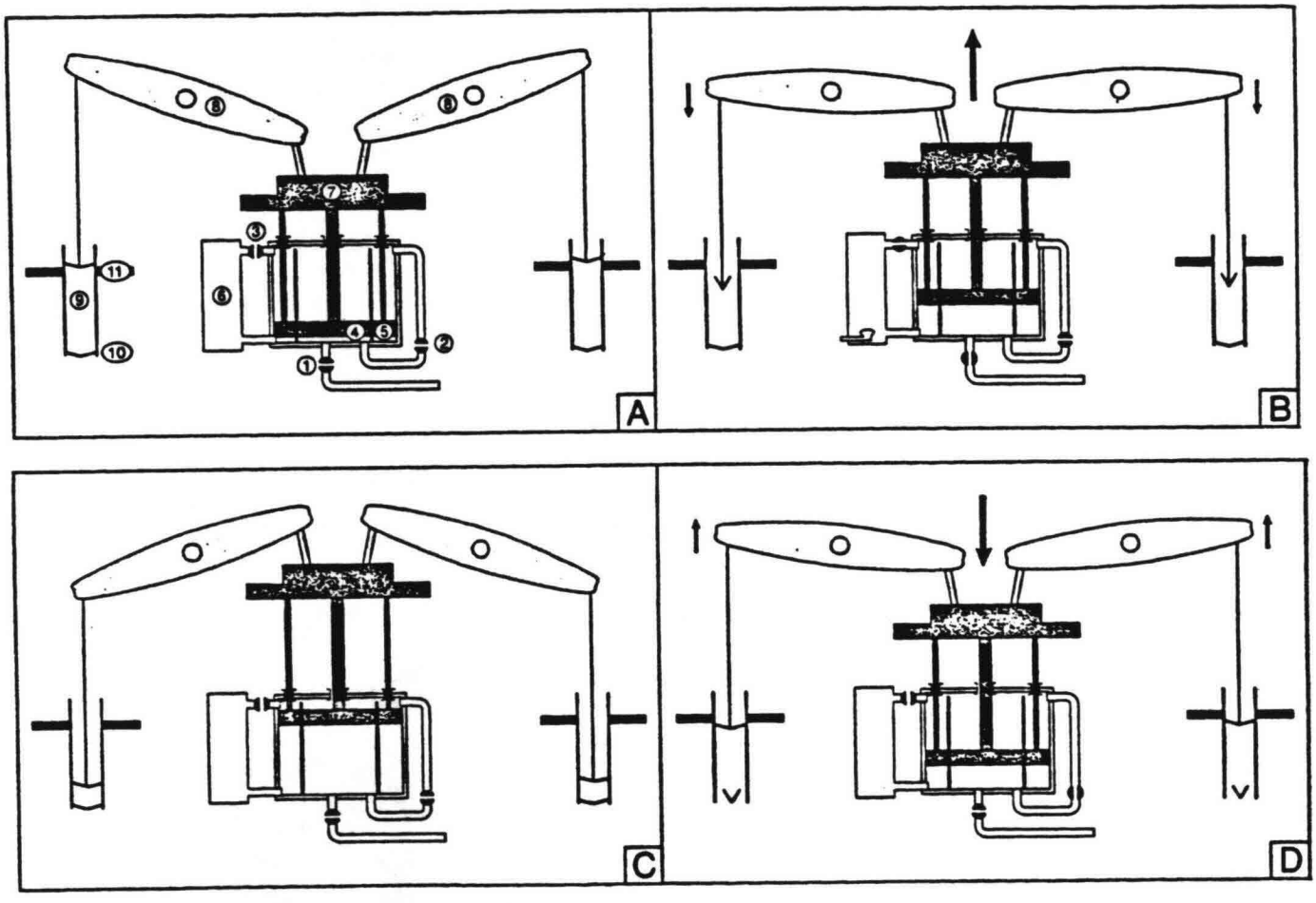


Fig. 16.4. Operation of the pumping station "Cruquius".
1. steam-valve, 2. balance-valve, 3. exhaust-valve, 4. centre-piston (high pressure), 5. ring piston (low pressure), 6. condensor, 7. head, 8. beam, 9. pump, 10. water in polder, 11. timber floor.
The water was raised 3 metres by pumps from the polder to a timber floor built around the pumping station. Sluice gates which could only open outwards released the water into the ring canal which had been dug around the Haarlemmermeer. The engine has two cylinders, one in the middle with one piston-rod surrounded by a ring cylinder with four piston-rods. All five piston-rods are coupled to one head (7). To this head eight beams are coupled which drive the eight suction pumps, diameter 1,85 m. The pumps are open from above. One engine operated the eight pumps.

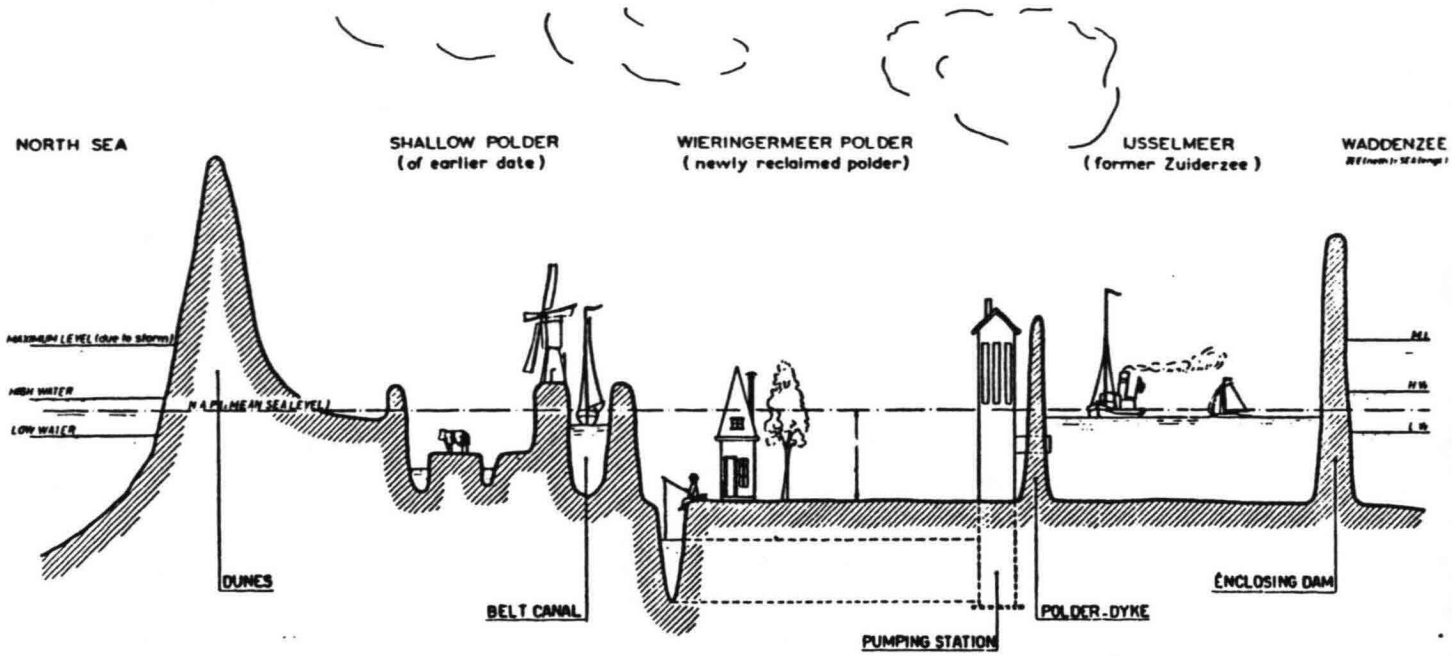


Fig. 16.5. Cross section of Noord Hollands's Polder-landscape.

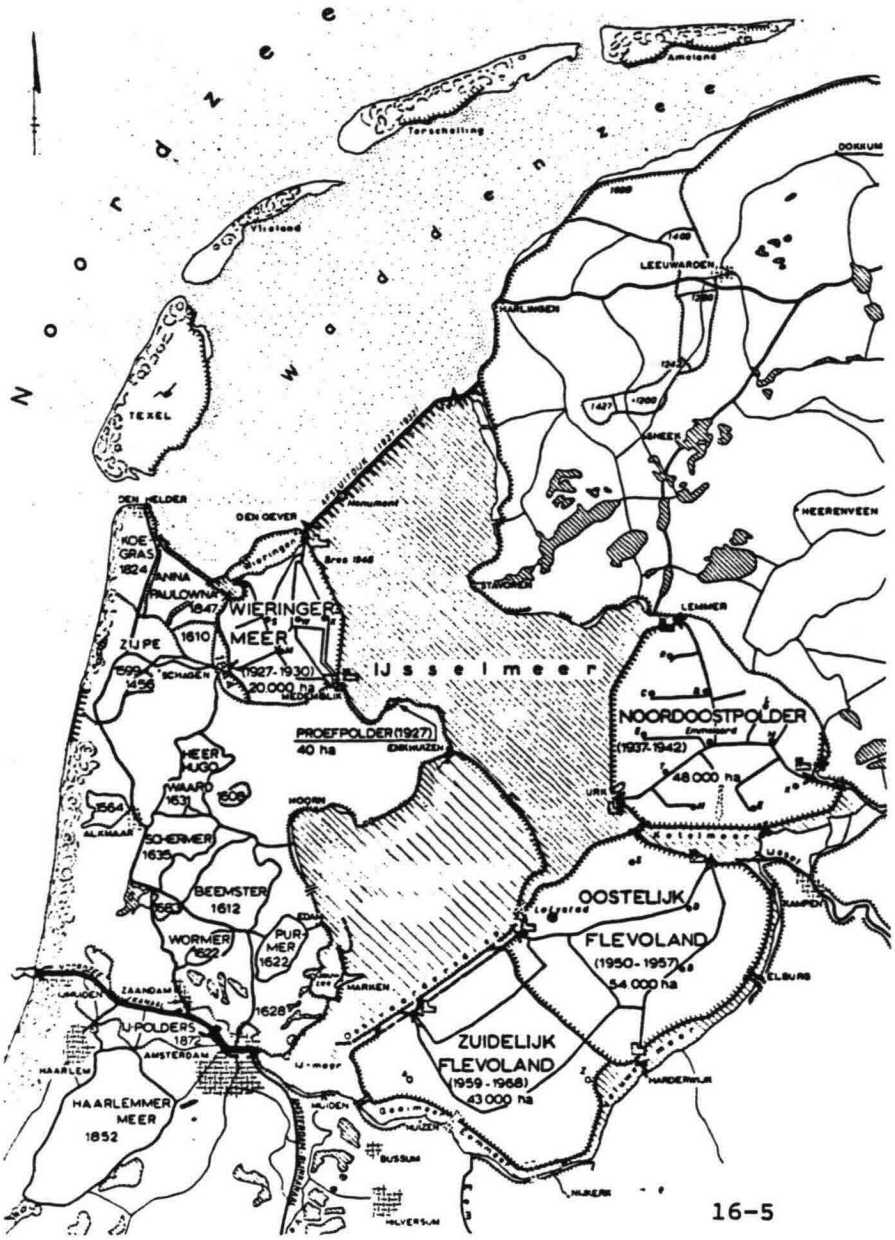


Fig. 16.6.
The Zuyder Zee works showing the newly reclaimed polders: Wieringermeer, North East polder, Eastern Flevoland, Southern Flevoland, the Barrierdam and lakes in the province of North Holland which has been drained mainly in the 17th century.

Reclamation works:	
Beemster	1612
Wormer	1622
Purmer	1622
Heer Hugowaard	1631
Schermer	1635
Anna Paulowna	1847
Haarlemmermeer	1842
Y-polders	1872
Proef(trial) polder	1927
Wieringermeer	1930
Noordoostpolder (North East polder)	1942
Eastern Flevoland	1957
Southern Flevoland	1968

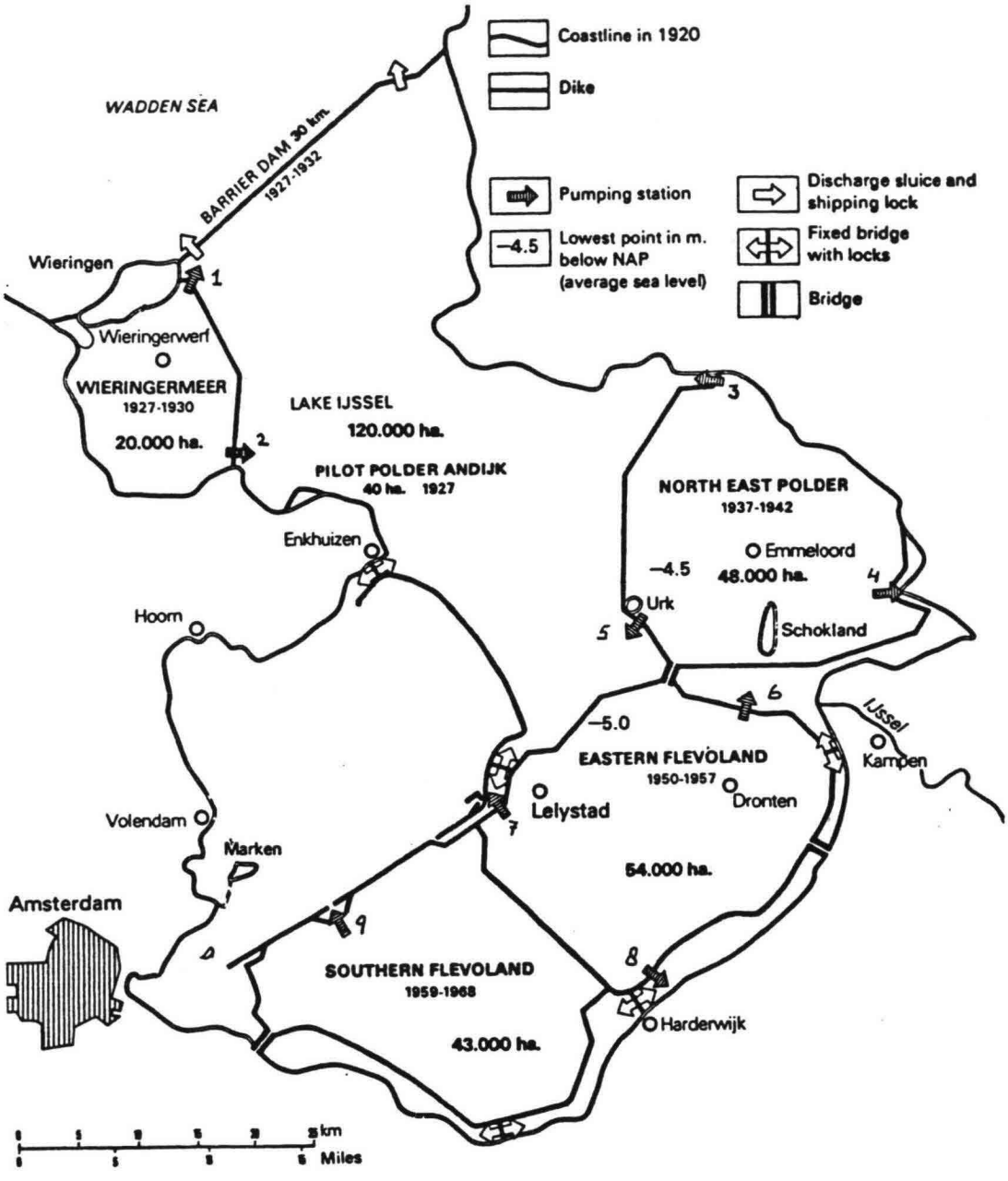


Fig. 16.7. The Zuyder Zee Works showing the various pumping stations built.

Polder	Name	Operational	Type	Engine	Capacity hp	Discharge m^3/min	He m
Wieringermeer	Leemans (1)	1929	CH	diesel	1200	250	4.4/5.9
	Lely (2)	1929	CV	electric	3000	400	5.1/5.9
North East Polder	Buma (3)	1941	CV	electric	2550	520	5.5
	Smeenge (4)	1941	S	electric	1600	600	4.3
	Vissering (5)	1942	CV	diesel	3210	545	5.5
Eastern Flevoland	Colijn (6)	1956	CV	electric	2700	580/500	5.0/6.0
	Lovink (8)	1956	CV	electric	1800	580	5.0
	Wortman (7)	1956	CV	diesel	4000	500	6.0
Western Flevoland	De Blocq van Kuffeler (9)	1967	CV	diesel	4800	845/700	5.0/6.0

CH = centrifugal pump + horizontal shaft, CV = centrifugal pump + vertical shaft, S = screw pump.
Forty years after the Lely and Leemans started operations 26 pumps have been commissioned with a total discharge capacity of 13,500 m^3/min, that is 10% of the average Rhine discharge.

The last big pumping station driven by steampower, the **'Wouda'**, west of Lemmer in Friesland, was built in 1920. For the drainage of the Wieringermeer diesel and electric driven engines were selected. For reliability reasons two pumping stations were chosen instead of one only near Medemblik: in the deepest part of the polder and located at the leeside of the dominant westerly wind. In order not to become fully dependent on the availability of coal (electricity) or diesel, one pumping station is diesel-driven the other one electrical-driven. As power grids at the time were not existing in Wieringen, Urk, Lelystad, Oostvaardersdiep diesel-engines were the only option at these locations.

The pumptype used for the **'Lely'** pumping station in the Wieringermeer suited best; this type was used for the other pumping stations that were built later (with the exception of the 'Smeenge'). Hereby the reinforced concrete of the building, the suction pipe, the pump house and the discharge pipe is integrated into one structure. The only steel parts from the engines, the gearbox, the shaft and the impeller (see Fig. 16.7).

All pumping stations have been built in **building pits**, in the sea, outside the dikes of the old land. The building pit was encircled by a temporarily dike and kept dry by pumps. After the completion of the structure, the polder dike is extended over the discharge pipes after which the dikes of the building pit are excavated. Before the start of the pumping the water out of the new polder, the inside water level is much higher than the future polder level. The consequence is a high uplift pressure during the initial stage. In case of the **'Leemans'** the room reserved for a possible third pump was ballasted during this phase.

The pumping stations have **trashrack** before the opening of the suction pipe to prevent floating -debris entering the pump system. These racks have to be cleaned regularly. Nowadays this is done by mechanical installations.

Due to subsidence of the soil after becoming dry the final polder level will be lower (some 0.5-0.8 m) than the initial polder level. As the pumping stations are founded on piles and the level therefore fixed the design level of the suction mouths, in particular the roof part, has to be based on the final polder level.

Some 700 million m^3 water had to be removed from the **Wieringermeer**; 85 millions m^3 could be discharged by gravity through the Oostoeversluice, the remainder had to be pumped. As the discharge head was rather small in the beginning special impellers were fitted suitable for this low heads. The water level was lowered by about 20 mm per day. It took eight months (January-August 1930) to pump the water out.

North-east polder: 1500 million m^3 water, 60 million m^3 by gravity discharge, 1000 million m^3 by pumping station **'Buma'** from Jan. 1941 to April 1942 and 375 million m^3 by pumping station **'Smeenge'**. The water level was lowered from - 0.07 m to - 4.00 m.

Eastern Flevoland : 1600 million m^3 water removed by three pumping stations - Sept. 1956 to June 1957 (-4 m.).

Southern Flevoland : 1400 million m^3. water by one pumping station in 1967 - 1968.

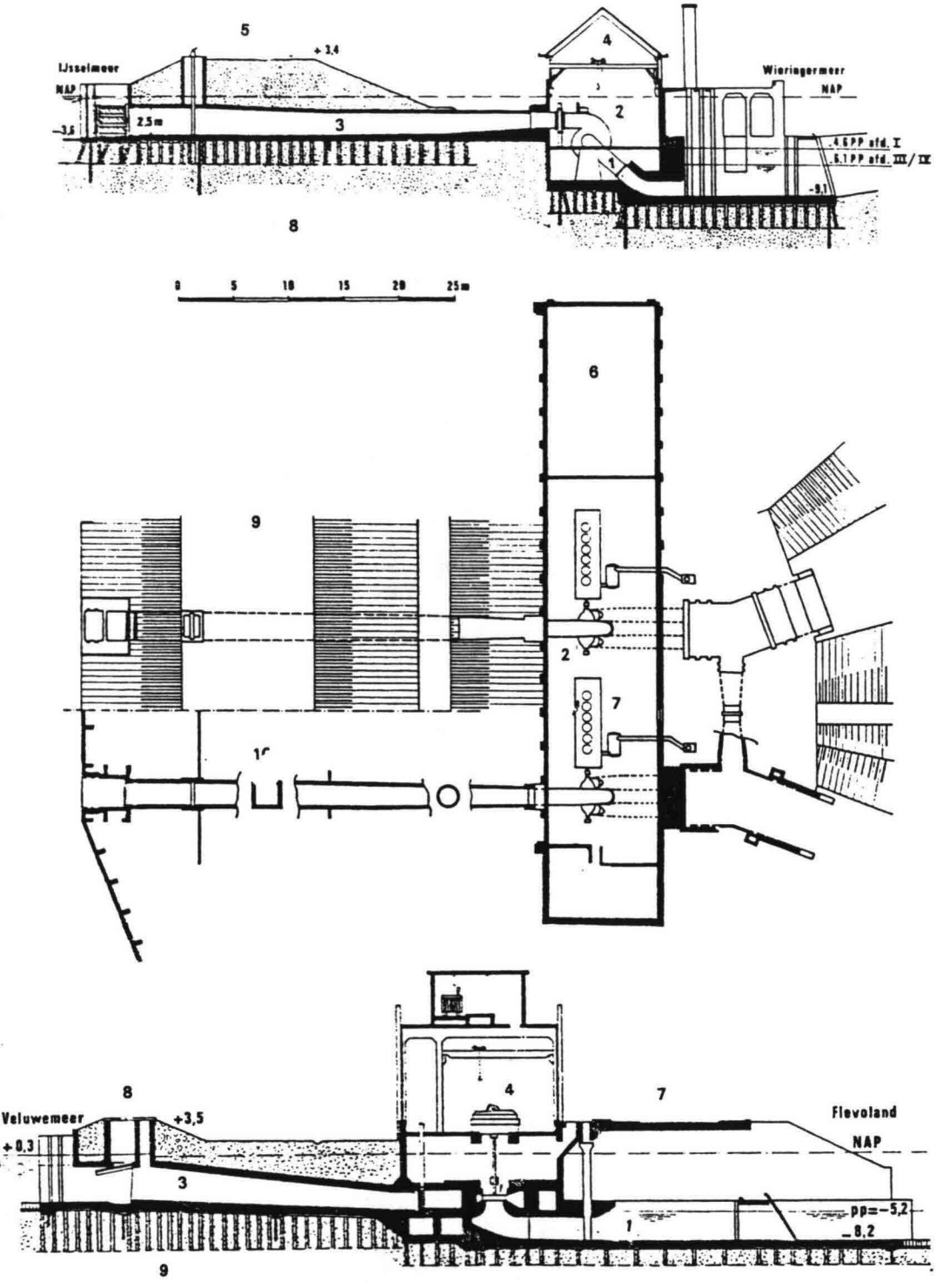


Fig. 16.8.
Pumping station Leemans.
Horizontal pump shaft

1. suction pipe
2. pump
3. discharge pipe
4. travelling crane
5. dike with road
6. room for third pump
7. engine
8. longitudinal section
9. cross section over engine hall
10. cross section over discharge pipe and above the suction-pipes.

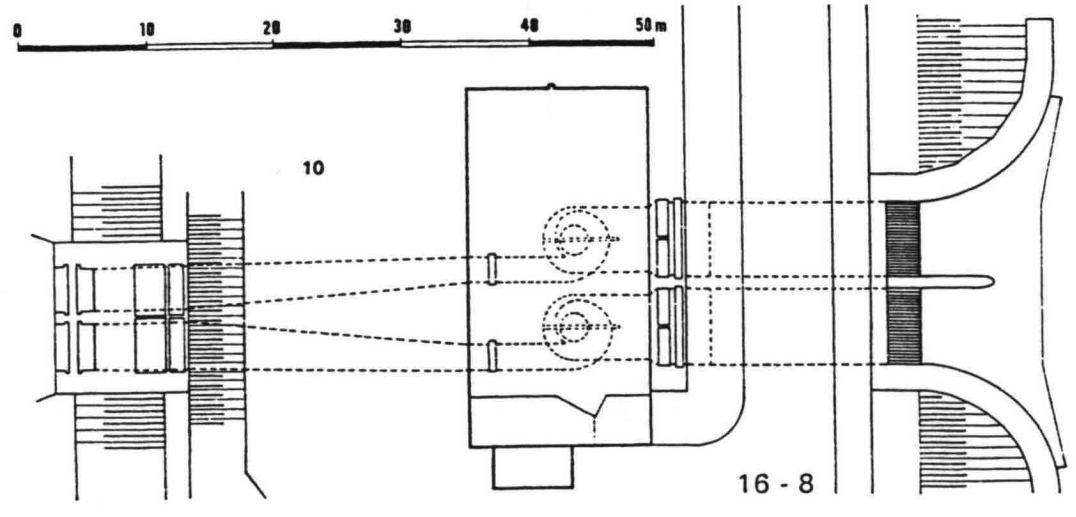


Fig. 16.9.
Pumping station Lovink.
Vertical pump shaft

1. suction pipe
2. pump
3. discharge pipe
4. engine
5. travelling crane
6. transformator
7. road
8. dike
9. longitudinal section
10. cross section over engine hall.

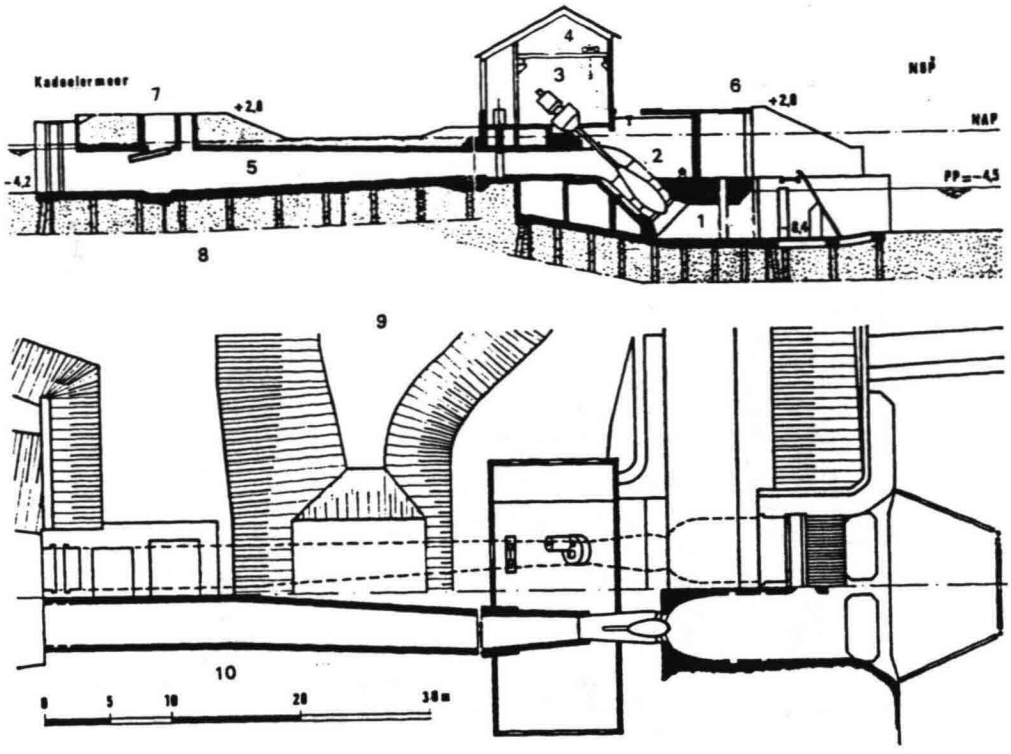


Fig. 16.10.
Pumping station Smeenge.
Inclined pump shaft

1. suction pipe
2. pump
3. engine
4. travelling crane
5. discharge pipe
6. bridge
7. quaywall
8. longitudinal section
9. cross section over engine hall
10. cross section over suction and discharge pipe.

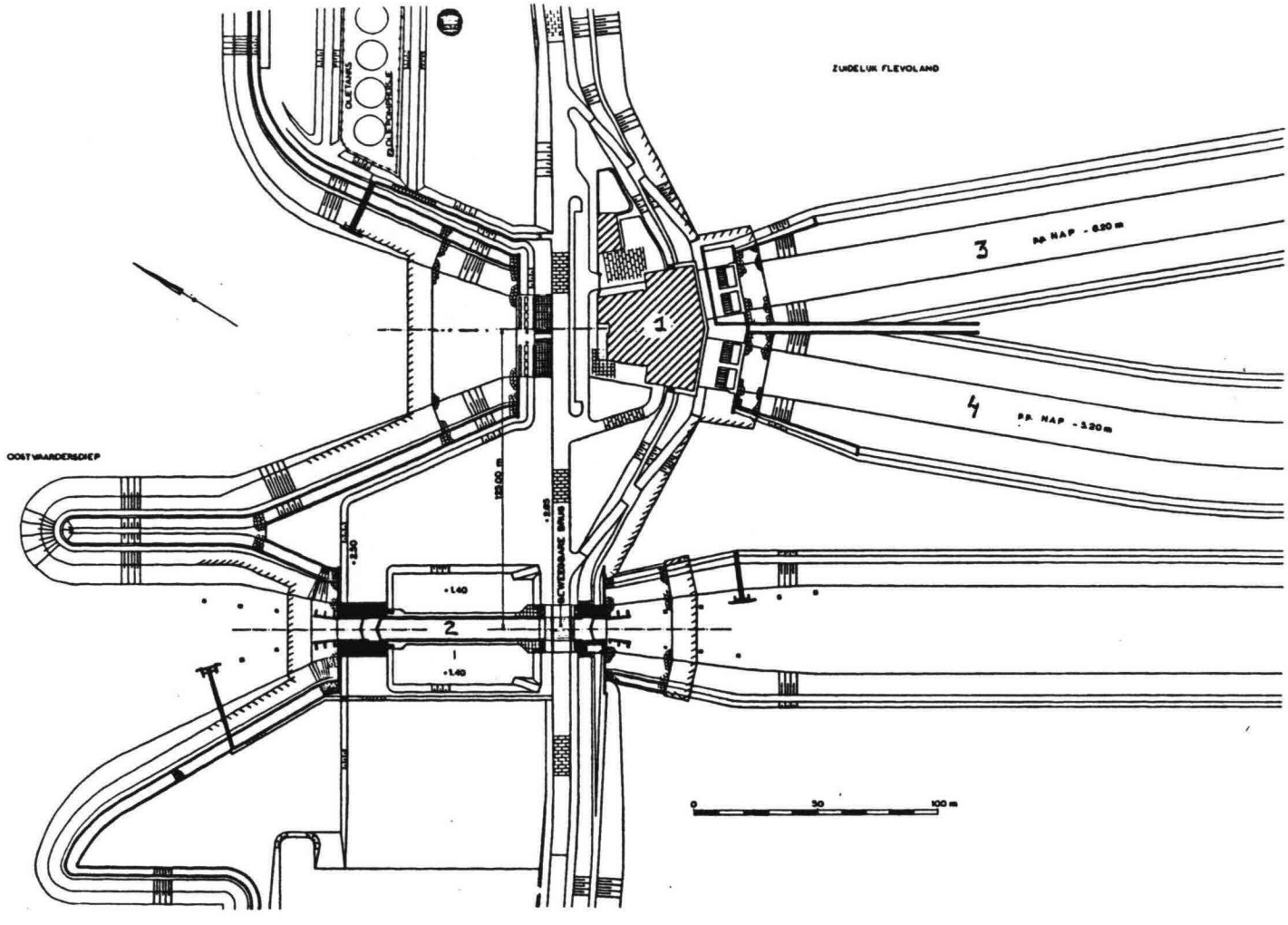


Fig. 16.11. Situation diesel pumping station "De Blocq van Kuffeler" in Southern Flevoland. 1 = pumping station, 2 = navigation lock, 3 = main canal low area, 4 = main canal high area..

The diesel pumping station "De Blocq van Kuffeler" drains the 43,000 ha polder area of Southern Flevoland. This polder consists of two area, a high section with a polder level of - 5.20 and a low section with a polder level of - 6.20 m. Both areas are connected with different main canals to the pumping station. The (design) level of the outside water is - 0.20 m.

Pumps : 4 identical units - 2 for each section; normal operation 85 rev./min.
high section static head 5.2 m, capacity 14 m^3/s
low section static head 6.2 m, capacity 11.6 m^3/s
total capacity of 4 pumps : 51.5 m^3/s equal to 10.35 mm on polder area per 24 hours or 1.2 litre/sec per ha.

Type of pump : screw-centrifugal with vertical shaft, pump - casing of reinforced concrete spared in foundation of building; diameter of impeller : 2930 mm; weight : 7500 kg.

Diesel engines : 6-cylinder 4-tact werkspoor TMAB 396 - capacity 883 kW (1200 HP) by 259 revs/min. Gearing-box (Wülfel) connecting horizontal motor-shaft 259 rpm with vertical pump shaft 85 rpm. From October 18th, 1967 till May 29, 1968 the water was pumped out of the polder. The water level was lowered in this period from about N.A.P. to 5 metres below. During the lowering of the first 2 metres the rotation speed was reduced, the discharge was approx. 80 cum/s.

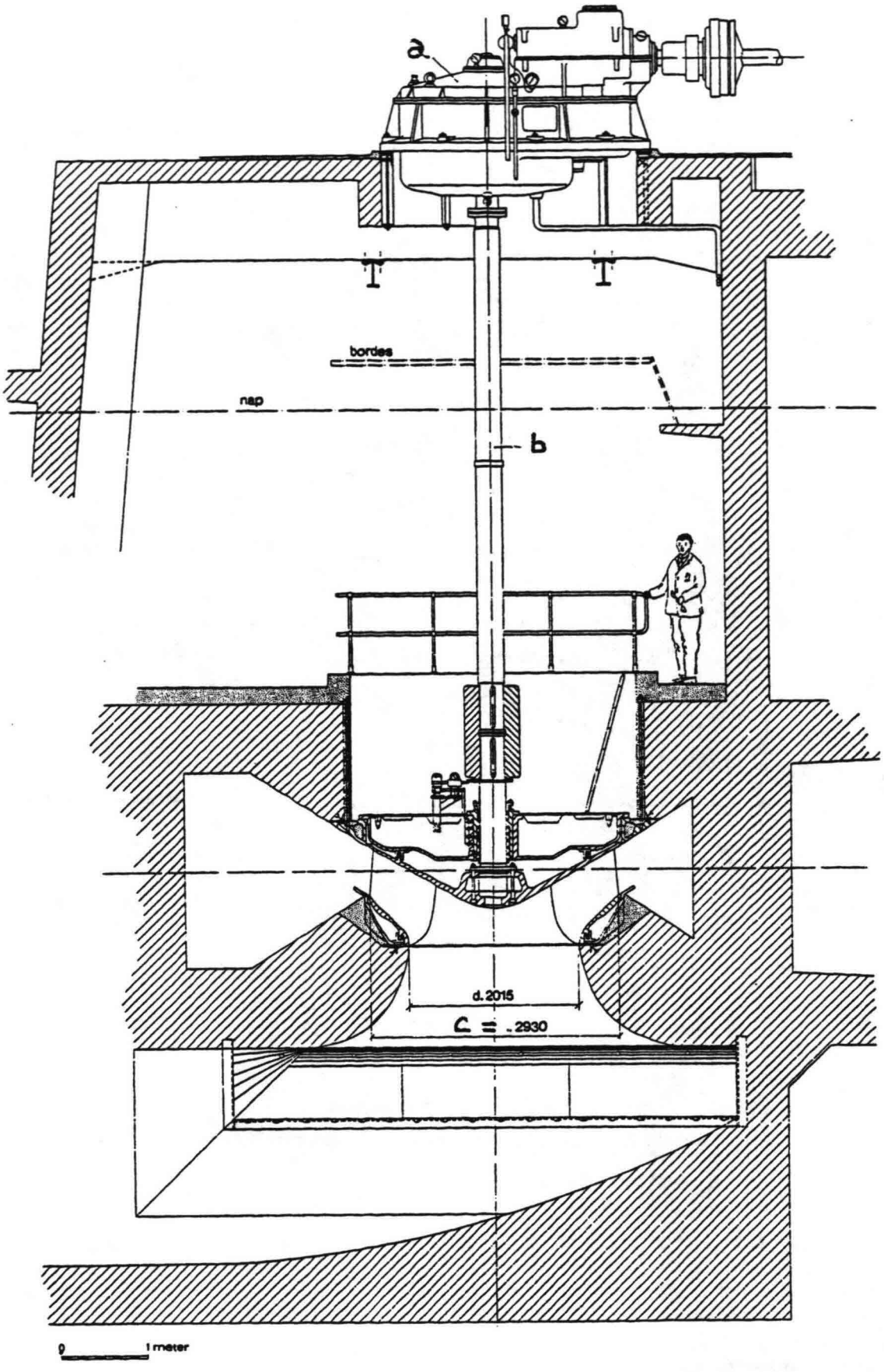


Fig. 16.12. Pumping station "de Block van Kuffeler", Flevoland, the Netherlands.
Arrangement gearbox and centrifugal pump: a = gearbox, b = shaft, c = diameter impeller (2930 mm), diesel engine: capacity 1200 hp at 245 rpm, centrifugal pump: discharge 800 m^3/min., average head 6m, 65 revs per min.

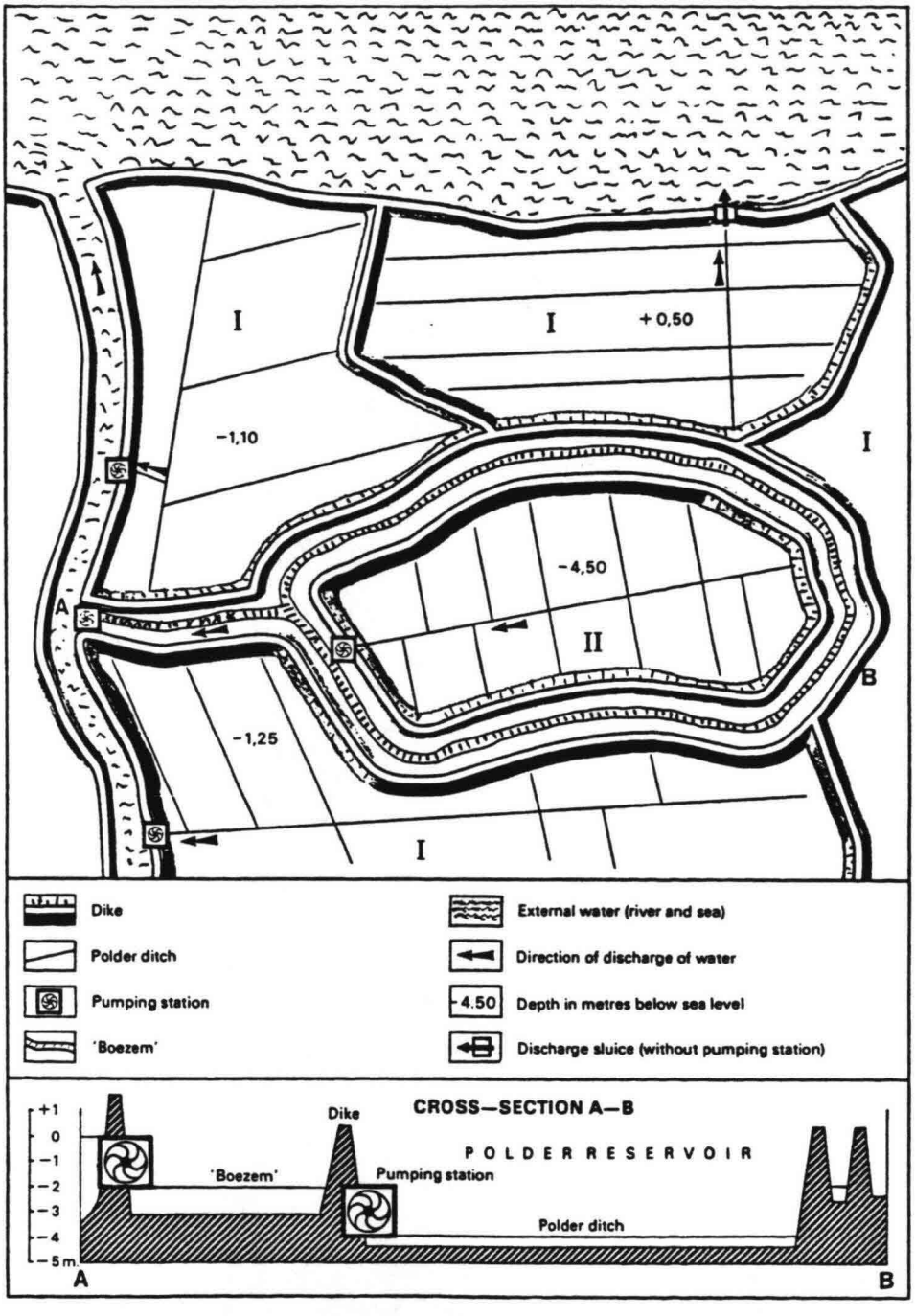


Fig. 16.13.
Drainage techniques of polders.

Two main types of polder can be distinguished on the map:

I - polders surrounded by a dike and

II - a polder surrounded by two dikes separated by a ring canal

To the first type belong, inter alia, the river and marine polders, situated respectively along a river or on a coast. Here the surplus water can generally be discharge directly to the 'outside water' (sea or river), with a pump and/or discharge sluice.

To the second type belong the drained lakes, which are usually very low-lying polders. The inner dike, situated around the former lake, is called the ring dike. It is surrounded by the ring canal, which borders the dikes of the neighbouring polders.

The cross section shows a much-applied system of water control. Each polder has a specified groundwater level relative to ground level - the polder datum. The desired level is determined mainly by the use to be made of the ground. If the water rises too high at a particular moment the pump is started and the surplus water is raised to the "boezem", situated at a higher level. From there the surplus water is discharged into the outer water, either naturally or by further pumping.

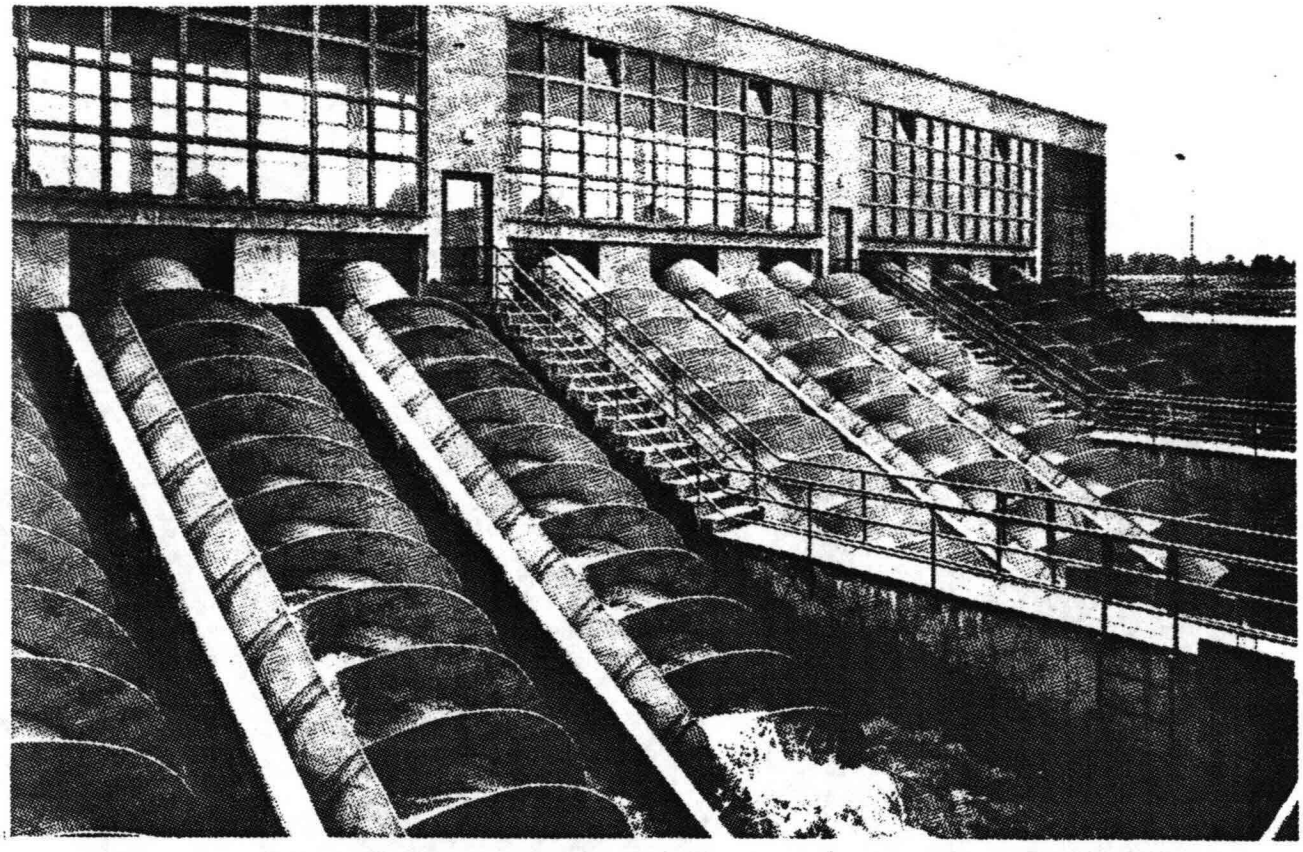


Fig. 16.14. Pumping station of the city of Köln (Germany) with Archimedean screw-type pumps. The pumping station is used as main station of the sewerage system; it has a total capacity of 20,000 litre/s divided over three canals and a total installed engine capacity of 1800 hp.

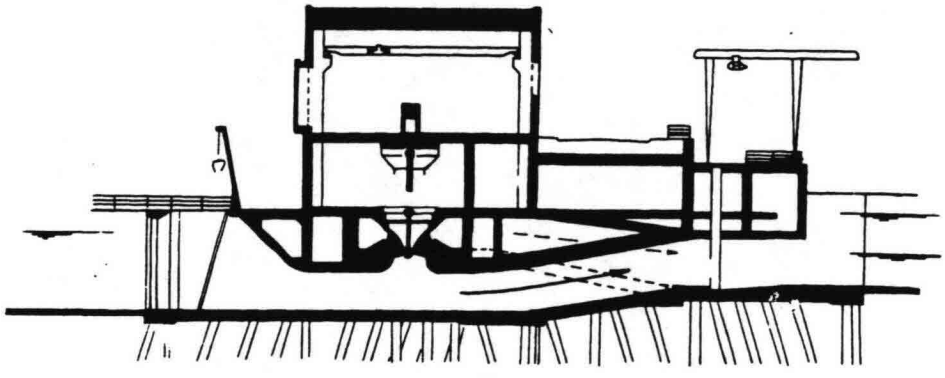


Fig. 16.15. Pumping station with free discharge (The Netherlands). As a result of settlement of the soil (subsidence) and the raising of the average sea level drainage by discharge sluices was not adequate anymore. This has led to the combination of natural discharge by gravity and pumping. This type can be seen along the large rivers and the large pumping stations along the coast.

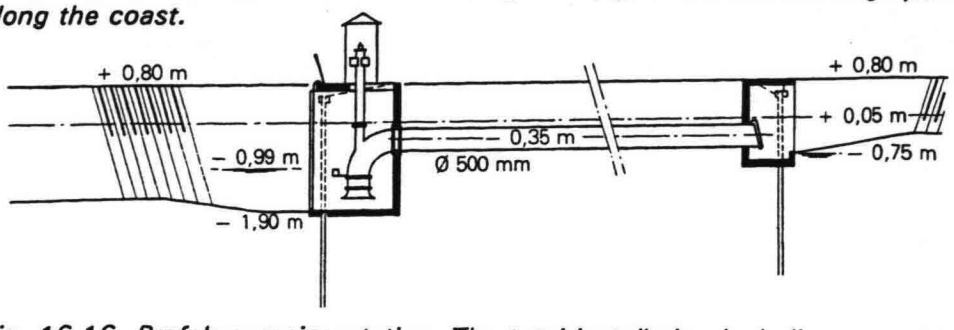


Fig. 16.16. Prefab pumping station. The total installation including pumps and machinery can now be delivered and placed by one supplier up to a capacity of 100 m³/min. They work automatically by remote control (unmanned).

17. Water power (hydropower) stations (hydroelectric)

Water power schemes are some of the largest, most expensive and most interesting civil engineering structures. Famous schemes are the Aswan High Dam across the Nile in Upper Egypt, the Volta River project in West Africa or the Snowy Mountains schemes (Australia). Only some 10-12% of the water power resources of our globe are at present being utilized.

Hydropower has been developed together with the electrification. The hydraulic energy of the water is transformed into mechanical energy by turbines. These turbines drive the generators that produce electricity. Famous types of turbines were developed by **Francis** (1849), **Pelton** (1890) and **Kaplan** (1913).

Classification of Hydropower Stations

1. Conventional hydropower stations : using the energy of flowing water
2. Tidal hydropower stations : energy from tidal currents
3. Wave energy (until present only experimental)
4. Depression power plants : the head available is due to elevations of areas located below main water body
5. Pumped storage schemes : conversion of hydraulic energy in electric energy and vice-versa

Quality of energy produced (conventional hydropower)

1. Run-of-river (with or without pondage)
   There is no storage of water, the energy production is therefore a function of the river flows. Other sources of electricity are needed during the dry season.
   This means that the produced energy has a low quality as you cannot plan generation.

2. Storage plants for equalizing
   Pondage = small scale storage
   Can provide firm power the whole year (base load)

3. Storage plants for peaking
   The storage capacity is large enough to transfer the fluctuation of the incoming water to the fluctuation of the power demand.

Available power :

$$P_A = \rho g \cdot Q_n \cdot H_n \cdot \eta$$

Q_A = net discharge
= total discharge - min. required river discharge downstream - losses
H_A = net head
= gross head - friction losses (intake losses, rake track losses)
η = overall efficiency
$= \eta_{turbine} * \eta_{generator} * \eta_{transformer} \approx 0.815$
with $g = 9.81\ m/s^2$ and $\rho = 1$

$$P_A = 8 \cdot Q_n \cdot H_n$$

Turbines
The threee basic types of turbines for large scale power generation are:

1. **Radial flow (Francis) turbine.**
   This turbine might be regarded as a centrifugal pump operating in '**reserve**'. The water enters the outer part of the casing and flow radially inwards through the rotating element (runner). The water then exists the casing through the cental outlet passage (see Fig. 17.1).

2. **Axial flow turbine (Kaplan).**
   This type may similarly be regarded as the reversal of the axial pump. Modern units incorporate mechanical means of varying the propeller blade angle of the runner, so that high runner efficiency may be maintained over the specified range of discharge and power input. Variable angle designs are called '**Kaplan**' turbines (see Fig. 17.2).

3. **Pelton turbine.**
   The Pelton wheel is applied where a high pressure water supply is available. By contrast with the other turbines the casing does not run full of water.
   The water enters the casing through one or more nozzles as a high velocity jet. This jet drives a series of flow deflectors or buckets (see Fig. 17.3).

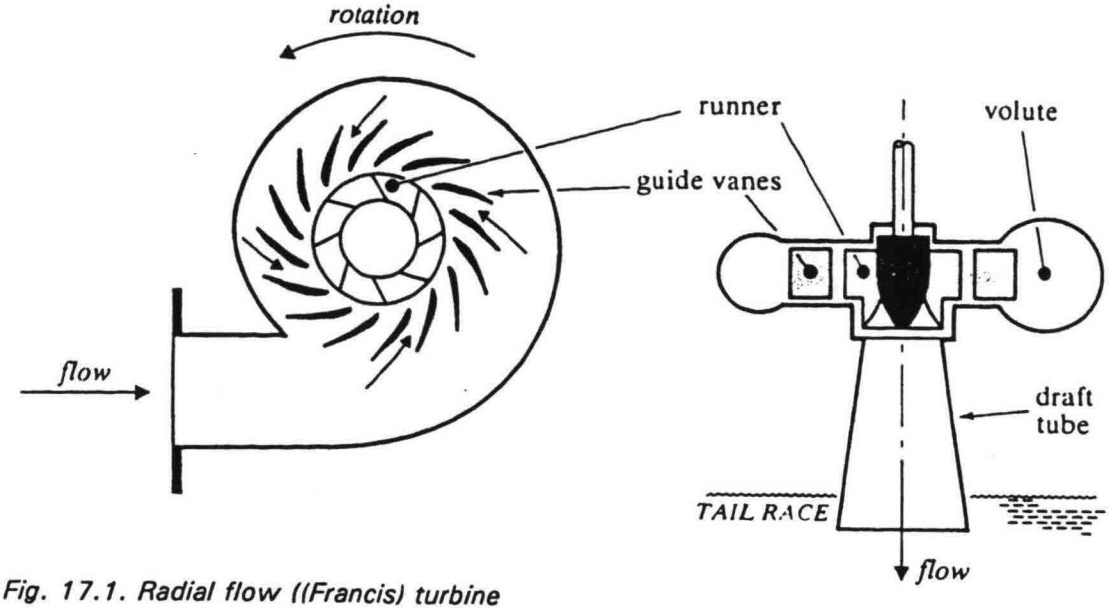


Fig. 17.1. Radial flow ((Francis) turbine

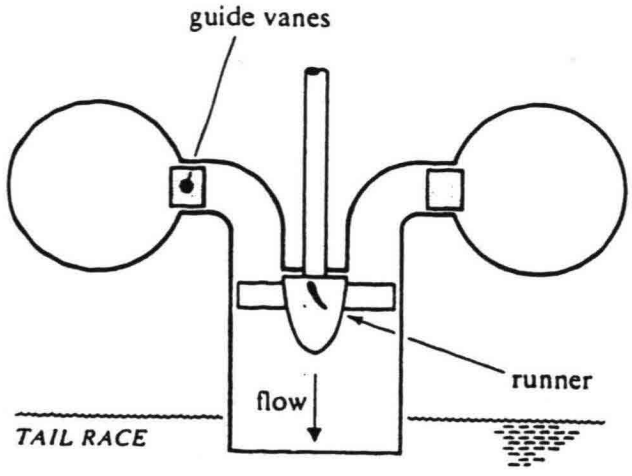


Fig. 17.2. Axial flow turbine.

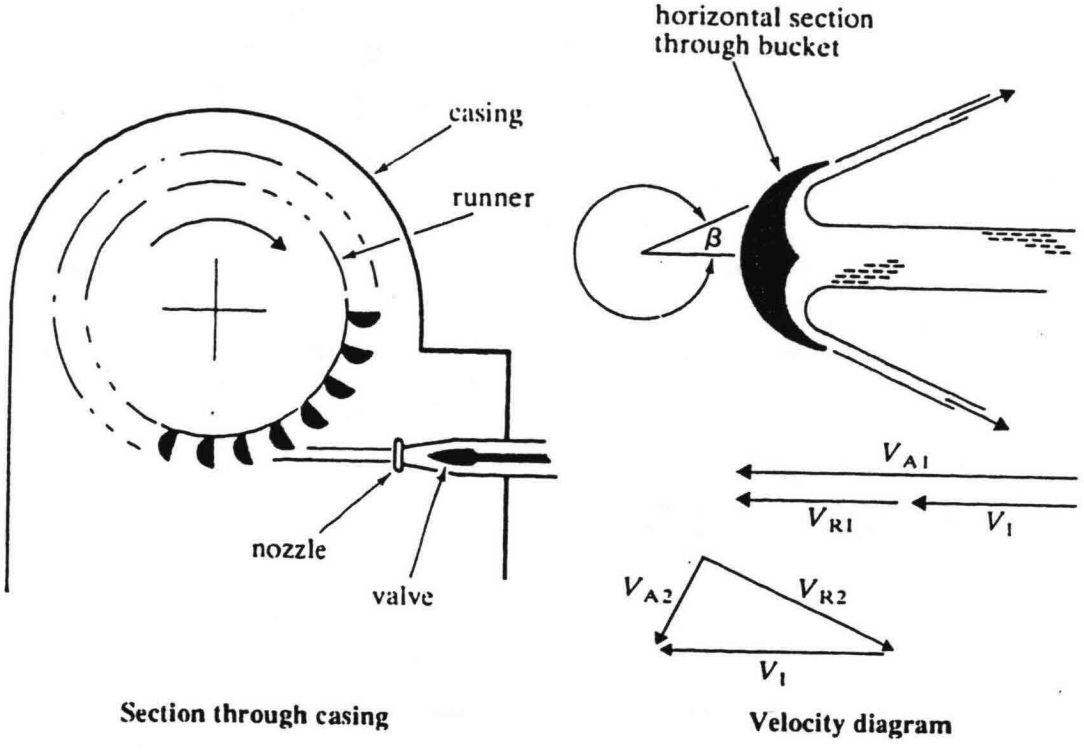
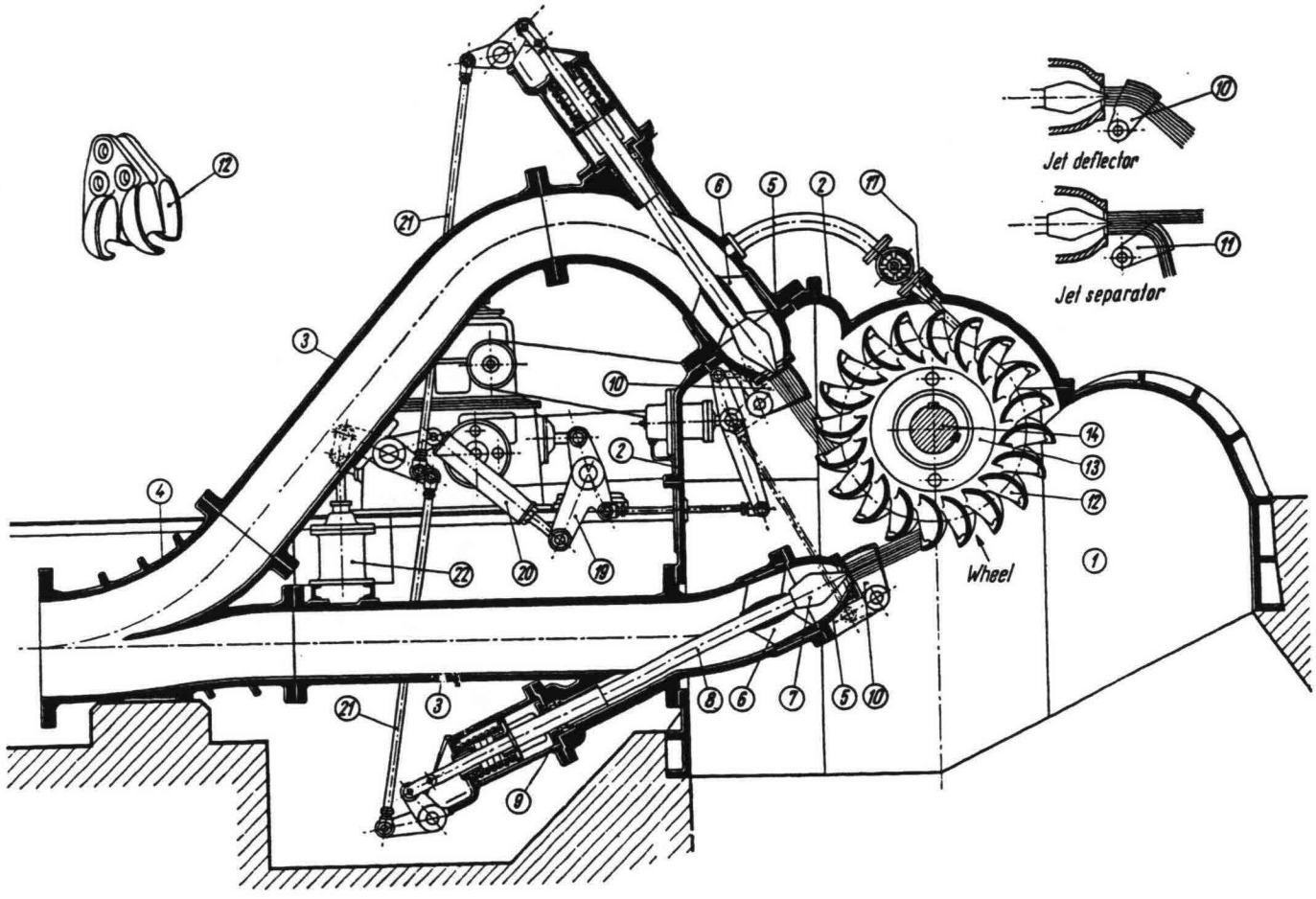
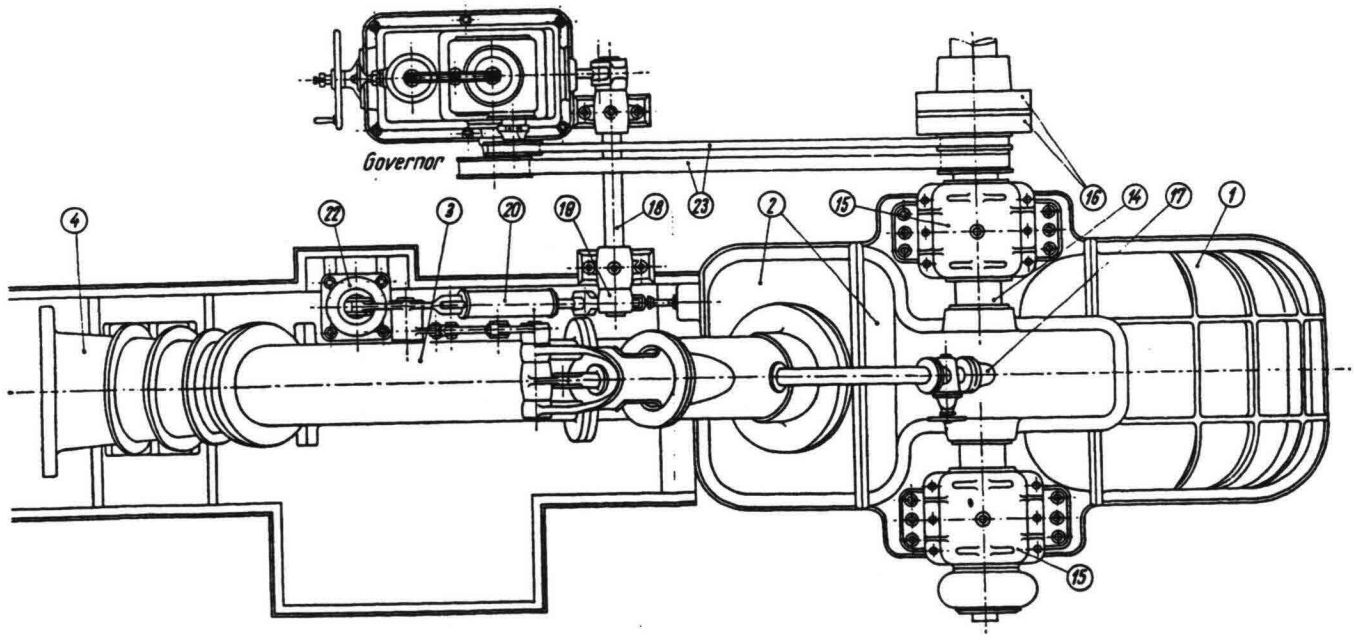


Fig. 17.3. The "Pelton Wheel" turbine.

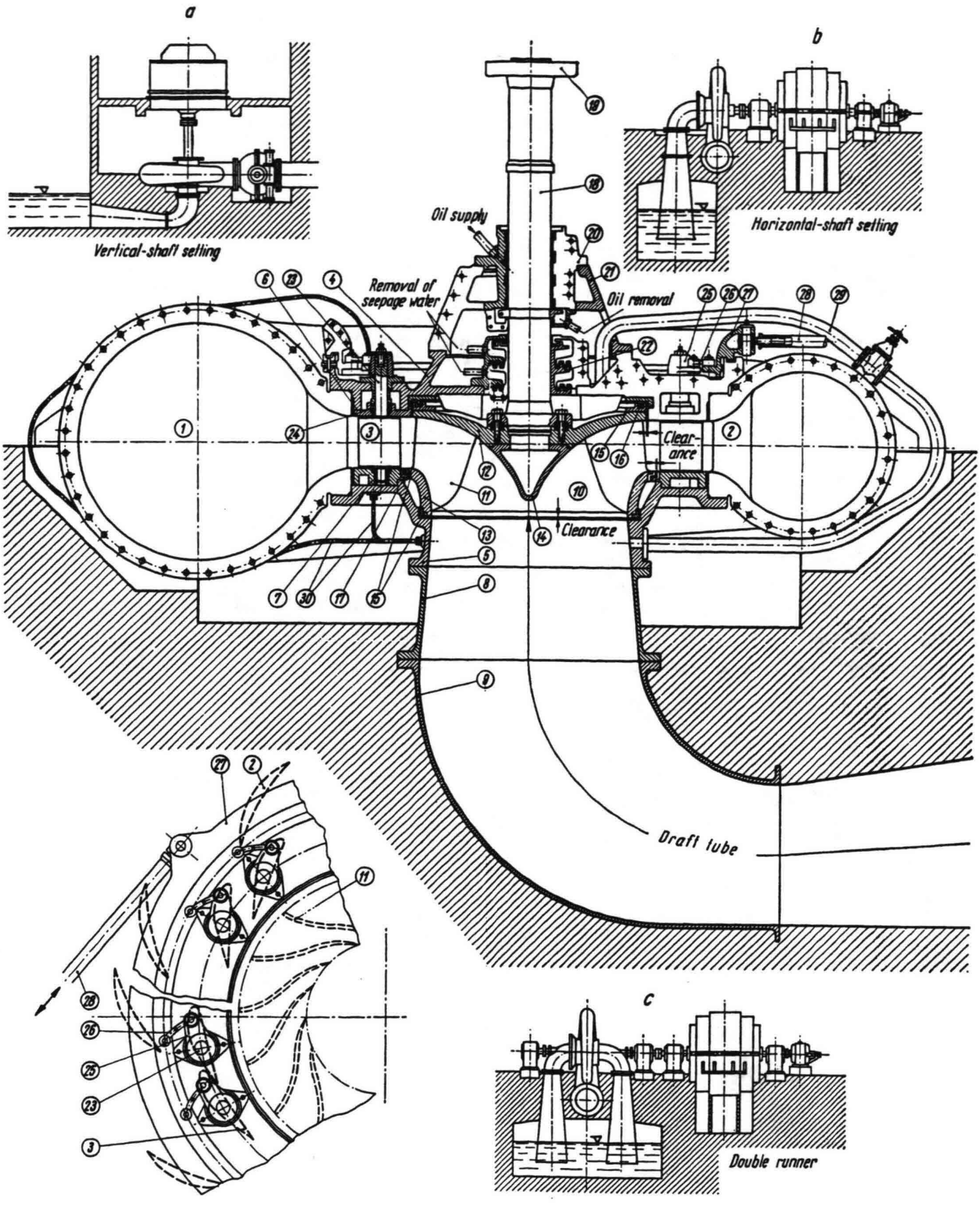
PELTON TURBINE





1. Casing
2. Cover
3. Nozzle pipe
4. Wye branch
5. Nozzle tip
6. Star guide
7. Needle tip
8. Needle stem
9. Stem packing
10. Jet deflector
11. Jet separator (in detail only)
12. Bucket
13. Runner disc
14. Shaft
15. Bearing
16. Shaft coupling
17. Braking jet
18. Regulating shaft
19. Regulating lever
20. Spring drawbar
21. Needle control rod
22. Dashpot
23. Belt drive

FRANCIS TURBINE



1. Scroll case
2. Stay vane
3. Guide vane
4. Head cover
5. Discharge ring
6. Upper cover inset for the wicket gate
7. Lower cover inset for the wicket gate
8. Throat liner
9. Draft-tube liner
10. Runner
11. Runner vane
12. Runner crown
13. Runner shroud
14. Runner cone
15. Wearing ring on runner-shroud
16. Wearing ring on head cover
17. Wearing ring on discharge ring
18. Turbine shaft
19. Coupling flange
20. Guide bearing
21. Guide bearing web
22. Labyrinth shaft sealing
23. Guide-vane stem
24. Guide-vane packing
25. Regulating arm
26. Shearing pin
27. Shifting ring
28. Drawbar
29. Pressure-equalizing pipe
30. Drain pipes

Water power plants may be classified according to the available head:

. Low head $H_{max} < 15$ m
. Medium head $H_{max} = 15$ m - 50 m
. High head $H_{max} > 50$ m

Some general characteristics:

	Low head	Medium-head	High-head
1. Topographical condition	flat land	hilly country	mountainous region
2. Relative share of discharges in capacity	large	moderate	small
3. Most suitbale solution of backing-up water in the river bed	river barrage with movable gates	river barrage with both movable gates & overflow dams	large dam
4. Conveyance of water for power generation	diversion canal run-of-river	diversion canal or run-of river	diversion canal, diversion tunnel, occasionally at toe of dam
5. Structures	entrance flume machine hall and substructure	entrance flume or headworks penstock machine hall and substructure	surge tank or headworks penstock machine hall and substructure
Powerhouse	block type	built in one block or separately	built separately
6. Turbine	Kaplan or fixed blade impeller; high-speed Francis	normal speed Francis; Kaplan (fixed blade propeller)	low speed Francis; Pelton wheel
7. Storage	no or daily pondage	daily or weekly pondage	seasonal annual storage
8. Power generation	fluctuating	slightly fluctuating	steady, hardly fluctuating
9. Operation mode	base load or peak load	base load or peak load	base load peak load

Note: peak loads require storage possibility.

17.1. Low head Water Power Stations

Classification of low-head hydropower stations on the basis of the general arrangement of the power house proper and the type of the axial turbines (according to Mosonyi)

1. Unit block and twin arrangement
All the machine sets can be accommodated in a single powerhouse (unit block), or for some reasons (fluvial, nautical, political) the station consists of two powerhouses, divided by the weir (twin arrangement)

1.1. Conventional solution
The powerhouse is equipped with vertical-shaft spiral-case axial turbines (Kaplan and/or propeller machines) directly coupled to the generators (conventional aggregates) in normal setting (i.e. the spiral case roof is at a lower elevation than the minimum headwater level).

1.2. Siphon setting
Conventional axial turbines are installed in a siphon-type location (spiral case roof above min headwater level) to reduce excavation depth.

1.3. Powerhouse with spillway conduits
Closed flumes are constructed in the substructure between the conventional machine sets, aiming the release of excess water and/or the diminution of surges.

1.4. Powerhouse with tubular aggregates
Different machine types:

a. bulb-type units,
b. wreath-(ring-) generator machine sets and STRAFLO aggregates respectively,
c. so-called S-machines and Tube-Units respectively.

2. Pier-head power station
The machine sets are accommodated into the piers of the weir, aiming the reduction of the width of the entire project.
Generating equipment:

2.1. conventional units
2.2. tubular aggregates

3. Submersible power plant
The machine sets are located inside the body of the overflow weir. The general solution is the installation of

3.1. tubular sets, however, also projects with
3.2. conventional units have been implemented.

4. Underground layout (mainly Swedish practice)
Under special topographical and geological circumstances the underground or burried location might be chosen. The underground (cavern) powerhouse can be equipped with

4.1. conventional or with
4.2. tubular machine sets.

5. Tidal power plant
Two kinds of operation: single directional or two-directional. Two types of tubular sets are suitable:

5.1. bulb-type or
5.2. ring-generator-type (STRAFLO) machines.

Types of low-head hydroelectric plants.

1. Diversion canal types of power plant
   - 1.1. Artificial power canal
   - 1.2. Natural power canal (river branch, main course dammed)

2. Run-of-river plants
   - 2.1. Single plants and lowest on canalized river sections
   - 2.2. Intermediate plants and those located uppermost on canalised river branches.

3. Power plants situated in river branches (with main course open)
   - 3.1. Plants at the upper end of the branch
   - 3.2. Plants at the mouth of the branch
   - 3.3. Plants located intermediately
   - 3.4. Two cooperating plants

Mode of operation: continuous or intermittent

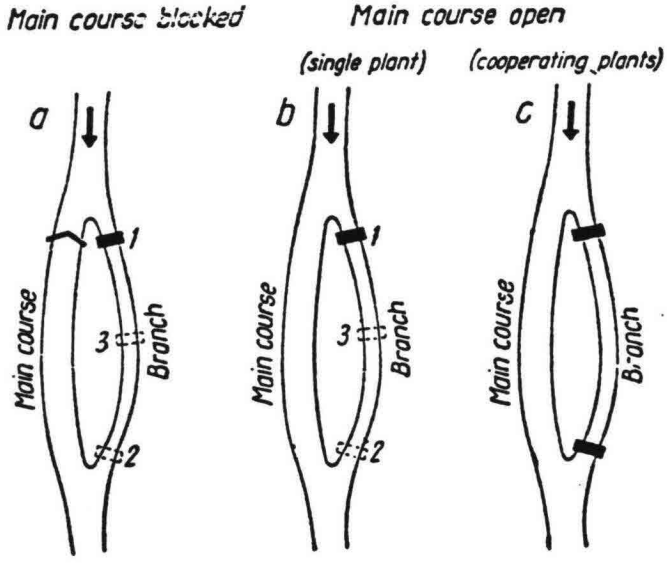
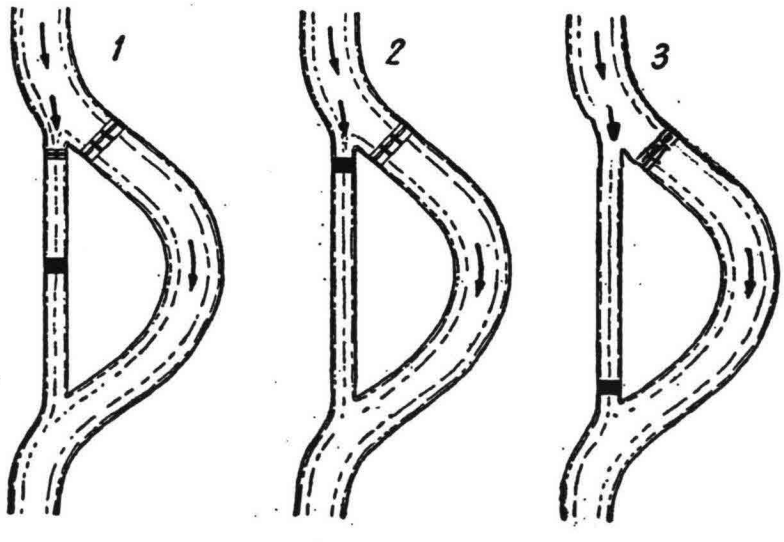


Fig. 17.4 Utilization of river branches



Location of the power station in the canal

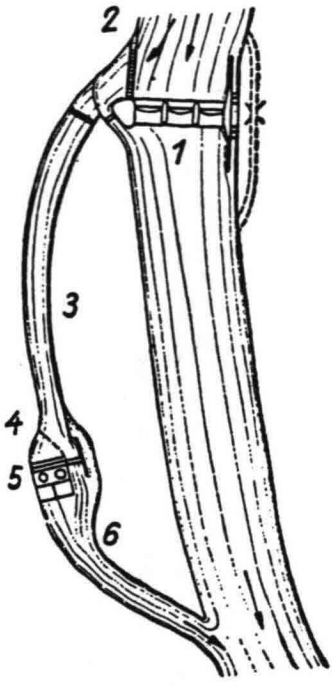


Fig. 17.5.
General layout of diversion canal type plant.
1. Weir
2. Intake
3. Headrace
4. Forebay
5. Power station
6. Tailrace

Structures involved in diversion canal development may be classified into the following groups:

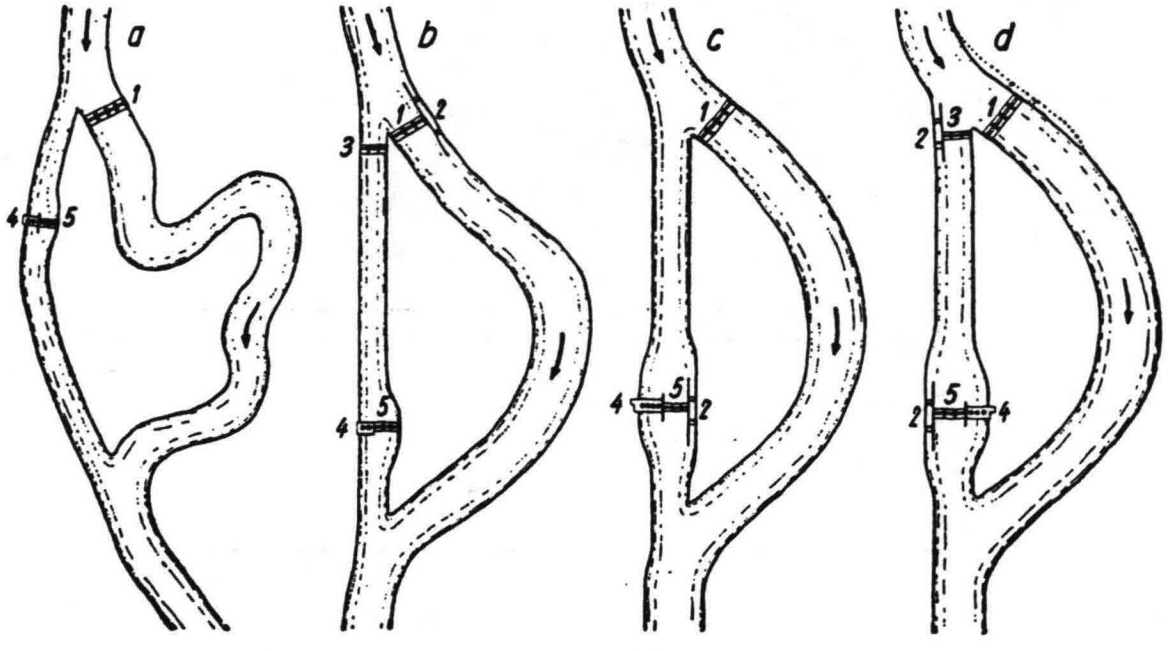


Fig. 17.6. Alternatives for diversion canal type layouts.
1. weir, 2. navigation lock, 3. intake, 4. power station, 5. spillway and bottom outlet.

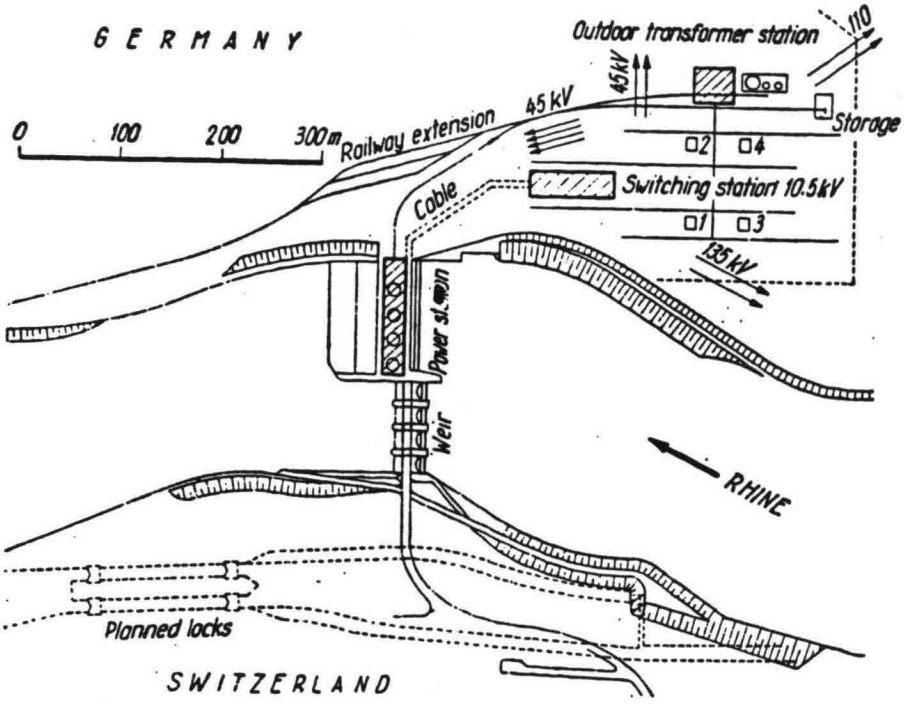


Fig. 17.7.
Stream-bed development. Example of block-type run-of-river power station without bed enlargement, Rhine River.

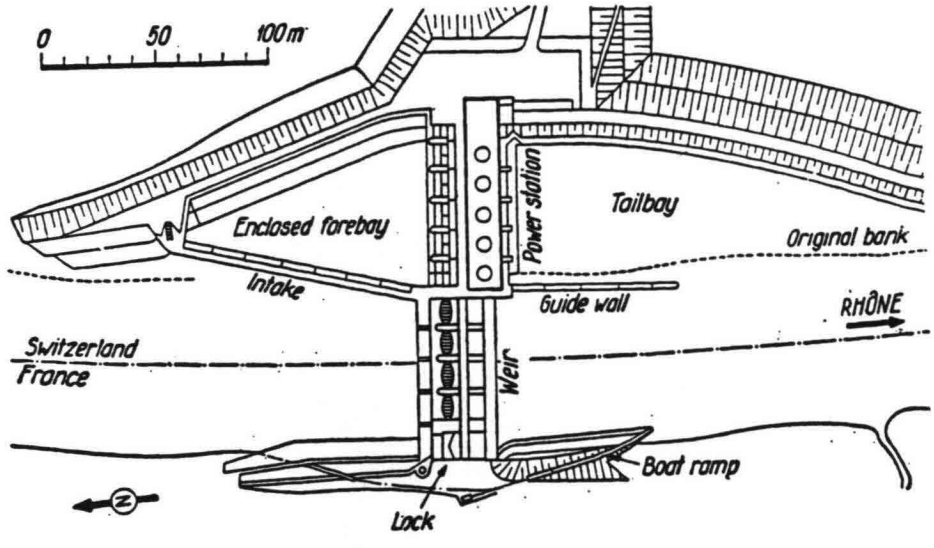


Fig. 17.8.
Run-of-river plant with closed bay: layout incorporating a separate intake structure at the entrance to the bay. Example: Khancy-Pougny power station, Rhone River, France.

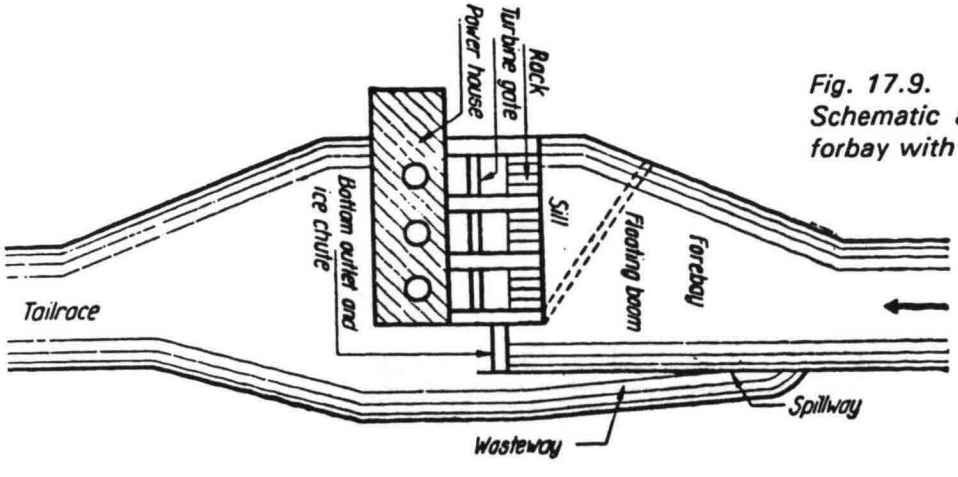


Fig. 17.9.
Schematic arrangement for forbay with spillway.

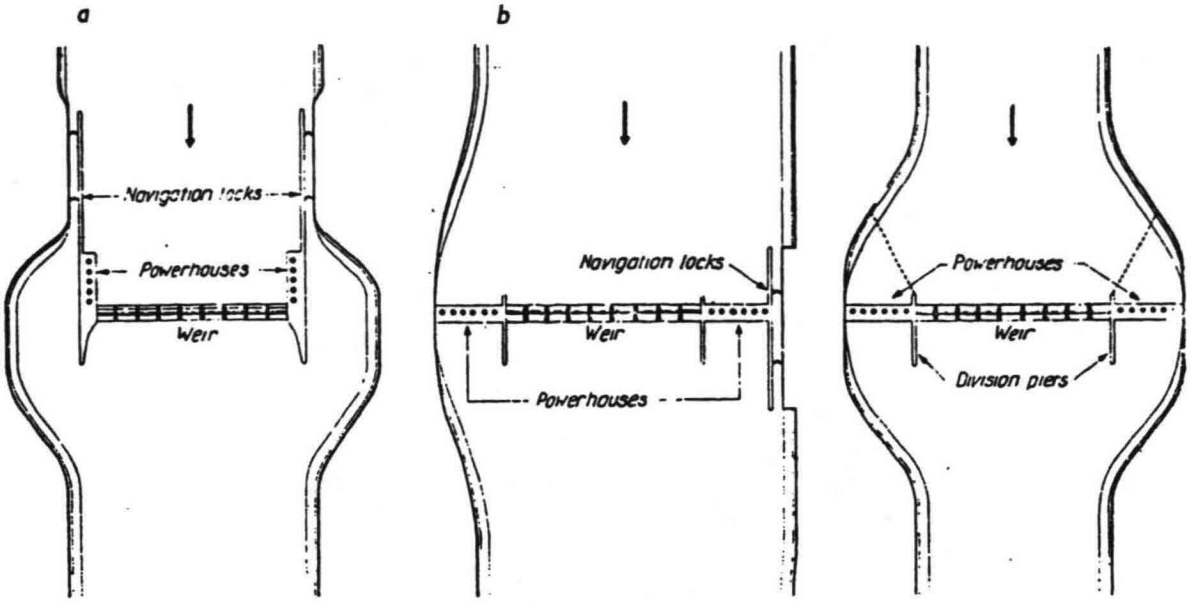


Fig. 17.10. Typical layouts of twin power stations.

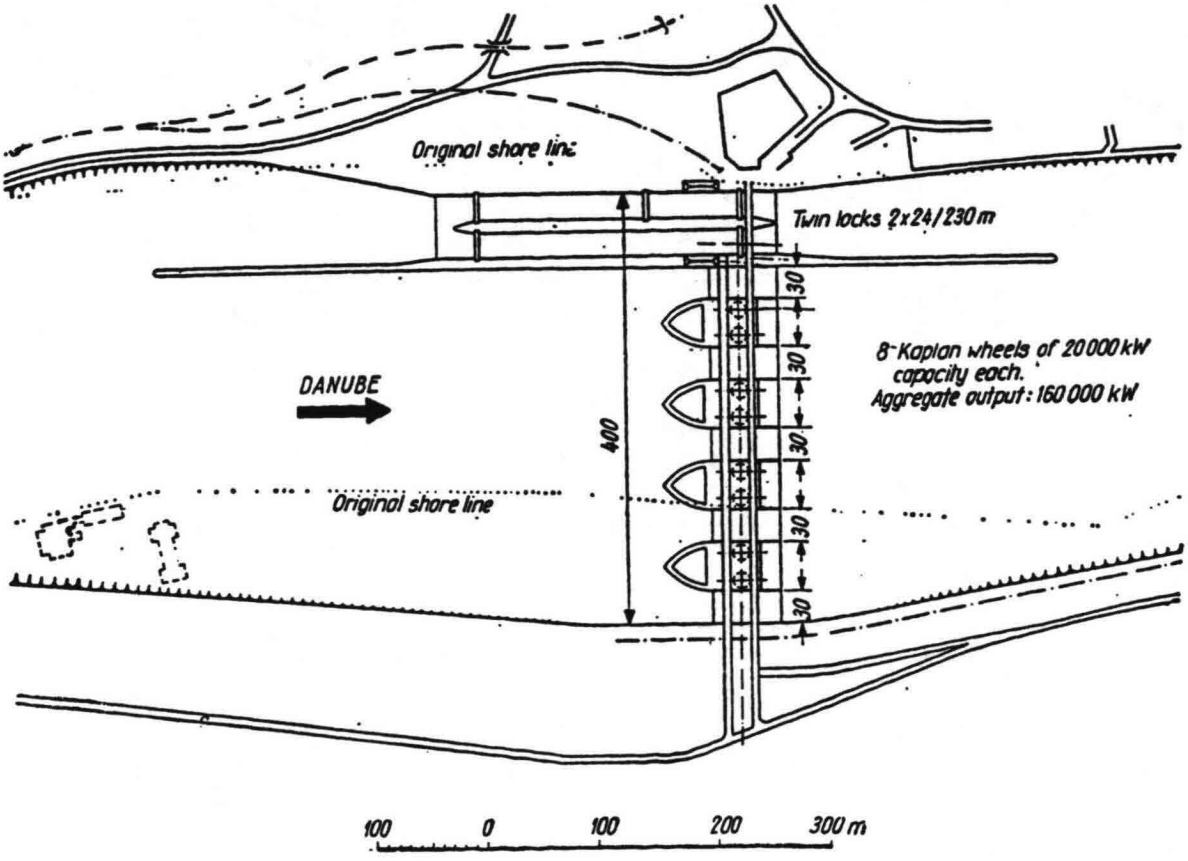
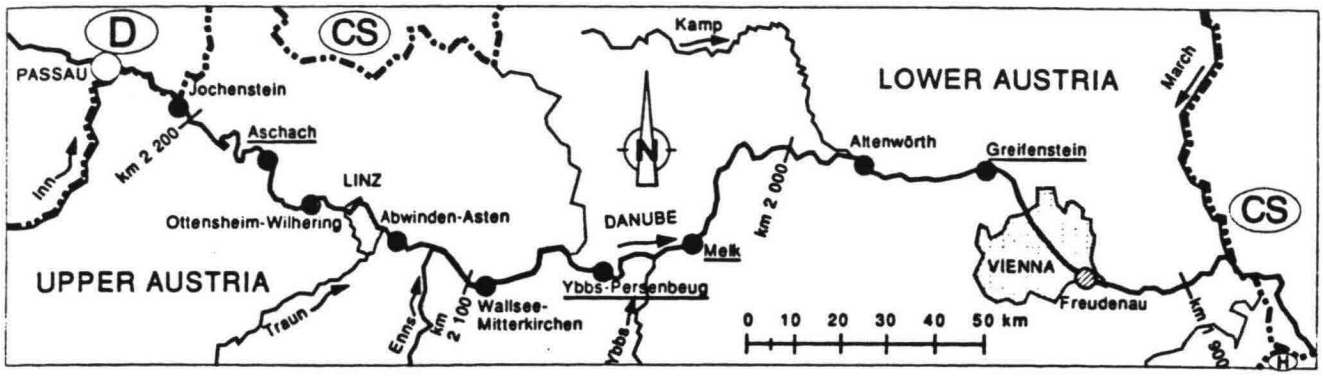
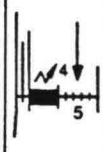
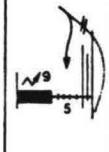
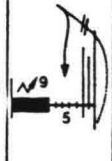
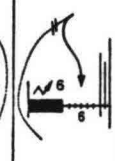
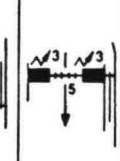
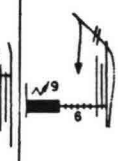
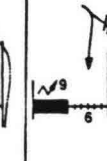
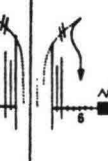
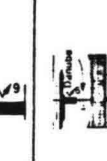


Fig. 17.11. Pier-head power station.



Power station		Jochenstein	Aschach	Ottensheim-Wilhering	Abwinden-Asten	Wallsee-Mitterkirchen	Ybbs-Persenbeug	Melk	Altenwörth	Greifenstein	Freudenau
Owner		DKJ	DoKW	DoKW	DoKW	DoKW	DoKW	DoKW	DoKW	DoKW	DoKW
Operation since		1955	1963	1973	1979	1968	1957	1982	1976	1984	Project
Stationing	km	2 203.3	2 162.7	2 146.7	2 119.5	2 093.6	2 060.4	2 038.0	1 979.8	1 949.2	1 921.05
Storage level	m	290.3	280.0	264.0	251.0	240.0	226.20	214.0	193.5	177.0	161.35
Q_{mean}	m³/s	1 430	1 450	1 450	1 600	1 730	1 750	1 807	1 830	1 882	1 700 *
Flow Q_{max}	m³/s	8 900	8 900	8 900	9 500	11 100	11 100	11 170	11 170	10 750	8 200 *
Q_{used}	m³/s	1 750	2 000	2 250	2 475	2 600	2 100	2 700	2 750	3 150	3 000 *
Head H_{mean}	m	10.20	15.30	10.70	9.30	10.90	11.0	8.4	14.80	12.6	8.5
Capacity (MC)	MW	(130) 65	286	179	168	210	200	187	328	293	165
Energy (AAE)	GWh	(850) 425	1 648	1 143	1 028	1 320	1 282	1 180	1 950	1 720	1 017
Layout											
Spillway/Weir											
Bays width	m	6×24	5×24	5×24	5×24	6×24	5×30	6×24	6×24	6×24	4×24
Pier width/height	m	5–6/31	7.10/41	7.50/37	6.0/37	7.50/37	7.50/34	6.0/31	7.0/37	6.0/31	6.0/33
Q max_{100}	m³/s	8 900	8 900	5 940	8 450	8 600	11 100	11 170	9 785	8 650	6 530
Gates		hook-double	hook-double	hook-double	tainter + flap	hook-double	hook-double	tainter + flap	tainter + flap	tainter + flap	tainter + flap
Powerhouse											
Construction, type		high	medium	low	low	low	medium	low	low	low	low
max height	m	52	53	39	40	42	42	39	46	44	47
Turbines, number and type		5 Kaplan ↓	4 Kaplan ↓	9 Kaplan →	9 Kaplan →	6 Kaplan ↓	6 Kaplan ↓	9 Kaplan →	9 Kaplan →	9 Kaplan →	6 Kaplan →
Backwater area											
Length	km	27	40	16	27	26	33	22	32	31	28
Overflow flood	m³/s	–	–	2 960	1 050	2 500	–	–	1 385	2 100	2 200**

* Danube River only

** New Danube – Flood diversion

Fig. 17.11. Danube power stations in Austria.

Development of the Austrian Danube was commenced when the demand was consistent with the large energy potential available which actually was not the case until the period after the Second World War. This led to the most welcome result that, prior to the construction of the first power station at Jochenstein, a masterplan covering the whole Austrian reach of the Danube available. The number planned power stations was reduced from fifteen to twelve by combining projects for reasons of economy. The plan was implemented in the period 1955-1984, practically for 30 years, in stages. The Masterplan covers a river length of 350 km and a total head of approx. 150 m.

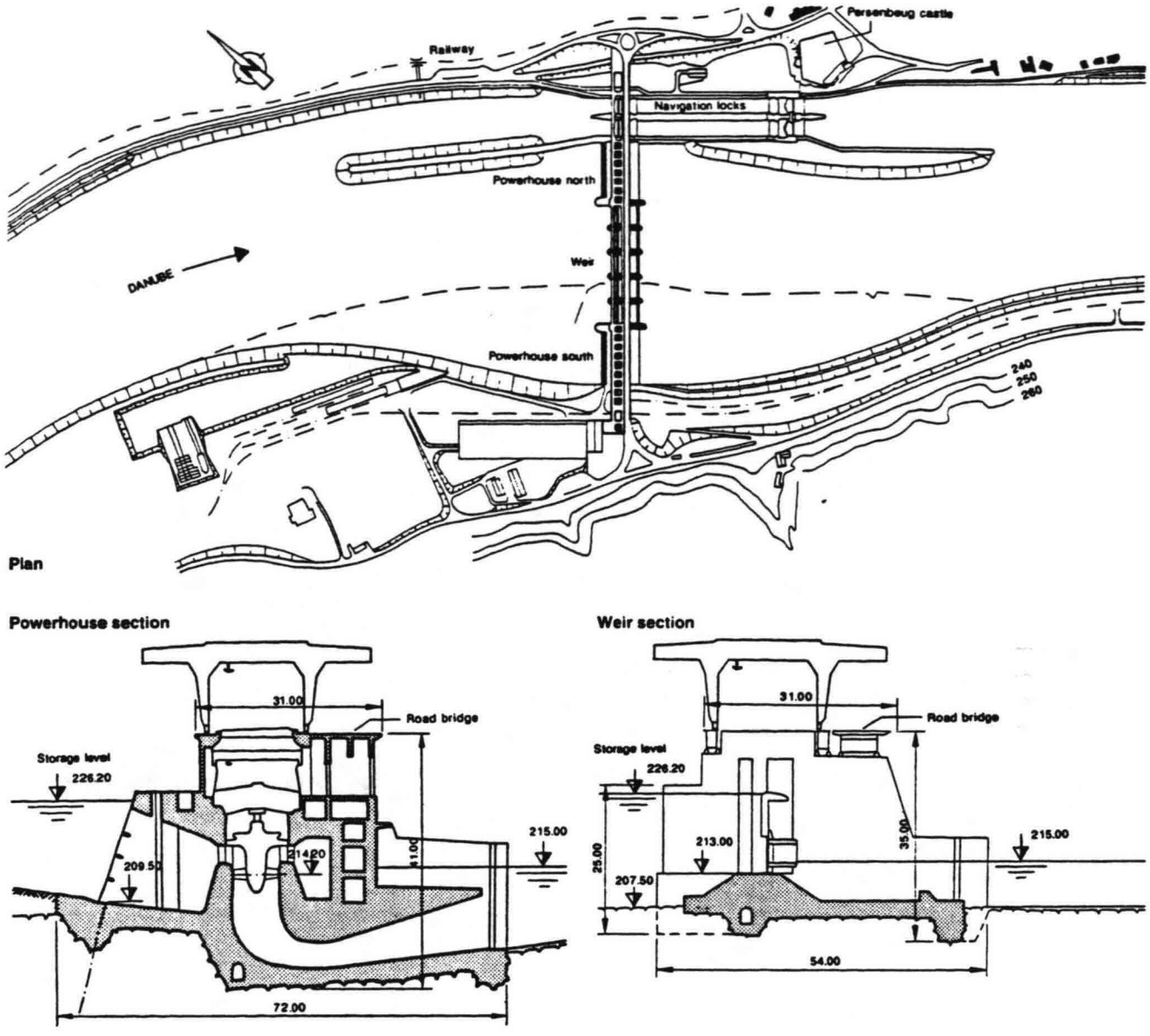


Fig. 17.12. Twin power stations, island arrangement, Ybbs - Persenbeug river Danube, Austria. Operating since 1957.

Purposes: hydropower, inland navigation and flood protection.
The barrage consists of a weir with 5 openings of 30 m each, 2 power stations and a lock with 2 chambers, 24 x 230 m.
Total length: 5 x 30 + 4 x 7.5 (weir) + 2 x 93 (power stations_ = 366 m.
Apart from developing the river for power generation and important purpose of the project was to improve navigation conditions in the Strudengau, a narrow part of the Danube Valley. The two power houses (north and south) flanking the weir are each equipped with three vertical-shaft Kaplan turbines of a rated discharge of 350 m^3/s and a maximum capacity of 35 MW in total).
Flow: Qmean = 1750 m^3/s, Qrated = 2100 m^3/s Qmax = 11,100. Meanhead 11.2 m
The turbine runners 7.4 m in diameter are located 0.8 m below mean tailwater level.

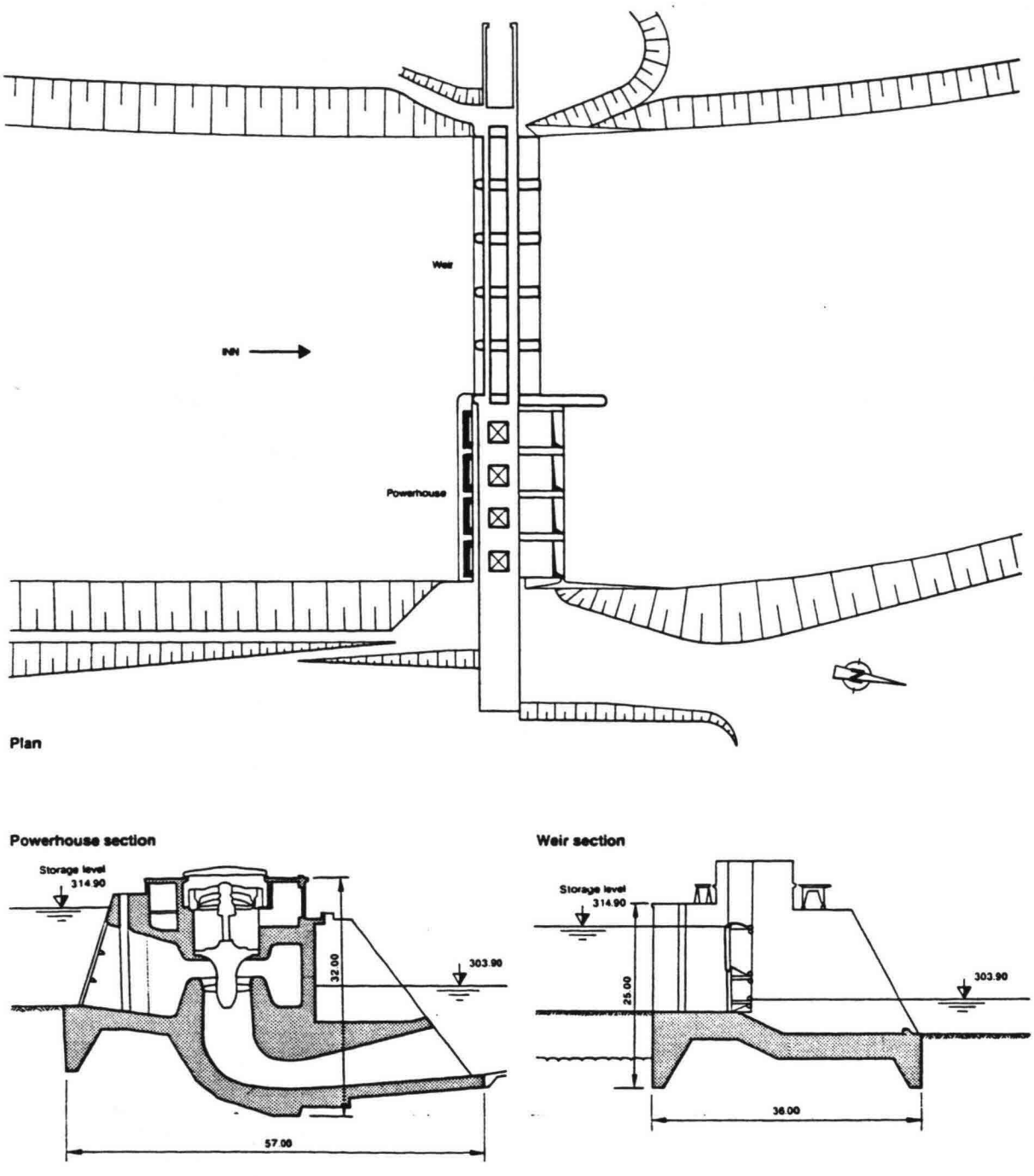


Fig. 17.13. Unit-block power station Schärding-Neuhaus, river Inn, German-Austrian border.

Operating since 1961.
Purposes: hydropower and flood protection
The barrage consists of a weir with 5 openings of 23 m each, a power station with four vertical shaft Kaplan turbines with a rated discharge of 250 m^3/s and a capacity of 24 MW in total).
Flow: Qmean = 732 m^3/s, Qrated = 1000 m^3/s Qmax = 6,800. Meanhead 11.2 m
Construction of the barrage was carried out in two successive pits in the river channel widened at places. The five openings of the weir have hook type double-leaf gates 13.8 m in height.

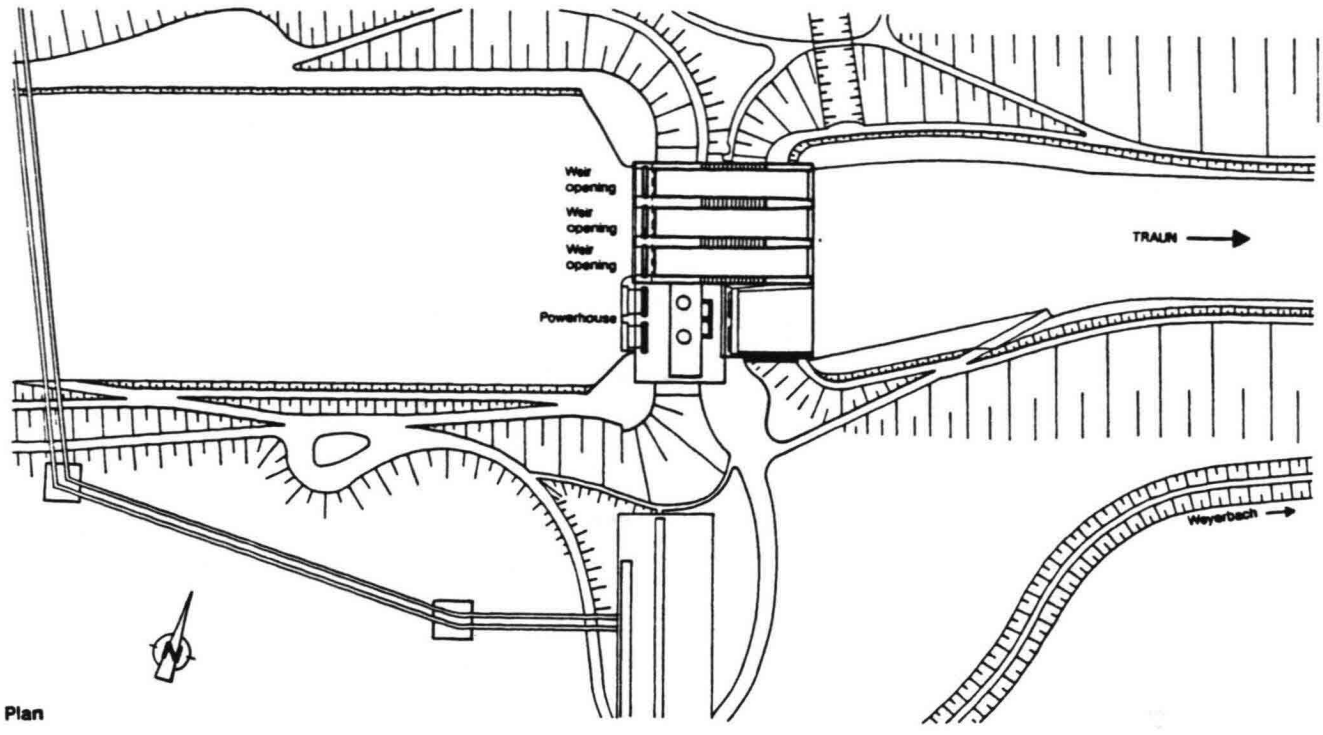


Fig. 17.14. Unit-block power station Traun, near Linz, Austria. Operating since 1982.

Purposes: hydropower prevention of further degradation of the Traun river bed, stabilisation of the phreatic surface.
The barrage consists of a weir with 3 openings of 13 m in width each and a power station with 2 vertical shaft Kaplan turbines with a rated discharge of 103 m^3/s and a capacity of 23 MW (46 MW in total).
Total length : 48 m (weir) + 55 m (power house) = 103 m.
Flow: Qmean = 128 m^3/s, Qrated = 206 m^3/s Qmax = 2,300. Meanhead 25.5 m.
The weir has three openings closed by tainter gates with upset flaps of 15.5 m in overall height. The turbine runners, 4.7 m in diameter, are situated 2.2 m below mean tailwater level.

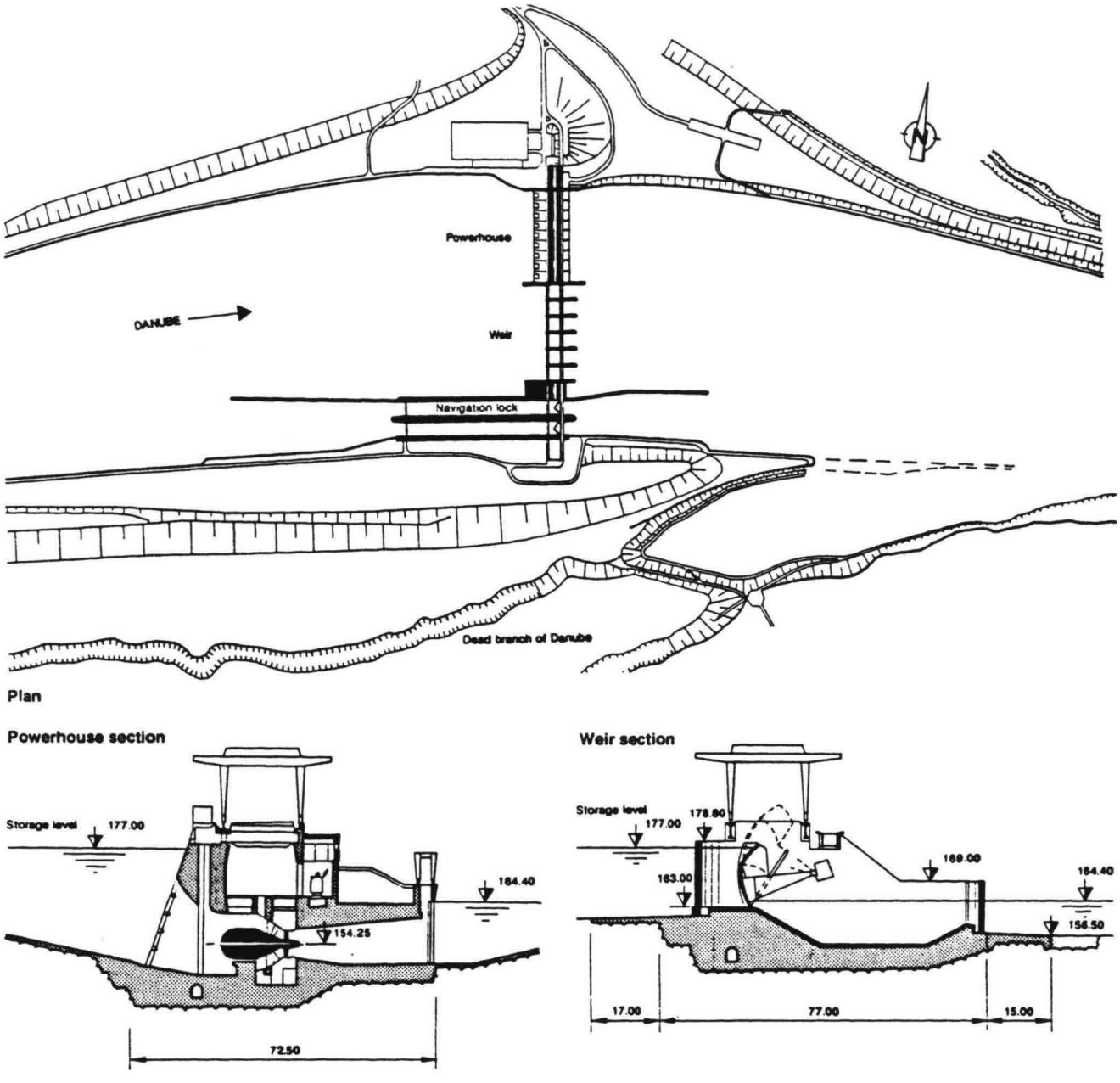


Fig. 17.15. Unit-block power station Greifenstein in the river Danube, Austria. Operating since 1984.

Purposes: hydropower, inland navigation, flood protection, ecological effects (riverside forest irrigation system).
The barrage consists of a weir with 6 openings of 24 m, a power station with 9 bulb turbines, 293 MW in total and a lock : 2 chambers 24 x 230 m. The backwater area is 31 km long; embankments and flood banks had to be constructed on both sides of the river.
Flow: Qmean = 1882 m^3/s, Qrated = 3,150 m^3/s Qmax = 10,750 m^3/s.
Meanhead 12.6 m.
The powerhouse on the left bank accommodates nine turbo-generator sets, each consisting of a horizontal Kaplan bulb turbine with an output of 34 MW, a runner diameter of 6.5 m; the units each take up 19 m in width.
Main dimensions: power house - 44 m high and 9 x 19 = 171 m long weir - 31 m high and 6 x 24 + 5 x 6 = 174 m long, total length : 345 m.

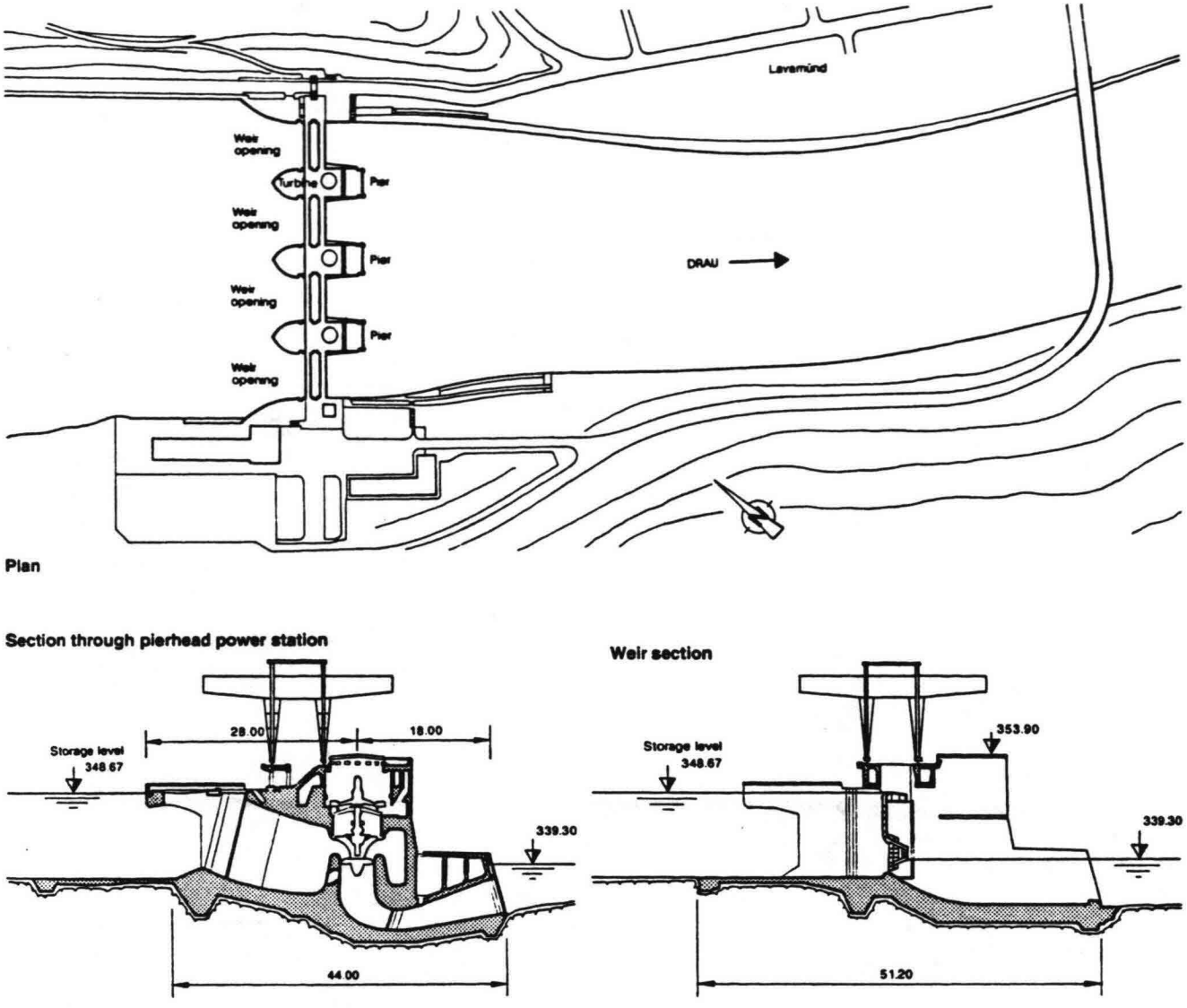


Fig. 17.16. Pier-head power station in river Drau near Lavamünd, Austria.

Purpose : hydropower

Lavamünd is the first pierhead power station in the world. Operating since 1944 (2 turbines), 1949 (3 turbines). Weir : 4 openings, each 24 m in width (Q = 5000 m^3/s) spaced by 3 piers with one 9.6 MW turbine each, 28 MW in total. Total length : 4 x 24 + 3 x 18 = 150 m.

To utilize the facility as soon as possible, engineers developed the pierhead type of power station which allowed operation already after the first stage of construction (1 pier, 2 weir openings) had been completed and when the backwater was not yet fully developed. The station was built in the river bed, the customary method at that time. Each pier holds a turbogenerator set consisting of a four-blade Kaplan turbine and attached umbrella-type generator. The four openings (24 m each) of the weir are closed by hook type double-leaf gates of 11 m in height.

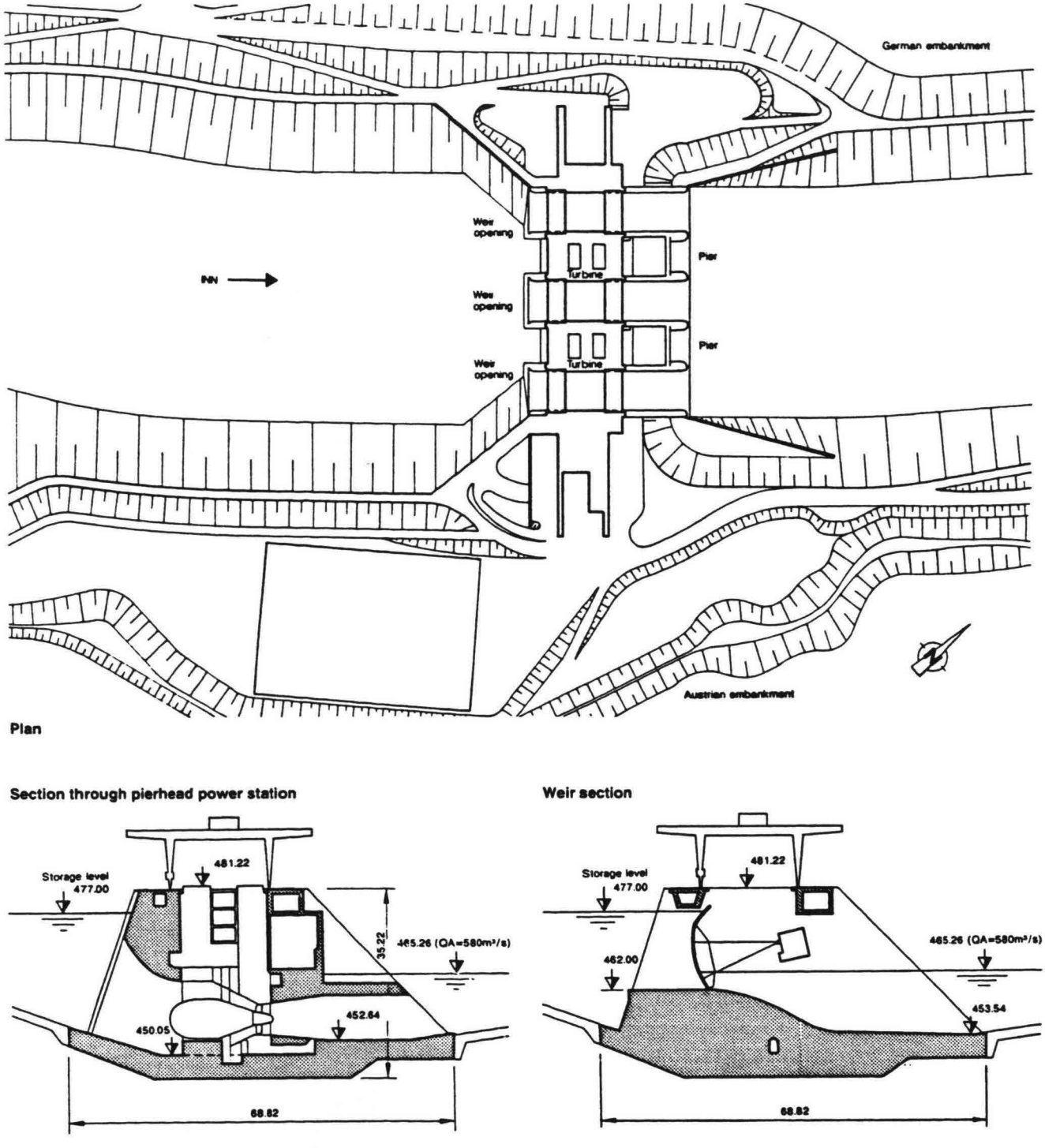


Fig. 17.17. Pier-head power station in river Inn near Kufstein, Tirol, Austria on border with Germany.

Purposes : hydropower, flood protection and prevention of riverbed degradation. The project is scheduled for completion in February 1992.
Construction period: 1988-92. The barrage consists of a weir with 3 openings, 16 m width each spaced by 2 piers of 20 m width and one 30 MW horizontal bulb turbine each, 60 MW in total. This power station is the final link in the chain of Inn power stations downstream of Kufstein. It will complete the development of 220 km stretch of river until its confluence with the Danube in Passau (Germany/Austria); 16 power stations in total.
The backwater area stretches for 16 km and required embankments of up to 8 m. Diaphragm walls (max. 20 m) were provided for cut-off.
Excavation in barrage area : 2,000,000 m^3; Meanhead 11.7 m.
Flow: Qmean = 305 m^3/s, Qrated = 580 m^3/s Qmax = 2,150 m^3/s.
Total length of barrage : 88 m.

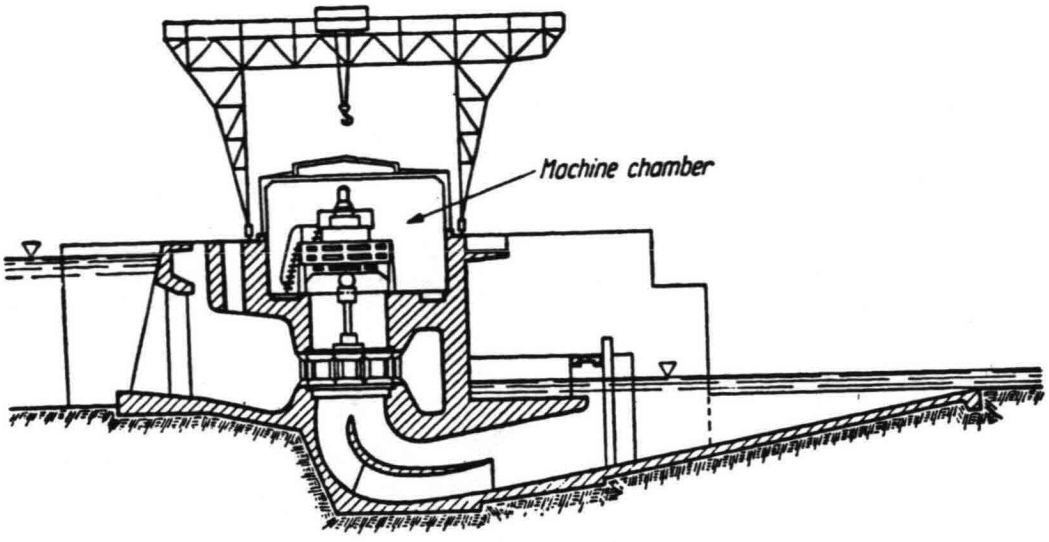


Fig. 17.18. Low-head powerhouse with semioutdoor superstructure. Wetter power station, Ruhr River, Germany.

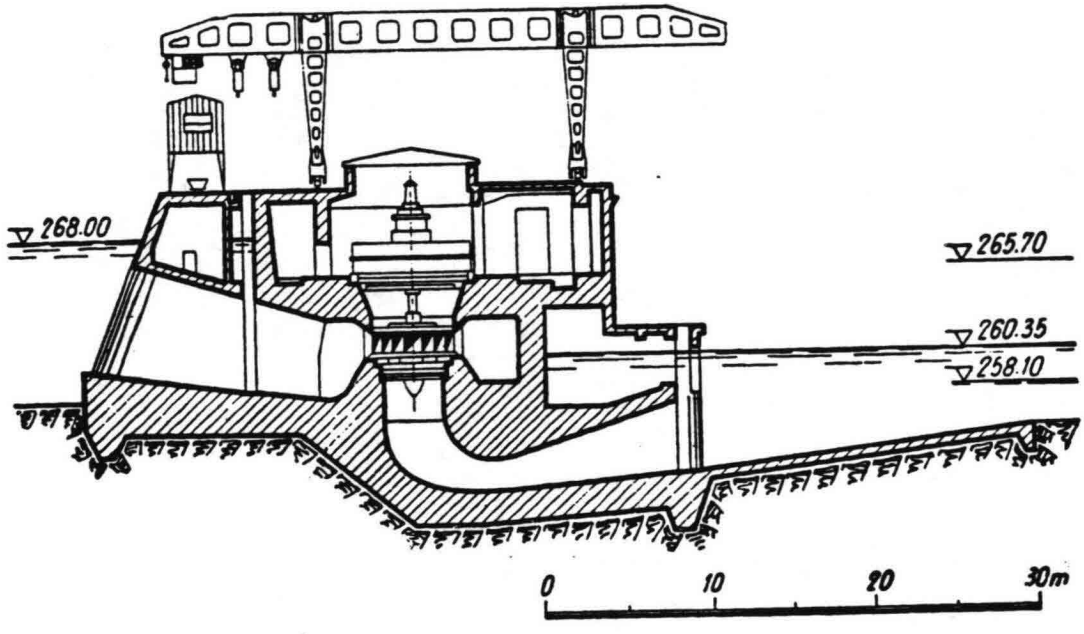


Fig. 17.19. Low-head powerhouse with semioutdoor superstructure. H_{max} = 10 m. Mühlrading power station, Enns River, Austria.

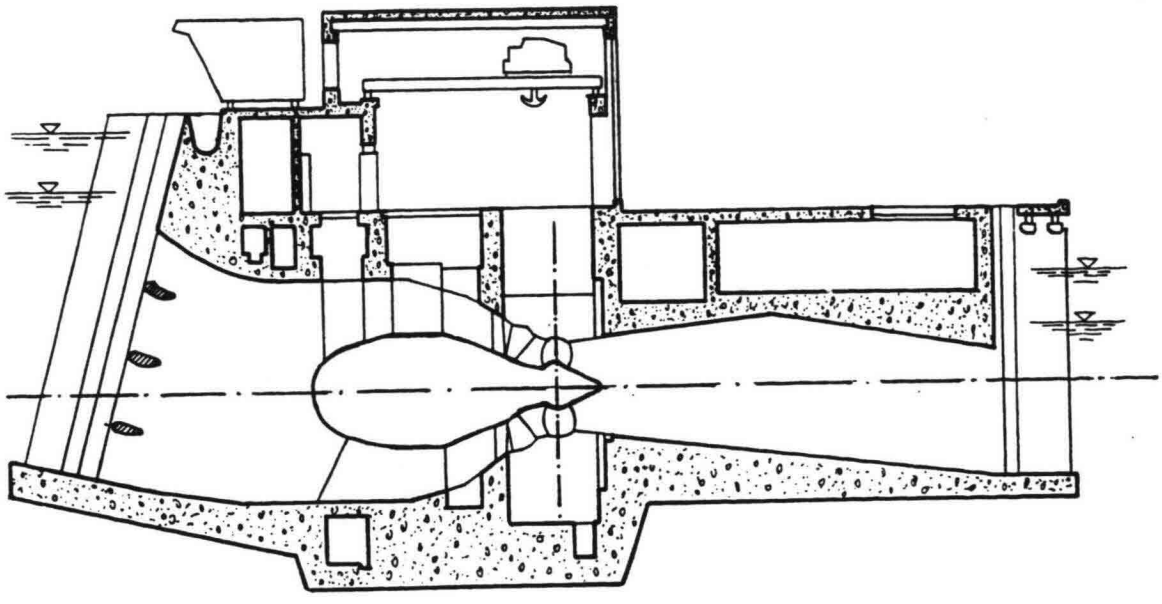


Fig. 17.20. Kaplan turbine, bulb- generator setting.

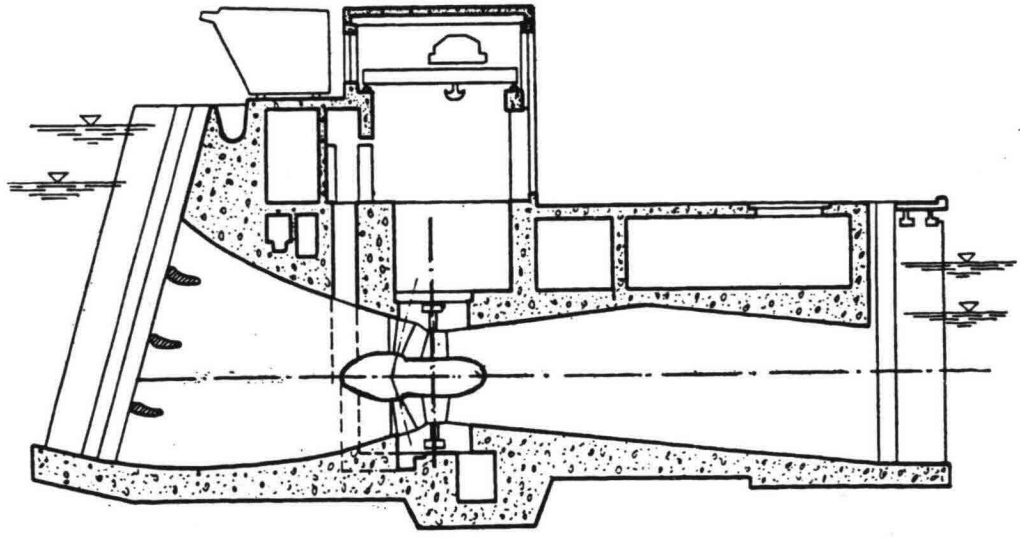


Fig. 17.21. Kaplan turbine, Ring generator setting.

Comparison between ring- generator turbine setting and Bulb unit setting with the same output and head.

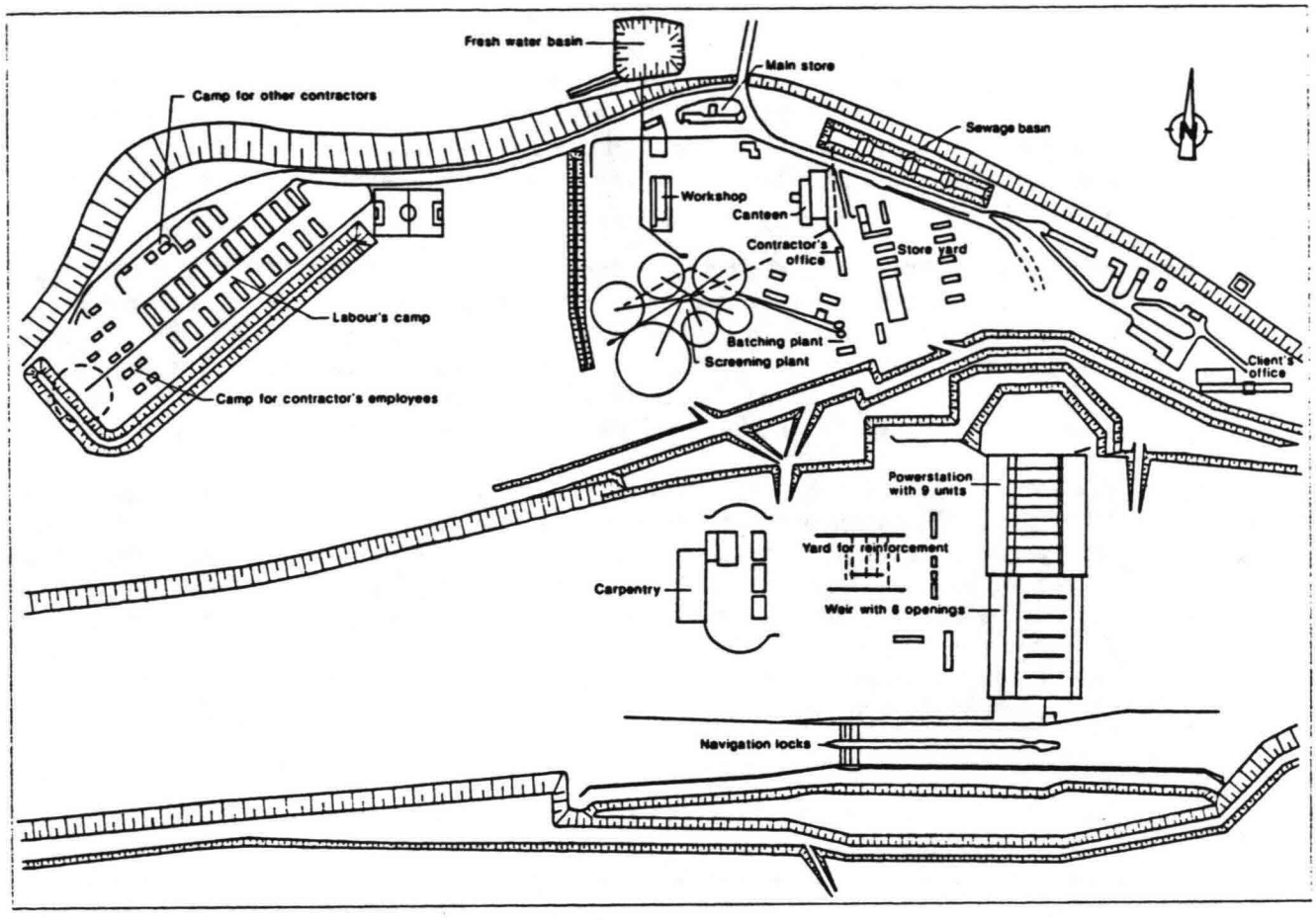


Fig. 17.21. Location plan of site installations for Greifenstein run-of-river power project on the Danube river (Austria) during the construction phase (1981-1984).

In the period 1950-1990 eight run-of river power stations have been constructed on the Austria part of the Danube river. Together with the experience gained at construction sites on the rivers Ems, Drau, Mur and Inn this has developed the construction techniques. In the first one-third of this period (1950-1963) railmounted vehicles were the dominating means of transport; railways led to the very centres of activity. During the mid-sixties and onwards, as large construction equipment was developed and road construction techniques were improved, roads of increasing capacity were constructed to provide direct access from the public road network.

The growing use of construction equipment has allowed increasing cuts in number of personnel from about 3000 workers at Ybbs-Persenbeug (1953-1957) to about 1000 at the Melk run-of-river station (1978-1982). In spite of that, motor capacity per man has increased from 12 kW to about 20 kW. The Greifenstein barrage was built in the chord of the bow of the original riverbed; this location permitted construction in a single large, unconfined and flood-protected building pit without impairing navigation. The 15.7 million m^3 of excavation was done; 1.7 million m^3 of this being bedrock (flysch) which was ripped out by three 86-t bulldozers more than 500 kW in engine capacity and equipped with ripper teeth (max. monthly performance: 300,000 m^3).

The treatment of mass concrete holds a prominent place in the construction techniques for large barrages. This is why great efforts have always been spent on the development of the construction techniques and the corresponding machinery in this particular field. At the first hydro schemes on the Danube concrete was placed by pumping. However, this extremely efficient method of concrete placement had to be abandoned for reasons of concrete technology, as the high cement content needed for the pumping led to excessive heat build-up in the mass concrete. Later heavy tower cranes were used with the concrete being placed y buckets. The development then led to the use of high-velocity belt conveyors mounted on the booms of telescopic cranes. At Greifenstein, as much as 70% of the total concrete quantity was already placed by conveyor belt.
Great progress has also and above all been made in the field of formwork technology. At the large barrages, formwork was lifted into place by tower cranes, later DOKA climbing formwork entered the path towards worldwide application. By using this type of climbing formwork it was also possible to accomplish a substantial reduction in the number of hours spent per m^2 of formwork.

An essential factor in the good functioning of construction operations is adequate project management and organization. As almost all the large hydropower projects are constructed by joint ventures, the first step has always been to conclude a joint-venture contract between the participating firms. A clear site management organization chart with the Site Manager at the top and a subdivision organization spheres of competence at the second level has been successfully used for the execution of the individual construction projects

The eight power stations on the river Danube, constructed in the three decades between 1955 and 1985, demonstrate the importance of rationalization at large run-of-river stations. The two main items in this type of structures are earthworks and concrete construction.
Total man-hours per project was reduced from 15-20 million to about 5 million for the last three power stations. Man-hours related to output unit of work for the first project 22.8 and 2.8 for the last project.
The reduction in man-hours has for the greater part been reflected by the construction cost per unit of work done, by more than a factor 3.

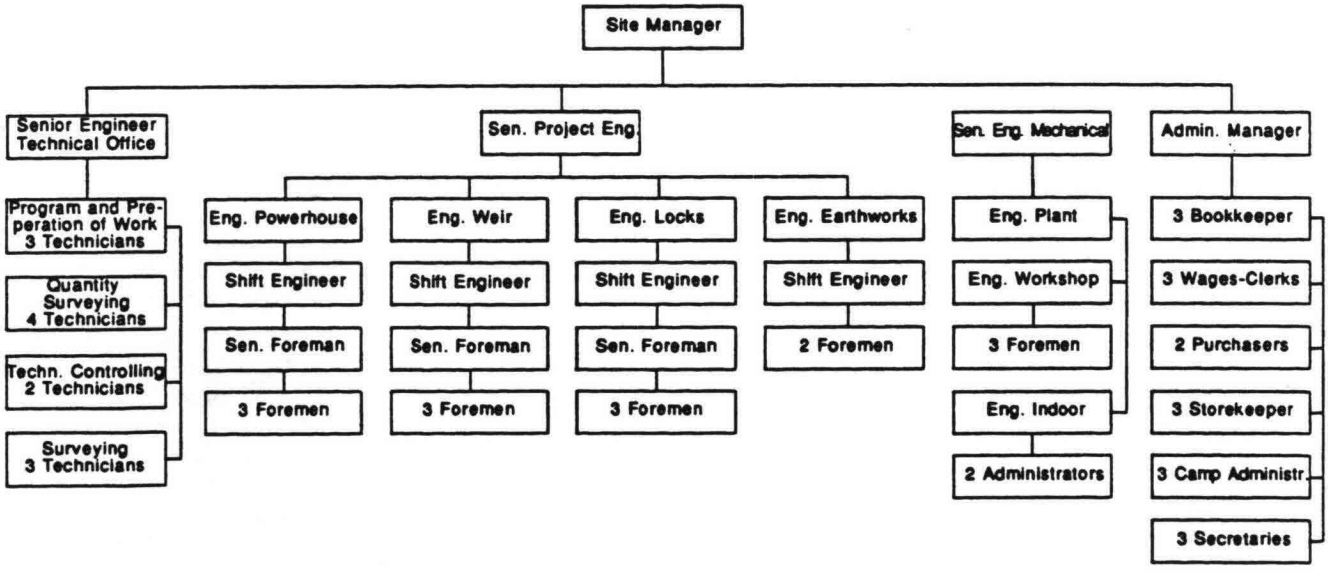


Fig. 17.22. Organization chart at the climax of construction operations at the Greifenstein project.

17.2. High-head Water Power Stations

Types (after Mosonyi):

1. Diversion canal (free flow) development

The water is diverted from the river at a dam or weir (diversion dam, diversion weir) and led through an intake to a canal. A settling basin is needed to remove coarse sediment. The canal transport the water over considerable distance to a location where enough head is available (steep slope) to build a penstock and a powerhouse. The water is led through the tailwater canal back to the river.

The main parts of a **high-head diversion canal type plant** are:

a. weir and appurtenant structures (log chute, raft sluice, boat lock and sometimes ship lock, fish ladder or fish canal)
b. canal intake
c. headrace (headwater canal) and appurtenant structures (aqueducts, power tunnels, highway and railway bridges, siphons and aqueducts for intersecting water courses across the canal) or a freeflow tunnel
d. headpond with spillway and gate or valve chamber
e. penstock or a pressure shaft
f. powerhouse or an underground (cavern) station
g. tailrace (sometimes tailwater tunnel)
h. switchyard or an underground (cavern) switch gear equipment.

2. Pressure tunnel development

Structural elements:

a. dam or weir
b. intake or headwork
c. pressure tunnel through mountain
d. surge tank(s)
e. penstock (with valve chamber) or a pressure shaft
f. power house or mostly an underground (cavern) station
g. tailrace or tailwater tunnel
h. switchyard or an underground (cavern) switch gear equipment

What is very popular nowadays is the underground development, which often is the cheapest solution. The powerhouse is installed in a cavern in the rock. The penstock is in this case replaced by a pressure shaft.

3. Dam development

A high dam with the power station at the toe of the dam. This arrangement could be termed plant with concentrated fall or dam station development. The maximum head for the power station approaches the height of the dam. The main parts of this type of power plant:

a. water intake (generally built on the upstream face of the dam)
b. pressure conduit (transversing the dam body or bypassing the dam through the adjacent rock)
c. powerhouse
d. switchyard

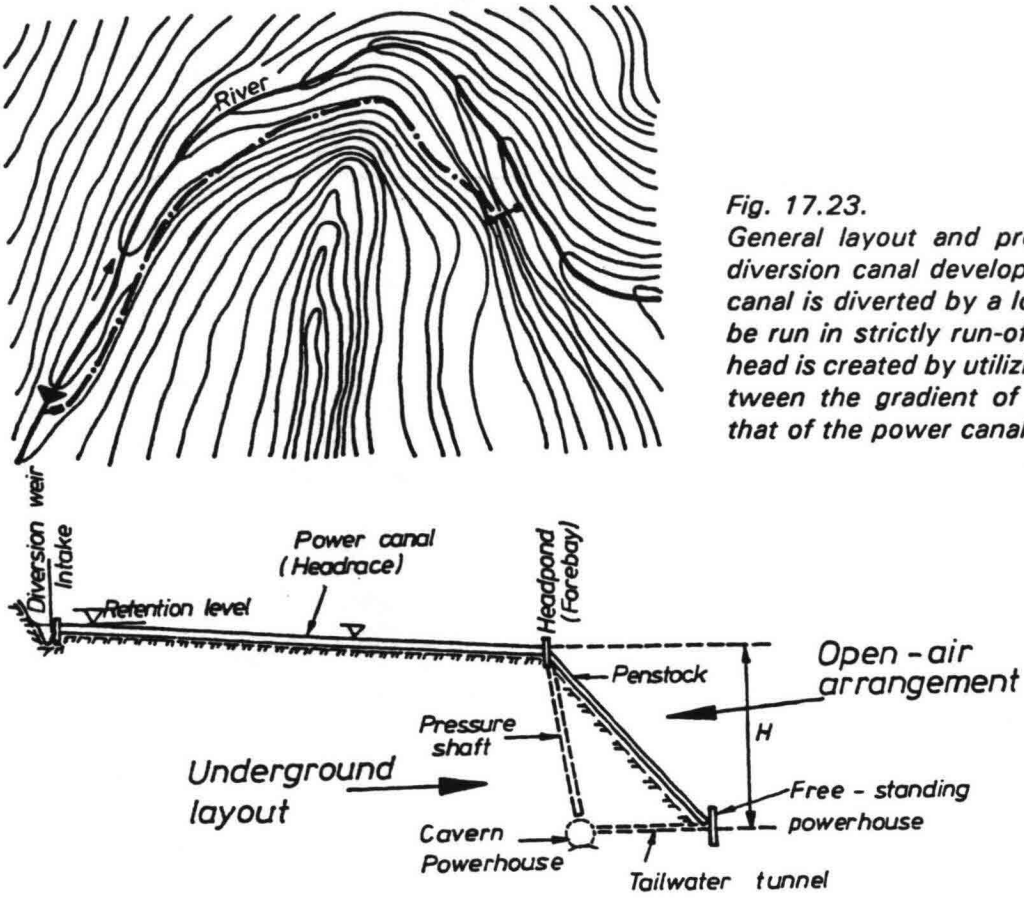


Fig. 17.23.
General layout and profile of a high-head diversion canal development. Water for the canal is diverted by a low weir. Can usually be run in strictly run-of-river operation. The head is created by utilizing the difference between the gradient of the river valley and that of the power canal.

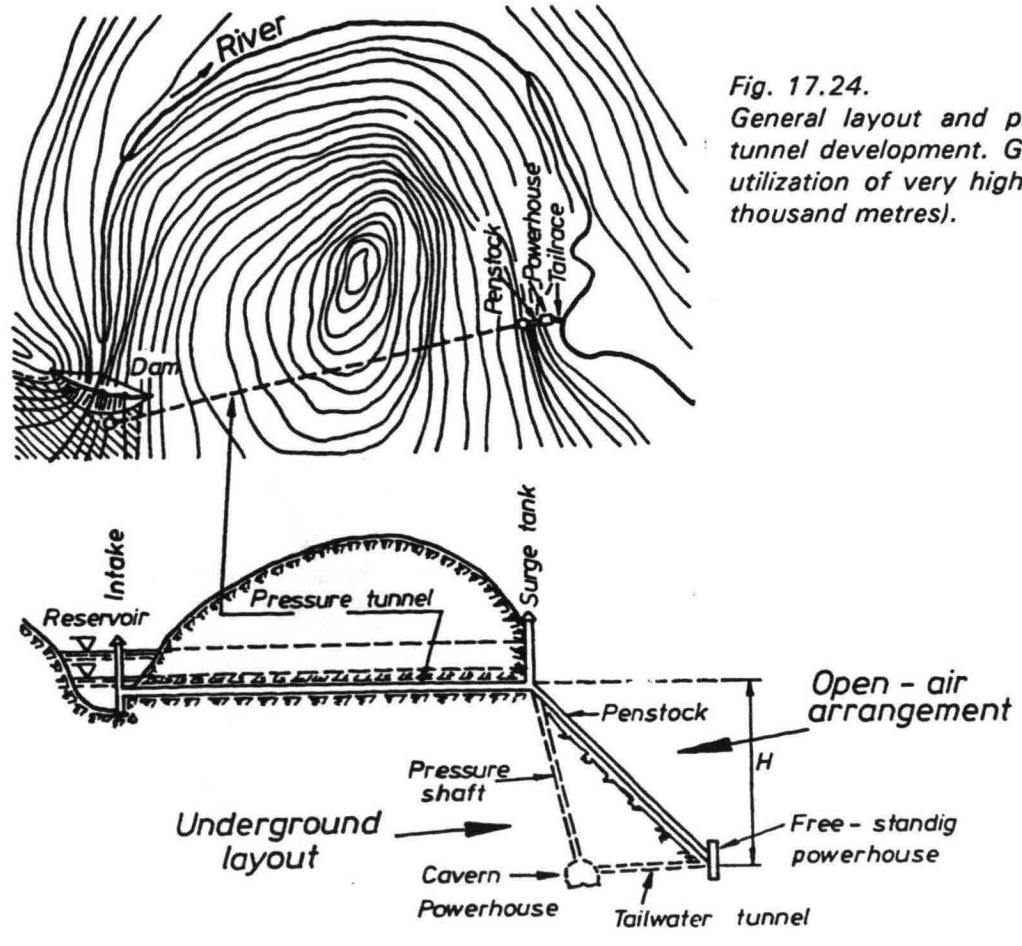


Fig. 17.24.
General layout and profile of a pressure tunnel development. Generally used for the utilization of very high heads (even over a thousand metres).

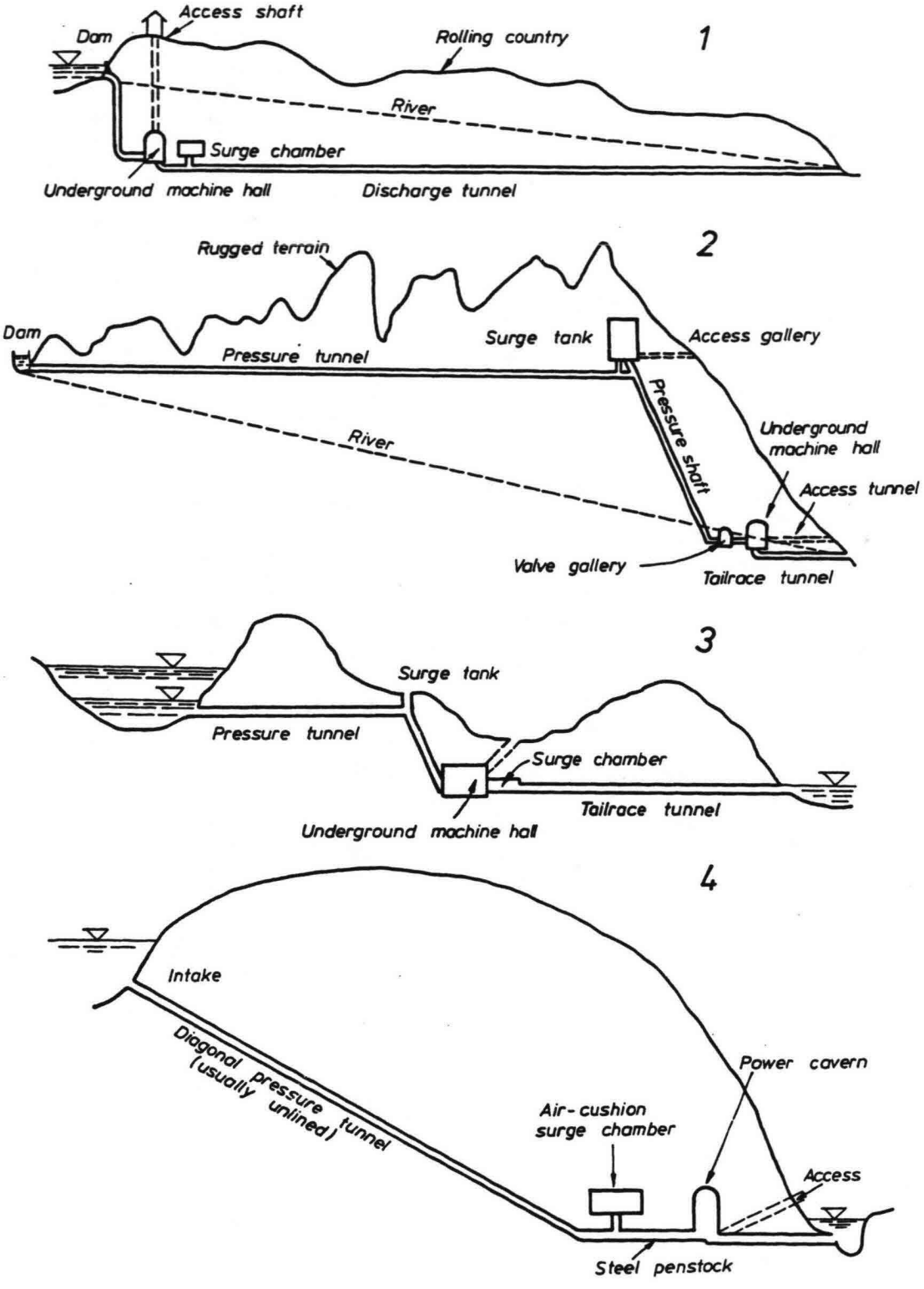


Fig. 17.25. Underground power stations.

Characteristic types of underground power developments
1. Upstream station arrangement (called Swedish type of development)
2. Downstream station arrangement (called Swiss type of development)
3. Intermediate station arrangement (Italian arrangement)
4. Diagonal tunnel alignment with air-cushion surge tank (Norwegian solution).

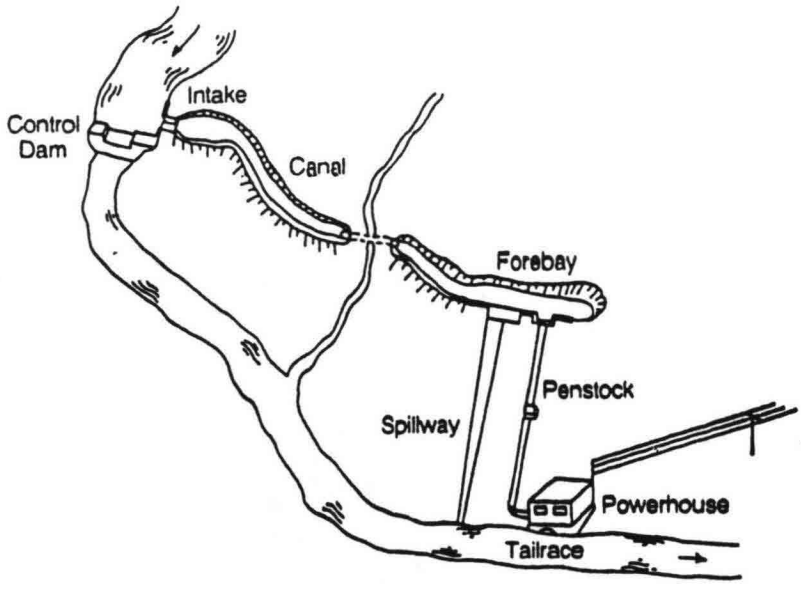
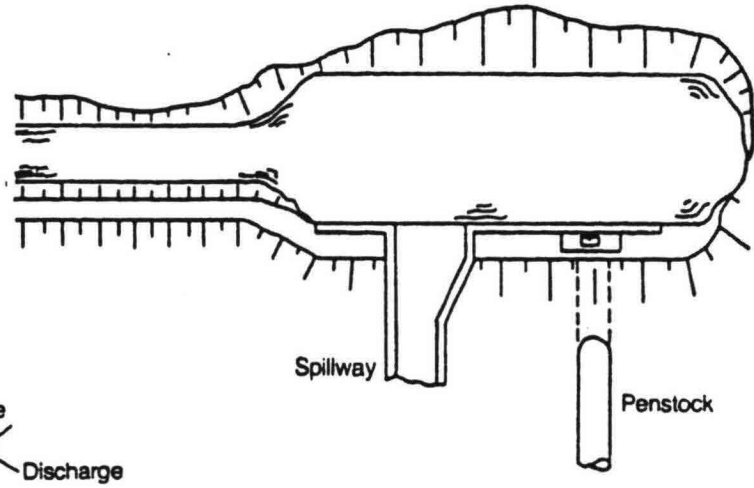
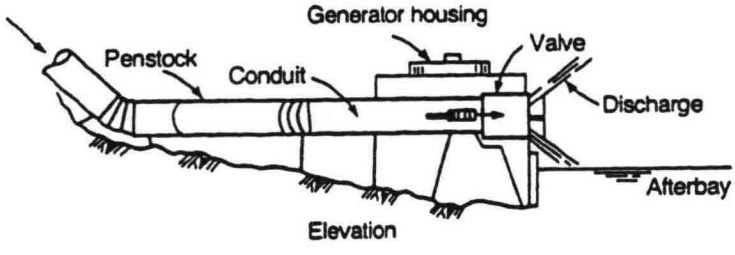


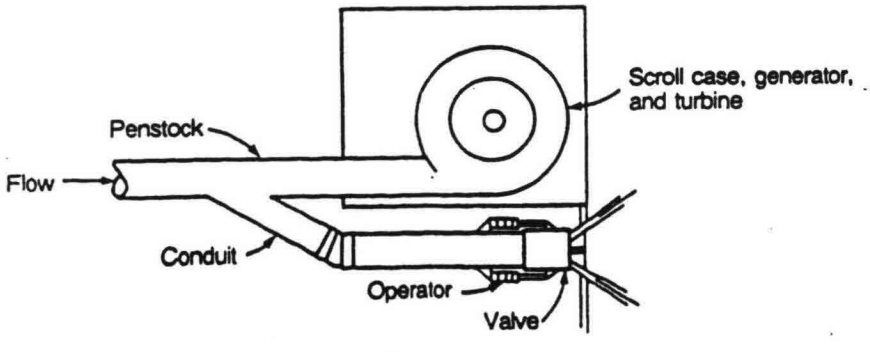
Fig. 17.26
Typical configuration for a high-head site.
Small high-head hydropower projects will almost always utilize a penstock or tunnel to develop the necessary head. A small diversion dam is generally required (concrete, masonry) and this dam will be designed for overflow. A canal conveys water diverted from the river to the upper end of the penstock.

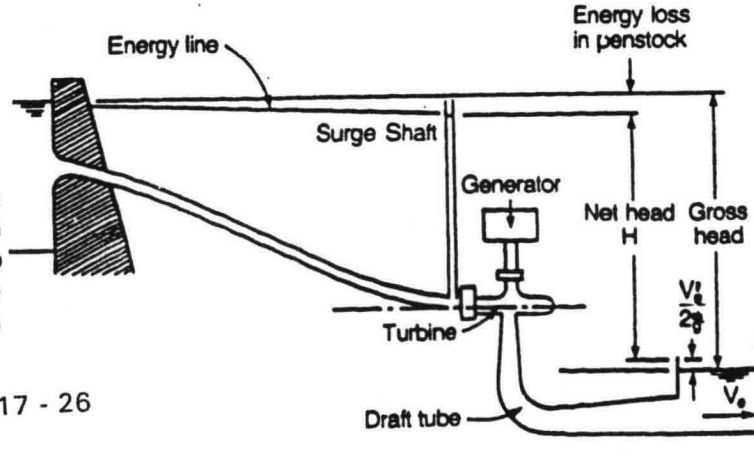
Typical canal section and a typical expanded section at the downstream end of the bay: the penstock forebay. The spillway provides flood discharge and keeps the canal from overflowing if the turbines are shut down but flow into the canal continues. Diversion to the penstock should be made through a gate structure constructed in the side of the canal.



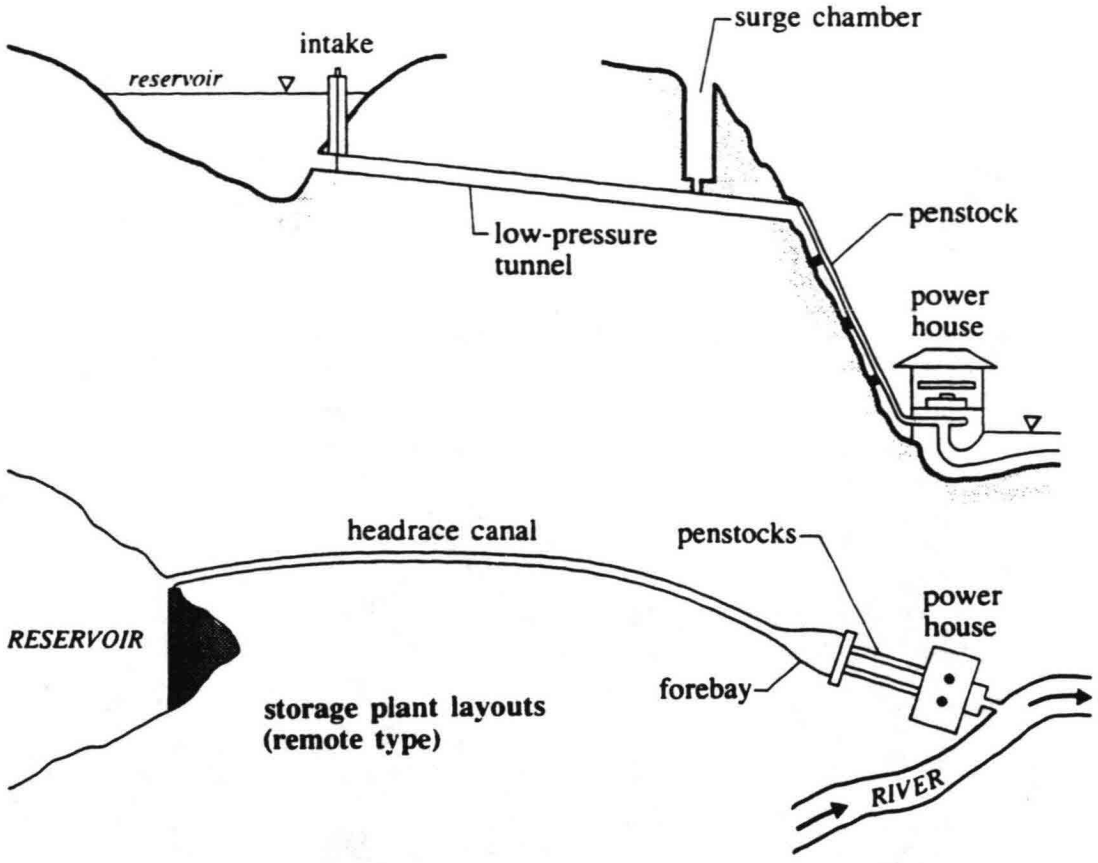


Arrangement of a synchronomous bypass valve; they are arranged and controlled so that flow is slowly established in the penstock by opening the valve prior to starting up the turbine. As the turbine is brought up to speed, the valve closes at a synchronized rate to maintain constant flow in the penstock. For shutdown the process is reversed.





Definition sketch for net head; it shows schematically the energy gradient through the system. The net head H is equal to the difference of the static water surface elevations upstream and downstream minus the sum of all losses.



Storage plant layouts (remote type).

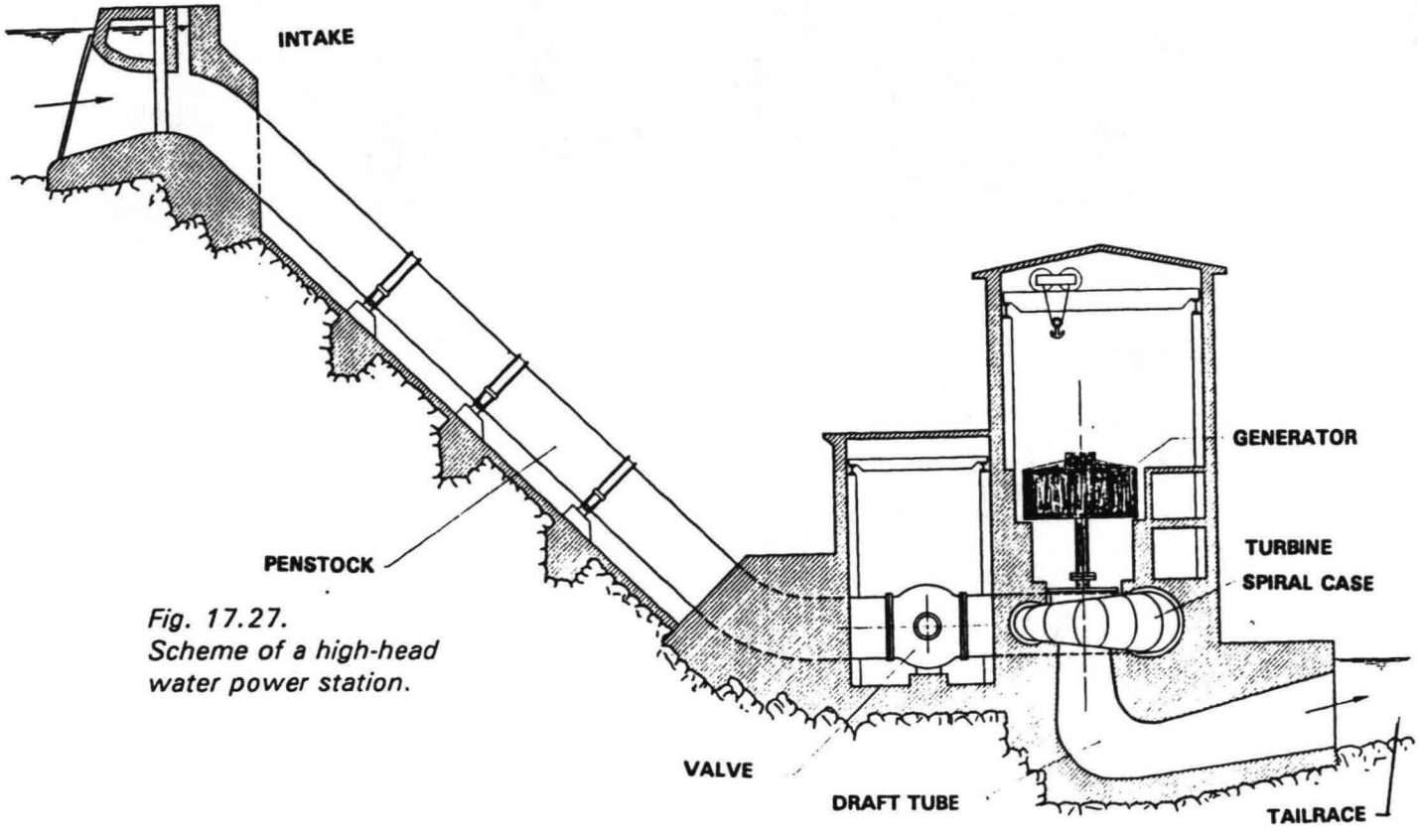
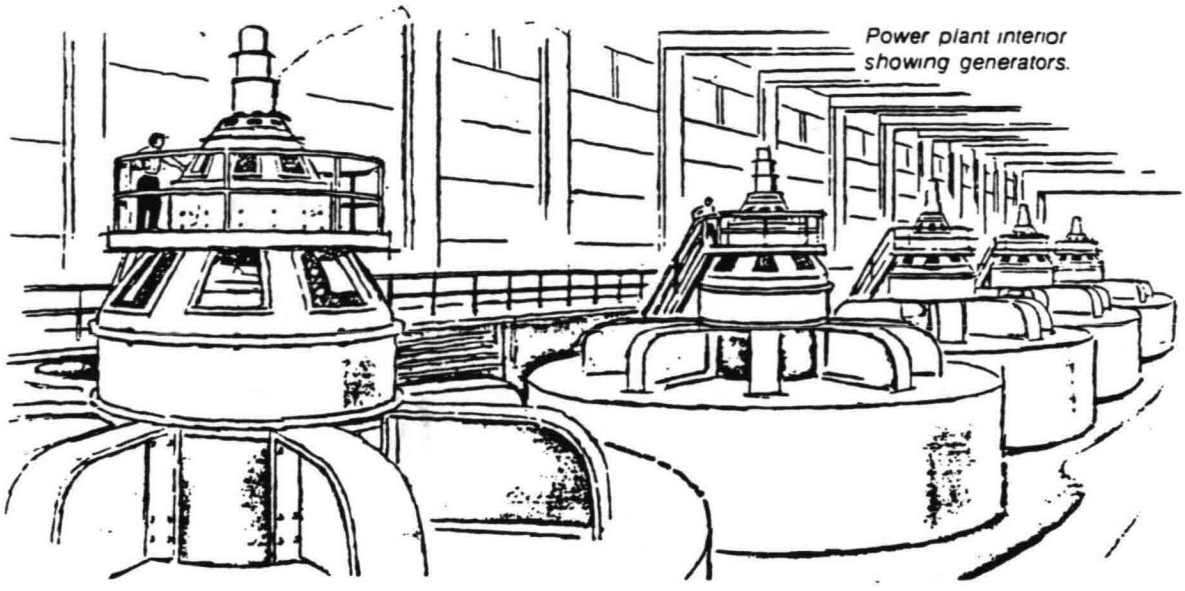
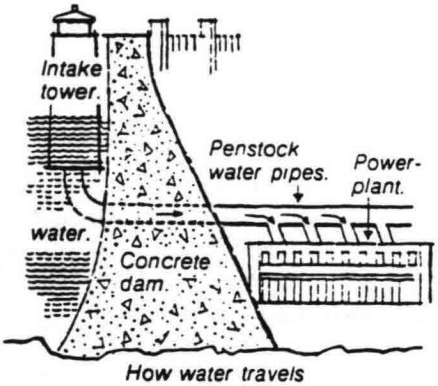


Fig. 17.27.
Scheme of a high-head water power station.

Fig. 17.28 Hoover (Boulder) dam and twin power station, Colorado River, Nevada-Arizona, USA. A huge reservoir, Lake Mead, having a storage capacity of 38,000 million m^3 was created by the max. 222 m high gravity dam. The total capacity installed at the twin power station is 1,320,000 kV.
In narrow gorges, where the space is limited and the possibility of locating the high-capacity power station at the toe of the dam is excluded, the power house may be constructed in the gorge walls either as a single block or divided into two. This arrangement is exemplified by the Hoover (formerly Boulder) development.

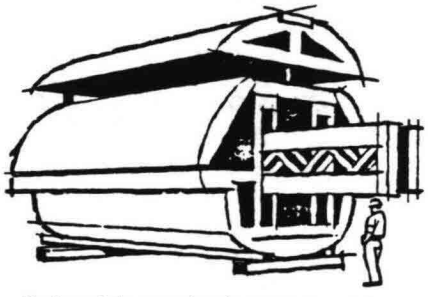


Power plant interior showing generators.



How water travels to the turbines.

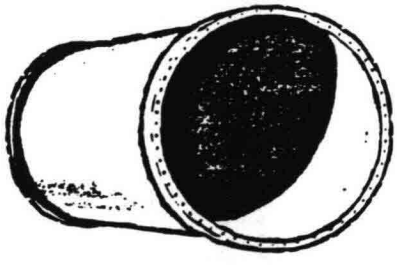
PENSTOCKS



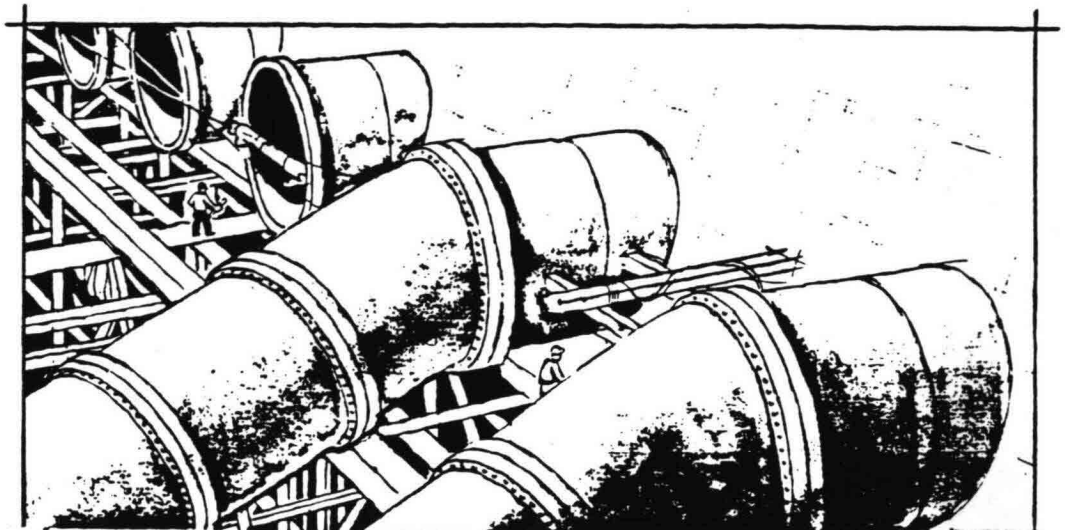
Collapsible wooden forms are used to make openings in the dam when concrete is poured around them.

Concrete is poured over round wooden forms that are shaped like long tin cans. After the concrete has hardened, the forms are removed and big steel pipes called *penstocks* are put in the holes left in the concrete.

These penstocks will carry the water through the dam to the generators.



After forms are removed huge steel pipes are placed in the holes.



17.3. Pumped-storage Schemes

The basic principle of pumped storage is to store the surplus electric energy generated by a power plant, or available in a system in off-peak periods, in the form of hydraulic potential energy in order to regain it in periods when the peak demand on the system exceeds the total capacity of the generating station.

A pumped storage project consists basically out of two basins, a lower basin (tail pond) and an upper basin (head pond), and a pressure conduit with a power station. The water is released from the upper basin to drive a turbine during periods of peak demands (high tariff). During night time, when electricity is cheap, the water is pumped back to the upper basin. Pumped storage schemes can therefor only be used to provide extra power during peak hours.

The most important progress in the last decades was the introduction of reversible water machines, the so-called pump-turbines. In recent years, a number of **'pumped-storage'** schemes have utilised a special type of Kaplan unit. This unit operates as a turbine during periods when the demand for electricity is high. However, when demand is low during the night, the blade angle is then reserved and the unit is motor-driven so as to act as a pump.

There are two main types of pumped-storage schemes:
a. pure pumped-storage plants
b. mixed pumped-storage plants.

The pure pumped-storage type serves the sole purposes of energy storage. There are various types of mixed-type but basically water is pumped up to a higher reservoir (delivery head H_1) and released to a separate power station on another location (with head H_2), whereby $H_2 > H_1$.

The 1800 MW Dinorwig pumped storage plant in North Wales, completed in 1983, is the largest U.K. example.

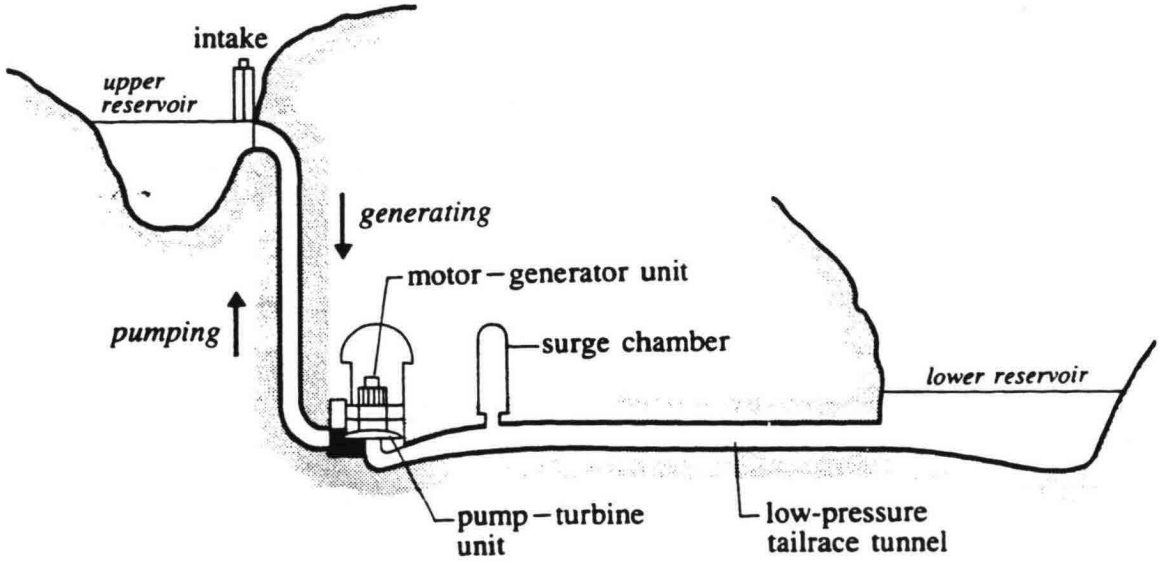


Fig. 17.29. Pumped storage plant

17.4. Tidal power

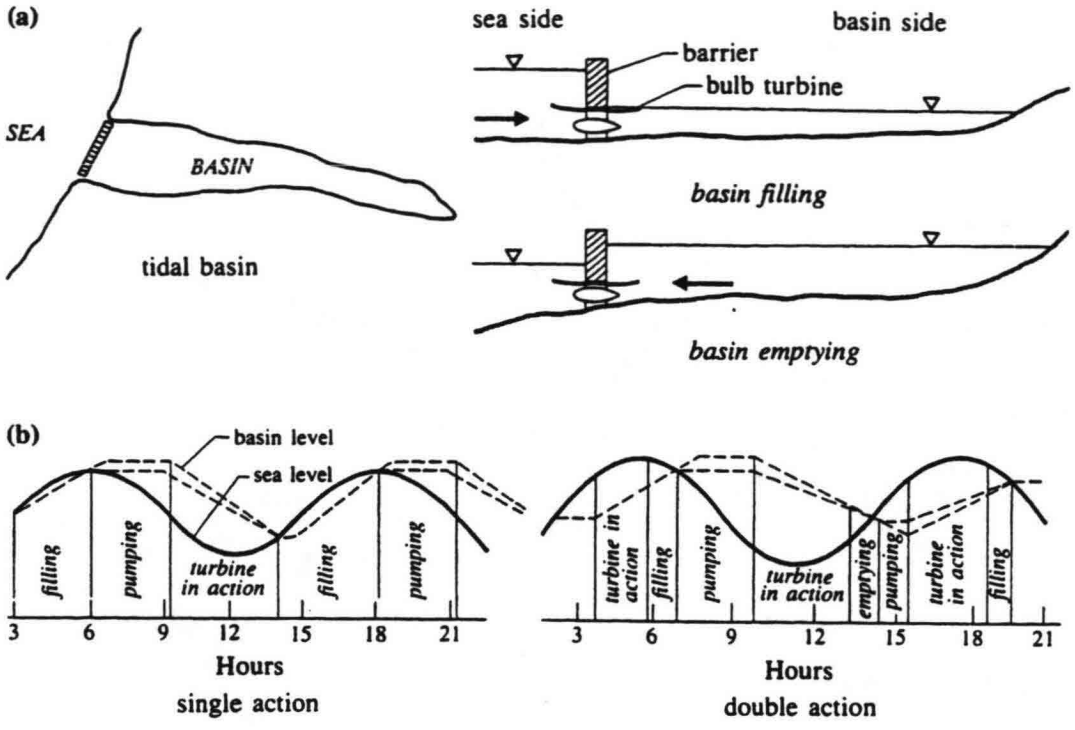


Fig. 17.30. Tidal power development

The use of tidal energy dates back to the 12th century, when tidal mills worked along the coast of Brittany, France. The basic principle of their operation was to form a storage basin by constructing a dyke closing off a cove; the basin filled through gates during the flood tide, and during the ebb tide it emptied through an undershot wheel, thus producing a driving force. The operational principle of a tidal power plant remains the same as that of the tidal mills, but the turbine units may produce power during filling as well as emptying of the basin.

Tidal amplitudes attain considerable magnitudes along certain coastal stretches (Canadian Atlantic coast 13.5 m, Bristol Channel, UK, 10 m, French Atlantic coast, 8 m; figures relate to mean annual ranges). In the Pacific region, e.g. along the coasts of China and USSR, mean amplitudes of 6-9 m have also been recorded.

Electric generation by tidal energy has been practised at the estuary of the River Rance near St.- Malo (France) since 1968 with 24 pump turbines of 10 MW each, generating a nett annual output of 540 GW-h, mostly on the ebb tide, usually for about two hours in 12. Pumped storage is used.

Watermills

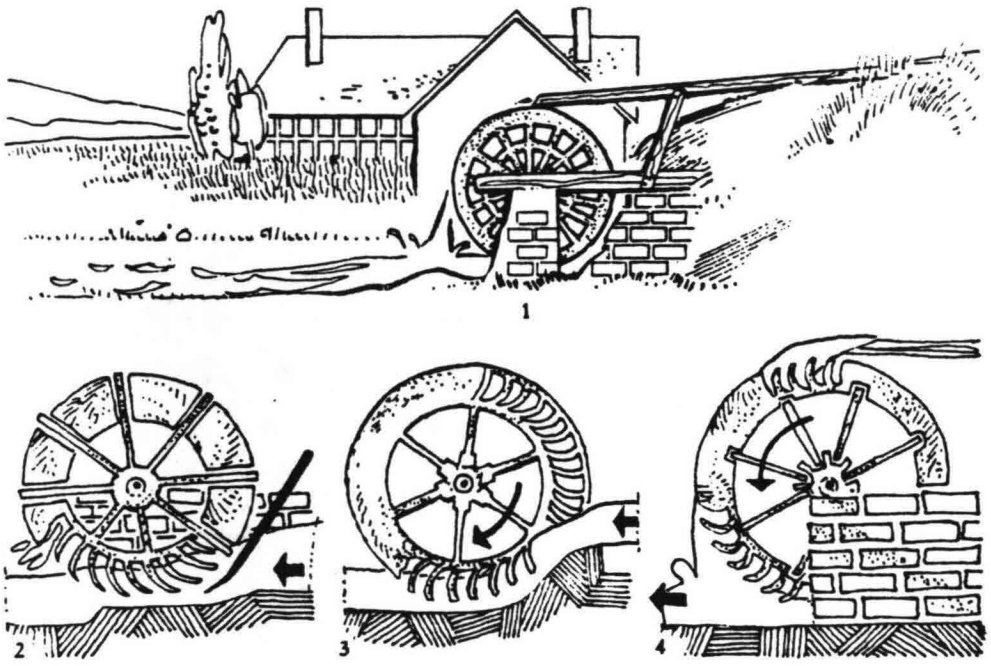


Fig. 17.31.
Watermills.
1. Overshot
2. Undershot
3. Breastshot
4. Overshot

Fig. 17.32.
Gallo-Roman mill of the 2nd/3rd century A.D. at Barbegal near Arles in France.

18. Docks

18.1. Dry docks

Dry docks are used for construction, repairing and maintaining of sea ships and are therefore usually situated near a port. The entrance to the dock must be near deep water. A dry dock is, generally speaking a chamber or basin separated from the adjacent harbour waters by a dock gate (Fig. 18.1). Its basic structure comprises a floor, side walls, head wall and dock gate which define the dock chamber.

The part of the dock structure located at the entrance, called the dock head, is the support for the dock gate and for all auxiliary devices necessary for docking a ship. The floor of the dock in that part is constructed as a sill and the side walls as vertical faces for the dock gate. Flooding and dewatering of the dock is carried out by means of pumps and other installations usually located in a pumping station.

When the gate is open, the water level in the dry dock is the same as in the harbour basin. It is then possible to bring in the ship, i.e. to dock it, or to float it out of the dock, i.e., to undock it. If the gate is closed, the water level in the dock can be lowered by pumping out the water so that the docked ship can settle on supports placed on the dock floor. These supports consist of keel blocks, side blocks and bilge block. Their number and spacing depend on the type and size of the docked ship.

Docks of great length can be partitioned off by inner gates, while in some cases a dry dock may have two entrances. The second entrance is especially useful when the dock has an inner gate. The modern dock has vertical walls with cantilever altars because many modern ships have vertical side walls. The main crane can now be situated closer to the ship.

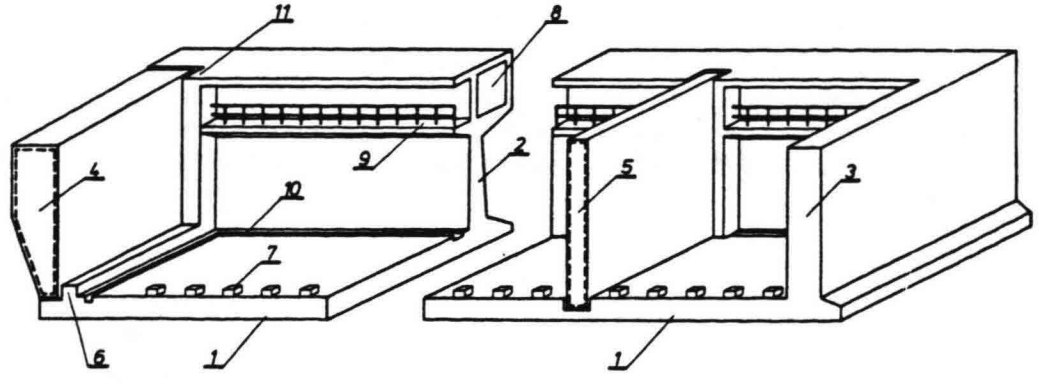


Fig. 18.1.
General scheme of a dry dock.

1. dock floor
2. side wall
3. head wall
4. dock gate
5. inner gate
6. sill
7. keel blocks
8. culvert
9. barriers
10. dewatering channel
11. gate supports

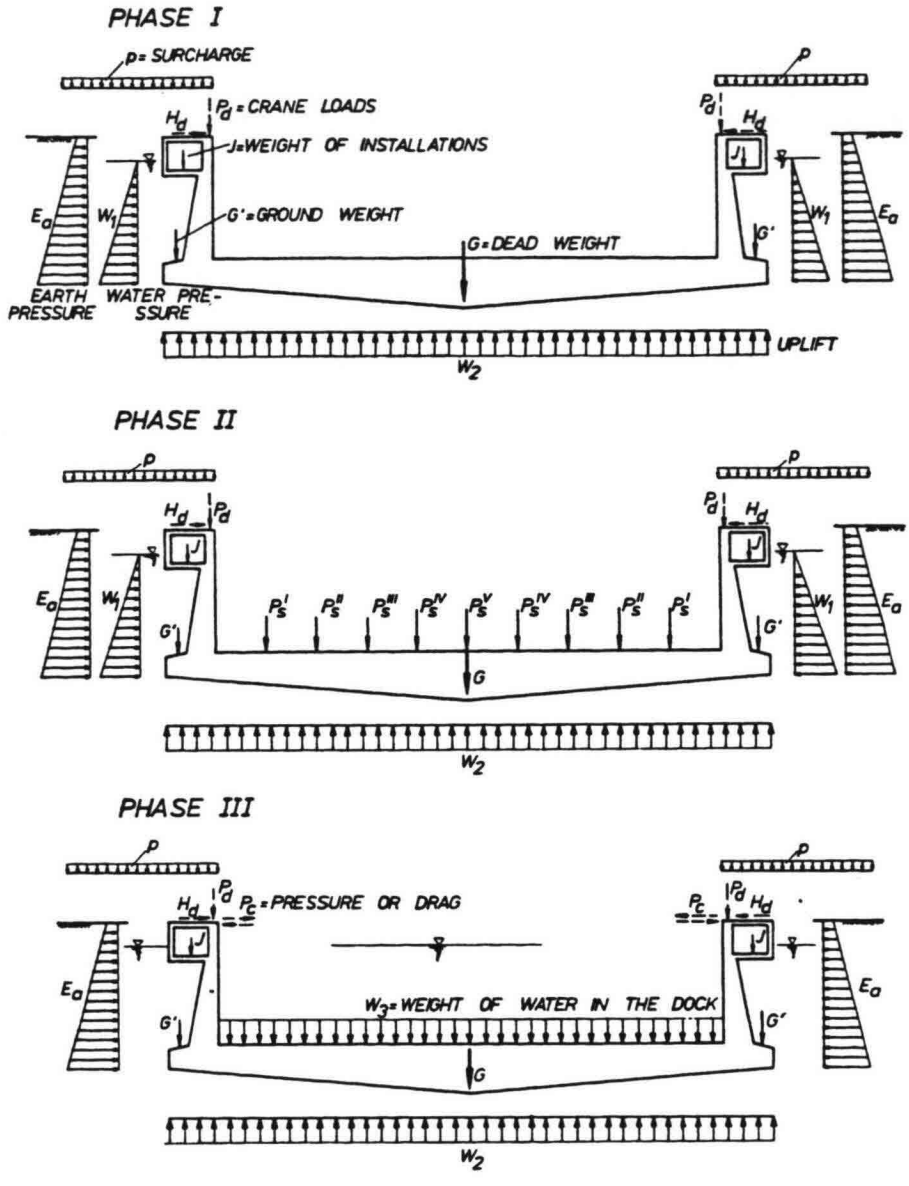


Fig. 18.2.
Loading states.
Phase I: dock in the dry without vessel inside.
Phase II: dock in the dry with vessel resting on central keel-blocks.
Phase III - dock flooded.

The simplest dock designs are those in which the dock is not subject to upward hydrostatic pressure beneath the floor and those with such conditions of permeability of the strata that relief of pressure by drainage is possible. Where the soil foundation is rocky, it is not difficult to bear the ship's loads and, in general, a thin layer of floor-concrete is sufficient to provide a good working surface.

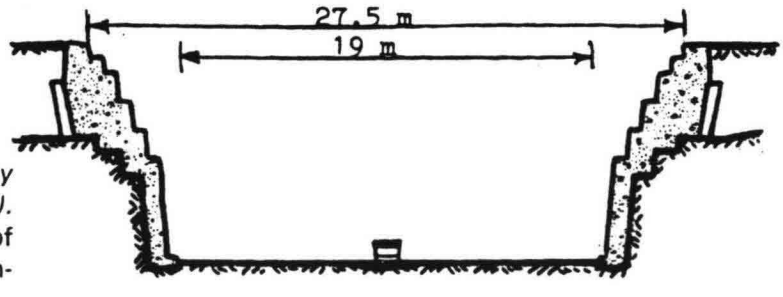


Fig. 18.3.
Cross-section of a dry dock at Milford (U.K.).
The walls are made of mass concrete construction.

The hydrogeological conditions fundamentally affect the structural solutions of the dry dock. For a dry dock constructed onshore under the influence of groundwater, or offshore, the basic dock load will be the hydrostatic uplift. The hydrostatic uplift may not act when the dock is sited in sound rock or very firm cohesive soils.

The structural solution of dry docks differs first of all in the method of counteracting hydrostatic uplift. Accordingly dry docks may be divided into three basic categories:
a) **Heavy or gravity docks**, in which the permanent loads are larger than the largest hydrostatic uplift.
- with separate flood slab and side wall which, however, interact in transferring the hydrostatic uplift
- with heavy floor slab balancing the whole hydrostatic with its own weight
- with monolithic frame structure of floor slab and walls.

b) **Anchored docks**, in which the largest hydrostatic uplift is totally balanced by the permanent weight of the dock structure and by a proper anchorage in the ground:
- with piles
- with anchoring plates
- with ground anchors.

c) **Drainage docks** in which the hydrostatic uplift is reduced by a proper drainage system under the dock floor:
- fully drain
- partly drained
- on impervious subsoil.

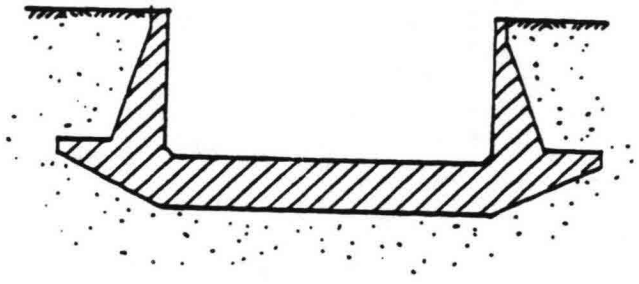


Fig. 18.4.
Many modern docks have been built in which a substantial part of the weight resisting uplift is supplied by fill. The construction is of reinforced concrete with toes.

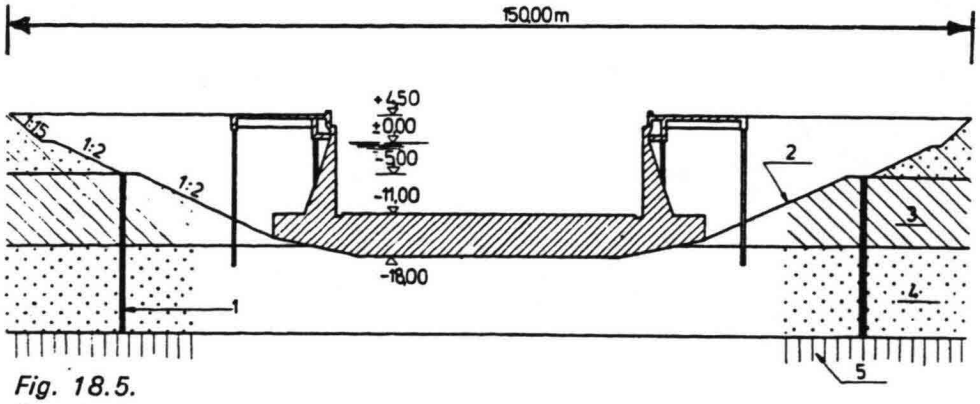


Fig. 18.5.
Dry dock structure as a concrete frame.
1. water tight curtain
2. excavation
3. mud
4. sand
5. clay.

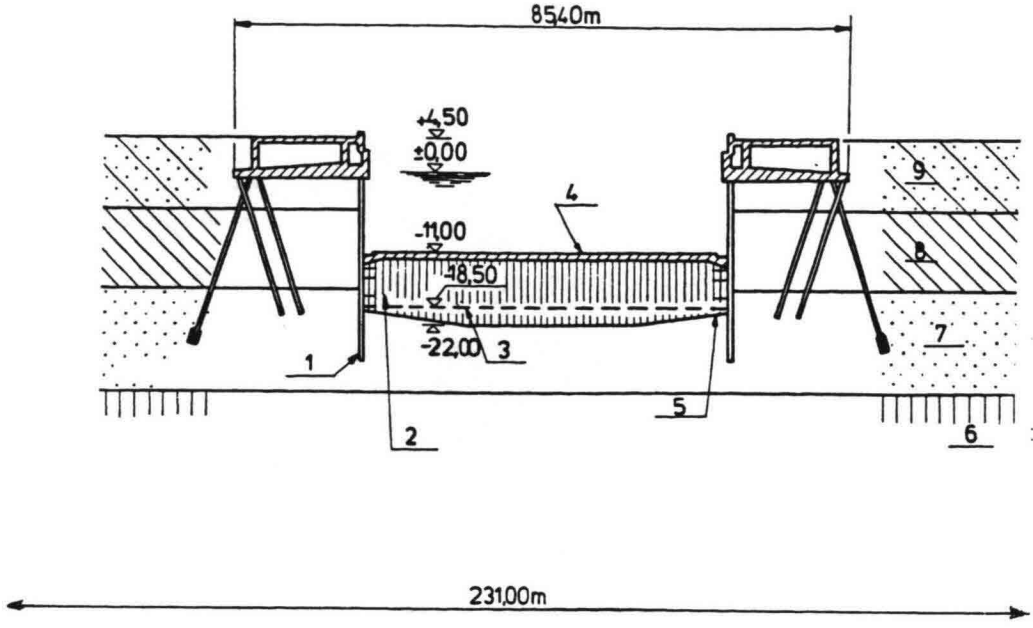


Fig. 18.6.
Dry dock structure as quay walls and underwater concrete floor. Heavy dock with separate floor slab and side wall.

1. steel sheetpiling
2. underwater concrete with gravel fill
3. floor thickness at underwater concrete with fill of lead slag
4. finishing layer to cast in dry
5. plastic foil
6. clay
7. sand
8. mud
9. sand.

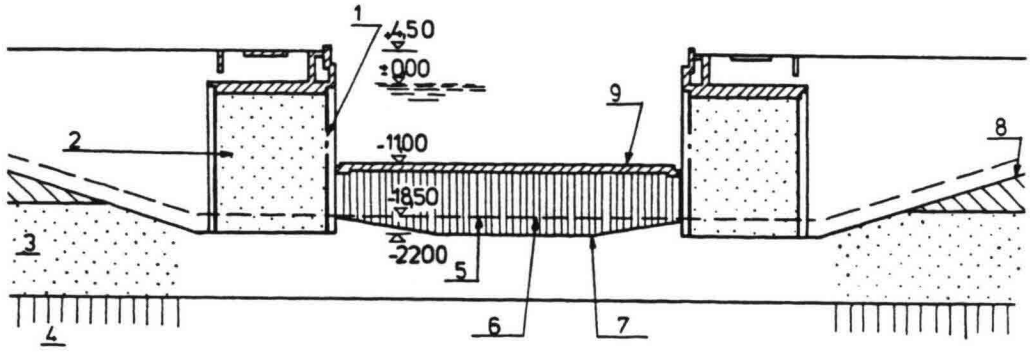


Fig. 18.7.
Dry dock structure as cellular cofferdams and underwater concrete floor.

1. drainage opening
2. cellular cofferdams with sand fill
3. sand
4. clay
5. underwater concrete with gravel fill
6. floor thickness at underwater concrete with fill of lead slag
7. plastic foil
8. dredging line
9. finishing layer to be cast in the dry.

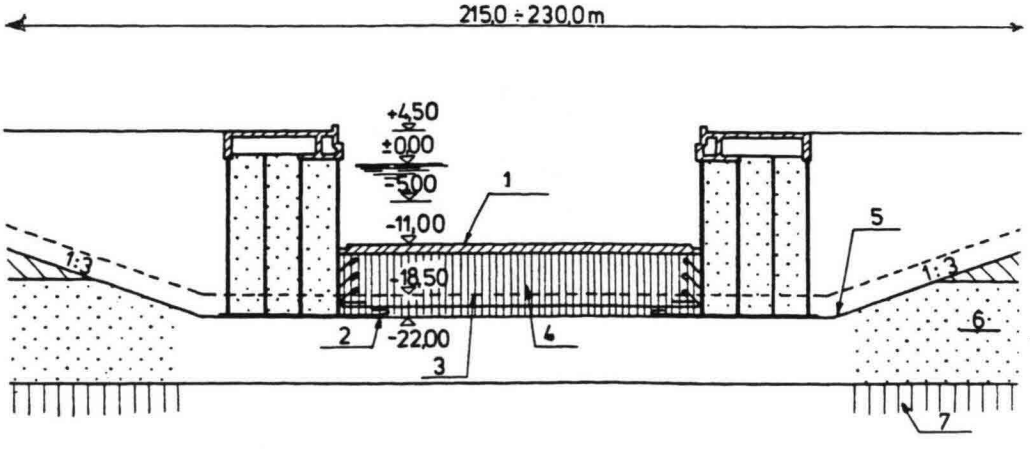


Fig. 18.8.
Dry dock structure as concrete caissons and underwater concrete floor.

1. walls and finishing layer to cast in the dry
2. plastic foil
3. floor thickness of underwater concrete with fill of lead slag
4. underwater concrete with gravel fill
5. dredging line
6. sand
7. clay

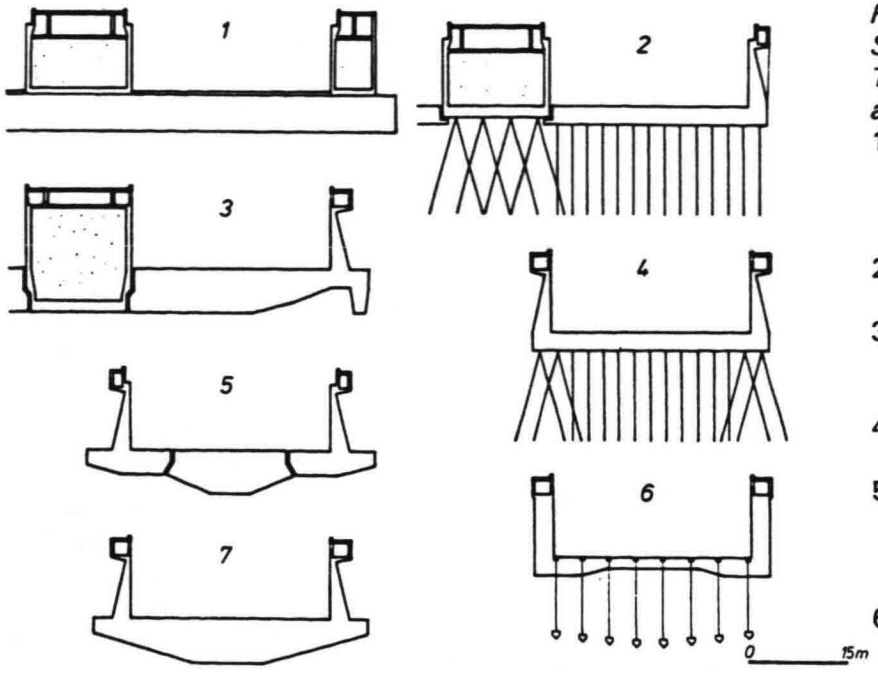


Fig. 18.9.
Several types of docks. The buoyancy is counteracted by various means.

1. heavy dry dock with concrete floor slab and walls as caissons filled with sand
2. anchored dry dock on piles
3. heavy dry dock with separate floor slab and concrete caisson
4. anchored dry dock as a concrete frame on piles
5. heavy dry dock with separate floor slab and walls as concrete retaining wall
6. anchored dry dock with concrete plates in ground
7. heavy dry dock as concrete frame.

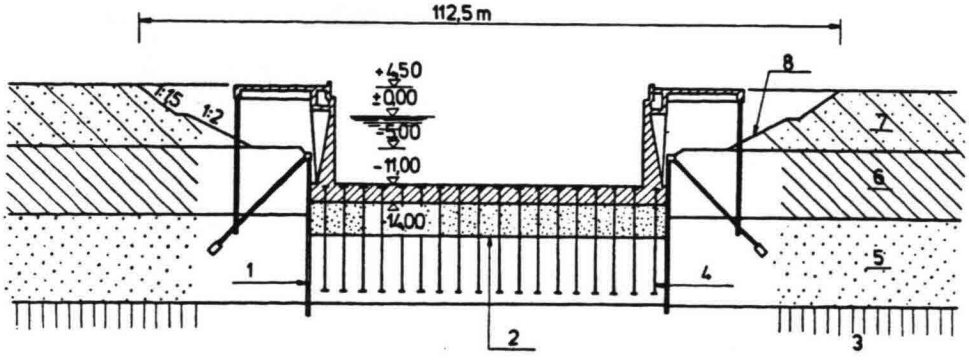


Fig. 18.10.
Dry dock structure as anchored concrete slab.

1. steel sheetpiling
2. soil improvement
3. clay
4. steel anchors
5. sand
6. mud
7. sand
8. excavation line.

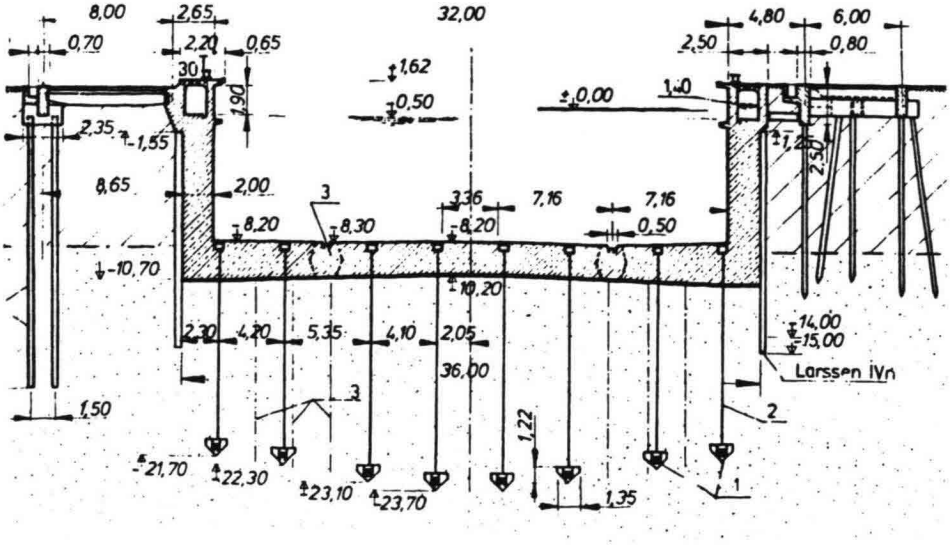


Fig. 18.11.
Cross-section of dry dock at Emden (Germany) (1954).
The floor is anchored by means of pre-stressed steel cables anchored to reinforced concrete blocks of cruciform cross-sections. The blocks were jetted into the sandy soil to depth of between 11 and 14 m.

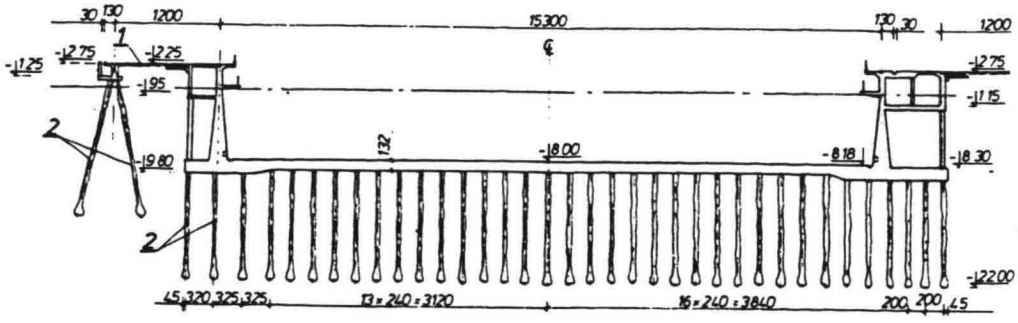


Fig. 18.12.
Anchored dry dock considered for the dry dock in Gdynia, Poland.

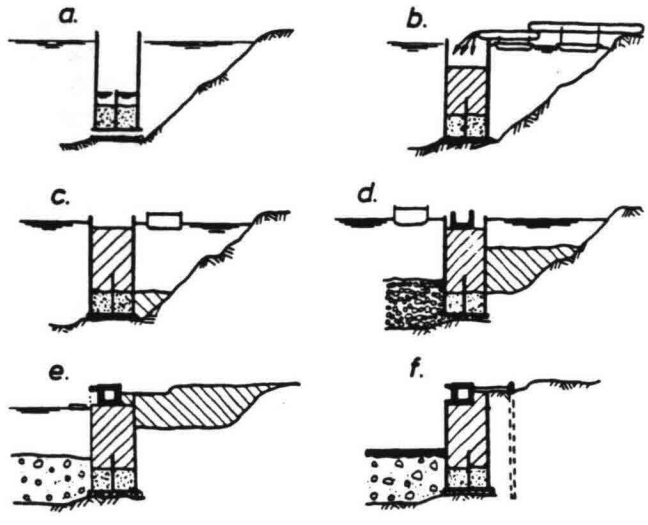


Fig. 18.13.
Placing caissons, filling and subway construction of the dry dock in Scaramanga, Greece.

a. caisson floated over prepared bed sunk by water ballast
b. caisson filled with soil by conveyor
c. partial backfill behind caisson by barge
d. floor stone placed, further backfill and subway construction started
e. floor grouted, overheight of front wall of caisson demolished, backfill continued from land
f. floor topping laid, crane track piles and beam constructed.

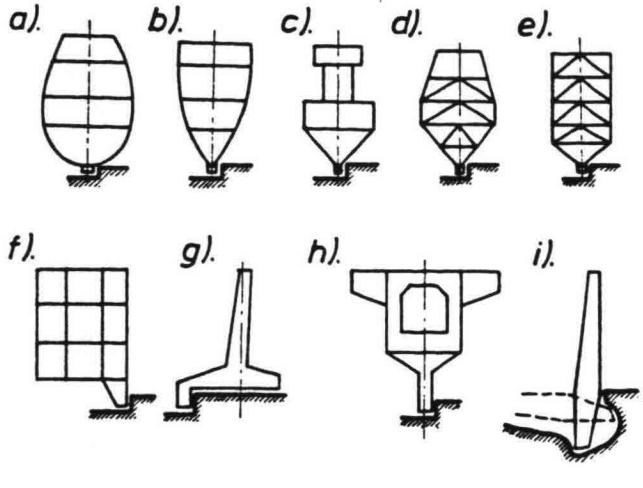


Fig. 18.14.
Shapes of dock gate transverse cross-sections.

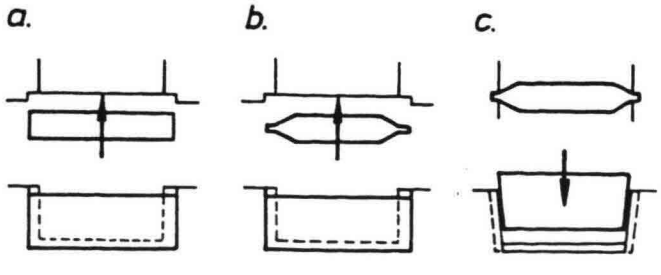


Fig. 18.15.
Shapes of dock gate longitudinal cross-sections.

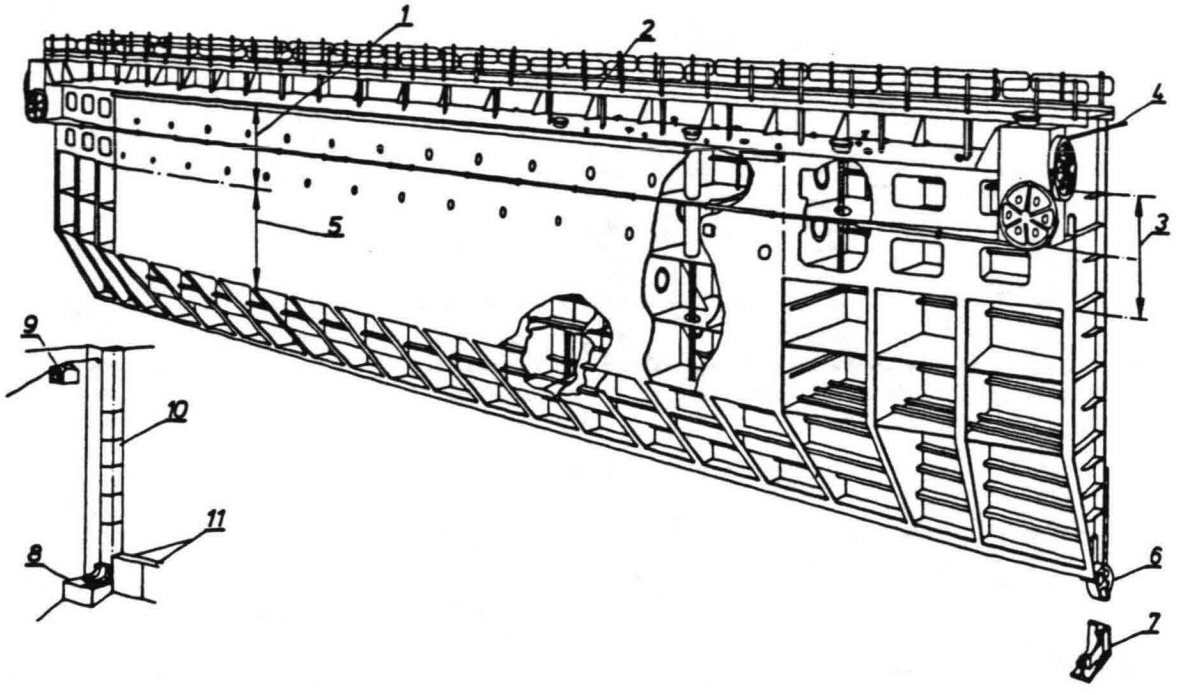


Fig. 18.16.
General arrangement of a flap gate.

1. free-flood area
2. removable walkway
3. tide range
4. gate haulage rope
5. buoyancy and water ballast chambers
6. pivot
7. bearing
8. hinge pedestal
9. wall shave
10. quoin stones
11. sill stones

Dry dock Largest World

The largest dry dock in the world is that at Koyagi, Nagasako, Japan completed in 1972. It measures 990 m long; 100 m in width and has a maximum ship building capacity of 1,000, 000 tons deadweight.

Dubai Dry Dock

In 1971 a plan for building a dry dock in Dubai, United Arab Emirates was approved by the Ruler of Dubai. At that time plans for building a dry dock were motivated by the upturn in the tanker market, which was going in for bigger and bigger ships. In 1973 a $300 million contract was signed to build the world's largest and most sophisticated ship repair complex, the Dubai Dry Dock. The Dubai Dry Dock and Ship Repair Facility was opened in 1979. It consists of three docks:

- one of 525 x 100 m for tankers of 1,000,000 DWT
- one of 415 x 80 m for tankers of 500,000 DWT
- one of 370 x 60 m for tankers of 350,000 DWT

The walls of these docks are constructed with slip formed reinforced concrete caissons of 31 m long, 17 m wide and 18 m high; 162 number of these caissons were needed. These caissons, weighing 3500 tons each, were constructed on land and with aid of a synchro-lift launched to be transported over water to its final place. The harbour basin was in the meantime dredged to -11.5 m.

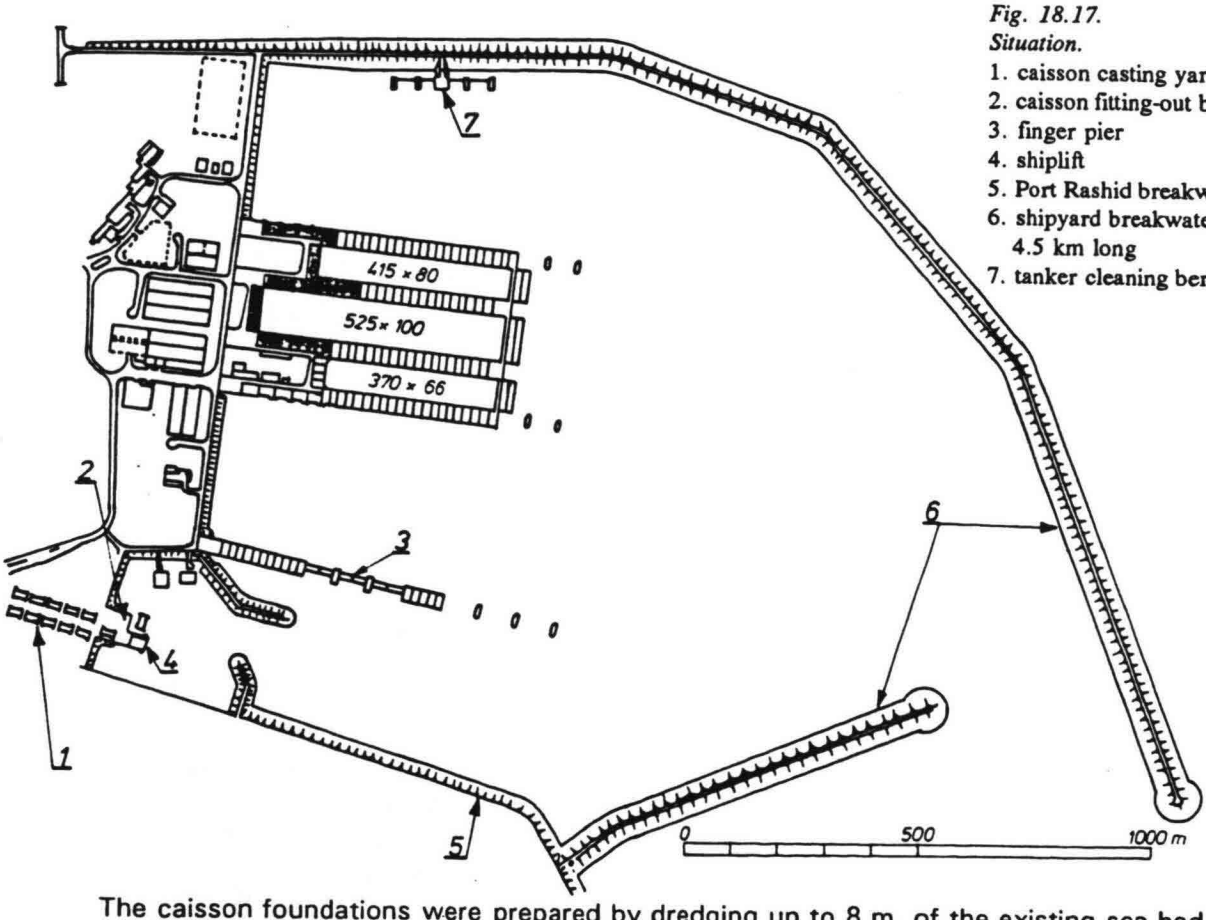


Fig. 18.17.
Situation.
1. caisson casting yard
2. caisson fitting-out berth
3. finger pier
4. shiplift
5. Port Rashid breakwater
6. shipyard breakwater 4.5 km long
7. tanker cleaning berth

The caisson foundations were prepared by dredging up to 8 m. of the existing sea bed to give clean firm rock at a depth of 14.2 m. After final positioning, the caissons were ballasted with water to sink them on their foundations. The void between the caisson underside and the seabed was pumped full of cement grout.

The docks are dewatered by five main 2,650 hp pumps installed in two pump houses located at the seaward end of each of the two central piers. The 1.5 m thick dock floors required 170,000 m^3 of concrete. The three gate sills each consists of a heavily reinforced concrete beam up to 7 m thick and 22 m wide. A 10 m deep reinforced concrete barrier has been built below the toe of the sill to from a cut off to prevent water seepage under the sill.

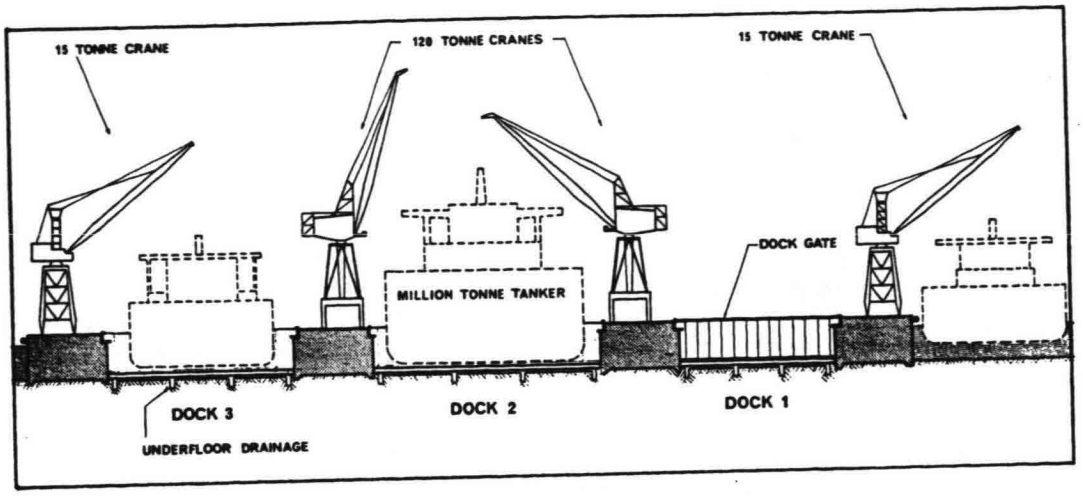


Fig. 18.18. Cross-section.

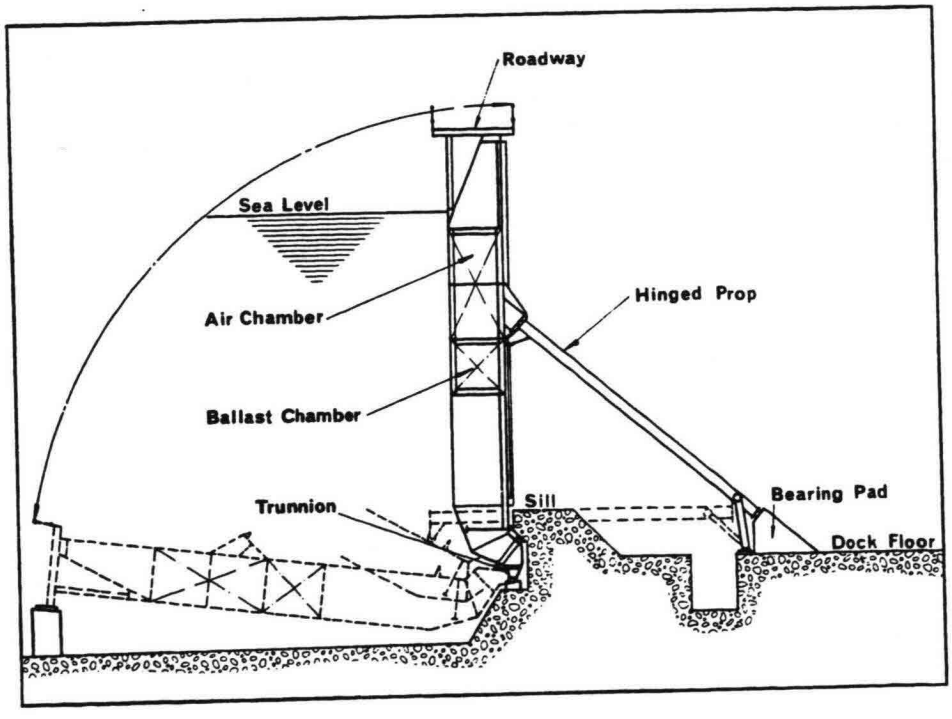


Fig. 18.19.
The gates, which are 100 m, 80 m and 66 m long are flap gates made from steel. The gates are formed by buoyancy modules, with stiffened plates between. The modules are hinged outside and below the dock entrance.

18.2. Construction docks

Construction docks are only used for building a ship hull. They have less depth, and are therefore cheaper than a dry dock. In a construction dock, only the hull of the ship is constructed which has less draught (draft) than a completed ship. After finishing the dock-activities, the ship hull is transported in floating condition to a working pier, where the ship is completed and the draught increases.

Dry docks for fabrication of offshore structures are, in general, larger structures than those for shipbuilding. These facilities are equipped with cranes and other support facilities required for fabrication or construction of large and heavy structures, which are outside the capacities of the fabrication yards, and can be floated out for transport to offshore locations.

The dimensions of Kishorn dry dock, Scotland, are 180 x 170 x 11.5 m deep. This facility, with its deep-water mooring site and various fabrication and paint shops, is a typical example of the dry dock suitable for fabrication of large steel jackets.

18.3. Floating docks

Floating docks consist of a steel pontoon composed of sections with vertical walls. Bottom and walls are divided in water tight compartments. By supplying the compartments with water, the dock sinks and the ship sinks and the ship enters the dock between the vertical walls. By emptying the compartments the floating dock rises till the ship rests on the keel- and bilge blocks. At last the dock floor is dry, and the bottom of the ship can be repaired. The advantage of floating docks is the possibility of changing the location; they can easily be towed to another place.

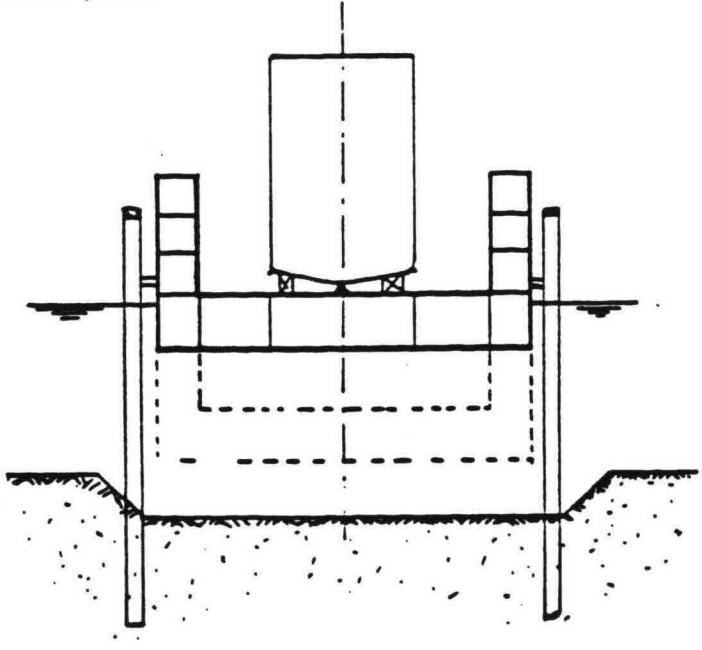


Fig. 18.20.
Floating dock.

18.4. Harbour docks

A harbour basin with a lock at its entrance is also called a dock, a so-called **'wet-lock'**. In case there is a tidal range of a few meters or more a harbour basin is provided with a lock system to ensure a constant high water level inside the harbour basin. Ships can only enter or leave this dock at high water.

Famous dock system exist in London, Southampton (U.K.). These docks were constructed in marshy low-lying areas of rivers. The dock walls are quay walls, there is no bottom and the dock system cannot be used as a dry dock.

19. Caissons

Caissons are hollow substructures designed to be constructed on or near the surface and then sunk as a single unit to their required level.

19.1. Caisson foundations

The types of caisson foundation are :

1. A **box caisson**, which is closed at the bottom but open to atmosphere at the top.
2. An **open caisson**, which is open both at the top and bottom.
3. A **compressed air or pneumatic caisson**, which has a working chamber in which air is maintained above atmospheric pressure to prevent the entry of water and soil into the excavation.
4. a **monolith**, which is an open caisson of heavy mass concrete or masonry construction containing one or more wells for excavation.

Caissons are often required to carry horizontal or inclined loads in addition to the vertical loading. As examples, caisson piers to river bridges have to carry lateral loading from wind forces on the superstructure, traction of vehicles on the bridge, river currents, wave forces and sometimes floating ice or debris.
Caissons in berthing structures have to be designed to withstand impact forces from ships, mooring-rope pull, and wave forces.

A caisson will be safe against overturning provided that the bearing pressure beneath its edge does not exceed the safe bearing capacity of the foundation material, but it is also necessary to ensure that tilting due to elastic compression and consolidation of the foundation soil or rock does not exceed tolerable limits.

The walls of caissons are frequently subjected to severe stresses during construction. These stresses may arise from launching operation (when caissons are constructed on a slipway and allowed to slide into the water), from wave forces when floating under tow or during sinking, from racking due to uneven support whilst excavating individual cells, from superimposed kentledge, and from the drag effects of skin friction.

Box caissons

Box caissons are designed to be floated in water and sunk onto a prepared foundation bed. The stages of sinking are shown in fig. 19.1. The foundation bed is prepared under water by divers, and the caisson is lowered by opening flood valves to allow the unit to sink at a controlled rate. Box caissons are suitable for site conditions where the bed can be prepared with little or no excavation below the sea- or river-bed. Thus, they are unsuitable for conditions where scour can undermine a shallow foundation. They are also unsuitable for conditions where scour can occur during the final stages of sinking by the action of eddies and currents in the gap between the base of the caisson and the bed material as the gap diminishes.

Box caissons can be of relatively light reinforced-concrete construction, since they are not subjected to severe stresses during sinking. Light construction is desirable to give the required freeboard whilst floating. After sinking they can be filled with mass concrete or sand if dead weight is required for the purpose of increasing the resistance to overturning or lateral forces.

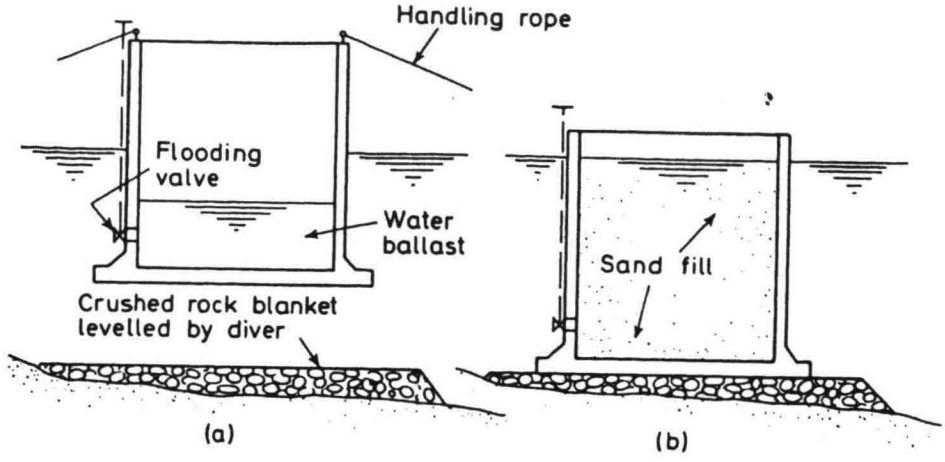


Fig. 19.1.
Stages in sinking a buoyancy raft.
(a) Flooding valve opened to admit water ballast;
(b) caisson sunk in final position.

Open caisson

Open caissons are designed to be sunk by excavation while removing soil beneath them through the open cells. They are designed in such a manner that the dead weight of the caissons together with any kentledge which may be placed upon it exceeds the skin friction of the soil around the walls and the resistance of the soil beneath the bottom (cutting) edges of the walls.

To aid sinking, the soil may be excavated from beneath the cutting edges, or kentledge may be placed on the top of the walls to increase the dead weight. The skin friction around the walls can be reduced considerably by injecting a bentonite slurry above the cutting edge between the walls and the soil.

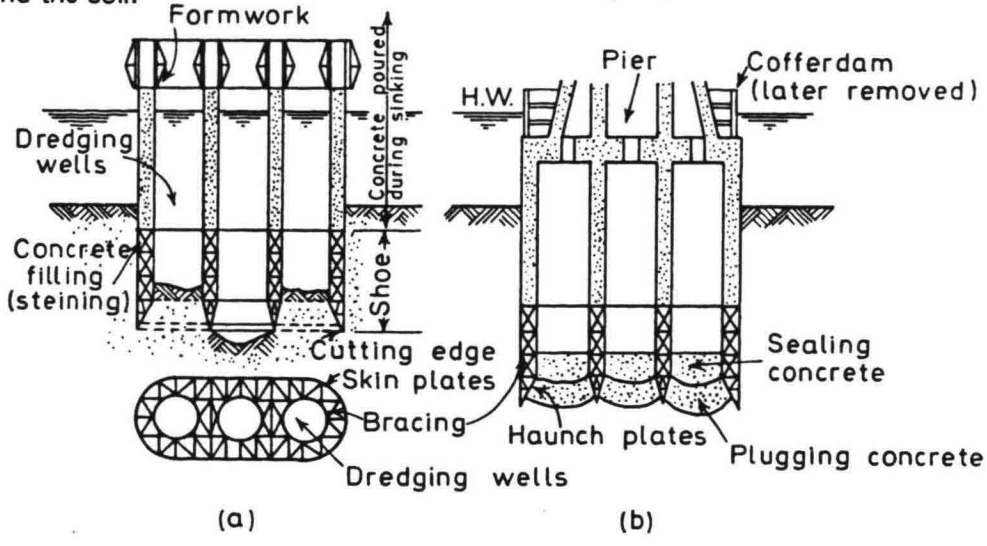


Fig. 19.2.
Stages in sinking an open caisson.
(a) Grabbing from cells and concreting in walls;
(b) plugging and sealing concrete in place with caisson at final level.

On reaching foundation level, mass concrete is placed to plug each cell after which any water in the cells can be pumped out and further concrete placed to form the final seal. The portions of the cells above the sealing plugs can be left empty, or they can be filled with mass concrete, sand, or fresh water depending on the function of the unit and the allowable net bearing pressure.

The lower part of an open caisson is known as the **shoe**. This is usually of thin mild steel plating stiffened at the edges. Concrete is placed in the space between the skin plates of the shoe to provide ballast for sinking through water and thereafter more concrete and further strakes of skin plating are added to obtain the required downward forces. While the top of the shoe is still above water level, formwork is assembled and the walls extended above the shoe in reinforced concrete. Thick walls are needed for rigidity and to provide dead weight.

The introduction of bentonite injection techniques to aid sinking has made it possible to design caissons entirely in reinforced concrete and to sunk them to great depth. Circular caissons were sunk to depths as much as 105 m below the bed of the Jamuna River (Bangladesh).

The soil is excavated from within the cells and, where necessary, from below cutting edge by mechanical grab. On reaching foundation level any kentledge placed on the walls is removed to arrest sinking and mass concrete is quickly placed at and below cutting edge level in the corner cells to provide a bearing on which the caisson comes to rest. The remaining outer cells are then plugged with concrete followed by the completion of excavation and plugging of the inner cells.

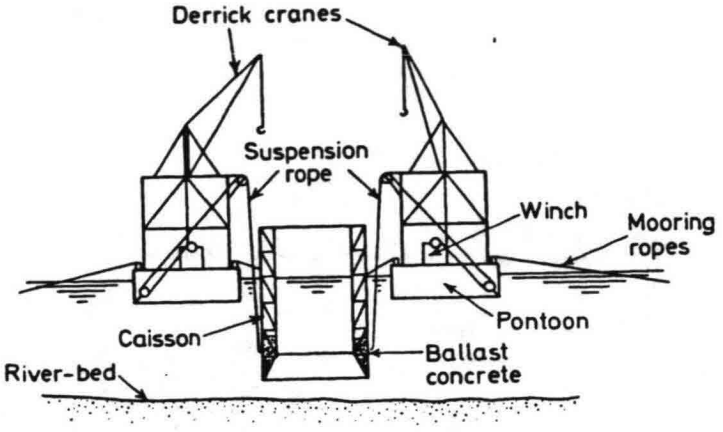


Fig. 19.3.
Lowering caisson from moored pontoons.

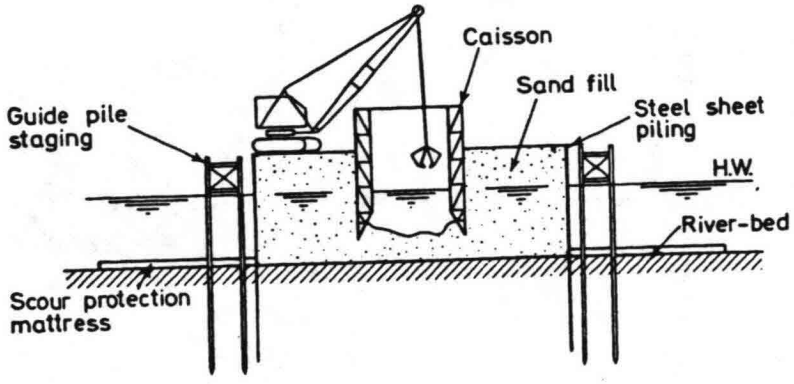


Fig. 19.4.
Sinking caisson through a sand island.

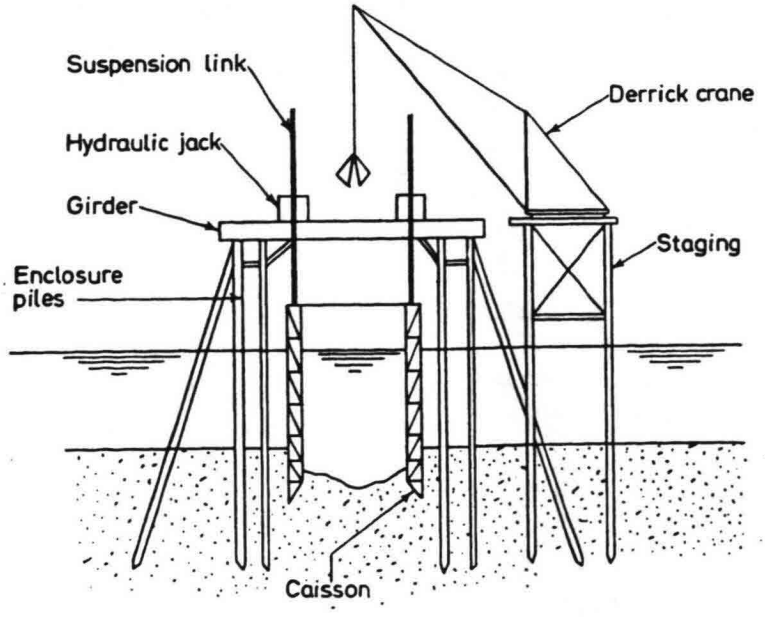


Fig. 19.5.
Lowering caisson within a piled enclosure.

Open-well caissons are best suited to sinking in soft or loose soils to reach a foundation level on stiff or compacted material. They are unsuitable for ground containing obstructions which cannot be broken out from beneath the cutting edge.

Open caissons can be provided with air domes. These are provided by airlocks and are designed to be placed over individual cells as required.. Then compressed air is introduced to expel water, after which workmen can enter through an airlock to remove obstructions or to prepare the bottom to receive the sealing concrete. Air domes provided on all cells can be used as the means of floating an open caisson to the sinking site and for controlling the sinking operation (**flotation caissons**). The domes of flotation caissons are not normally provided with an airlock.

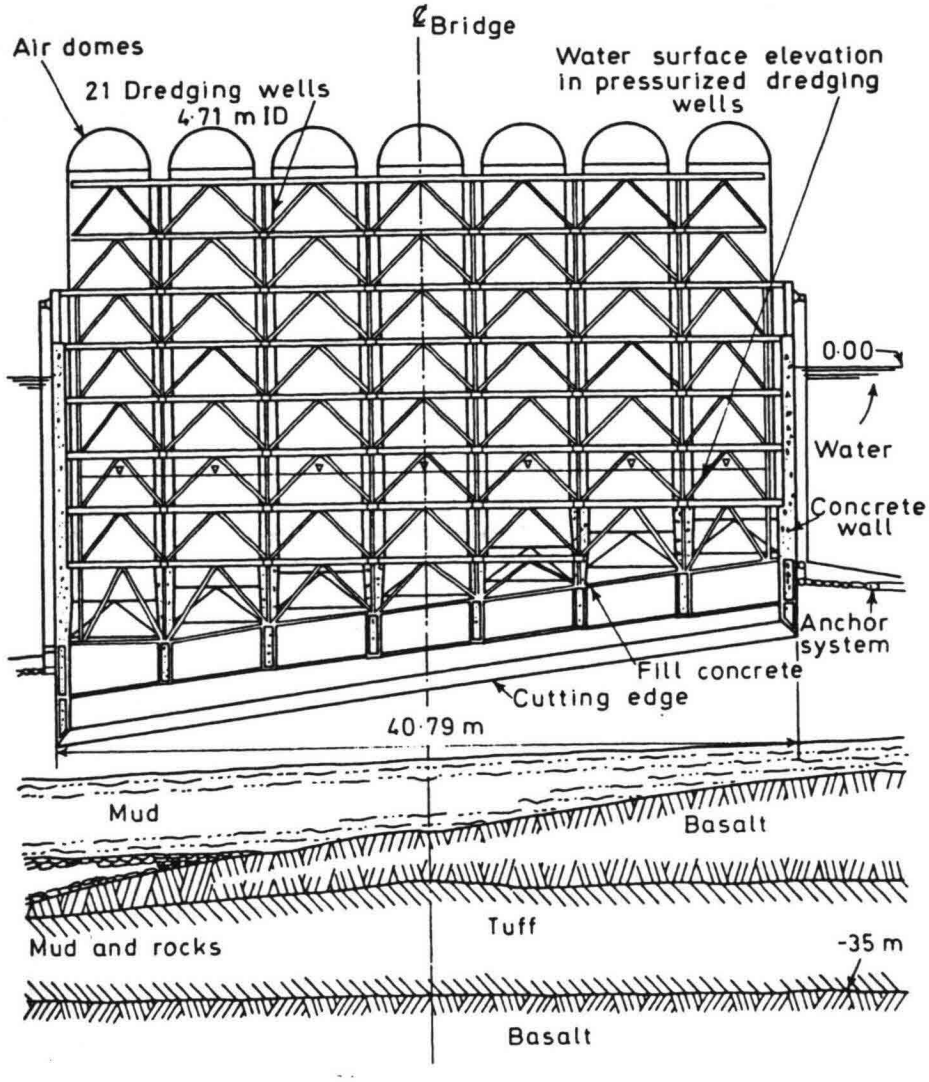


Fig. 19.6. Flotation caisson for the Tagus River bridge.
The cutting edge of this caisson was 'tailored' to suit the profile of the rock surface on which the caisson was landed (1965).

Pneumatic caissons

Pneumatic caissons are designed to be sunk with the assistance of compressed air to obtain a 'dry' working chamber.

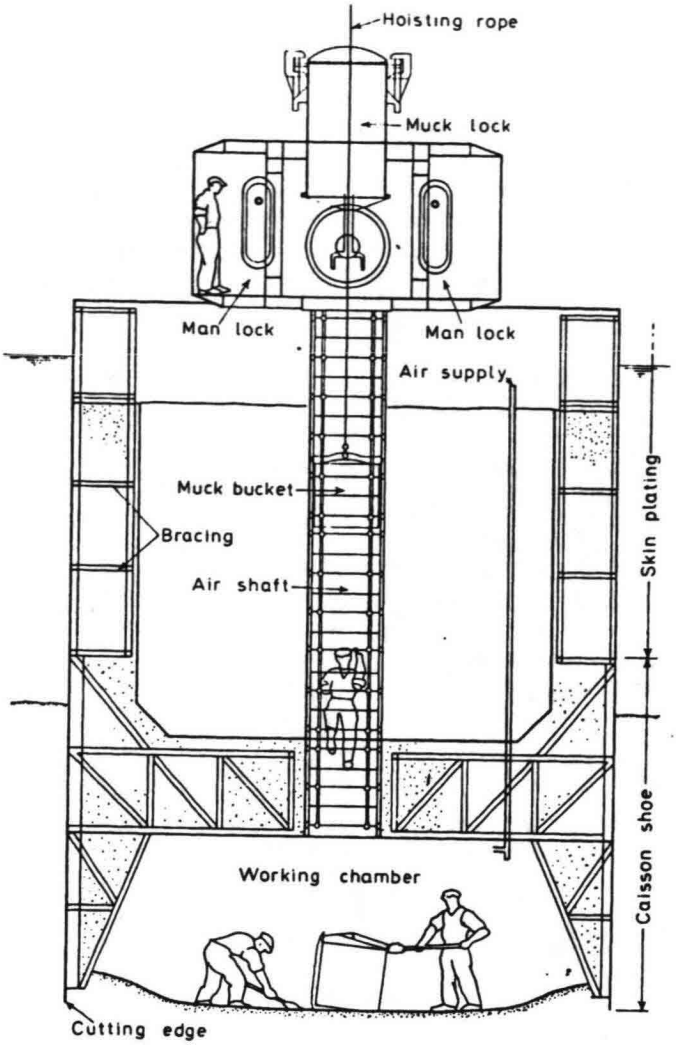


Fig. 19.7.
Compressed air caisson.
The caisson consist of a single working chamber surrounded by the shoe with its cutting edge, and a heavy roof. The high cost of compressed-air sinking generally precludes pneumatic caissons for all but special foundations where no alternatives are feasible or economically possible.

Monoliths and cylinders

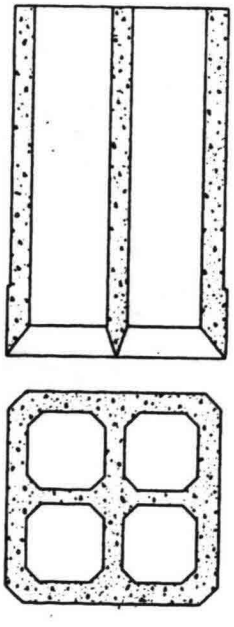


Fig. 19.8.
Concrete monolith.
Monoliths are open caissons of reinforced concrete or mass concrete construction and are mainly used for quay walls where their heavy weight and massive construction are favourable for resisting the thrust of the filling behind the wall.

Open caissons of cylindrical form and having a single cell are sometimes referred to as cylinder foundation.

Caisson breakwater

An alternative method form of construction of a vertical-faced breakwater is the caisson, constructed partly on shore and partly afloat in a building dock, floated into position and sunk on to a prepared foundation. The main advantage of caissons is that very large units can be handled and with adequate space for building them progress can be rapid. However, their use is limited by the depth in which they can be floated (see chapter 22).

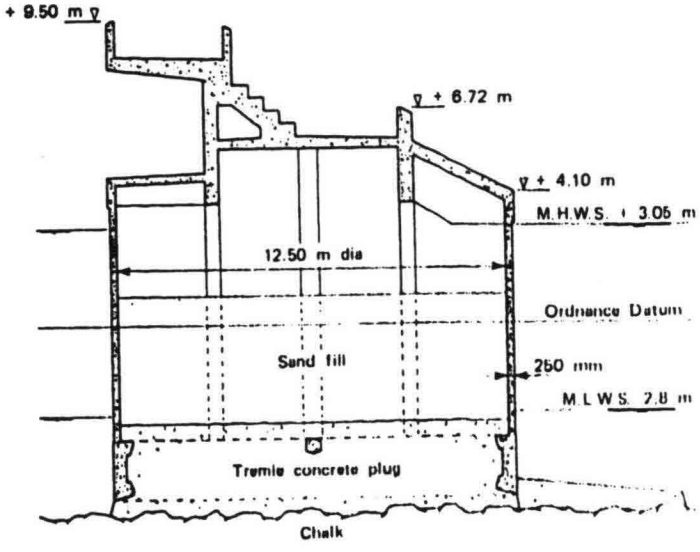


Fig. 19.10
Cellular breakwater at Brighton.
A caisson system which does not have the floating depth limitation. The caissons are 12-m diam. concrete cylindrical units weighing up to 600 t each, and open at their lower ends. They are constructed on shore and transported to the end of the completed part of the breakwater on a self propelled trolley where they were suspended in position clear of the seabed by a special crane while tremied concrete was placed in the bottom to form a massive foundation plug.

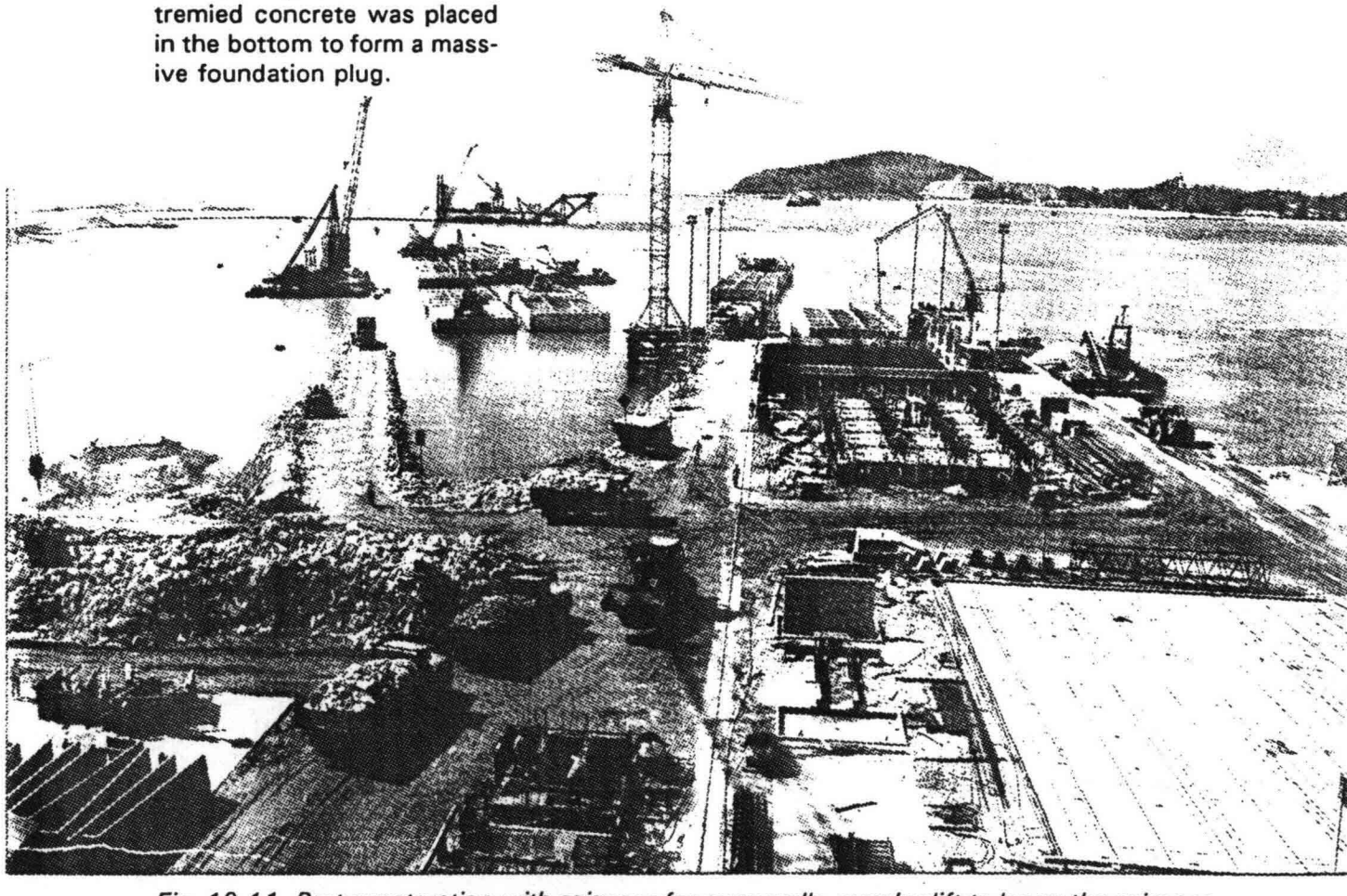


Fig. 19.11. Port construction with caissons for quay walls, synchrolift to lower the caissons (when half finished) in the water and cutter suction dredger in the background.

Schelphoek dike-breach (The Netherlands)

The largest dike-breach caused by the storm flood of the 1st of February 1953 was that near the Schelphoek. Due to the strong tidal currents running into and out of the polder, the width of the gap grew to 500 m, the depth being more than 30 m. The tidal range at the seaside of the gap was 2.8 m, while the tidal range in the polder was considerably smaller. Narrowing the gap, the current velocity would increase to about 5 m/s.

Therefore the closure was planned with a dike of which the centre-line followed a wide curve far inland from the breach. The total length of this dike was more than 4 km. after making some parts (1600 m) of this dike in a normal way with clay and sand a closing gap was left with a length of 2400 m. The closing gap consisted of two shallow parts and two deep gullies.

It was of great importance to restrict the current velocities in these gullies as much as possible, ir order to prevent an extraordinary deepening, which might cause dangerous slides in the loose sand bottom. In the north-west gulley a Phoenix-caisson was sunk with a length of 62 m, a width of 19 m and a height of 18 m. The north-east gulley was closed three days later (August 21) with a smaller caisson. After that the extended shallow parts of the closing gap had than to be closed. Here caissons were used with a length of 55 m and 66 m and with a height of 2 and 3 m.

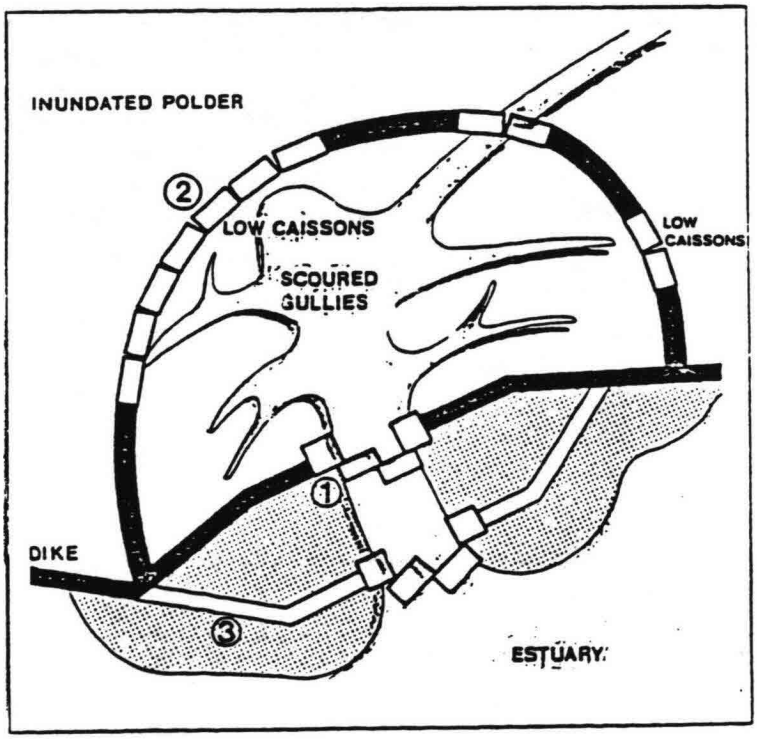


Fig. 19.12.

1. By reconstructing the damaged dike along its original course, which involves the closing of gullies.
2. By constructing a 'horse-shoe' dike round the gap on the land side, which involves a wide gap on ground level.
3. By constructing a 'horse-shoe' dike along the shallows outside the original dike, which involves the closing of gullies.

19.2. Sluice caissons

When building a dam to close off a tidal basin, the inflow and outflow through the narrowed gap will be reduced, causing a decrease in the tidal range in the basin, but higher current velocities through the remaining opening. As a consequence, the scouring effect on the bottom near the dam will be increased, which endangers the stability of the bottom. This implies that the bottom, when consisting of easily erodable sand, must be protected. A special problem arises from the method of closure. If the velocities become higher than 2 to 3 m/s heavier materials than sand or gravel should be used. One of the methods to close the final gap is the "sudden" closure by means of caissons. These structures allow the whole gap to be closed rapidly. Normally closed caissons are placed in the gap during a slack-water period and thus closing the whole gap at once. Closed caissons are useful in small gaps, say no larger than one to three caissons, or in areas where there is only a very small tidal motion.

Another solution is the use of sluice caissons which are positioned during several successive slack-water periods and kept open during the period in which all the sluice caissons are positioned. After the placement of the final caisson, all the caissons are closed at the same slack-water period by means of gates. Sluice caissons should be used when the gap to be closed is a large one and the tidal motion is considerable.

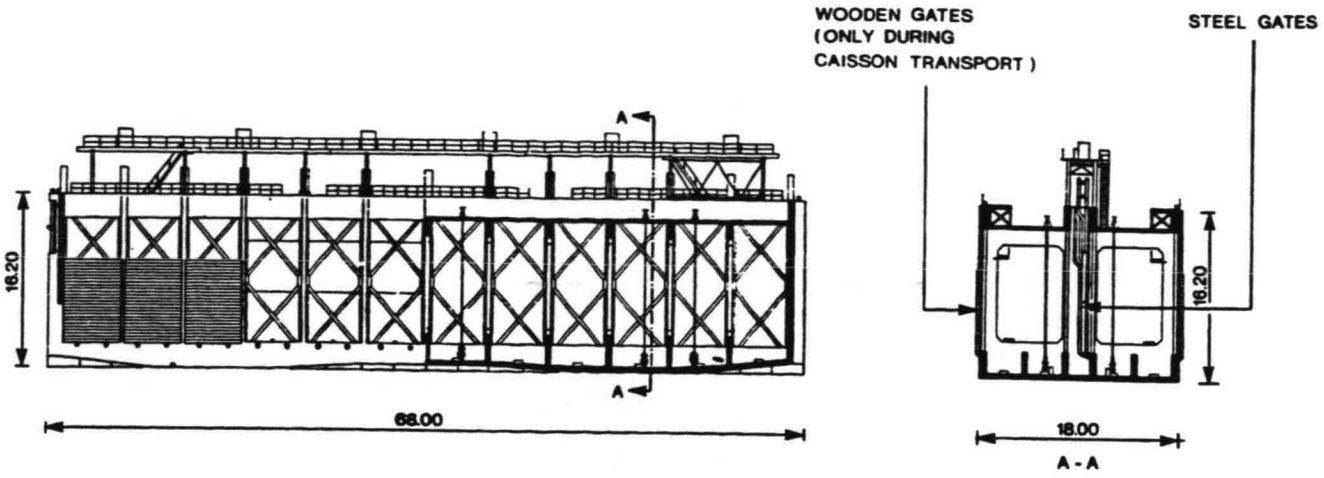
In 1953 caisson units were built for the first time as a means of closing dike breaches in the Netherlands caused by the flood disaster of that year. A striking example of the application of "standard" or "unit" caissons on a high "sill" was the closure at ground level at Schelphoek (island of Schouwen Duiveland) in August 1953. The 525 m. breach in the coastal dike with a maximum depth of 37 m. was closed by a "horseshoe" shaped dike of 4 km. long round the gap on the land side for which, in total, 235 standard caissons were used. When closing the gap at Schelphoek, 462 metres of caissons (42 units, 11 m. long) were placed within 24 hours.

Sluice caissons have been used:

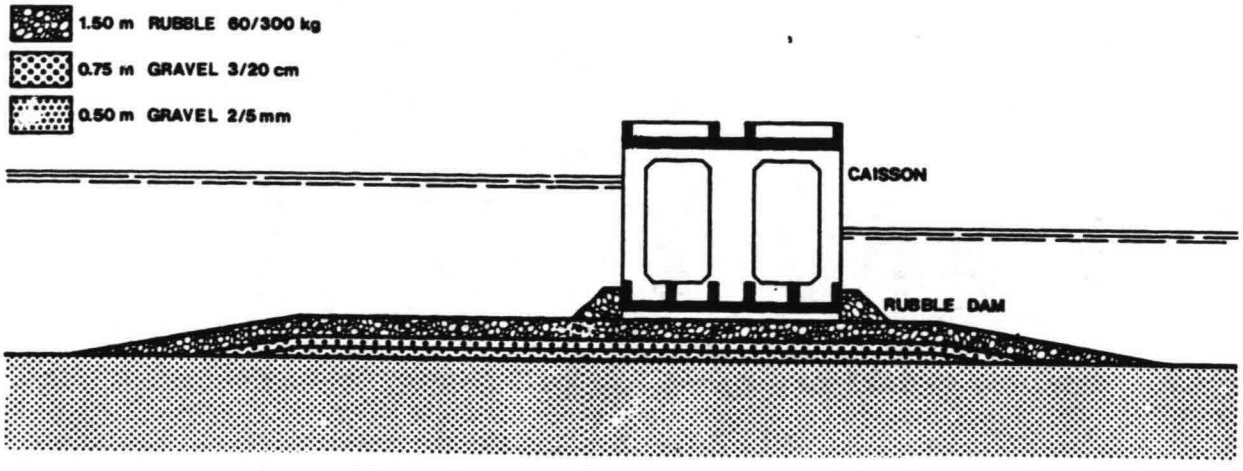
Veerse Gat	1961	depth of sill: - 8 m
Lauwerszee	1969	depth of sill: - 6 m to - 6.5 m.
Volkerak	1969	depth of sill: - 7 m
Brouwersdam	1971	depth of sill: -10 m

For the closing of the northern channel of the Brouwershavensche Gat twelve "sluice" or "culvert" caissons were used, each 68 m. long, 18 m. wide and 16.2 m. high. Their draught was 6.10 m. and their displacement some 7,000 m^3. Each caisson had twelve sluices or culverts 5 m. in width. Each of these openings could be closed off by two gates placed along the length of the caisson. During towing and sinking the openings were closed by wooden gates. Each caisson was fitted with sixteen 500 mm. diameter values for filling the caissons during sinking. The closure gap was bordered on both sides by slopes with a gradient of 1:5. On these slopes two closed abutment caissons were placed. The caissons were built in a construction dock some 12 km. away.

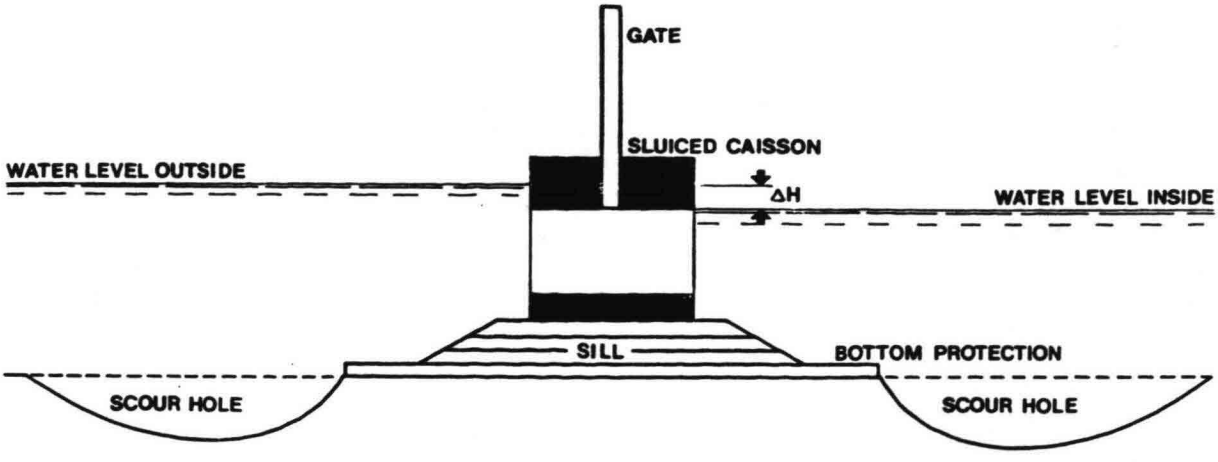
Six tugboats with a combined power of 8,600 hp were required for the positioning of a sluice caisson. All sluice caisson were placed in April 1971, one per day at low water slack.



Sluice caisson



Caisson with rubble dam on a sill



Construction stages of caisson closure

Fig. 19.13. Sudden closure by means of caissons.

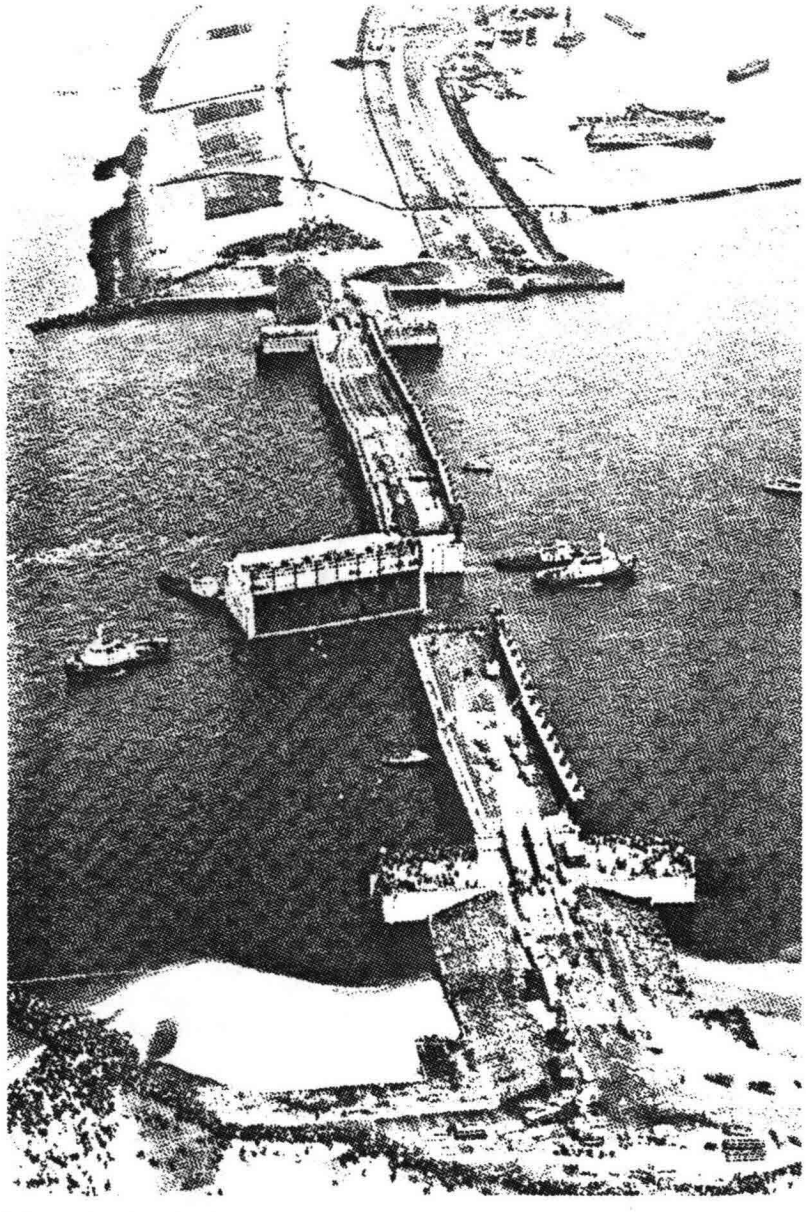


Fig. 19.14. Closing the Veerse Gat in 1961.

After the storm surge barrier in the Hollandse IJssel was completed, a start was made on the first major dams in the turbulent tidal inlets. In line with the principle of working from small to large, the first project was the Three Islands Project, which entailed the construction of dams in the Zandkreek between North and South Beveland and in the Veerse Gat between North Beveland and Walcheren. During the construction of the Zandkreek dam asphalt was used for the first time to cover the sill which would support the caissons, huge concrete structures which can be positioned quickly to dam the tidal inlet. The Zandkreek dam was completed in 1960.

The seven caissons used a year later to dam the Veerse Gat were even larger. Sluices in the caissons allowed the tide to ebb and flow without obstruction. This prevented the current from becoming too strong while the mouth of the channel was gradually being dammed. When all seven were in position the steel sluice gates were closed simultaneously and the caissons filled with sand. A substantial part of the dam had already been built on a sandbar in the Veerse Gat by pumping sand onto it. The Veerse Meer, the lake formed after the dam was completed, is partly brackish due to dilution by rainwater. Land that has been reclaimed is now used for farming or recreational purposes and a few sandbars have been left entirely to nature.

20. Fendering, mooring and berthing structures

Fendering or guiding structures serve to guide navigation through bridge-openings and locks and for mooring ships temporarily during waiting time. The fendering must protect the ship as well as the (hydraulic) structure.

Dolphins have a protective function by mooring-operation. Usually dolphins are constructed with driven piles (steel or hard wood) which are connected to each other. Driven piles are rather elastic, so therefore capable to absorb kinetic energy in case of ship collision. Reinforced retaining walls are rigid and not suitable to absorb the kinetic energy of the moving vessel; in that case the wall may be protected by rubber fenders, which can be replaced easily when damaged. A **fender** protects the ship and the quay wall, jetty, pier etc. There are many types of fenders.

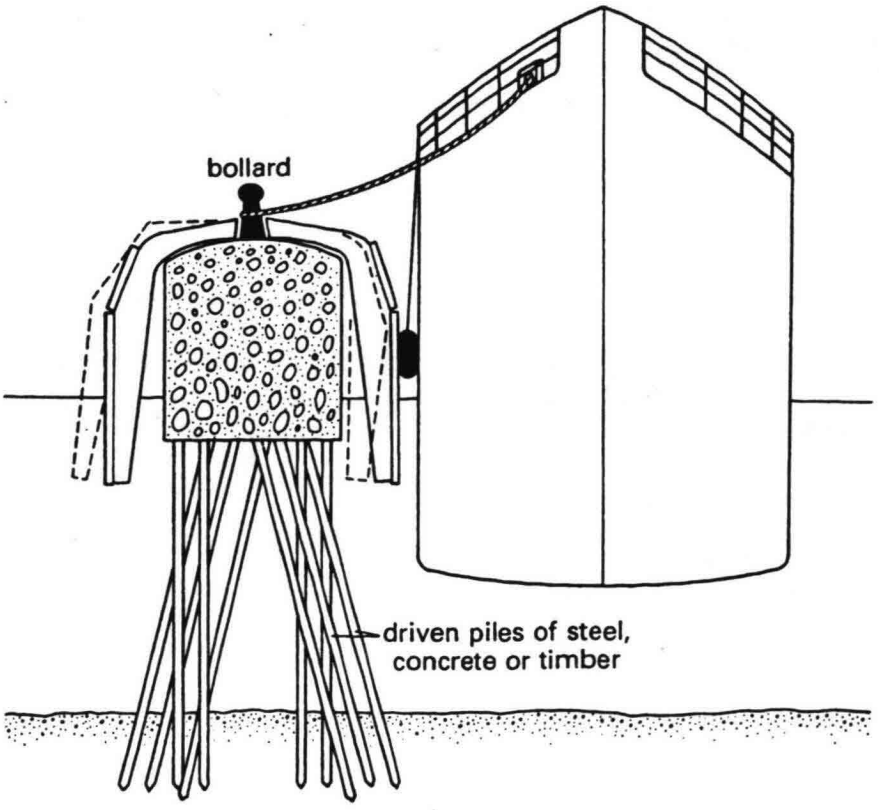
The last decades the size of ships has increased considerably. The developments in navigation has resulted in larger navigation locks, wider and deeper shipping canals, higher ship speeds (larger waves), heavier quay walls and jetties and longer breakwaters with larger cross-section as they were built in deeper water, Also **berthing structures** has increased in size.

Berthing dolphins are usually flexible constructions that are able to withstand the impact forces during the berthing of the ship and the loading by the ship due to wind, wave and current forces.

Mooring dolphins are rigid constructions that should be able to take the loading by the mooring cables.
A **berthing** is a horizontal beam on piles for berthing a ship instead of berthing dolphins.

Fig. 20.1.
Baker bell dolphin for mooring at sea. The dotted line shows allowable movement of bell.

A large bell-shaped steel or concrete fender, suspended on a cluster of piles in open sea for the mooring of vessels, first used at Heysham jetty.



Fendering

Fender systems are designed to protect both the vessel and the breasting structure from damage caused by berthing impacts. They range from timber rubber-strips fixed to a quay face, to purpose-built, free-standing energy-absorbing structures. The factors determining the type and capacity of a suitable fender system include the nature and the size of the berthing vessel, the form of the structure to be protected, the environmental conditions (i.e. wind, waves, currents etc.) the operational requirements and the consequences of damage to the vessel or structure.

The berthing force is often the predominant lateral load imposed on a quay or jetty structure and its effect is largely controlled by the fender system adopted. The design of the fendering system must therefore form an integral part of the structural design. A fendering system may be defined as a structural element or a combination of elements which ensures the safe disposition of a vessel's kinetic energy whilst it berths in a controlled manner. Most systems incorporate an elastic energy-absorbing unit but occasionally a plastic or friable unit is included, the deformation of which provides additional protection against marginal overloading.

Fendering systems may be divided into two main categories:

1. **detached systems**, in which the berthing forces do not act on the main quay or jetty structure.
2. **attached systems**, in which the fender elements are attached to the main quay or jetty structure. The structure then provides the reactive force.

The category of **detached fender systems** may be subdivided into :

1. detached quay fender systems, and
2. breasting dolphins.

A common example of a detached fender system is a row of free-standing piles (usually steel or timber) driven into the sea- or river-bed in front of the face of the main structure. Berthing energy is absorbed mainly by deflection, the capacity for energy absorption being determined by the size, length, penetration and material properties.

Breasting dolphins are berthing structures independent of the service structure provided for the vessel. Their disposition about the service quay or jetty is such as to effect the most suitable compromise for the range of vessels envisaged. With **flexible** dolphins the energy absorption is provided in part by deflection of the entire structure and in part by energy-absorbing units attached to the dolphin face. With a **rigid** dolphin, all the energy dissipation is achieved by units similar to those used in attached fender systems.

An **attached fender system normally** consists of energy-absorbing units bolted to, or suspended from,a quay face or strong points on a jetty (see figure). Most of the units are made from synthetic rubber and the required energy absorption capacity is achieved by deformation in compression or shear. Some types, such as the hollow cylindrical rubber fenders, the arch type, and the pneumatic fenders, can be allowed to make direct contact with the vessel's hull whereas others require face panels to reduce contact pressure.

Gravity fenders are those in which kinetic energy is converted in potential energy by means of raising a large mass. They relieve the main structure of the berthing load but impose considerable dead load and a horizontal force depending on the movement of the fender block. The **berthing beam** principle combines the absorption capacities of cantilevered piles and gravity systems whilst avoiding the dead-load penalty of the latter.

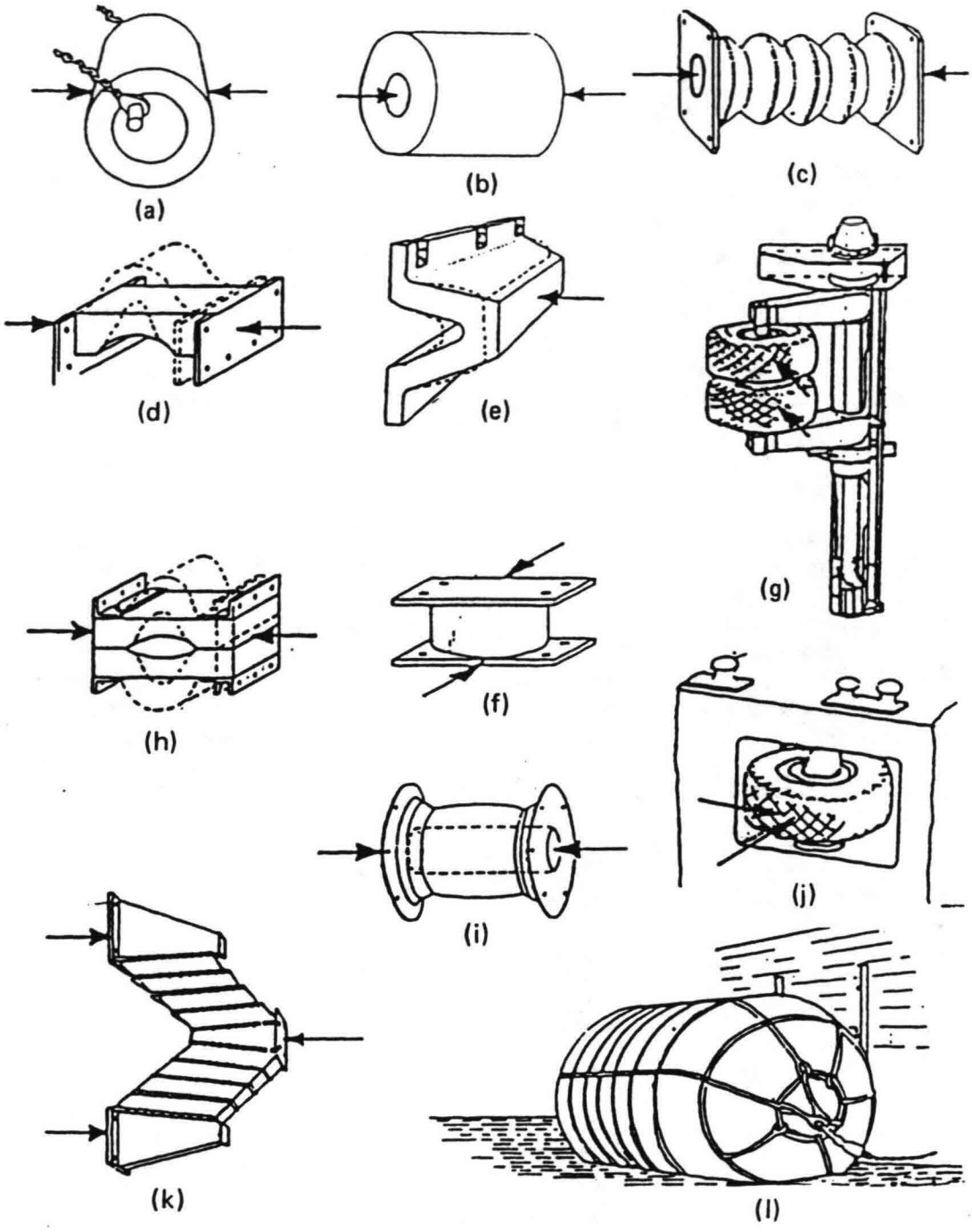


Fig.20.2. Typical replaceable energy-absorbing units.

Dolphins

Dolphins are of two kinds:
1. **breasting dolphins**; and
2. **mooring dolphins.**

The use of both types is shown in the figure.

A **breasting** dolphin is an isolated structure designed to fulfil two distinct functions:
1. it must absorb the kinetic energy of the berthing vessel; and
2. it must assist in restraining the vessel at the berth.

Two berthing structures, one each side of the service platform, are generally sufficient, but if the berth is to be used by ships of widely varying size two additional dolphins closer to the platform may be required.

There are two basic types of breasting dolphin: (1) **rigid**, and (2) **flexible**. The former will be either a massive structure (such as a blockwork or caisson construction) or an open multiple-pile structure rigidly held together at the top by a massive deck or a steel jacket. The flexible dolphins usually takes the form of parallel flexible steel tubes (or a single tube), with a high elastic limit, which absorb most of the energy by deflection up to a maximum of 1.5 to 2 m.

Mooring dolphins are isolated structures to which mooring lines are attached to restrain the ship at the berth. They are not normally subject to impact of a berthing vessel and do not therefore need fendering or to be flexible to absorb energy. They must, however, be designed to resist the horizontal load from the mooring lines over a wide angular range, which arises from both wind and current load on the moored vessel and the ranging of the vessel from wave action. They must also be designed for uplift, to resist the vertical component of the force in the mooring line. They are normally rigid piled structures.

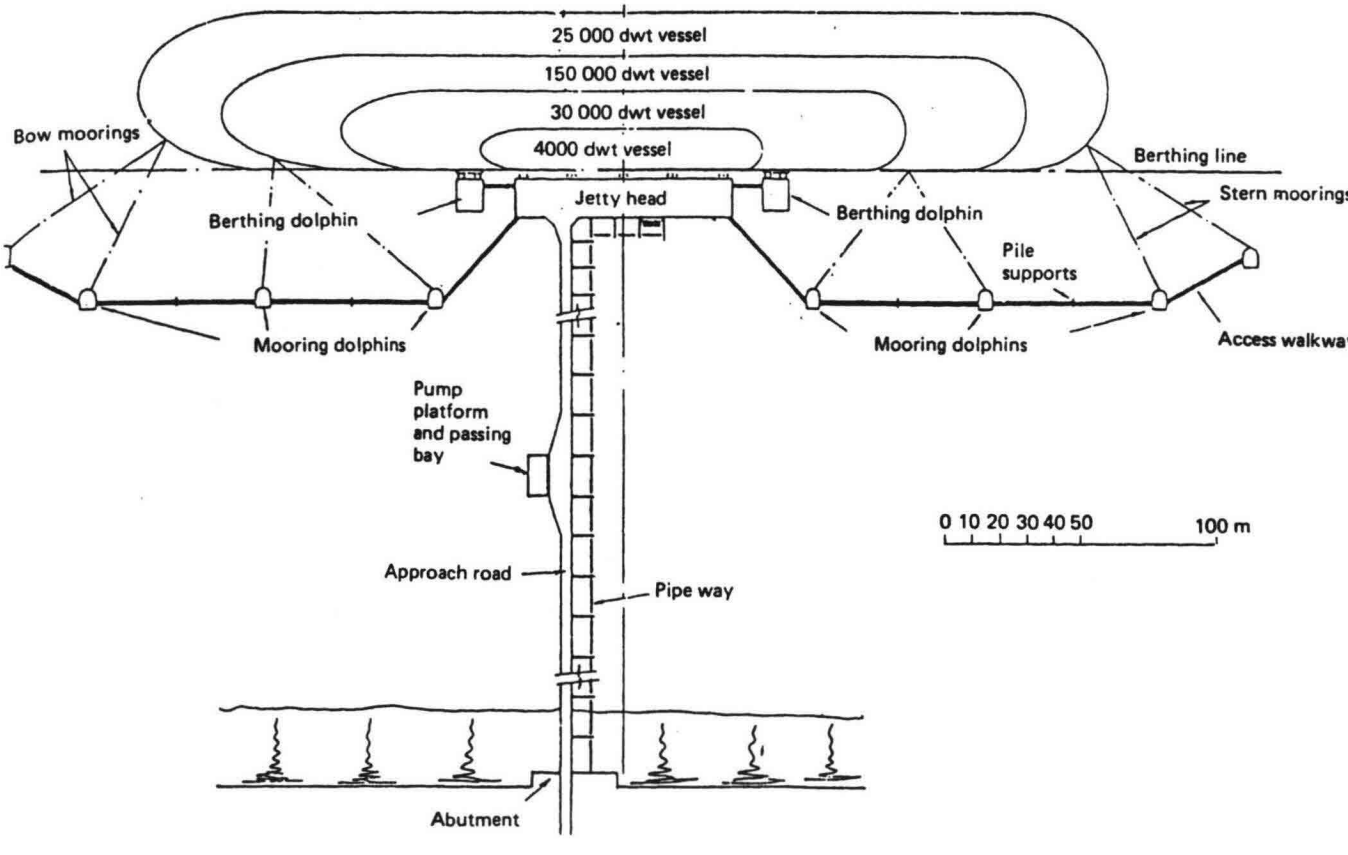


Fig. 20.3. Typical plan of oil-jetty berth.

Single buoy mooring

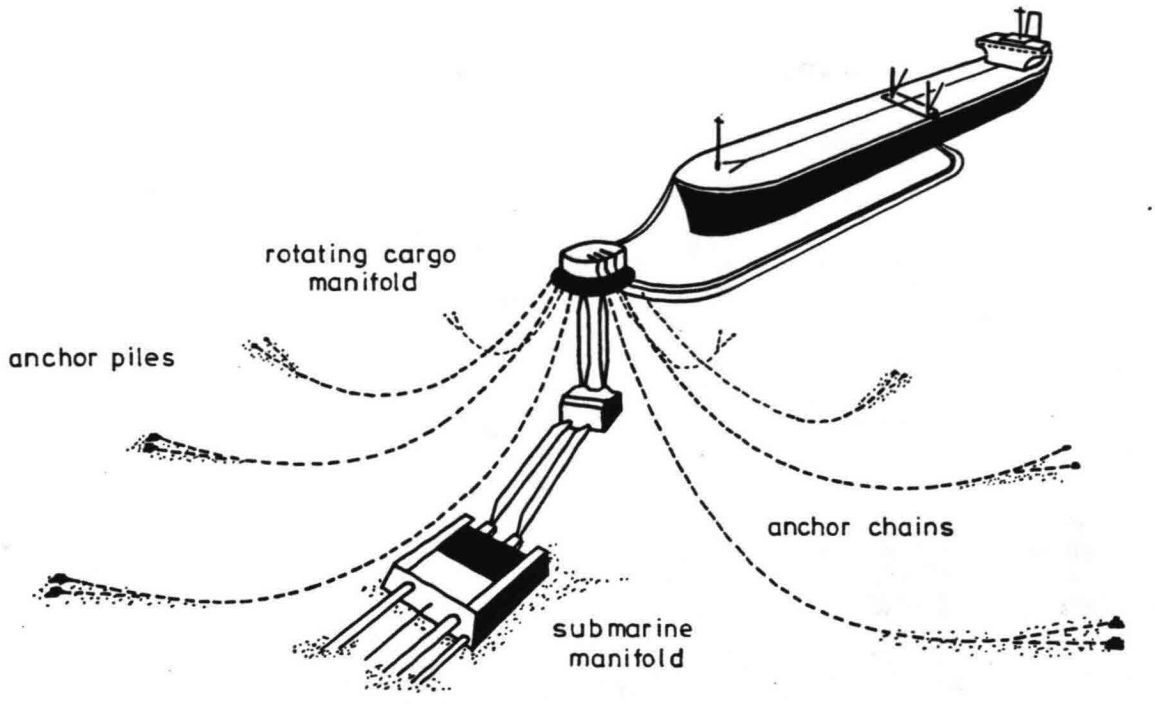


Fig. 20.4. A Single Point Mooring (SPM) is an offshore berth which provides a link between an undersea pipeline and a moored vessel for the transfer of fleet cargoes and to which the vessel can be secured and can weather vane during the cargo transfer as dictated by the environment (wind, current, tides, etc).

The single point mooring structure generally consists of two types: floating and fixed. The **floating structure** generally consists of a buoyant hull which provides a platform for mooring attachment points which supports the anchor leg(s) that transmit mooring forces to the seabed and which many carry cargo piping.

The **fixed structure** generally consists of a truss-like tower, a single pile or a group of piles fixed to the seabed which supports the mooring system and transmit the mooring forces to the seabed.

A **manned SPM** refers to a SPM that is fitted with living accommodation and is intended to have personnel on board; all other SPM are considered unmanned.

The **swing circle** is the area swept by the vessel as it revolved about the mooring point. The radius of the swing circle is defined as the sum of the horizontal displacement of the SPM from its centre position under operating hawser load and minimum tide, the horizontal projection of the length of the mooring line under operating hawser load, the length overall of the largest vessel for which the SPM is designed and a safety allowance of 30 m.

21. Seadikes and dams

Dikes have been built for centuries to protect low-lying property against flood - to keep water outside an area or to control water quantity - keep water inside an area.

Levees (river dikes) have protected land since primitive times. In China of dikes started many centuries before Christ. In the first centuries of the Christian era the people, living in the flood areas of **The Netherlands** learned to protect themselves and their stock against storm tides by building their homes on artificial mounds of clay (socalled **"terps"**). Many of such early settlements became a nucleus of existing villages and towns. In the ninth century the first dikes were built. At first the aim of dike building was defensive, to protect the land where people lived. In a later stage the construction of dikes was used in an offensive way viz. reclaiming land from the sea. **Polders** are reclaimed low lands protected by dikes against high water levels and waves.

A **sea dike** has to withstand intense actions for a short time (storms). Horizontal and vertical tidal movements are important phenomena. Now and then a storm flood occurs with a very high water level and with a strong wave attack. The different circumstances result in a strong defended type of dike. The height of the dike is determined by the maximum expected storm surge level (1 per 10 years, 1 per 100 years, 1 per 1000 years).

In general we can define a dike as a long-shaped two-dimensional body consisting of clay or sand or a mixture of both. Sometimes the construction is protected by a revetment construction. The subsoil and the banks below the water are part of the total construction. A **classification** can be made by:

a. function
b. construction material.

The design of the sea-defence in The Netherlands was based on the highest recorded storm surge water level and observed wave run-up and attack.

21.1. Dike construction

Clay was used as construction material for dikes in The Netherlands. This material is impermeable and locally available. As the need for better protection of the land required heightening of existing dikes and construction of new dikes, the quantities of clay available became inadequate.

Sand, which was abundantly available, was used as an alternative. Originally the sand was used in combination with clay to provide the impermeability of the dikes. Gradually when experience with sand dikes increased the clay was replaced by membranes e.g. asphaltic concrete.

With the use of sand as construction material, the **hydraulic fill construction method** was introduced. Small bunds are made along the base of the dam to be constructed and a first layer of sand is pumped in this area. The sand thus placed is used for the construction of new bunds and a new layer is placed. Depending on sand characteristics and the stability characteristics of the subsoil layers of 0.75 - 3 m are placed in this manner. So far sand fill dams have been built to a height of about 25 m.

If sand deposits are available nearby and a modern dredger is used production rates of 2000 - 3000 m^3/hour are possible. The typical concept of the **Dutch modern dike** is a huge body of sand with gentle slopes to avoid stability problems even in very poor soil conditions. This sand body, constructed as hydraulic fill by high capacity dredgers, is protected by relatively thin protection systems, decreasing the need for quarry products.

Dike construction often requires the **removal of soft underlayers** to improve the subsoil. These soft (clay) layers may be excavated down to the firm subsoil by a cutter suction dredger or a bucket dredger. The supplement by sand might be done with dump barges or split barges as long as water depth allows (draught requirement of the barges). After that sand will be placed hydraulically through a discharge pipeline.

These **protection systems** were also developed because of lack of good quality clay and brushwood (willow-fascines). Extensive use of geotextiles have been made to solve the filter problem. **Asphalt concrete** has been widely used as well as **asphalt mastic** laid insitu or used like **sand asphalt** and **fixtone** were developed. **Cement concrete blocks** of different shapes were prefabricated, either for use as a pitching construction or tied to a geotextile as in the **block mattress**. Gravel - or sand filled bags or tubes of geotextiles have also been used.

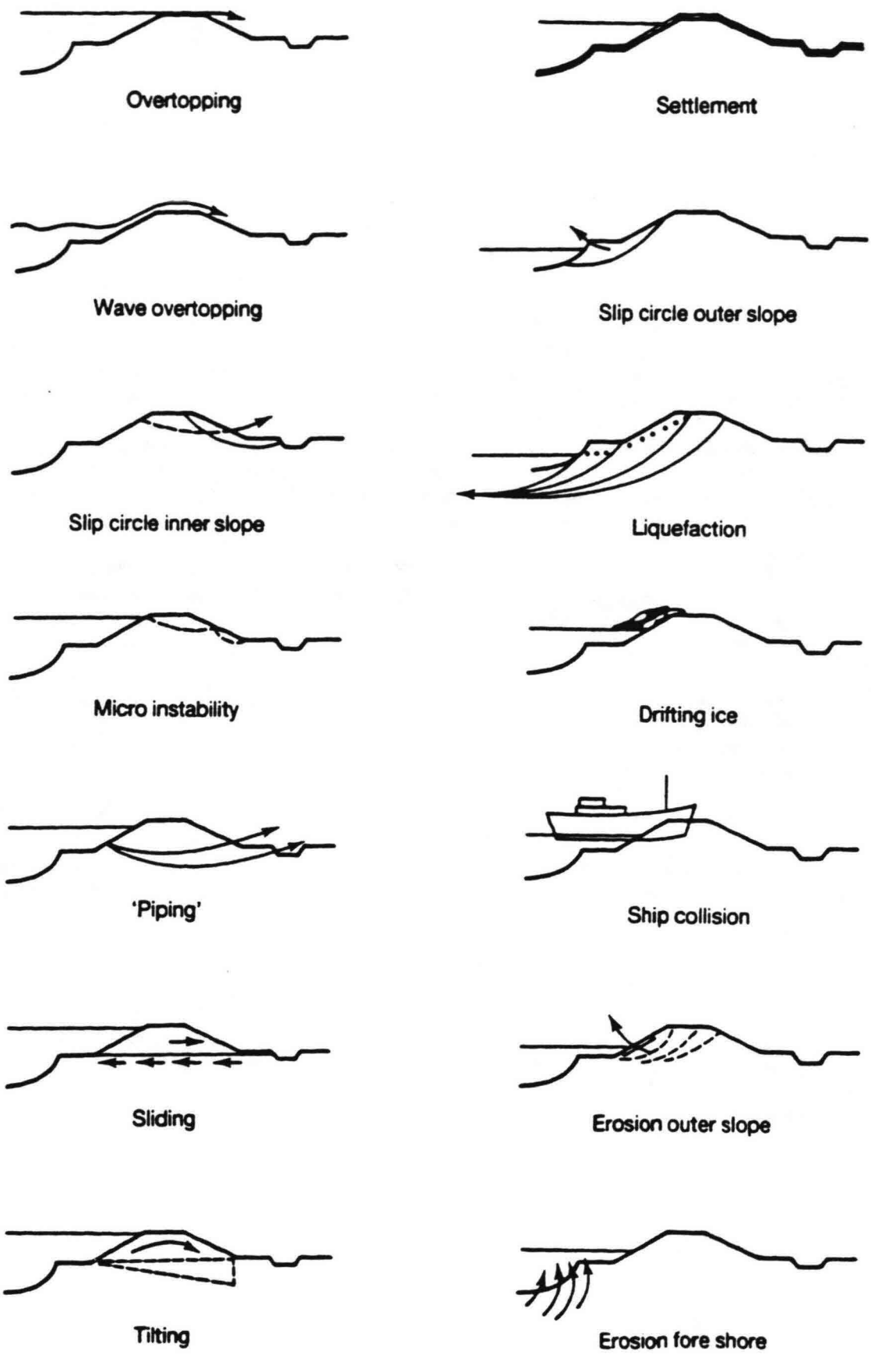


Fig. 21.1. Failure modes of dikes.

Saudi Arabia - Bahrain Causeway

In 1982 the construction of the Saudi Arabia - Bahrain Causeway commenced. The- Bahrain causeway consists of a number of bridges and embankments (seven sections). The seven embankments all have a similar basic design (see Fig. 1.).Two parallel bundwalls constructed by quarry run enclose a central core of hydraulically placed sandfill. In order to prevent loss of sand, the inner slopes of the bundwalls have a filter layer of fine aggregate which, in turn, is covered by a woven filter fabric.
Several graded layers of armour stone are placed on the outer face of the bundwalls and continue beyond the toe on the seabed up to a distance of 10 m. to provide protection against tidal scour.

Work on the Embankment-sections 1 and 2 at the western end (Saudi Arabia side) of the causeway started at the end of January 1984. Embankment 1 is, in fact, entirely onshore serving as an approach ramp to bridge 1 and was constructed by landbased equipment. Embankment 2 and 3 are located on shallow tidal bars. The bundwalls were formed by dumping stones from barges. The barges were loaded at the temporary work harbour established at Al Aziziyah in Saudi Arabia. Embankment 4, the middle of the causeway, is, in fact, an artificial island with an area of 66 ha. This island accommodate customs and immigration facilities together with coast guard bases. The bundwalls were also formed by dumping stones from barges.

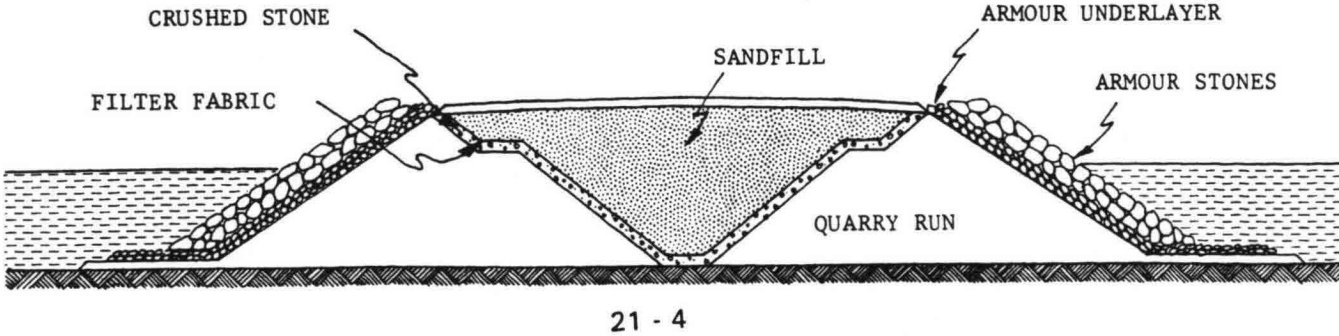
The bundwalls of Embankments 5, 6 and 7 (Bahrain side) were constructed from the shore by a fleet of trucks bringing the quarry run and the armour stone from a quarry located in Bahrain. The bundwalls have been constructed by end-tipping and were shaped and graded by excavations and draglines.

Hydraulically placed sandfill, forming the core of the embankments was dredged from adjacent borrow areas by the cutter suction dredger "Queen of Holland". In total over 8 million m^3 of sandfill was placed in the embankments as well as 1.4 million m^3 of stone and almost 2 million m^3 of quarry run.

Very extensive ground investigations were carried out to ensure stability of the bridge piers (455 boreholes, 6 km of core, 9,000 samples, 16,000 laboratory tests). Additional investigations were conducted including boring and the use of shallow seismic techniques in the vicinity of the causeway in order to locate adequate quantities of suitable sandfill for the embankment sections.

In early 1983 a 1 km trial length of the embankment was completely filled to enable dynamic compaction trials to be conducted. Dynamic compaction was applied to all embankment sections to ensure that the sandfill has a sufficient relative density. Compaction was achieved by using a 15 t weight dropped from a height of 20 m at carefully calculated intervals on a grid system. An extensive programme of quality control was undertaken to monitor the density improvement of the sandfill, and included the use of the electric and nuclear density probes and continuous penetration testing.

Fig. 21.2. Cross section of embankment.



Feni dam closure - Bangladesh

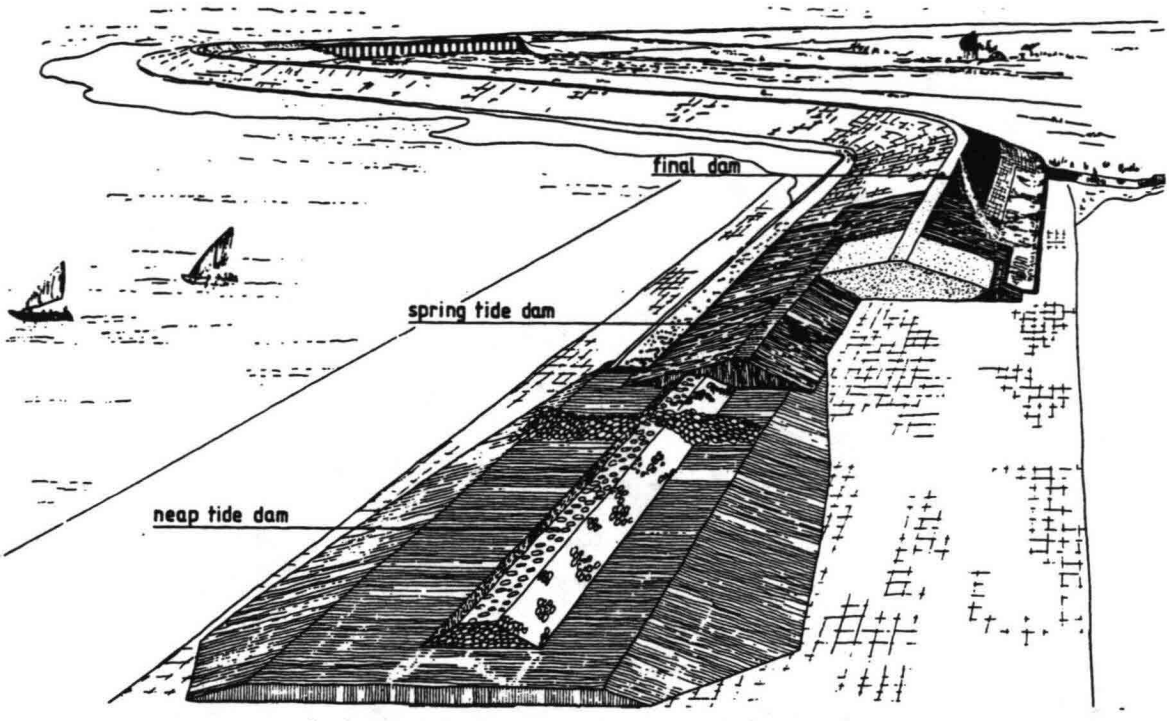


Fig. 21.3. Feni Dam, diagram showing build-up and dam after completion. Lacking money and machinery but rich in manpower, Bangladesh used the brawn of 15,000 men to close the mouth of the Feni River to control flooding and create a freshwater reservoir for irrigating rice. Crossing the muddy river bottom at low tide, workers carried 45-kilogram bags to 11 stockpiles. During a frenetic seven-hour intertidal marathon, they blocked on February 28, 1985 a 1300-metre gap, the largest dam yet built in this South Asian country.

Construction began with the sinking of huge mattresses to protect the river bottom from erosion. Boulder and clay bags were dumped in gullies to make a level sill. Over this base more bags were stockpiled until closure day. Then teams hoisted bags into the gaps creating the barricade (neap tide dam). Next earthmovers raised dam's height with clay (spring tide dam, final dam). The structure was faced with concrete and bricks.

The sand from the river could not be used to fill the jute bags as it was so fine that it sifted out of the bags. Therefore clay was dug in the Chittagong Hill Tracts trucked down and scooped in the bags. The clay-filled bags were stacked in four-metre high stockpiles, each stockpile containing 100,000 bags. The sea rising two metres in half an hour rushed through the gaps between the stockpiles at 2.5 m/s. The current could have dislodged the bags, undercut the artificial islands, and destroyed six months' work. But the islands held.

On the day of the closure thousand-man teams were directed to each artificial island, identified by a coloured flag. As the ride reached its lowest point the workmen began to shift the top layers of bags into the gaps alongside the stockpiles. Working from assigned and flagged stockpiles, labourers swung 600,000 bags into the gaps creating a bund or embankment three metres wide on top and two metres high in a record seven hours.

Twelve million bricks were baked in kilns near the work camp; then men pounded most of the bricks apart to make gravel and aggregate needed in the final concrete facing. The ten-metre-high dam proved so strong that it easily withstood the storm surges of a tropical cyclone that whipped out of the Bay of Bengal three months later. Even with the use of local labour and resources the entire 3,000 meter long structure, including the outlet sluice, cost more than 20 million U,.S. dollars.

21.2. Dikes and dams in The Netherlands

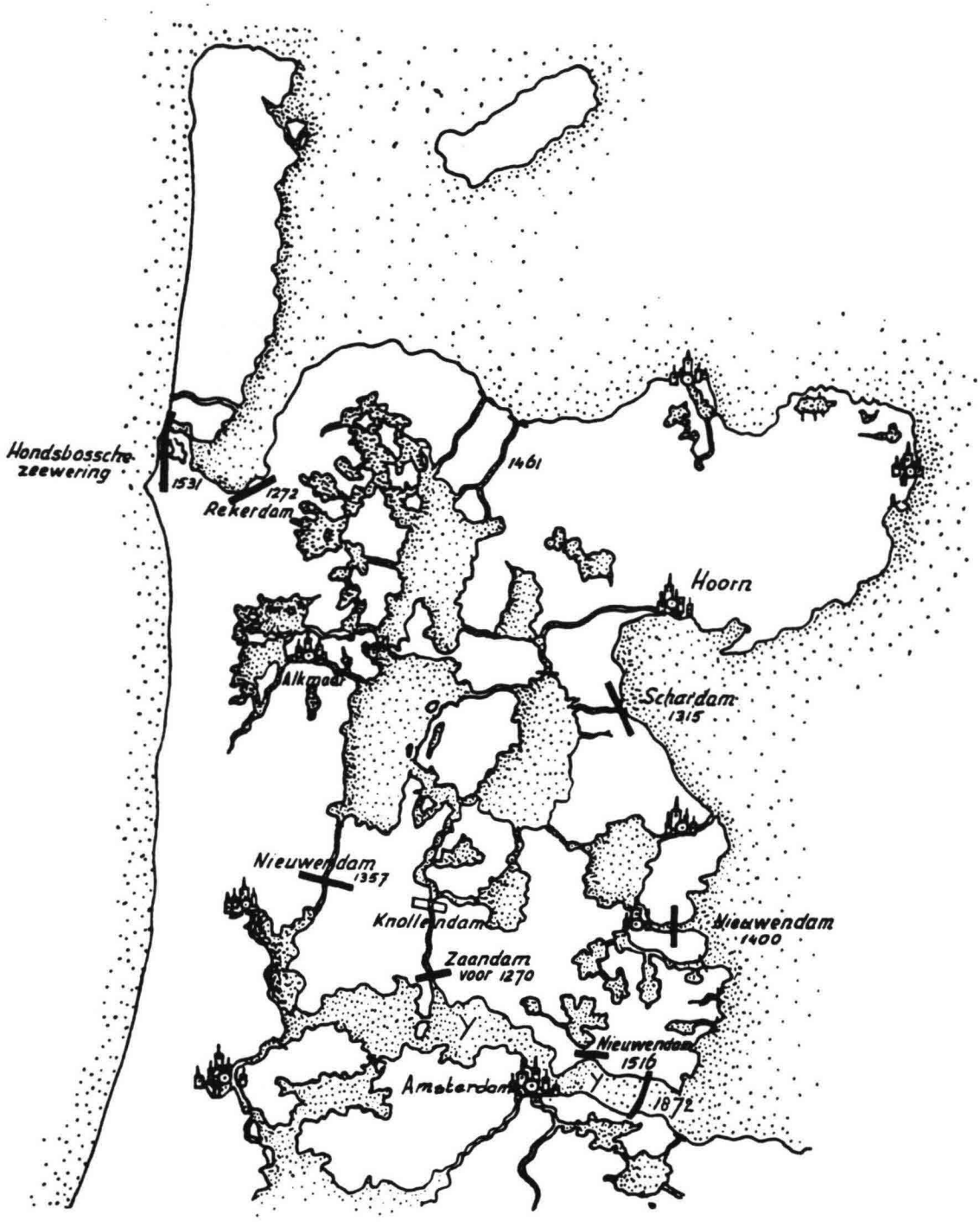


Fig. 21.4. Situation of Amsterdam, about 1550. The water direct north of Amsterdam, the IJ, is in open connection with the sea in the east. A dam is made in 1872.

The western part of The Netherlands, which is called Holland, "hollow land", lies below the level of the sea and that of the major rivers. In the course of the past centuries this low-lying area has been almost entirely encircled by dikes as a protection against high water levels. In the past there was far more water in Holland than there is today. Dikes were built round lakes and the water was pumped out to create a new polder and to increase the acreage of land.

Province of North Holland

Dikes are of vital importance of North-Holland, and for several reasons. North Holland is a low-lying area surrounded by water. The tidal difference is normally 1.5 m, but this is often increased by the prevailing westerly winds. A storm surge, for example, can raise the sea level more than 3 m. Another factor is the gradual rise of the sea level by 2.5 to 10 cm/ 100 years. And finally, many parts of North Holland are not only lowlying - the **Wieringermeer-polder** is as much as 5 m below sea level - but the land settles as well. A good sea defence is clearly an absolute necessity. Fortunately, the west coast has a natural sea defence in the form of dunes, which, when carefully maintained, provide an excellent protection against the sea. At some places, however, the wall of dunes is relatively thin, and between Camperduin and Petten there is a 5 km stretch without any natural sea defence. The coast would lay open to attack if the system of dikes called the **'Hondsbossche Zeewering'** has not been built. To the east, the open and often rough Zuiderzee threatened North Holland for hundreds of years. The coastline is completely protected by dikes. In 1932, when the Zuiderzee was closed by the **Afsluitdijk** and became the IJsselmeer, these dikes became a second line of defence.

The first dikes were made of mud, grass and **seaweed**. Seaweed was a suitable material since it was cheap and when compressed, strong and impervious to water. The seaweed dikes were later reinforced by **wooden piles**, until in 1731 the disastrous **pileworm** undermined the dikes by gnawing away the wood. The Hondsbossche Zeewering was not seriously damaged because it did not contain many piles. As a countermeasure against the pileworm, the piles were replaced by **boulders**.

On the Zuiderzee coast, West-Friesland and especially **Waterland** were struck by nearly all sea floods. On the North Sea coast, only the villages on the coast, such as Egmond aan Zee, Wijk aan Zee, Callantsoog and Petten, suffered the destructive effects of the sea, even through the dunes were often washed away. Inland was relatively safe behind a reserve system of inner dikes: the **dromerdijk** (the dreaming dike) and the **slaperdijk** (the sleeping dike). The **wakerdijk** (the wake dike) was a weak sand dike on the shore, but as the people thought the inner dikes offered enough protection, they neglected it. When it was severely damaged, they simply built a new dike behind the old one. It was not until 1767 that they made a serious start with the construction of a strong first line of defence.

Seafloods

1421 The **St.Elizabeth's Day** flood was a disaster for West-Friesland. Hontsbos and Petten disappeared beneath the waves.The dunes between Camperduin and Petten were severely damaged.
1552 **St.Pontian** flood caused great damage to the Hondsbossche Zeewering.
1570 The **All Saints** flood made three breaches in the Hondsbossche Zeewering; the Zijpe polder, reclaimed in 1552, was inundated; Callantsoog was washed away.
1573 The sea made one breach in the Hondsbossche Zeewering and reached the slaperdijk.
1586 Waterland and the Zaanarea were inundated.
1610 The sea dike of Waterland broke at three places. The encircling dike of the **Beemster**, where reclamation was almost complete, gave way.
1625 The Hondsbossche Zeewering was destroyed; large part of Petten disappeared.
1675 Several dike breaches; West-Friesland and Waterland inundated.
1704 Petten was severely damaged
1717 Egmond aan Zee was threatened by the waves. Dike breaches, the sea reached Alkmaar, Avenhorn and Waterland, but the reclaimed polders remained dry.
1730 Petten was destroyed.
1775 Hondsbossche Zeewering damaged; seadike of Waterland breached.
1825 Stone revetment of Hondsbossche zeewering destroyed;
1916 Dike breaches in Waterland and the Anna Paulowna polder. All sea defences were damaged. The serious situation triggered the amalgation of the long-established polder and drainage boards (waterschappen), and the foundation of a coordinating dike conservancy board.

Fig. 21.5. The "Hondsbossche Zeewering", the seadike between Camperduin and Petten (Province of North Holland) was built in the 15th century. At that time it lay 1200 m further to the west than today. During the past 500 years the dike was often severe damaged or even completely lost. Today, the Hondsbossche Zeewering" is 11.5 m high, 140 m wide and 4.5 km long, it has 30 breakwater (groins) and it is protected by tons of ballasting stone and numerous brushwork mattresses.

History of the Hondsbossche zeewering

The history of the Hondsbossche zeewering covers more than 500 years. Until 1421 a wall of dunes, formed in the first century, constituted a natural sea defence between Camperduin and Petten, 1200 m seaward of the present coastline. The dunes were steadily eroded by the waves.

1421 St.Elizabeth's Day flood swept away large parts of the dunes.
1432 A slaperdijk was built behind the remains of the dunes.
1446 Dunes were made and planted with marram grass, a sand dike was built.
1466 A dike in the dunes was built.
1477 Building, repair and maintenance of dikes; paid by Emperor Maximilian
1499 Extensive disintegration and weakening of the dunes and dikes; a new sand dike was built inland.
1506 A double row of piles were set up over a distance of 500 m; the first eight breakwaters were constructed, 85 m long and 3.5 m wide.
1507 Two more breakwaters were added.
1508 Two more breakwaters were built. The breakwaters were joined together by transverse frameworks of osier and stone.
1524 Sea caused considerable damage. Two more breakwaters built.
1526 Start of constructing of a slaperdijk from Hargen to the sea dike of Schoorl.
1547 The Hondsbossche Zeewering had 22 breakwaters.
1552 Nearly all breakwaters and transverse frameworks were damaged or destroyed.
1570 The waters of the All Saints flood made three breaches in the dike; inundation of Zijpe polder. The old sea dike was abandoned and a new dike was built behind it.
1573 The dike was breached in a heavy storm; because of the war with Spain, the gap was not allowed to be sealed until 1577.
1614 A dromerdijk was built to serve as an extra sea defence.
1625 A severe storm surge washed away a large part of Petten. All the defences of the Hondsbossche Zeewering were destroyed. Some 100 m behind its remains, a new Hondsbossche Zeewering was built without breakwaters or pile constructions.
1730 The old village of Petten was lost to the sea. The Hondsbossche Zeewering was drawn back to the line of the sea dike of Schoorl.
1793 After a number of heavy storms, there was hardly any form of sea defence left. A plan for reinforcing the dike was postponed.
1796 Construction of a reinforced dike and breakwaters which were 150 m long and 15 to 18 m wide. More breakwaters were built in 1806,1808 and 1811.
1825 The stone revetment, improved in 1823, gave way during a storm surge.
1834 Parts of the dike were swept away by heavy storms.
1837 Plans were made for a pile construction. This work was carried out in stages between 1839 and 1847. Where the breakwaters met the dike, they were joined together by a pile construction, called **kisting**. The kisting was formed by two parallel rows of piles: at the front heavy oak piles extending 3.5 m above sea level and at the back pinewood piles, which extended 2 m above sealevel and were joined together by boards. The 60 cm space between these rows of piles was filled with reed, rubble and limestone. On the seaward side, the kisting was protected by a 3 m wide osier revetment on reed, which was strengthened with stones and retained at the front by a palisade of 1-2 m long piles.
1840 The breakwaters were made 100 m long and 10 m wide.
1850 Seven new breakwaters brought the total to 29.
1875 A 30 m basalt revetment on rubble on a layer of clay, with a height of 5 m above sea level was constructed.
1916 The Hondsbossche Zeewering survived the violent storms of January.
1938 The crown of the dike was heightened to 8.5 m above sea level.
1953 During violent storms and the spring tide large parts of the dunes were washed away; 1500 m^2 of stone revetment were destroyed.
1977 The Delta act required the raising of the 8.5 m high Hondsbossche Zeewering to 11.5 m. Construction from 1977 to 1981. Foot of the dike widened to 140 m; 1.5 million m^3 of sand was placed. Cost: Dfl 32 million.

Zuyderzee Barrier Dam or enclosure dam.

The closing-off, drainage and reclamation of parts of the 'Zuyder Zee' had long been considered desirable. The three main objectives were :

- protection against high floods
- creation of fresh water storage basin
- reclamation of new arable land (agriculture).

The first plans, produced by **Hendric Stevin** (1667), the son of Simon Stevin, were not feasible because of lack of technical know-how at the time. The next plans for the closure and (or) partial reclamation of the Zuydersea date from the second half of the 19th century, that is, from the time the **Haarlemmermeer** and many other polders in Holland were reclaimed.

The first plan suitable for execution was made by Dr. **Cornelis Lely** in 1891. Food shortage during World War I and a severe flood in January 1916, causing dike breaches and inundation of large areas along the western shore of the Zuydersea, made it imperative that something should be done. In 1918 parliament passed the **Zuydersea Act** describing the closing off and partial reclamation of the Zuydersea, according to the plan of Lely.

The work started in 1920; during the first years it was slackened and even interrupted by an economic depression; in 1925 a second start was made.

The dikeline of the enclosing dam from the coast of **North Holland** to the western end of the island of **Wieringen** (2,5 km) and from the eastern end of this island to the **Frisian Coast** (30 km). The first 2,5 km dike was completed by 1924. Work on the construction of the Barrier Dam began in 1927, after the **Lorentz committee** had investigated the tidal currents to be expected during construction. A highest velocity in the closing gap at normal tides, not affected by wind, of 3,5 m/second and during storm tides of 6 m/second was predicted.

The **enclosure dam** had to be built in the open sea at an average depth of 3 m, except for channels, where depths of more than 12 m were measured. During construction some of these channels deepened to 20 m and more. The sea bed was, in general, adequate to carry the weight of the dam, but in short stretches layers of soft clay and peat had to be removed by dredgers and to be replaced by sand. **Boulder-clay**, a firm, tough loam originating from the morene's of glaciers during the ice-ages, was discovered in the vicinity of the place where the dam was planned and this clay loam proved extremely useful in resisting the current and the wave forces. As a heavily compact soil, consisting of particles of all sizes, from the smaller clay fractions up to boulders - the grain-size distribution diagram shows a nearly straight line through all sizes - it is practically impervious.

On the sea side of the dike a heavy dam of boulder clay was designed up to the design stormflood level. The dam was backed by a body of sand, brought into the dam by hydraulic fill. The whole body of sand was once again covered by a layer of boulder clay.

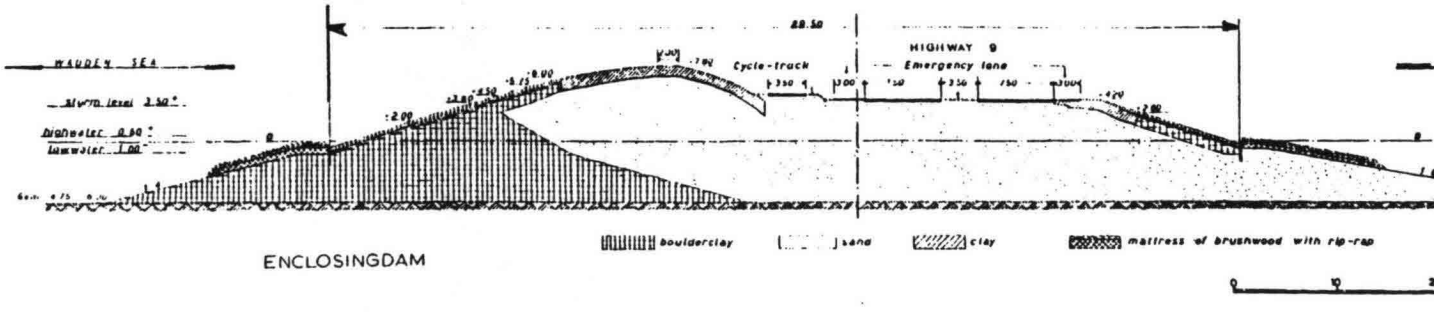


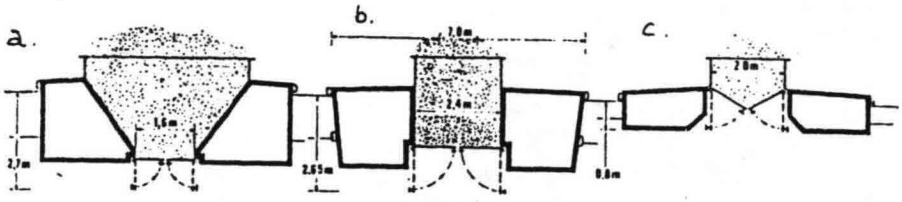
Fig.21.6. Cross-section of Barrier dam.

The clay loam has been of vital importance, transport distances varied from less than one to about eight kilometre. First, two parallel clay loam dams were built under water. The clay loam was dredged in the area and deposited into position from hoppers with gates in the bottom of their holds to release the material.

The bottom discharge barges of the time (fig.21.) were not suitable to discharge clay, as the clay loam tended to stick to the walls. Special barges had to be developed with vertical walls and the chains (for the operation of the gates) in separate, small compartments. As the gates were of course wider the hinge was moved upwards so that the draught remained the same. Another type of vessel was developed, the so-called **'top-bottom discharge barge'** with a draught of 0,8 meters only. (fig.21.).

The hinges were now installed as high as possible so that the gates -when open - were still inside the barge. The vessel had only a small loading capacity, 40-50 m, instead of 150 m, but was considerably long and wide. As the loaded draught was 0.8 metre only and the tidal amplitude about 1 metre, it was possible to make a dam up to the low waterline.

Fig. 21.7. Barges for dumping clay loam.



The clay was very efficiently handled with grabs, usually mounted on floating cranes. These **floating steam cranes** were used to raise the clay loam dam on the **Wadden Sea side** up to spring tide level. The clay loam was excavated by bucket dredgers, and loaded into elevator barges, which were towed by tugs to the crane pontoons. The grab cranes could move one load of 2 to 2,5 m per minute. Work on the dam was, however, slow and cumbersome: only one crane could work protected by the dam it had just made. The operation was somewhat speeded up by bringing the dam up to high water level, while a second crane followed, completing the clay loam dam.

Fig.21.8. Floating steam crane raising the clay loam dam up to spring tide level.

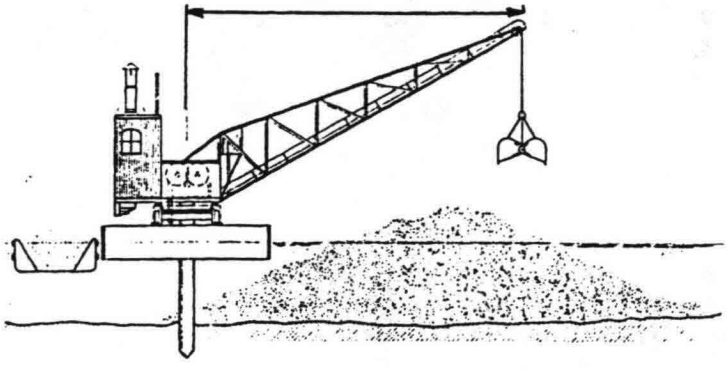
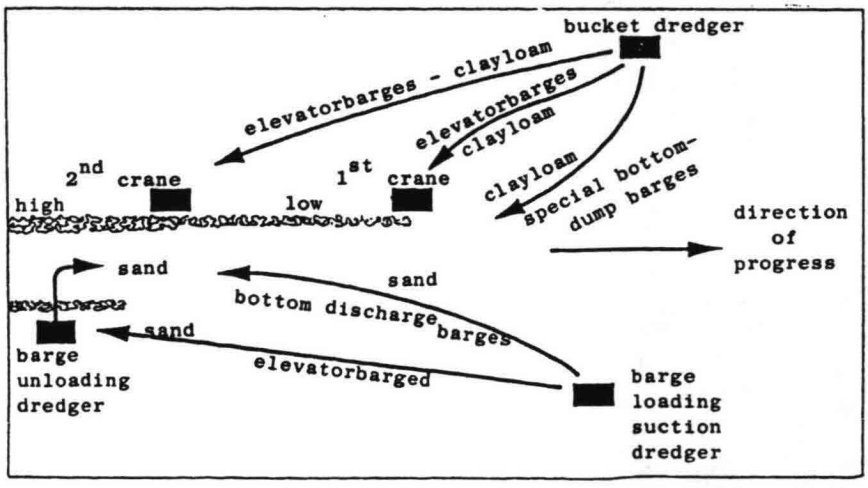


Fig. 21.9. Method of construction.



The sandbody was made at the innerside of the clay loam dam, outside the currents, first by bottom discharge barges and then brought above water by pumping, using barge unloading reclamation dredgers.

Work at the Enclosure dam restarted in 1927 at several places. At Breezand, a sandbank halfway between North Holland and Friesland, an artificial island was built with two working harbours, one allowing access from the Waddensea and the other from the Zuydersea. The building of the Barrier dam itself began at six different points, where the sea was not so deep. As the sections of dike grew towards one another, the channels between them became progressively smaller. The sea bed in the channels had to be protected against scour because of the high currents (5-6 m/sec.).

The final gaps were at **'Middelgronden'** (1500 m wide) and **'Vlieter'** (500 m wide). The tides flowed in and out of the Zuydersea with great velocity every day. The original intention had been to wait until the spring of 1932 before closing the two gaps, but the condition of certain osier mattresses affected by shipworm did not allow any postponements and the 'Middelgronden' was closed on November 22, 1931. On 28th may 1932 the final gap was sealed.

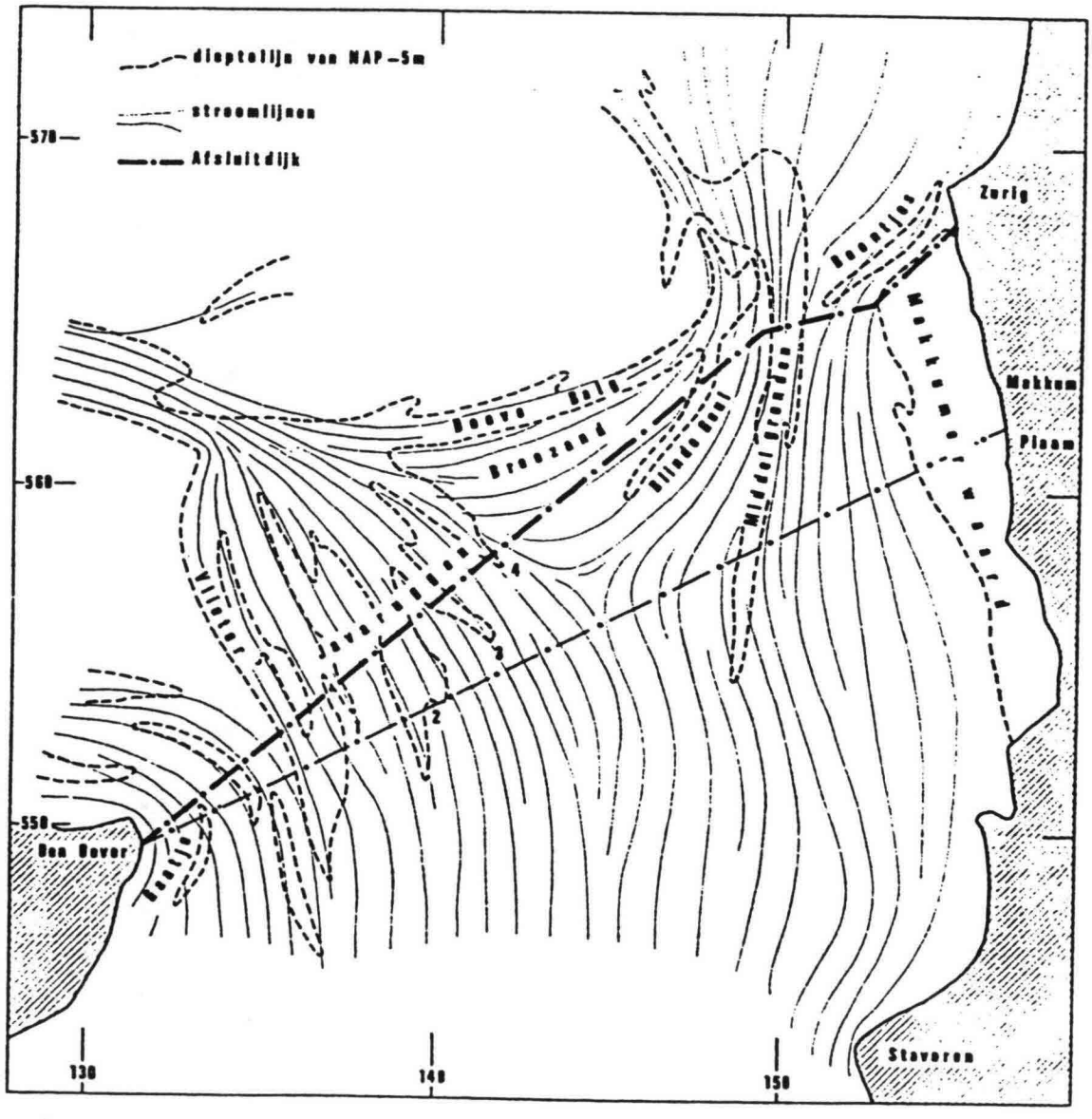


Fig. 21.10. Plan of barrier dam.

Contractor

The first Zuiderzee works were awarded to the lowest bidder. This system of tendering (the usual way of contracting in the Netherlands) was, however, not suitable for the contract of the Barrier Dam construction. The construction period was very long (perhaps eight years) and the conditions at the end of this period could not be foreseen. Furthermore it was not possible to predict an accurate description of the closing method of the final gaps. Splitting the works in separate contracts was undesirable.

It became clear as early as 1920, when the first dike was constructed, that special equipment was needed such as barges for the boulder clay transport and grabcranes. No contractor was prepared to invest large sums in this equipment without certainty of being awarded the total contract. For these reasons the M.U.Z.-contract was invented.

Four large Dutch contractors (A. Bos Pzn, L. Volker Azn, H.A.M. and M.J. van Hattum Ltd.) formed in 1922 the M.U.Z.-joint venture (company for the execution of Zuiderzeeworks). In 1926 an agreement was signed between the M.U.Z. and the government for the construction of the total dam; the various parts were negotiated later. A profit-sharing scheme was included; open accounts were checked by an auditor appointed by the minister of 'Waterstaat'. Up to 6 % profit was for the contractor; profits in excess of 24 % had to be returned to the State. Profits between 6 and 24 % were shared. In this way the client was protected against excessive profits. A profit was finally realized in the order of 12-18 %.

Equipment (at the closing of the 'Middelgronden' gap in 1931):

11 bucket dredgers	37 hopper barges
6 suction dredgers	60 elevator barges
7 double action suction dredger	35 pontoons
4 clay loam grab cranes	60 tugs and other equipment
4 other cranes	

Materials (1927-1932) measured in means of transport:

total length	: 28 500 m
sand	: 24,4 millions m^3
clay loam + clay	: 15.2 millions m^3
mattresses	: 1,4 millions m^2

Average quantity per m dike: 85,000 m^3 sand and 53000 m^3 clay loam.

Highest weekly production: 330,000 m^3 sand and 92,000 m^3 clay loam.

Work on the first polder, the **Wieringermeer**, had already started before the closure of the Zuydersea in 1932. An 18-km dike between Medemblik and Wieringen was constructed in 1927-1929. A total quantity of 12 million m^3 (of which 6 million m^3 clay loam) was used. Before the pumping out of the water in February 1930 began, the primary dredging of the main canals with heavy equipment was completed. In 1928-1929 some 15 to 20 large bucket dredgers were at work to remove 9 million m^3 for the main canals and 6 million m^3 for other canals. After the pumping out of the polder, which took 6 months (600 million m water), the secondary dredging, the removal of silt deposited in the canals during reclamation, had to be carried out. A total quantity of 6 million m^3 was placed at the side of the canals by some 20 small bucket dredgers.

In 1936 building of the dikes began for the second polder in the **IJsselmeer, the North East polder.** The ring dike was 54 kilometres long, 24 between the island Urk and Lemmer in Friesland and 30 km between Urk and Kadoelen. The subsoil of large parts of the dike trace needed improvements, the soft layers of clay and peat had to be removed by dredgers and replaced by sand. In October 1939 the Urk-Lemmer dike was closed, followed in December 1940 by the last gap near De Voorst. About 30 million m^3 was handled, of which some 6 million m^3 clay loam.

As in the case of the Wieringermeer, the first work to be done within the ringdike was dredging of canals and ditches. Some 84 km of canals had to be dredged of which 42 km main canals. These main canals had a bottom width of 12 meters and depth of 2 ~ 8 metres. This so-called primary dredging work, involving 14,7 million m^3 was started in early 1939 and finished in the winter of 1940, when the large equipment had to leave the polder (as the dike was closed). At the same time 2,3 million m^3 of soft soils was removed at the location of the future towns of Emmeloord and Marknesse and replaced by sand from the dredged canals. The secondary dredging work started in 1941 and involved some 12 million m^3.

The first large dam, the Zuyderzee Barrier Dam, was completed in 1932. Although its height of 19 m is not large, by present day standards, its length of 32 kilometres, gives the dambody, consisting of sand, clay and rock, an impressive volume of 63.4 million m^3.

In the decades following the completion of the dam large polders were constructed in the now relatively quiet water of the IJssel lake (former Zuyderzee); over 200 kilometres of dikes surround a vast poldered area of 165,000 ha.

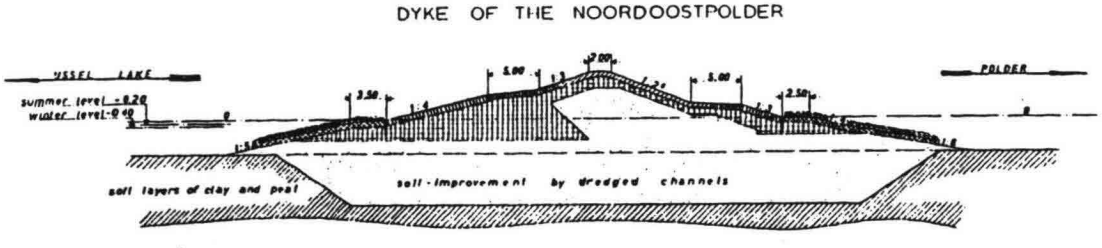


Fig. 21.11. Cross-section of a polderdike.

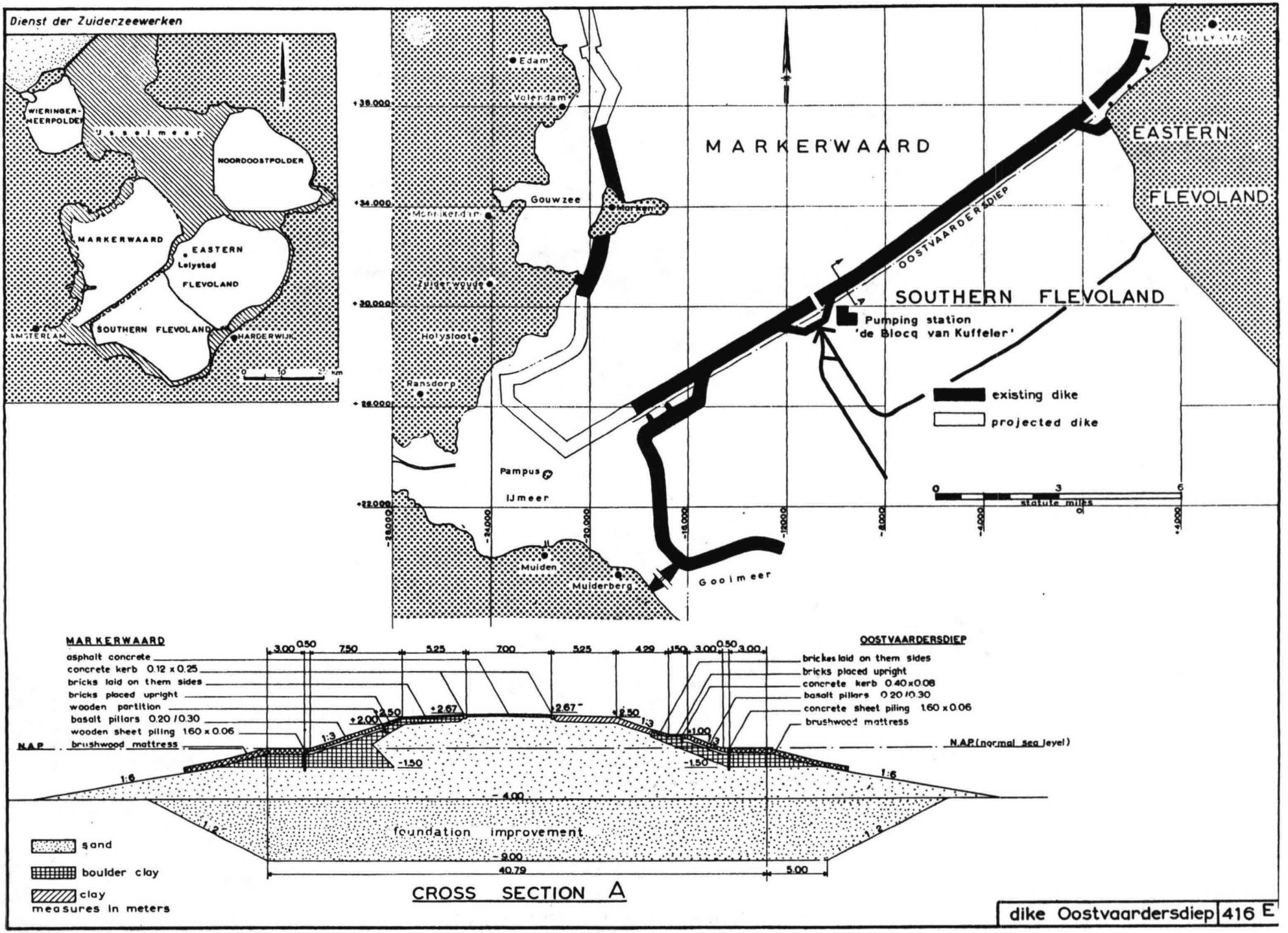


Fig. 21.12. IJssellake - Polderdike Southern Flevoland - The Netherlands.

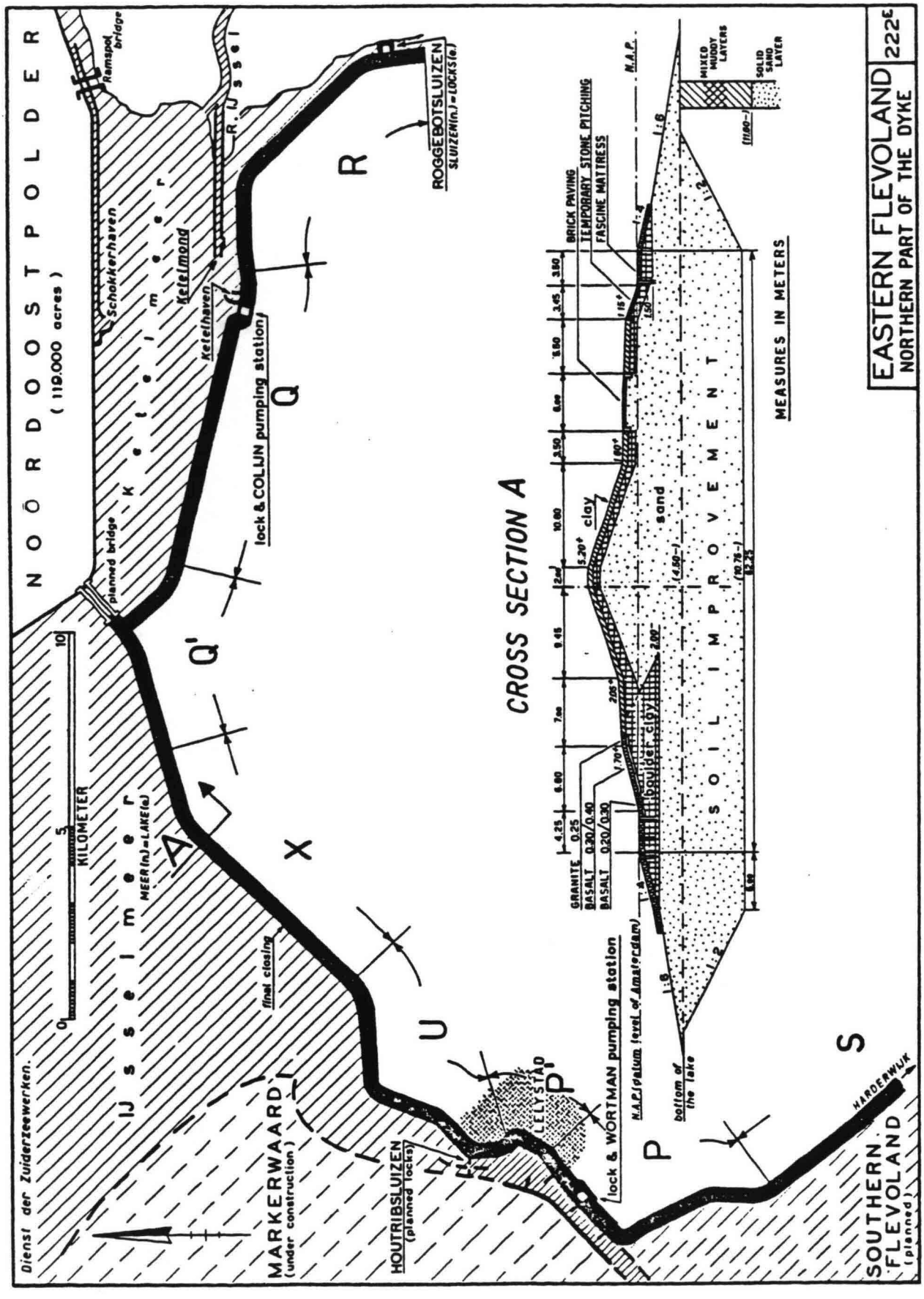


Fig. 21.13. IJssellake - Polderdike Eastern Flevoland - The Netherlands.

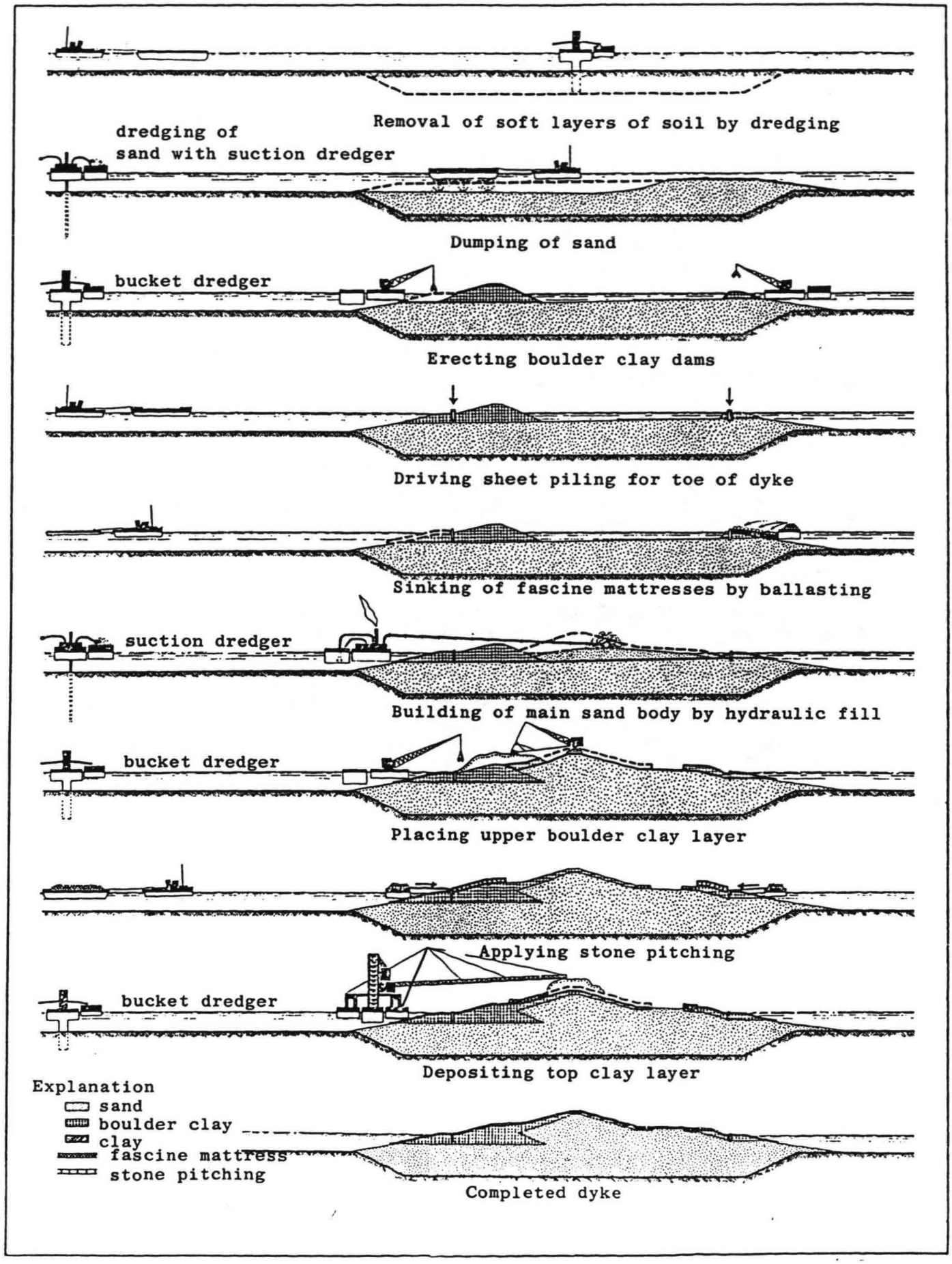


Fig. 21.14. Construction of a polderdike in The Netherlands.

Delta Project/ Delta works.

In the Delta region three great rivers flow into the North Sea. The region lies between the ports of Rotterdam and Antwerp (Belgium). There are three Dutch provinces in the Delta region : Zeeland, South-Holland and North Brabant. Nearly all the Delta region is below sea level, only close to the sea the land is higher than the sea, the dunes. Most people in the Delta live on the low-lying land, protected by dikes. The area is subject to tides. Heavy storms in the North Sea can cause a considerable set-up of the water level.

On the night of 1 February 1953 the combination of a spring tide and a persistent, violent northwesterly storm caused an extreme high water level. Waves destroyed the dikes and polders were inundated. Approximately 500 km of dikes were completely or partially destroyed, 200,000 hectares of land was flooded, 1,835 people drowned, many thousands of cows, horses, pigs and chickens were killed and some 47,000 houses and other buildings were destroyed. Great efforts were made to reseal the breached dikes. The last breach, near Ouwerkerk on Schouwen - Duivenland was resealed at the beginning of November 1953, just before the winter season.

The people of The Netherlands decided on the Delta Project to prevent such a catastrophe from ever happening again. The Project's principal goal was to improved the safety of the southwest Netherlands by considerably shorting and reinforcing the coastline. Dams were to be constructed across inlets and estuaries; primary and secondary dams with freshwater lakes behind the primary dams. Roads along the dams would improve access to the inland of Zeeland and South Holland. Dams could not be constructed across the New Waterway or the Westerscheldt, as these important shipping routes to the seaports of Rotterdam and Antwerp had to be kept open.

After the major flooding of the southwestern part of the Netherlands in 1953 three of the estuaries of the Rhine-Meuse Delta have been separated from the North Sea by means of large dams:

- **Haringvlietdam.** Has an overall length of 5500 metres, and a maximum height of 24 metres (20 million m^3 of mostly hydraulically placed sand; discharge sluice with 17 openings each 56.5 metres wide).
- **Brouwersdam.** Has an overall length of 6200 metres, a maximum height of 36 metres (27 million m^3 mostly hydraulically placed fill).
- **Eastern Scheldt storm surge barrier.** Overall length 9,000 metres, maximum height 45 metres. The main feature of this dam are the three sluice sections (overall length 2800 m.). Under normal circumstances the sluices are open but during storm surges in the North Sea they are closed.

Fig. 21.15. Delta works.

Closing tidal basins by means of cable way
(gradual vertical closure)

location	year	material	dumping capacity
Grevelingen dam, Northern channel	1963	rubble 60 - 300 kg	120 tonnes/hour
Haringvliet dam, Rak of Scheelhoek	1970	4 concrete blocks of 2.5 ton per telpher (own weight 20 tonnes)	300 tonnes/hour
Brouwersdam, Southern channel	1971	6 concrete blocks of 2.5 ton per telpher (own weight 17 tonnes)	1000 tonnes/hour

Table 21.1. Closures in south-west Netherlands by means of cableways.

A cableway consists of a fixed rail structure on each side of the gap to be closed. Two cables are then stretched between these structures. The cables were independent of each other, and were anchored at one end and tensioned at the other end by a tensioning weight or counterweight. Anchorage was achieved by means of a heavy concrete block, pre-tensioning cables and a cast-steel anchorage seating.

The construction of a cableway generally enables only one telpher to travel along a span at a time. To get a high dumping capacity, loading points are required on each shore. These loading points are installed in the fixed rail structures, where telphers pass along the flange of a rail section. The telphers could travel a circuit, i.e. making the outward trip along one cable, describing a half-circle turn at the other end and returning on the other cable.
The concrete blocks measured 1.04 m^3 and weighted 2.5 tonnes each.

For determining the exact position at which the telpher has to drop its load use may be made of a metrecounter in the telpher and a specially developed guide-light system.

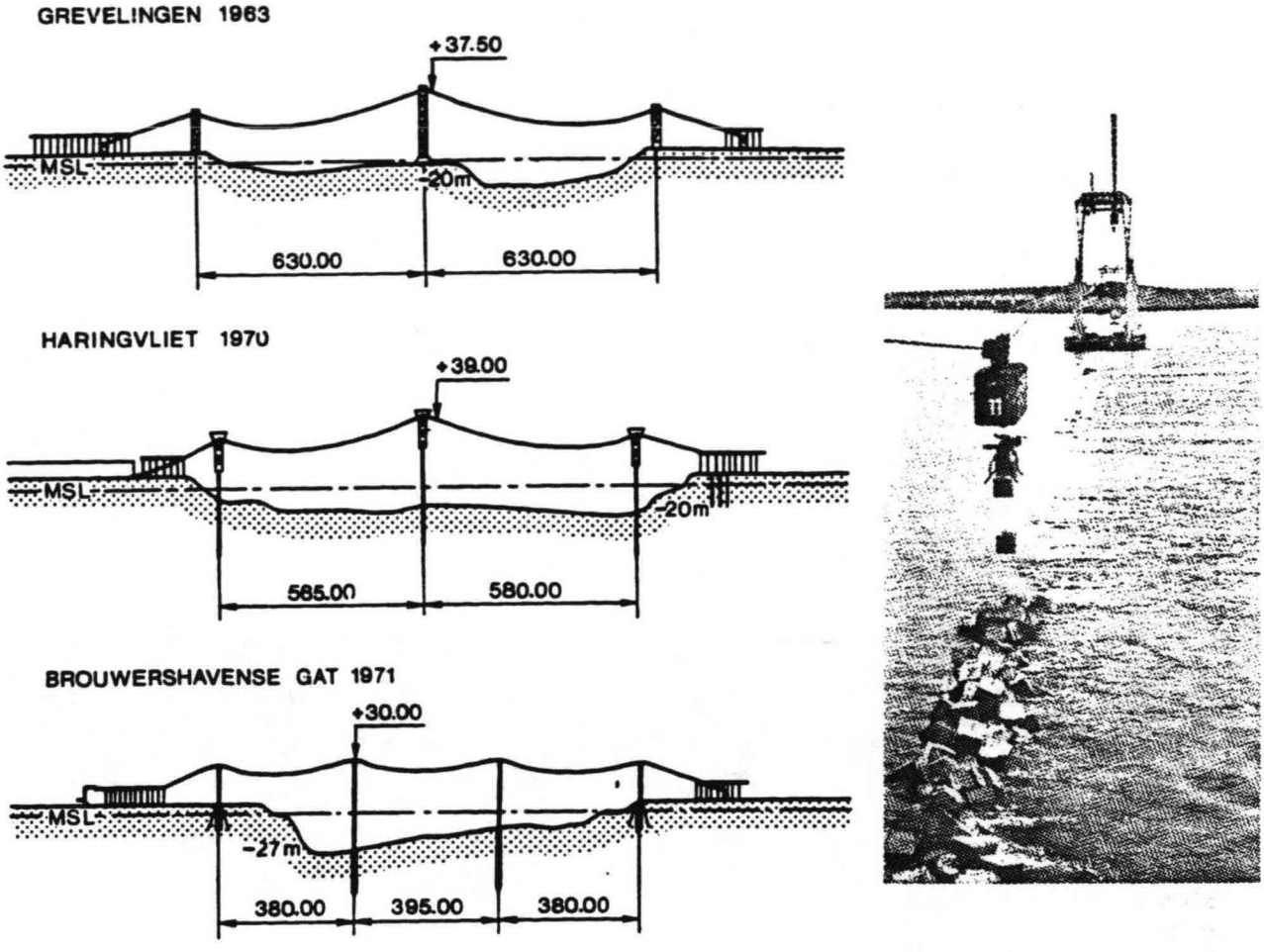
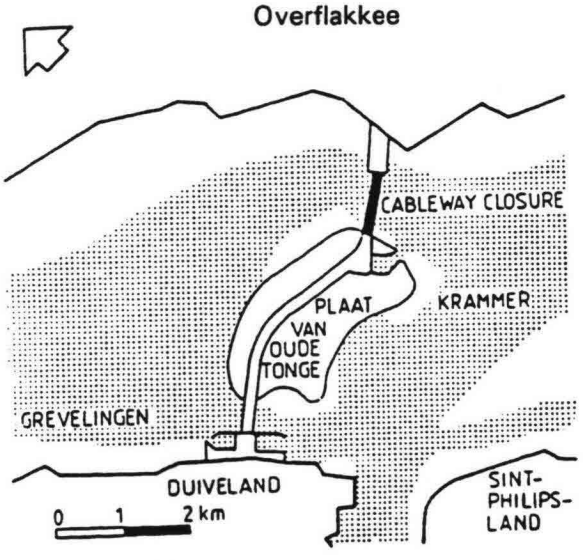


Fig. 21.16. Examples of cable way closures.

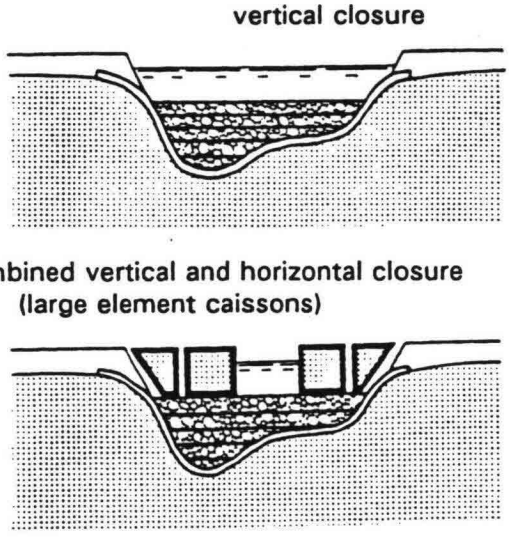
In view of the disadvantages of floating equipment, cableways were used for closing the Grevelingendam (1963), the Haringvlietdam (1970) and the Southern channel of the Brouwersdam (1971) in the Netherlands.

Grevelingendam

The construction of the Grevelingen dam (completed in 1965) entailed a more extensive use of the seabed protection techniques that had been tested in the Zandkreek and the Veerse Gat. The six-kilometre long dam between Schouwen-Duiveland and Goeree-Overflakkee crosses the channels on either side of the Oude Tonge sandbar. The section on the sandbar itself was built by pumping sand onto it. The southern channel was dammed by using the same type of small caissons that had been used in the Zandkreek dam, but a new technique was developed for the larger channel to the north: a cableway was spanned across the channel and cable cars tipped stone in the water.

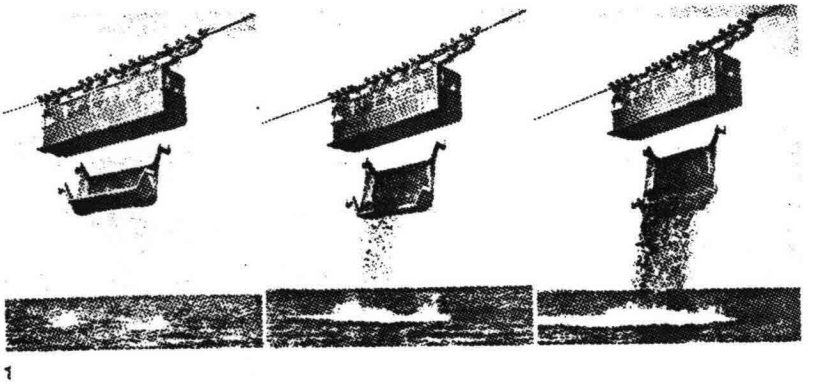


Position of Grevelingen.



Methods used for closing Grevelingen estuary.

Fig. 21.17.



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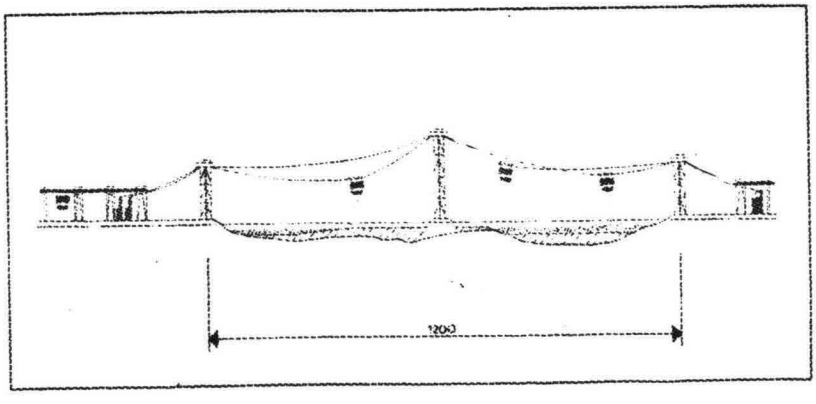


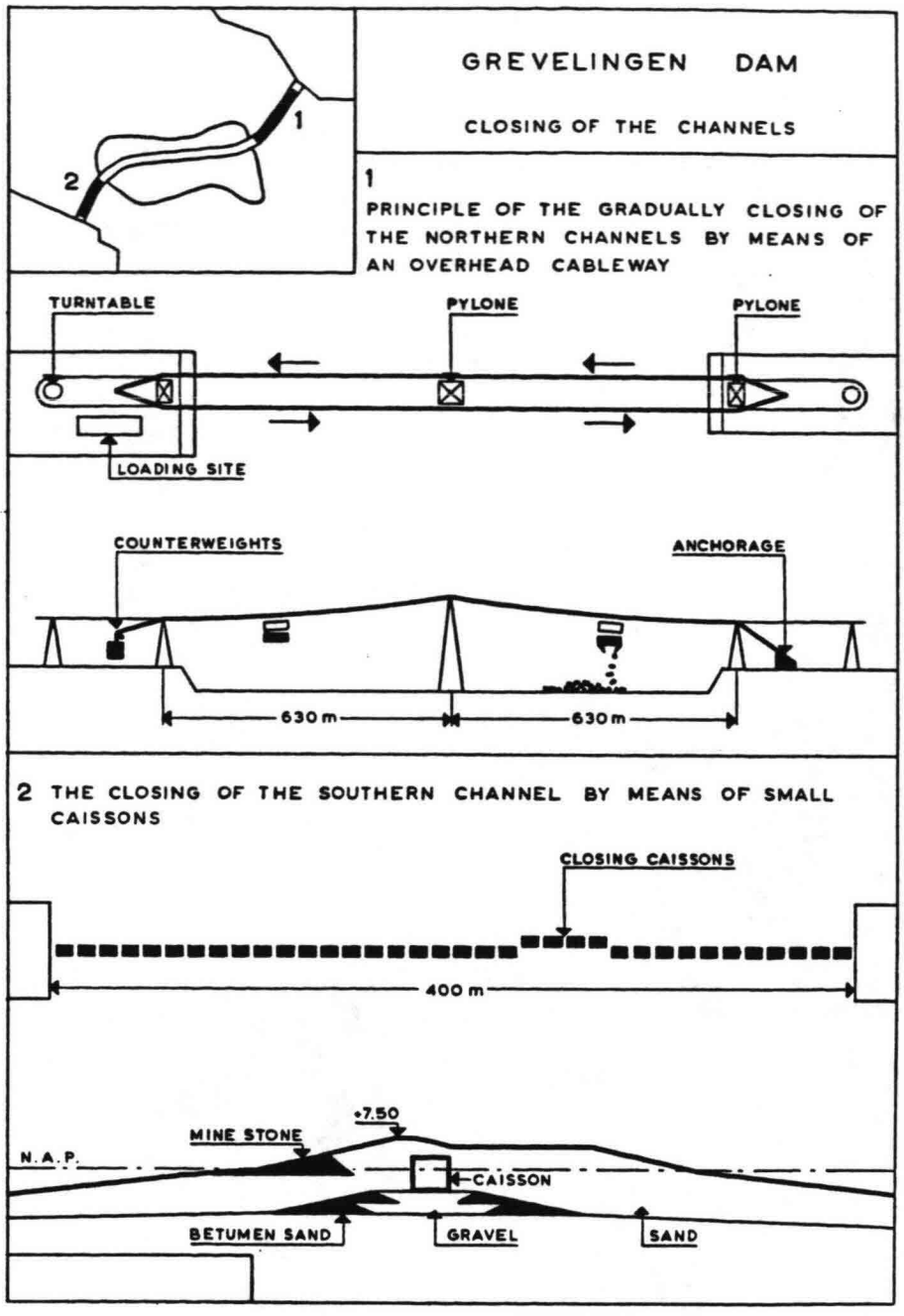
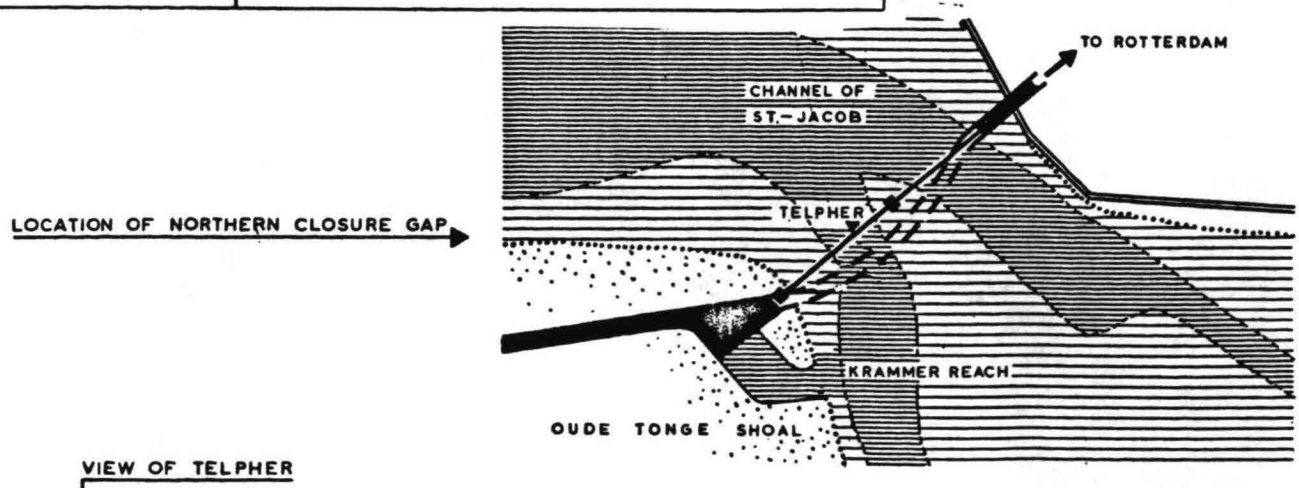


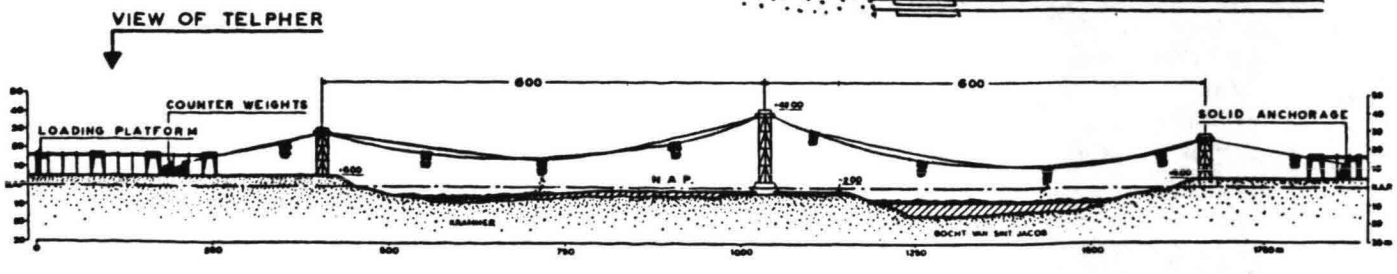
Fig. 21.18. Damming of the Grevelingen Estuary

The northern section of the Grevelingen was closed by dumping stones transported in pouches of chain netting suspended from trolleys running to and fro along a cable at an appropriate height above the site of the projected dam like a telpher.

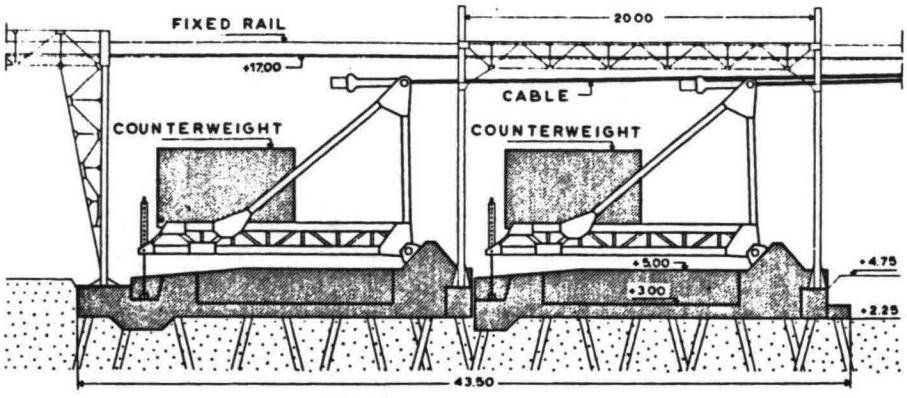
The stones are dumped at the required spot by releasing two of the four suspension cables. The motorized carriages move at a rate of about 9 m/sec. The carriages, which are each fitted with a diesel motor, are individually manned and are capable of transporting 10 tons of stone each. The nets are filled from stone dumps. The capacity of the telpher is about 300 tons per hour : at intervals of just over one minute the carriages leave with a full net, drop their load at the right spot in the water and return via the turntable at the other side to the loading quay.

The whole round-trip takes about 20 minutes. A total of some 110,000 tons of stone had to be handled this way, which took about 12 work-weeks. After that the dam was finished off with sand etc.

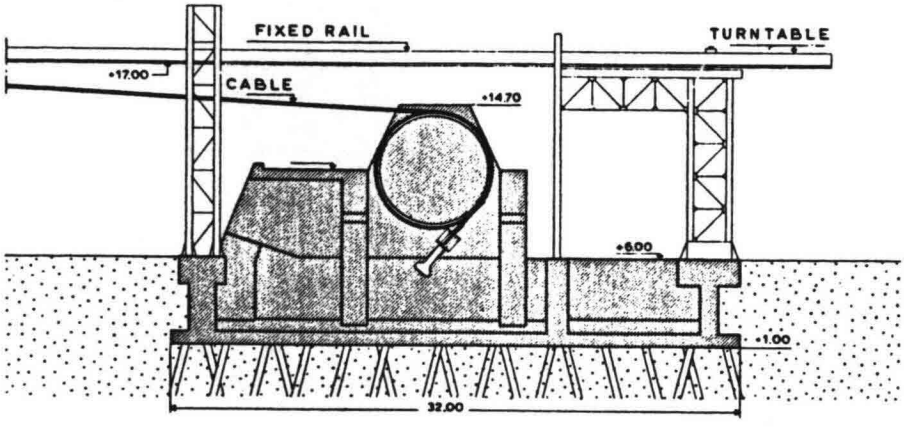




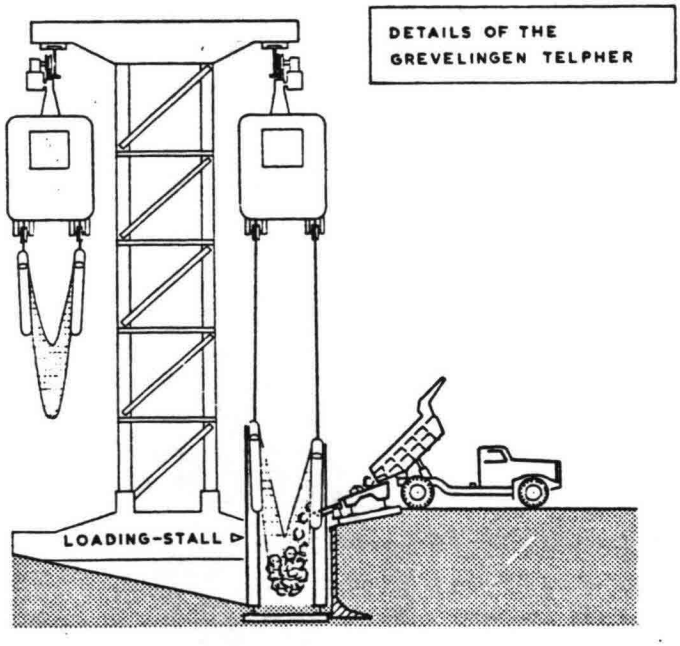
NOTE: ALL DIMENSIONS IN METRES
N.A.P.: NATIONAL ORDINANCE DATUM



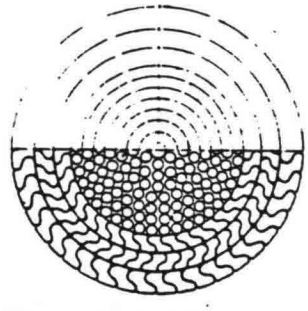
Details of the Grevelingen telpher.
On the South-West bank the cables are anchored to two tilting counterweights.



On the North-East bank the cables are anchored to a fixed point.

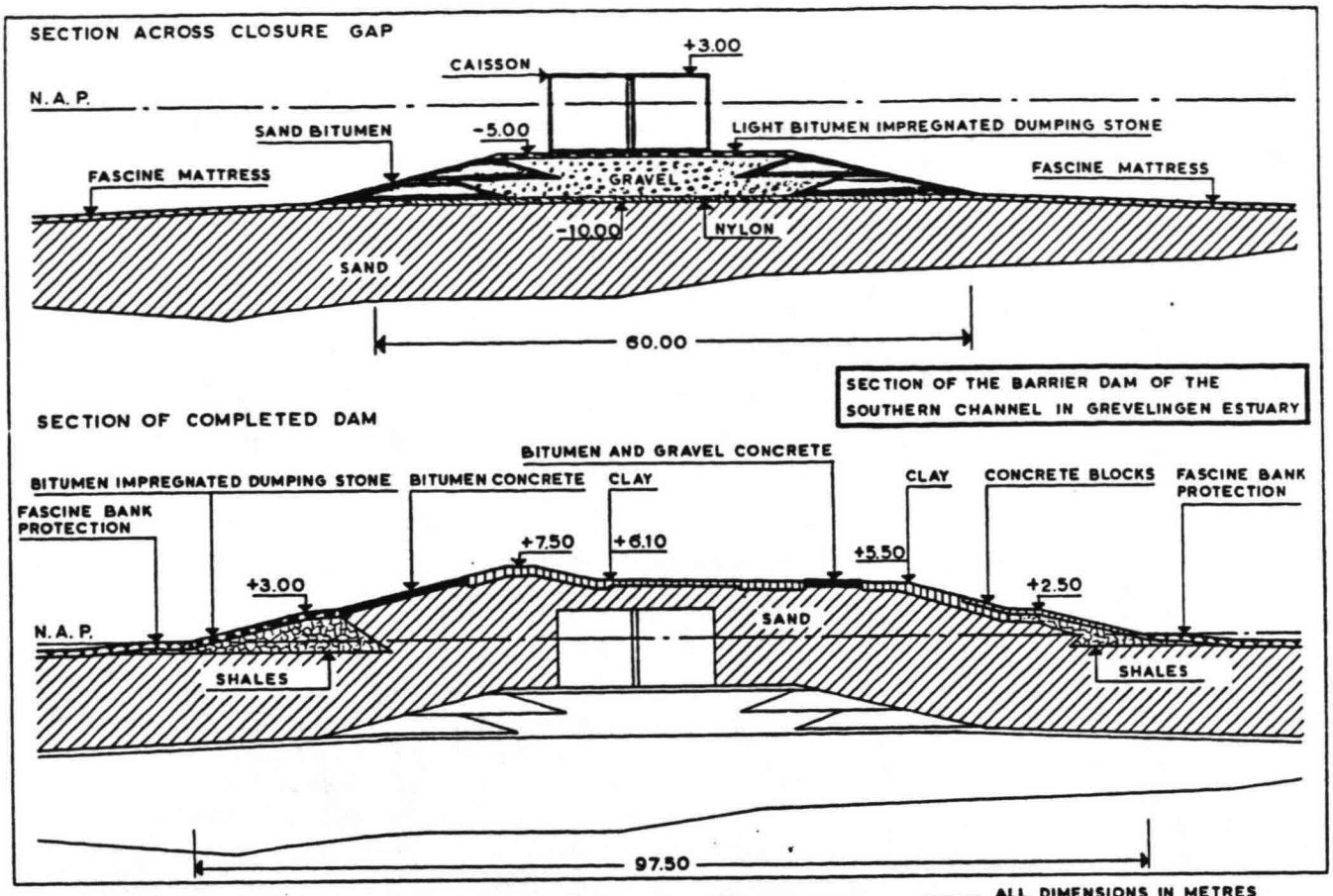


Filling the net in the loading-stall



The cable is 92.2 mm in diameter and weighs 47.1 kg/m. The wires of any one layer spiral at a certain pitch alternating from layer to layer from right to left. So the wires of one layer cross those of the adjoining layers and the connection becomes as firm as possible without any great internal tension. Moreover, the cable does not tend to unwind when subjected to tension stress.

Fig. 21.19. Closing Northern Channels Grevelingen by cableway.
The telpher conveyor is about 1,900 metres long. Two cables, span the gap and are supported by three pylons 630 metres apart. The cable of the Grevelingen telpher is heavier than that of any other telpher in Europe (at that time). When the Grevelingen telpher was installed on 22nd August 1963 one of the cables broke while being subjected to tension due to faulty tension. Tests confirmed that the cable itself was quite strong enough. The broken cable could not be used and it took 5 months to replace it (the new cable arrived early February 1964).



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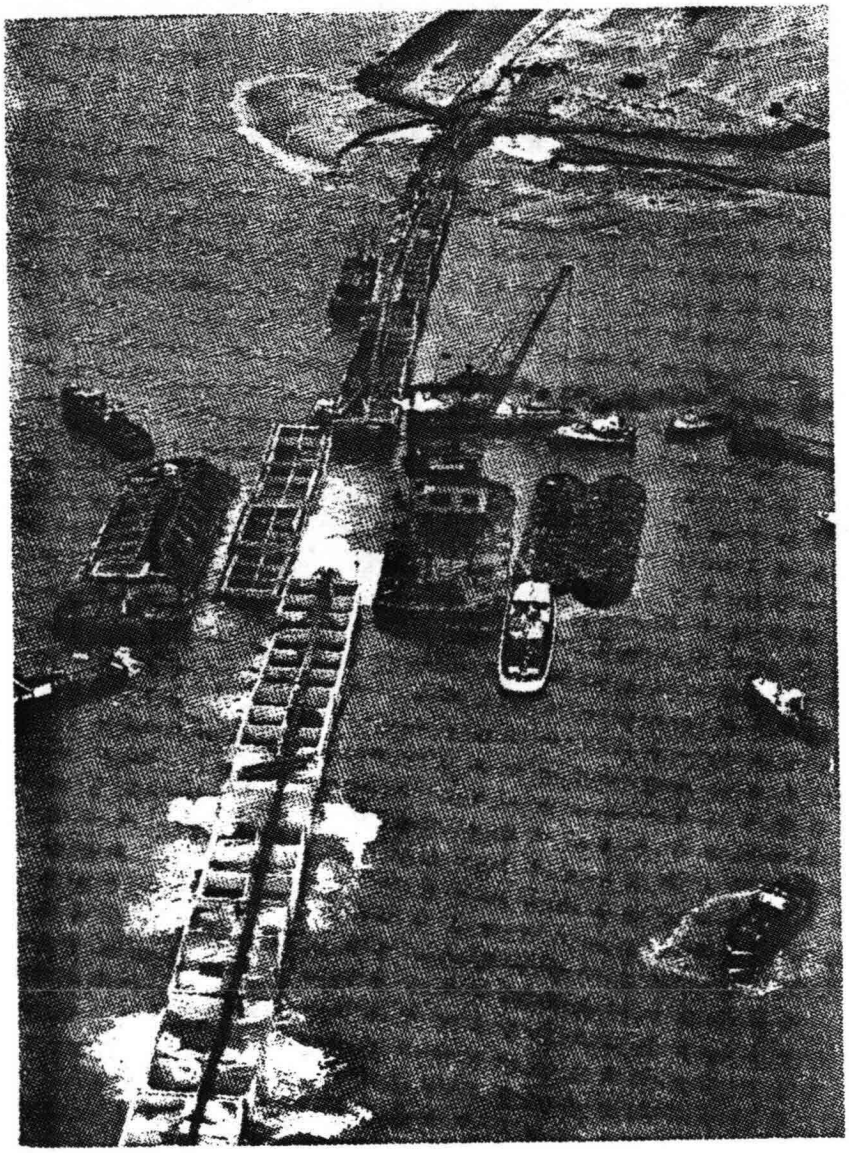


Fig. 21.20. Closing Southern Channel of Grevelingen by means of small caissons.

As the velocity of the current in these channel is relatively low, because the tidal currents from Brouwershavense Gat and the Eastern Scheldt meet more or less at the site of the dam. Consequently it was possible in the spring of 1962 to close the southern channel, the Grevelingen, by means of relatively small closed caissons (7.5 x 11 x 6 metres) surmounted with 2-metre-high blocks of concrete. In 1961 a start was made with the construction of the prefabricated standard caissons. For the closing of the 400 m wide gap 36 caissons and 33 crowning blocks were employed. In the period 3rd to 11th May 1962, 31 of them were put in place.

Photo: Situation - May 12, 1962

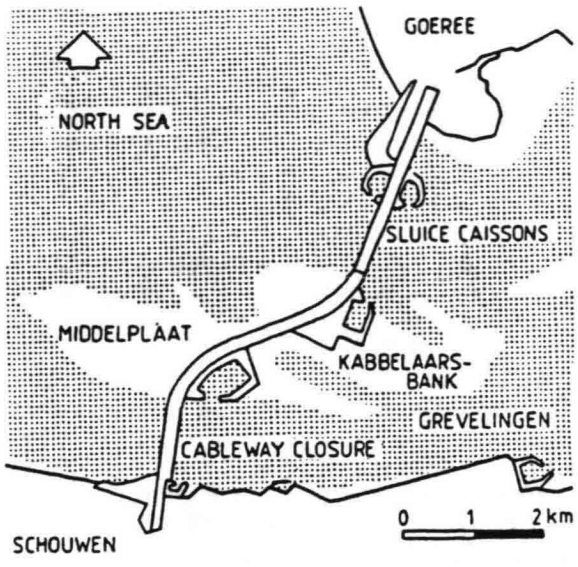
Haringvliet

The Haringvliet dam on the western side of the Haringvliet has two purposes. It must nor only protect the waters inland from high tides, but must also discharge excess water from the Rhine and the Maas. The construction of the 4.5 kilometre Haringvliet dam therefore began with the building of a large sluice complex. The 17 sluices can discharge an enormous volume of water. Closing or opening the sluices determines how much water will to be discharged by the alternative route via the New Waterway. By channelling as much water as possible through the New Waterway to the North Sea, saltwater is prevented from penetrating deep inland and making the groundwater saline. The sluice complex in the Haringvliet dam is called the 'tap of The Netherlands' because of its vital significance to the country's water management. The channel on the southern side of the discharge sluices was dammed with sand, and that on the northern side using the cableway method. The dam was completed in 1971. There is now no tide in the Haringvliet and it consist entirely of freshwater.

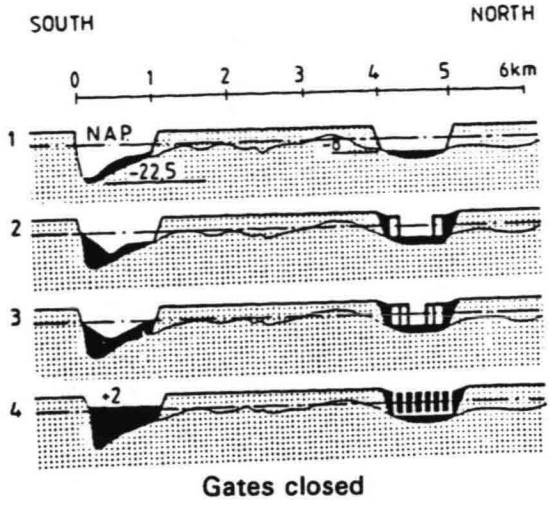
Brouwersdam

A substantial part of the dam built across the Brouwershavense Gat, which forms the western boundary of the Grevelingenmeer, was constructed on sandbars using the pumping method. The norther channel was dammed with sluice caissons. These remained open until the dam in the southern channel, which was constructed using a cableway, had risen well above the water level. Then the sluice gates were closed and the dam across the two tidal channels was completed using sand.

The Brouwersdam was completed in 1971, and the area behind it is now a lake, The Grevelingenmeer. The loss of the tide and the gradual reduction in the salt content of the water led to the death of many plants and animals. Within a few years, though, nature adapted to the new conditions. After a sluice had been constructed in the Brouwers dam to allow water to flow between the Gevelingenmeer and the North Sea, the lake developed into a very valuable area for nature conservation, the fishing industry and recreational purposes. In 1978 the construction of this sluice had been finished. It consists of two venturi-shaped culverts with the bottom on M.S.L. - 11 m.



Position of Brouwersdam



Gates closed

The simultaneous closure of the two channels of the Brouwersdam in 1971 (vertical measures in metres). Sill is coloured black.

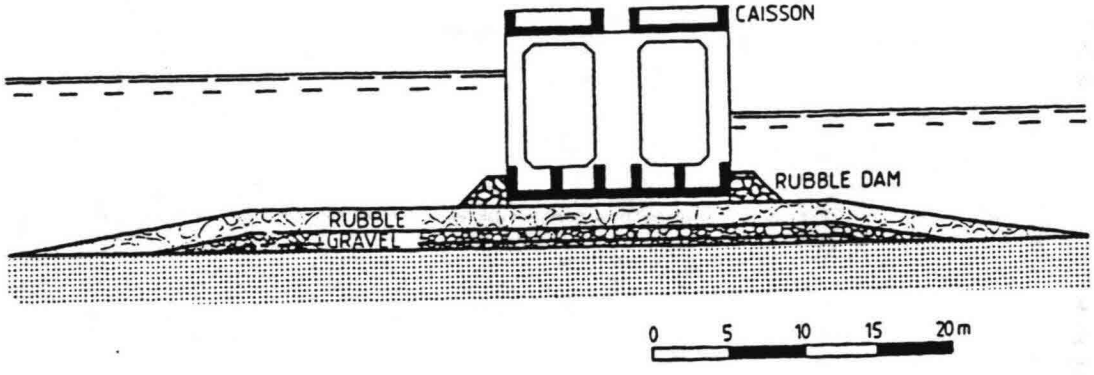


Fig. 21.21. Sluice caisson of Brouwersdam placed on a sill.

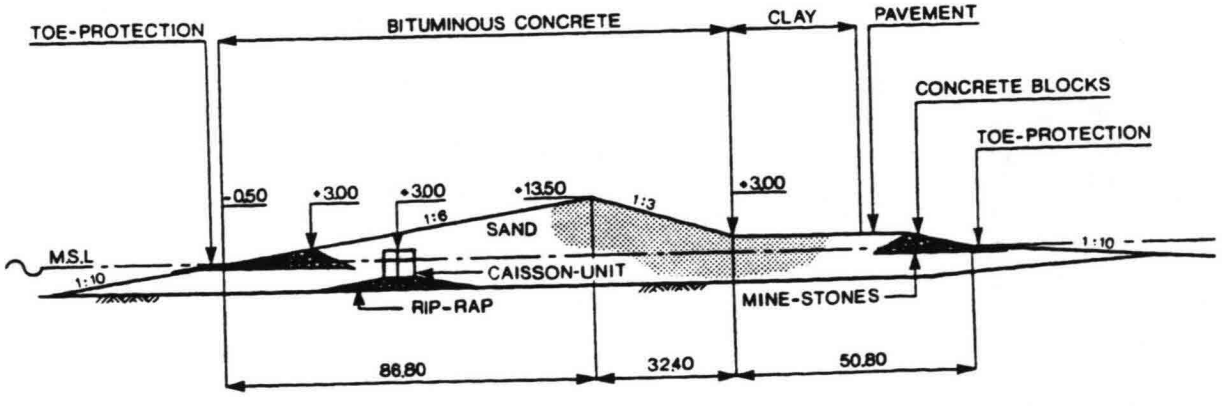


Fig. 21.22. Cross section of seadike Brouwersdam.

Seadike near Reide - Province of Groningen - The Netherlands

South of the spit of land of Reide (Dollard estuary, Province of Groningen, North Eastern part of the Netherlands) a new seadike has been constructed to replace the weak dike along the 'Johannes Kerkhoven' polder. The new dike is situated at the seaside of the old dike; between the two dikes there is room for a drainage canal for the East Groningen area.

The new seadike is mainly made from sand placed upon the tidal flats. The required sand was won in the mouth of the Dollard by the cutter suction dredger 'Utrecht'. By means of a 'spreader' pontoon barges were loaded. These barges transported the sand to the construction site, where the barge unloading dredger 'Oosterschelde' together with a booster station pumped the sand into the dike body.

Due to the low bearing capacity of the subsoil the work had to be executed in phases. Phase 1 and 2 were carried out in 1979. After the placing of the sandbody (hydraulic fill) during phase 1 vertical drains, made from synthetic material, were placed in order to accelerate the settlement. Depending on this consolidation phase 3 was started in the next year. Waiting time between phase 1 and 3 could be as much as 12 months. The total work was completed in 1981.

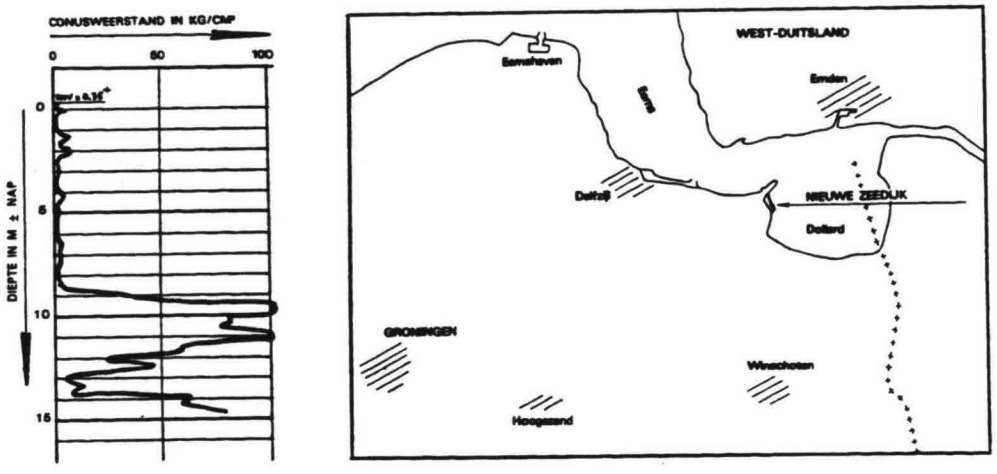


Fig. 21.23. Result of Dutch cone penetration test at the project site and the location of the new dike.

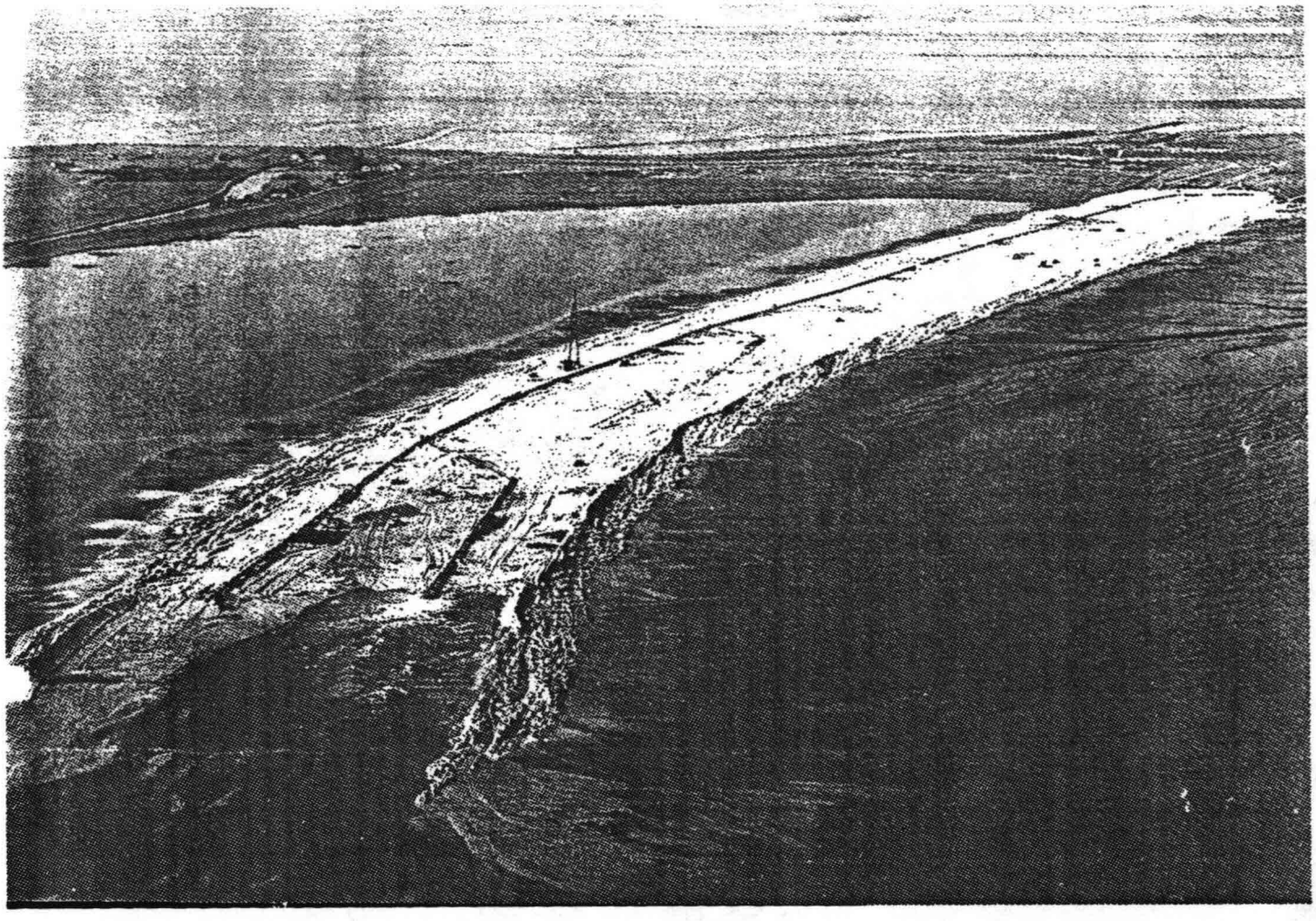
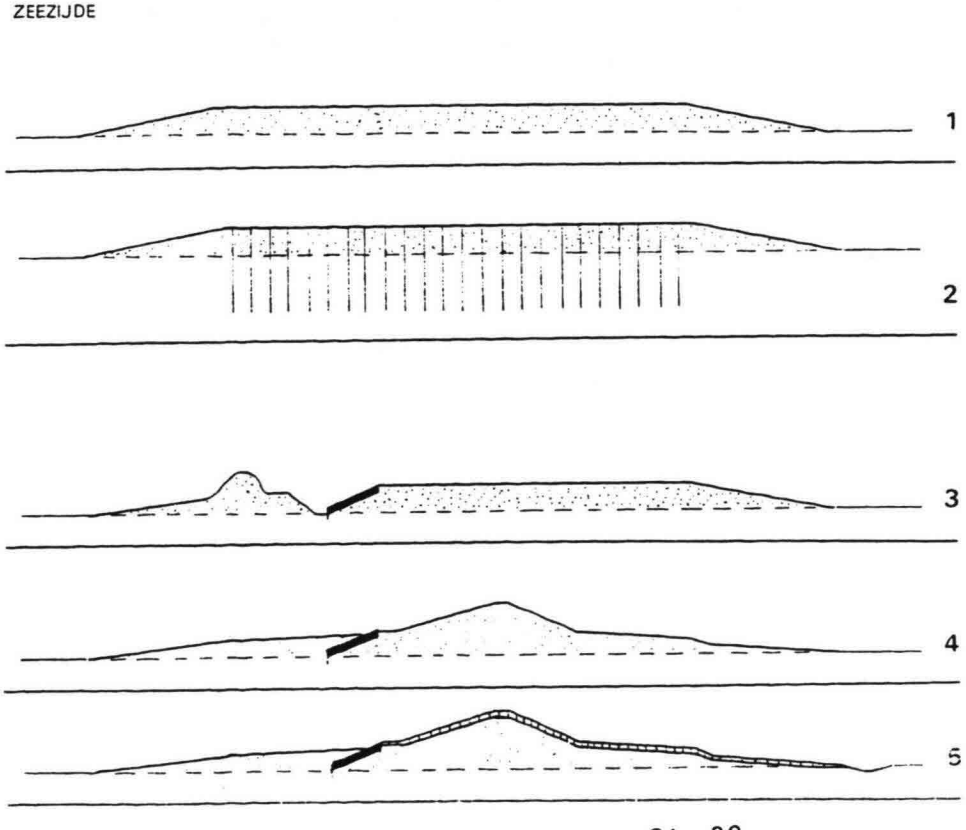


Fig. 21.24. Construction of new seadike near Reide - Dollard estuary in the North of The Netherlands. Phase 1+2: 1979, phase 3-5: 1980-81.



Execution phases:
Phase 1: placing of settlement beacons, reclamation of sandbody as a pancake 4 to 5 m thick.
Phase 2: vertical drainage by plastic drains down to the deep sandlayer to speed up consolidation of sublayer.
Phase 3: excavation of trench at seaside to construct foot of dike revetment.
Phase 4: shaping to profile by dry earth moving.
Phase 5: placing of 1 meter thick clay cover.

Westkapelle Seadike (Westkapelse Zeedijk)

This famous seadike was constructed in 1540 at a place where no dunes exist. During the second world war the dike was bombed by the British air force (October 1944) in order to flood the island of Walcheren and to flush out the German troops.

The dike was brought to Delta height in the period 1986-1988. For that purpose 1,400,000 m^3 of sand was dredged by the trailing section hopper dredger 'Geopotes XIV' at a sandbank some 1000 m from the coast. Then the sand was pumped into the dike body. the dike was raised at some places from 7 to 11½ m and at other places from +10 m to +12 N.A.P. The total length of this seadike, one of the heaviest ones in the Netherlands is 4 km. Total width: 130 - 160 m. Some 375 000 ton of stone was used for the protection of the sea side. On top of the dike there is a war memorial.

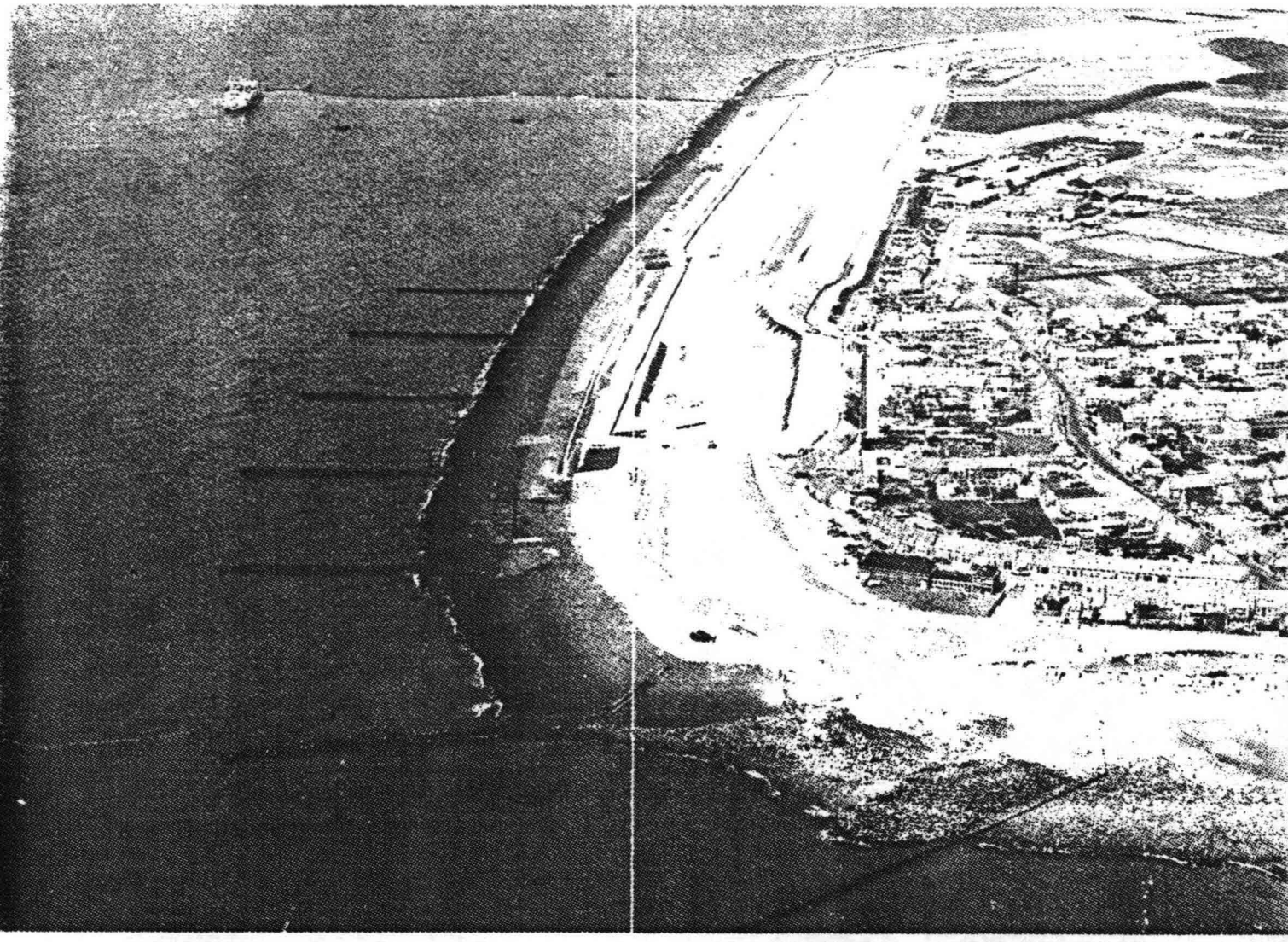
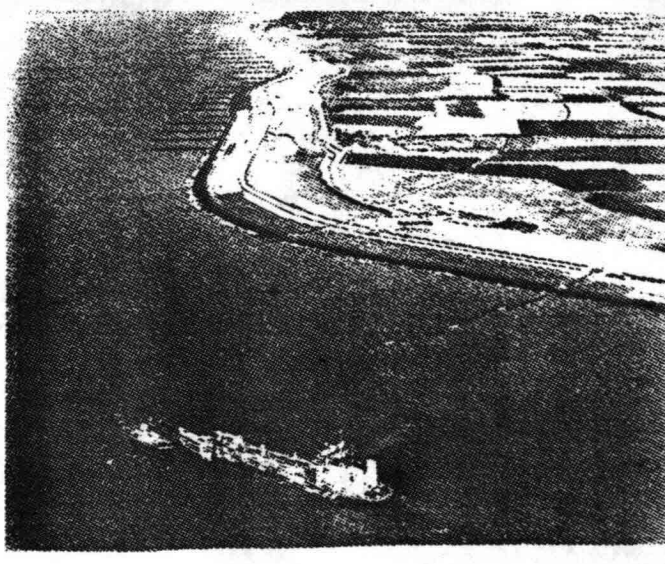
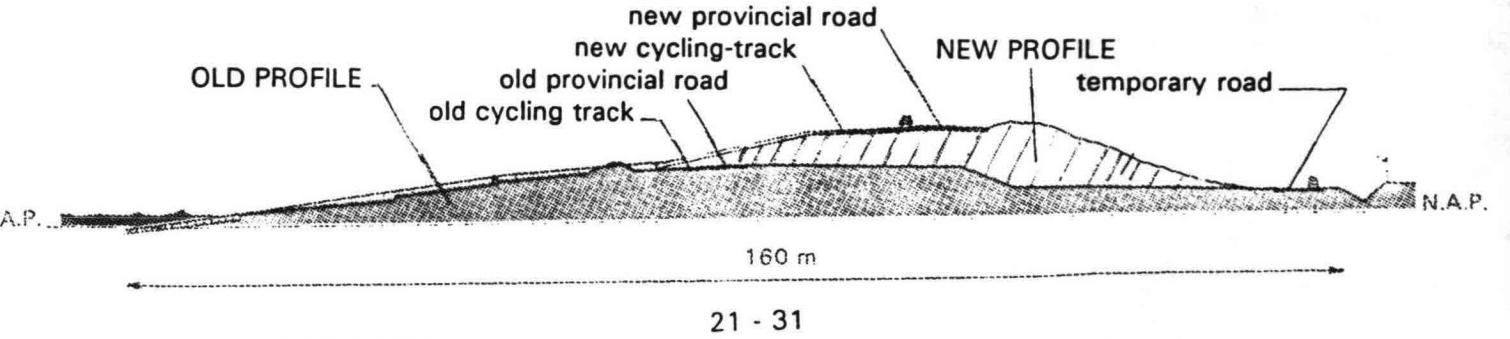


Fig. 21.25.
Heightening and widening of seadike at Westkapelle - Walcheren - S.W.part of The Netherlands (1986 - 1988).
Volume sand reclaimed: 1,400,000 m³
The trailing suction hopper dredger 'Geopotes 14' dredged the sand from the sandbank 'Bankje van Zoutelande', 1000 m in front of the coast and then pumps it ashore (reclamation work May- September 1986).





SCHAAL 1:1000

Biesbosch reservoirs.

The most economic way of storing drinking water in a densely populated flat area is by creating deep reservoirs surrounded by large earthfill dikes or dams. An example is the group of Biesbosch reservoirs with a capacity of 79 million cubic metres and made for storage of drinking water for Metropolitan Rotterdam and the provinces of Northern Brabant and Zeeland. The reservoirs are surrounded by 19 kilometres of high dikes (mainly constructed by hydraulic fill). A typical feature of these dikes is the thin flexible asphaltic membrane functioning as a watertight lining.

One of the 4 reservoirs is called 'The Gijster' with a 9 km. long surrounding dike. Most of the sand excavated in this reservoir (capacity of the Gijster: 32 million m^3) has been transported to the 'Moerdijk' to reclaim industrial areas (over 14 million m^3 sand). The enclosing dam was closed by the cutter suction dredger 'Linquenda III' by pumping sand from the reservoir (450,000 m^3). By doing so the dredger became "trapped" inside the dike. Although the dredger was of the demountable type, the contractor preferred to leave the dredger intact and to transport it by means of cylindrical rubber hoses, filled with compressed air, over the dike. (April 1979).

Slufter Project.

As onland storage of contaminated harbour deposits or the offshore dumping is no longer accepted, a large reservoir for storage of these deposits has been created offshore. This so-called Slufter Project located on the seaside of the Rotterdam harbour area, was constructed by excavating down to 28 m. below sea level. The outcoming sand was used to construct the surrounding dikes which rise to a level of 23 m. above sea level. Construction of the Slufter Project required earthworks (hydraulic fill) of 35 million m^3.

It became operational in 1988. It will be possible to store in this 260 ha site polluted dredged sludge, 150 million m^3 in total, up to the year 2000. Construction costs about 200 million guilders.

To keep the Rotterdam port area at a navigable depth, natural sedimentation has to be constantly removed by maintenance dredging. Major extensions to the port and considerably deepening of the access routes resulted in a great increase of maintenance dredging. The harbour silt coming down the rivers is so polluted that it may not be dumped at sea (10 million m^3 per year). It was not possible to find a location for a disposal area on the mainland, therefore one was designed in the form of a peninsula attached to the Maasvlakte. This plan is called the 'Slufter-plan'. It became operational in 1988. The sea-bed was excavated to a depth of -28 m below mean sea level within a ring-dike with a height of +23 m. It will be possible to store in this 260 ha site polluted dredged sludge, 150 million m^3 in total, up to the year 2000. This medium term solution costed about 200 million guilders.

Fig. 21.26. Slufter project.

22. Coastal structures

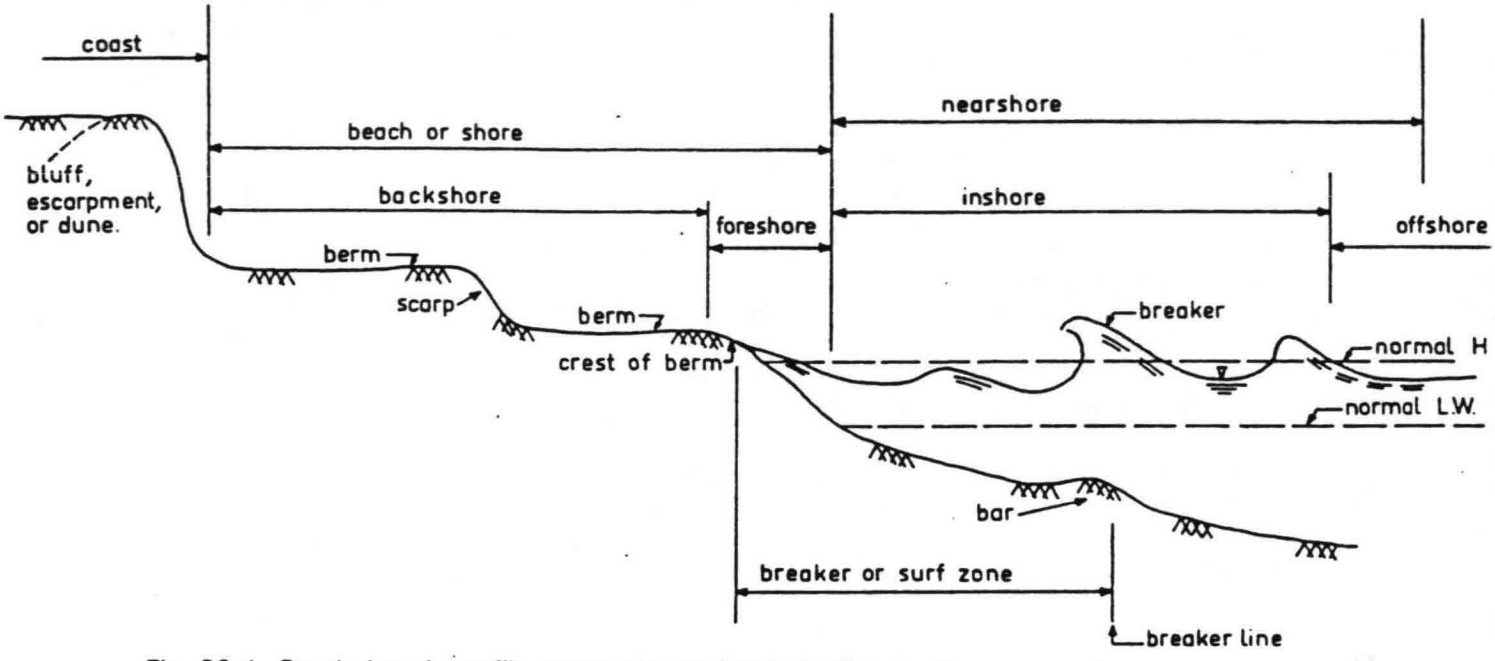
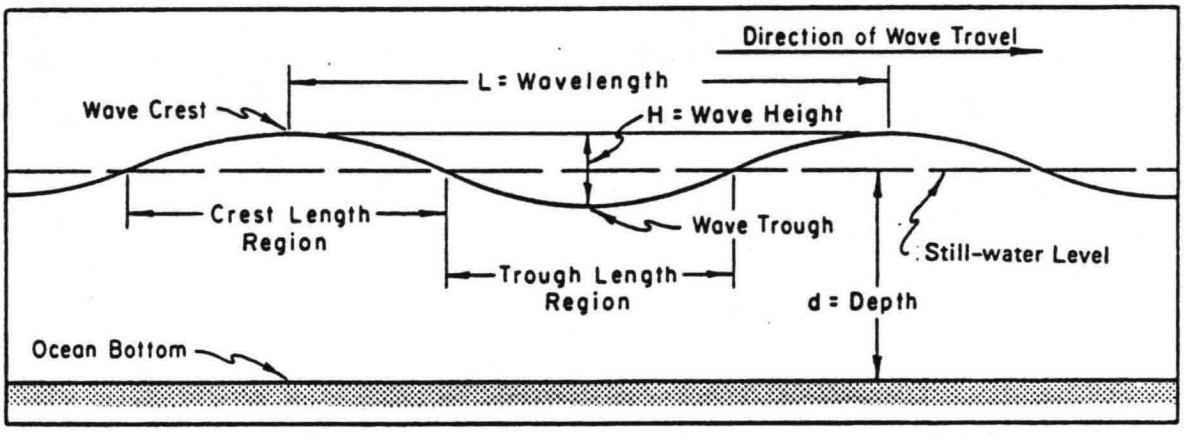
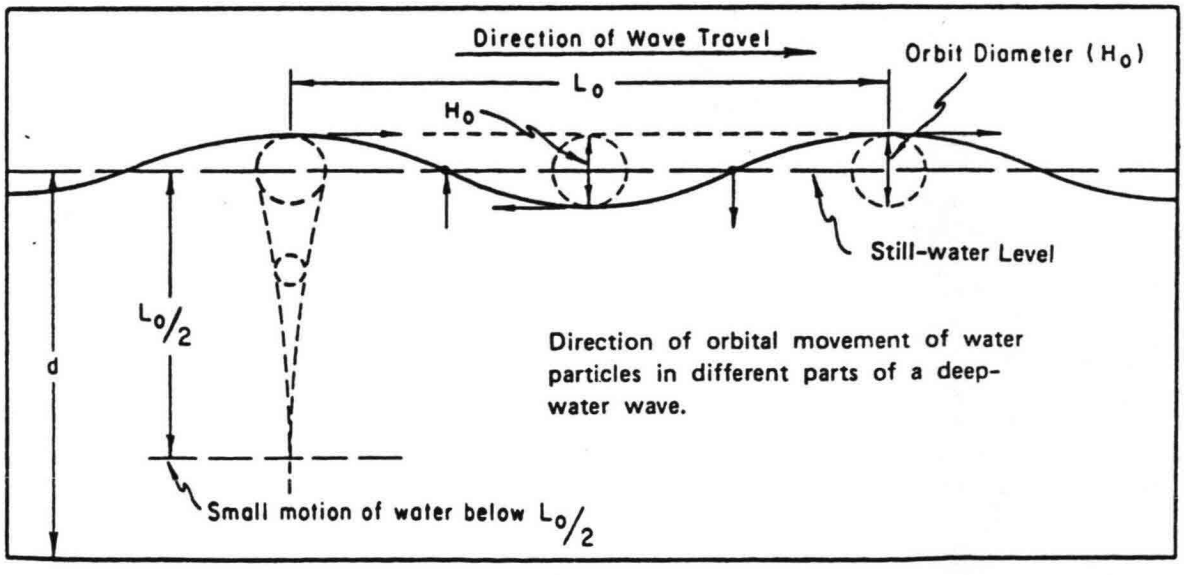


Fig. 22.1. Sandy beach profile nomenclature (scale is disturbed); Wave characteristics.





22.1. Breakwaters

The most obvious purpose of a breakwater is to provide protection against waves. The protection may be provided for an approach channel or even for a harbour itself. Thid type of protection is necessary in order to provide quieter water for ships to navigate and moor.

A breakwater can also serve to reduce the amount of dredging required in a harbour entrance. This can result from the cutting off of the littoral transport supply to the approach channel, or it can result from natural scouring action in an artificial narrowed channel.

At locations where little or no natural protection exists, breakwaters often serve as quay facilities as well. Such dual usage of breakwaters is economical in terms of harbour area but requires a different type of breakwater structure.

A fourth possible important purpose of a breakwater can be to guide the currents in the channel or along the coast.

The functions of breakwaters and harbour moles can be:

a. Protection against waves	(IJmuiden, Ashdod, Beirut)
b. Protection against shoaling	(IJmuiden, Abidjan, Maracaibo)
c. Provision of dock or quay facilities	(Assab, Takoradi, Saba, Antifer)
d. Guiding of currents	(Abidjan, N.breakwater Europoort).

Type of breakwaters :
Several types of breakwaters can be discerned, depending on their principle of operation and design. The most important types are:

a. **Rubble mound breakwaters**
A structure consisting of one or more layers of loose blocks of natural stone or concrete. The blocks can move with respect to each other and derive stability mainly from their weight with some additional help by interlocking effects. The structure is relatively porous, and absorbs therefore a greater part of the wave energy. The structure is flexible, not sensitive for uneven settlement. It remains functioning even when heavily damaged. Especially in deeper water rubble mound breakwaters require vast quantities of material.

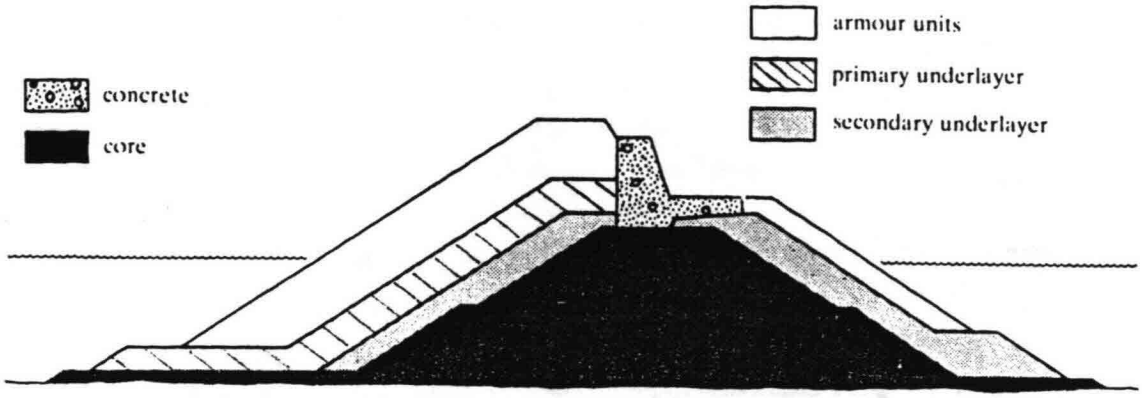
b. **Monolithic breakwaters**
A monolithic breakwater is a massive structure consisting of a small number of very large elements that are basically immovable with respect to each other. Such a monolithic breakwater can consist of concrete caissons, (with a vertical, sloping or porous front wall), cellular sheet piling, stacked block walls, etc. The most general appearance is a vertical front wall, therefore this type is often referred to as vertical wall breakwater. Wave energy is not absorbed but reflected. The structure is very sensitive to uneven settlement. Damage leads often to a complete destruction and loss of function.

c. **Composite breakwater**
Composite breakwaters consist of both, a rubble mound and a monolithic structure in one cross-section.

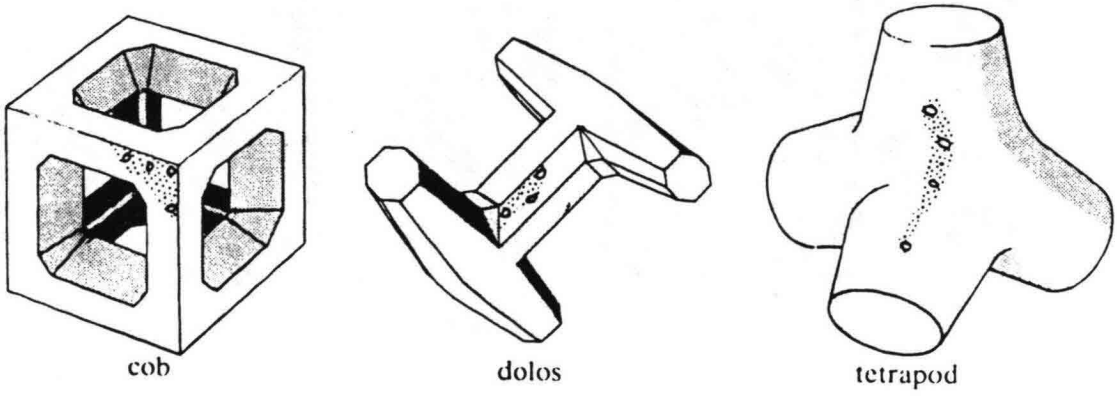
d. **Floating breakwaters**
A floating breakwater can be either rigid or flexible. In general this type of breakwater is cheap, quickly fabricated and thus well suited to provide temporary protection. The wave damping characteristics , however, are rather poor, especially in long waves.

e. **Hydraulic and pneumatic breakwaters**
Hydraulic and pneumatic breakwaters damp the wave action by discharging air or fluid from a submerged porous pipeline. The outflowing medium causes currents, which disturb the orbital movement in the waves and this initiates the breaking. This system of protection against waves is very energy intensive and not effective in long waves. The method is only feasible for temporary protection.

Breakwater Longest World. The world's longest breakwater is that which protects the Port of Galveston, Texas, USA. The granite South Breakwater is 10.85 km in length.

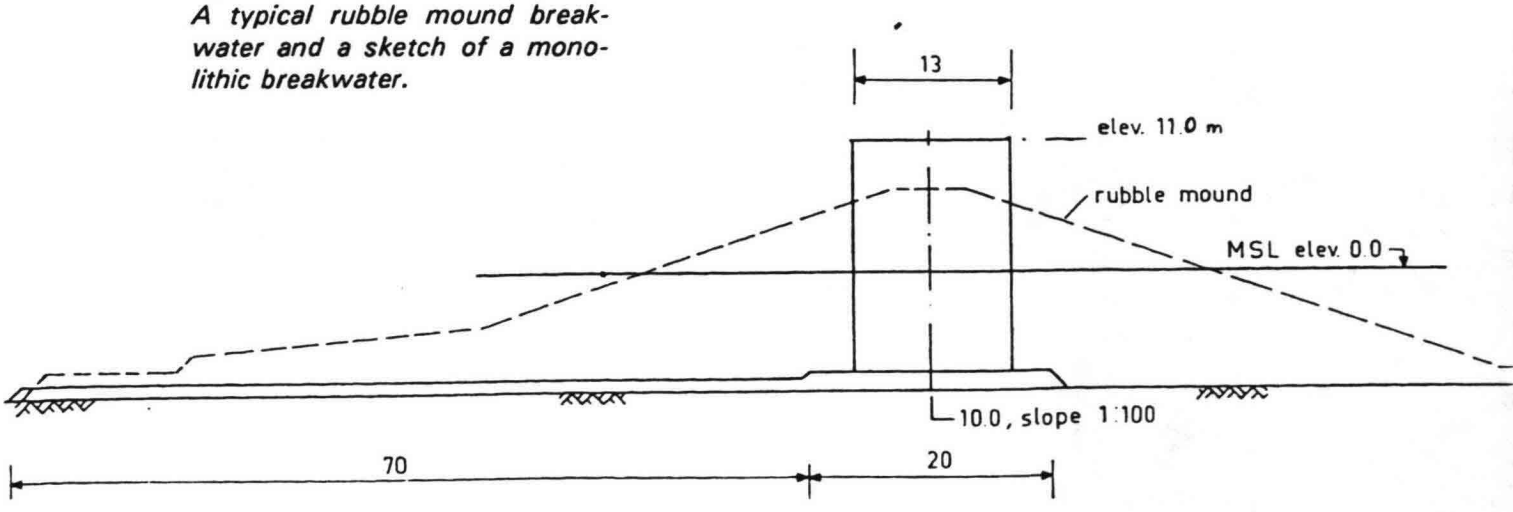


(a) Cross section



(b) Armour blocks

Fig. 22.2.
A typical rubble mound breakwater and a sketch of a monolithic breakwater.



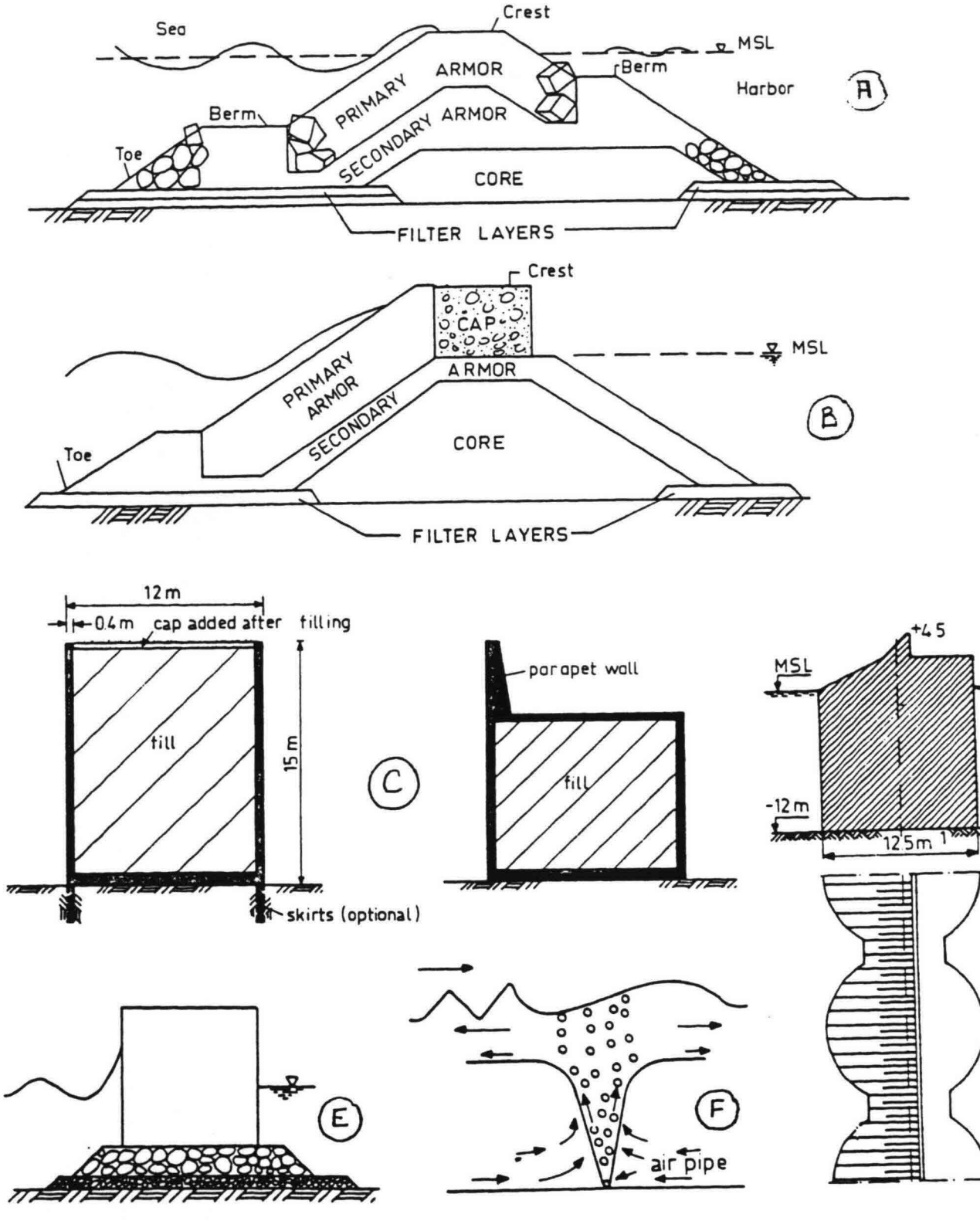


Fig. 22.3. Various types of breakwaters.

A. Overtopping rubble mound breakwater
B. Non-overtopping rubble mound breakwater
C. Monolithic breakwater - caisson type
D. Monolithic breakwater - Hanstholm breakwater
E. Composite breakwater
F. Air bubble curtain.

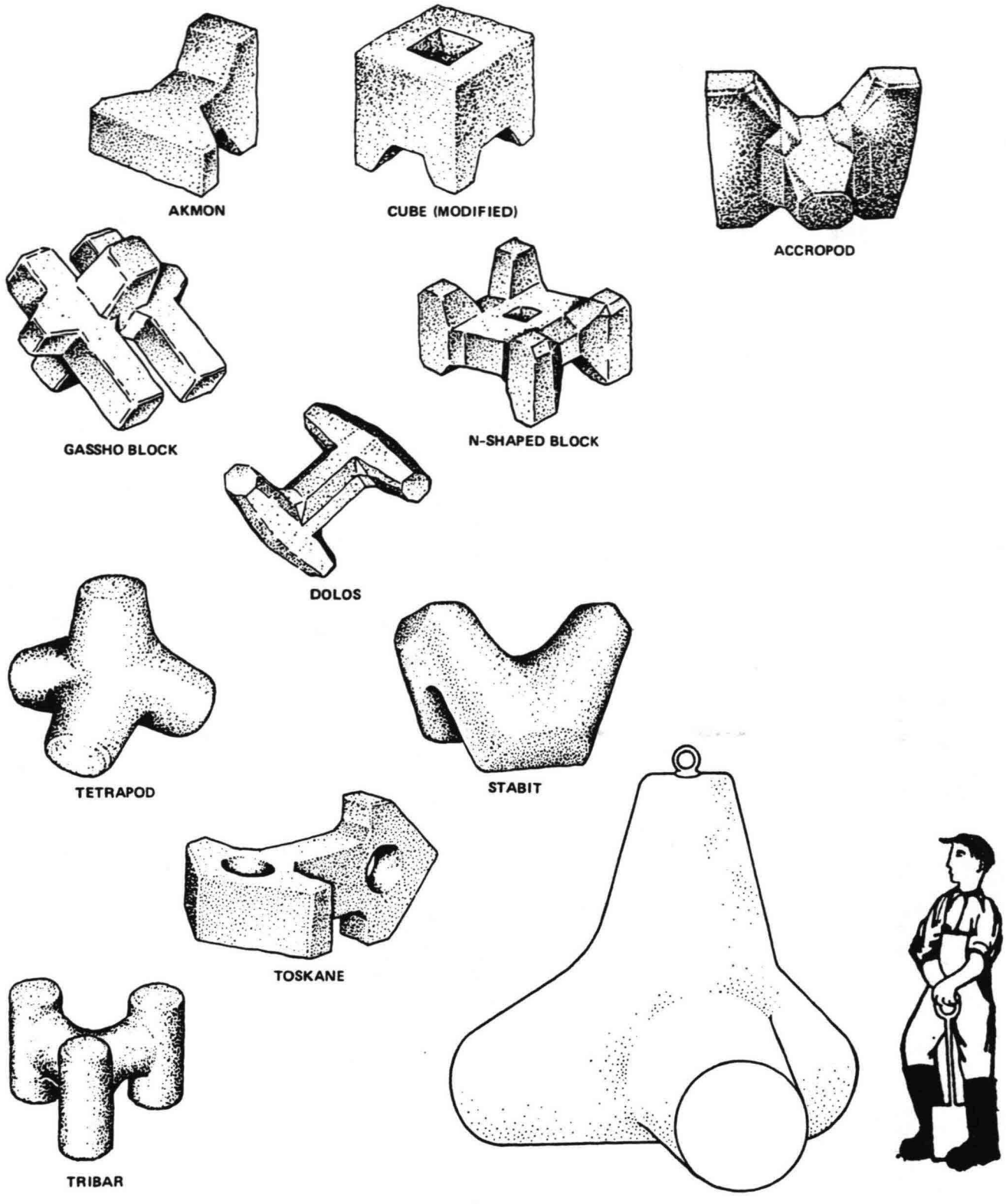
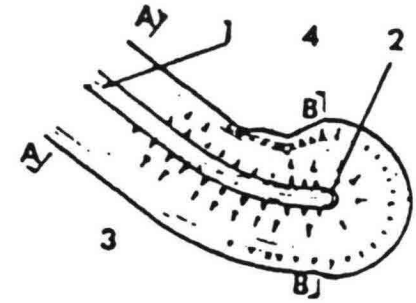
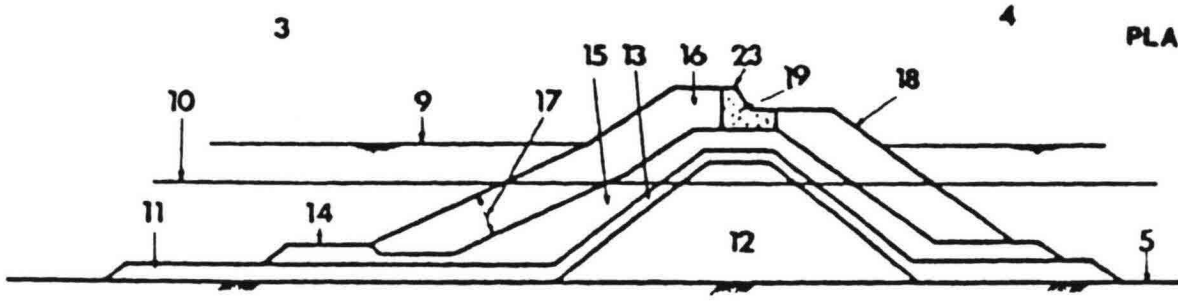


Fig. 22.4. Armour Unit Shape.
Since 1950 there has been a proliferation in the development of concrete armour shapes. Some examples are shown:

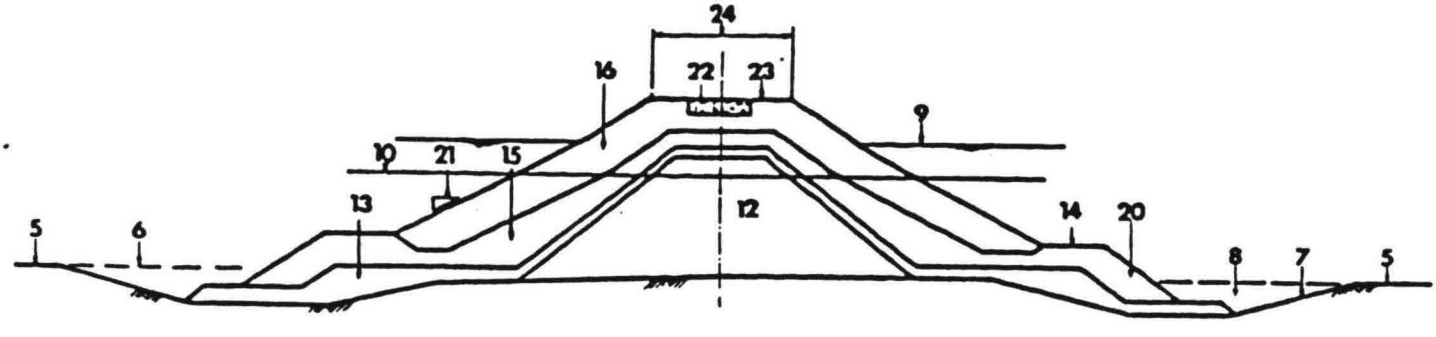
tetrapod	France	1950	akmon	Netherlands	1962
tribar	USA	1958	dolos	South Africa	1963
modified cube	USA	1959	toskane	South Africa	1966
N-shaped block	Japan	1960	Gassho block	Japan	1967
stabit	England	1961	accropod	France	1979

not shown: bipod - Netherlands; cob - England; dom - Mexico; hexaleg - Japan; igloo - Japan; sta-bar - USA; svee block - Norway; stabilopod - Romania; robloc - Netherlands; antifer cube - France.





SECTION A-A THROUGH BREAKWATER



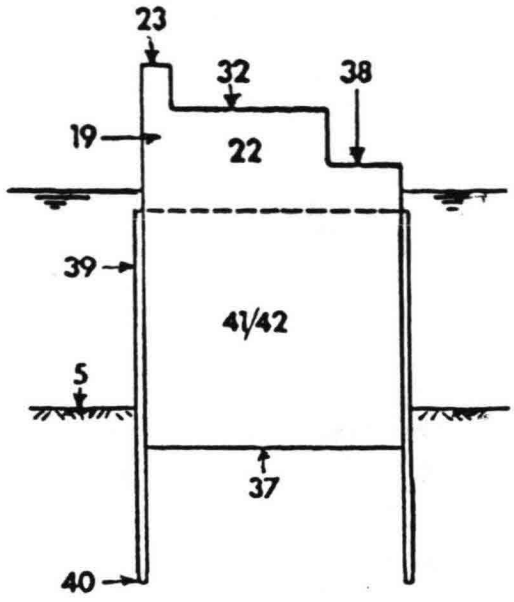
SECTION B-B THROUGH BREAKWATER

Fig. 22.5. Rubble mound breakwater.

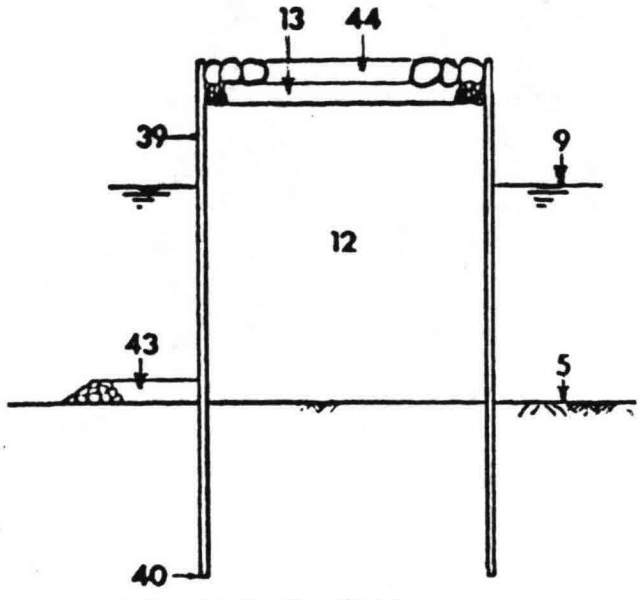
1. breakwater trunk
2. breakwater roundhead
3. seaward
4. harbour-side
5. seabed
6. original seabed
7. dredged slope
8. excavated trench
9. water level
10. datum
11. scour apron (bottom protection)
12. core quarry run
13. filter layer
14. berm
15. secondary armour
16. primary armour
17. layer thickness
18. rear face
19. concrete wave wall
20. toe bund
21. slope
22. concrete cap
23. crest
24. crest width

Fig. 22.6.
Breakwater types.

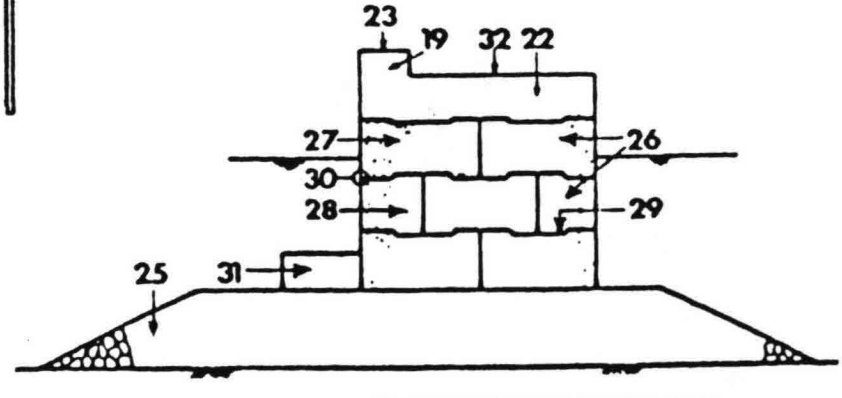
5. seabed
9. water level
12. core
13. filter layer
19. concrete wave wall
22. concrete gap
23. crest
25. rock mound
27. header blocks
28. stretcher block
29. key
30. sausage joint
31. blockwork toe
32. road
32. road
33. gap between blocks
34. chamfered edges
35. porous synthetic fibre
37. injected grout
38. pedestrian walkway
39. steel sheetpiling
40. pile toe
41. tremie concrete
42. colloidal concrete
43. anti-scour toe armour (toe protection)
44. rock armour



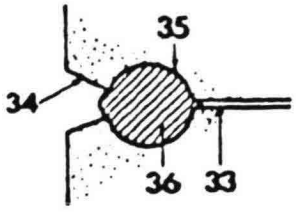
MASS CONCRETE BREAKWATER



ROCK FILLED PILED CELLS



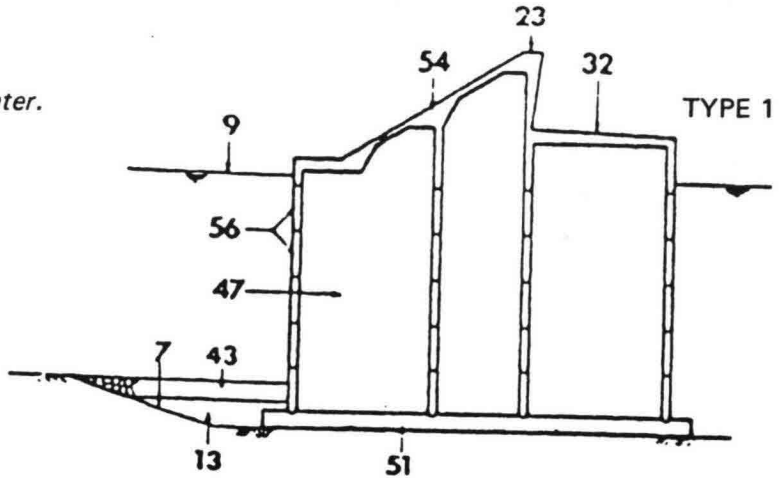
BLOCKWORK BREAKWATER

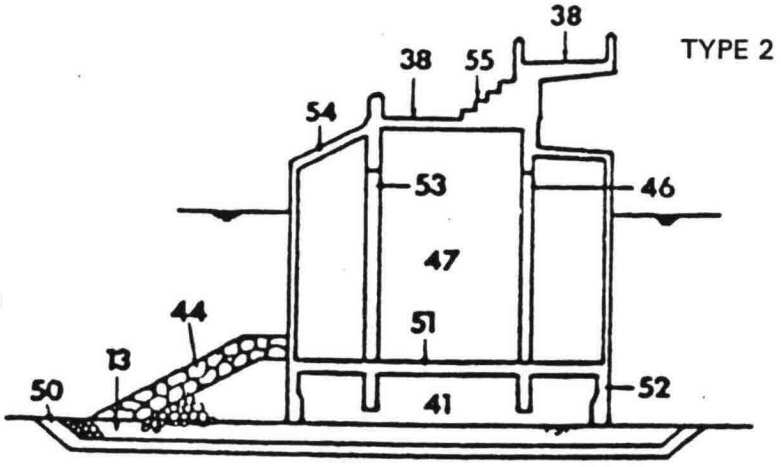


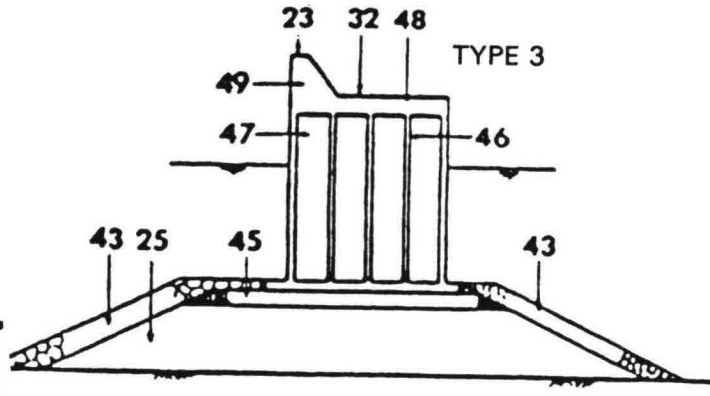
DETAIL

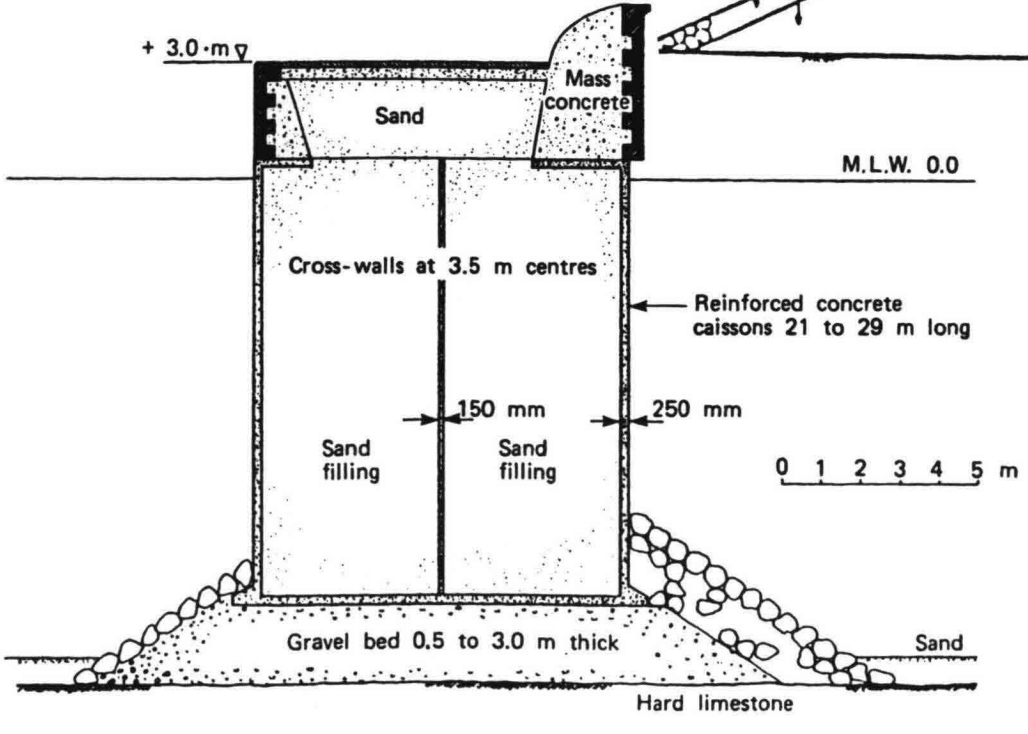
Fig. 22.7. Caisson breakwater.

7. dredged slope
9. water level
13. filter layer
23. crest
25. rock mound
30. sausage joint
31. blockwork toe
32. road
38. pedestrian walkway
41. tremie concrete
43. toe protection
44. rock armour
45. granular bedding
46. concrete caisson
47. sand filling
48. concrete slab
49. concrete wave wall
50. gravel infilling
51. base slab
52. base ring
53. internal diaphragm
54. concrete sloping wave
55. stepped wave wall
56. precast concrete segments.









Caisson breakwater at Helsingborg, Sweden.

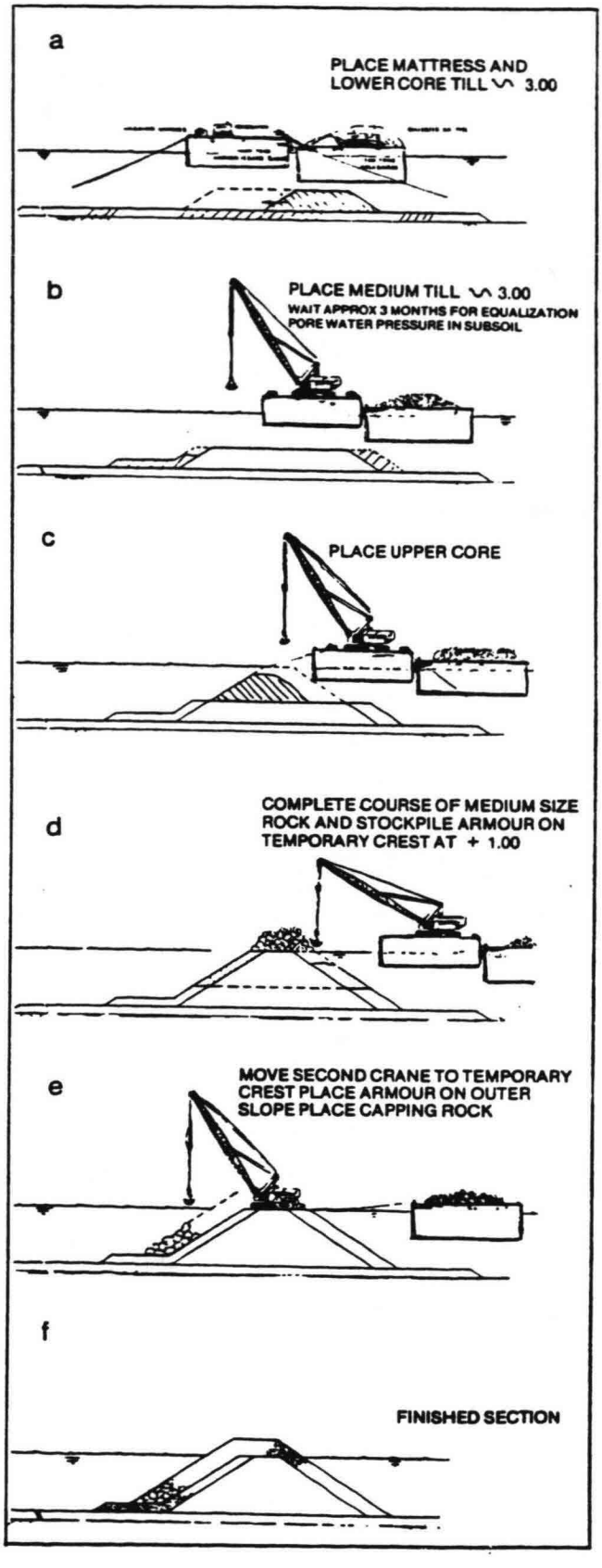
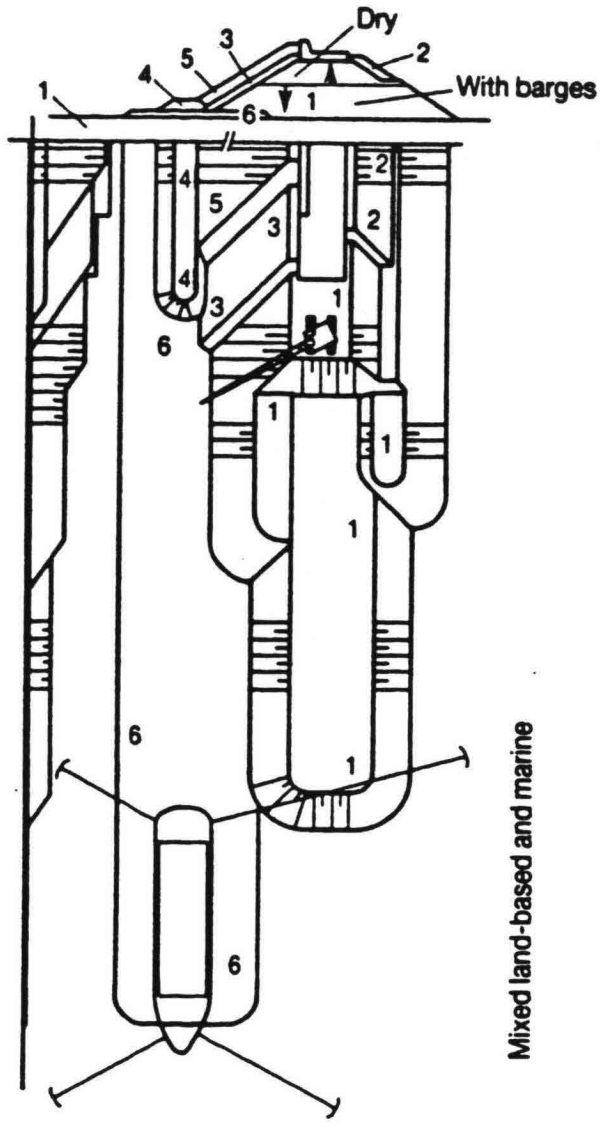
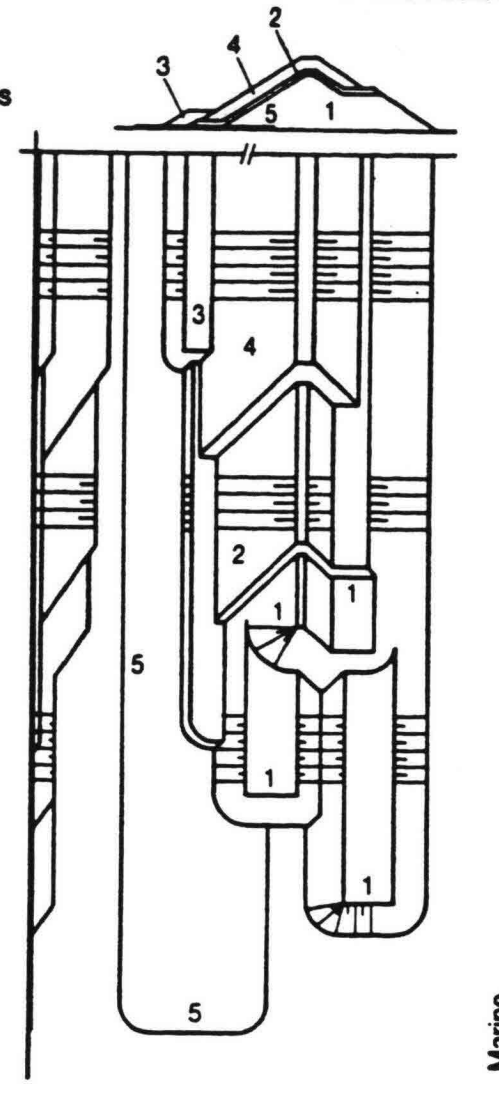


Fig. 22.8.
Breakwater construction method. Deep-seaport of Laem Chabang, Thailand (1968 - 72).

Most significant characteristics of land-based and marine execution





Transport

- Congestion problems on long breakwaters;
- Mostly standard equipment;
- Crest dimensions and elevation based on logistics.

Breakwater site

- Handling only one or two categories at a time;
- Crane reach often limitation for length of slope;
- Deep water poses problems due to long slopes;
- Delays due to heavy seas causing spray.

Transport

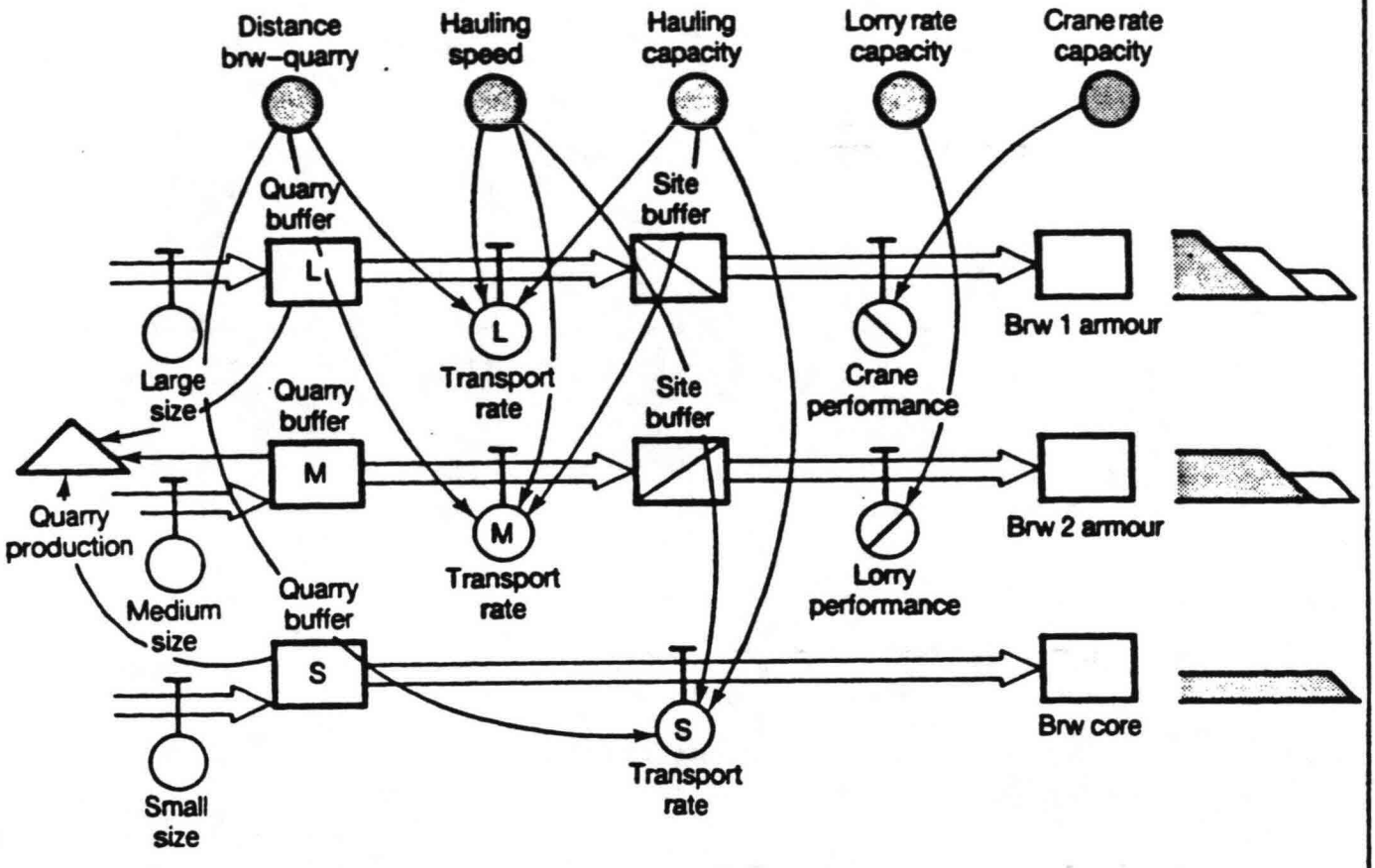
- Special floating equipment seldom locally available;
- Special terminal facilities required;
- Working phases more or less independent of each other.

Breakwater site

- Long work front;
- Downtime due to motions of barges in waves;
- Hydraulic requirements determine cross-section so minimal dimensions;
- Unit cost for marine execution can be three times the cost of load-based equipment

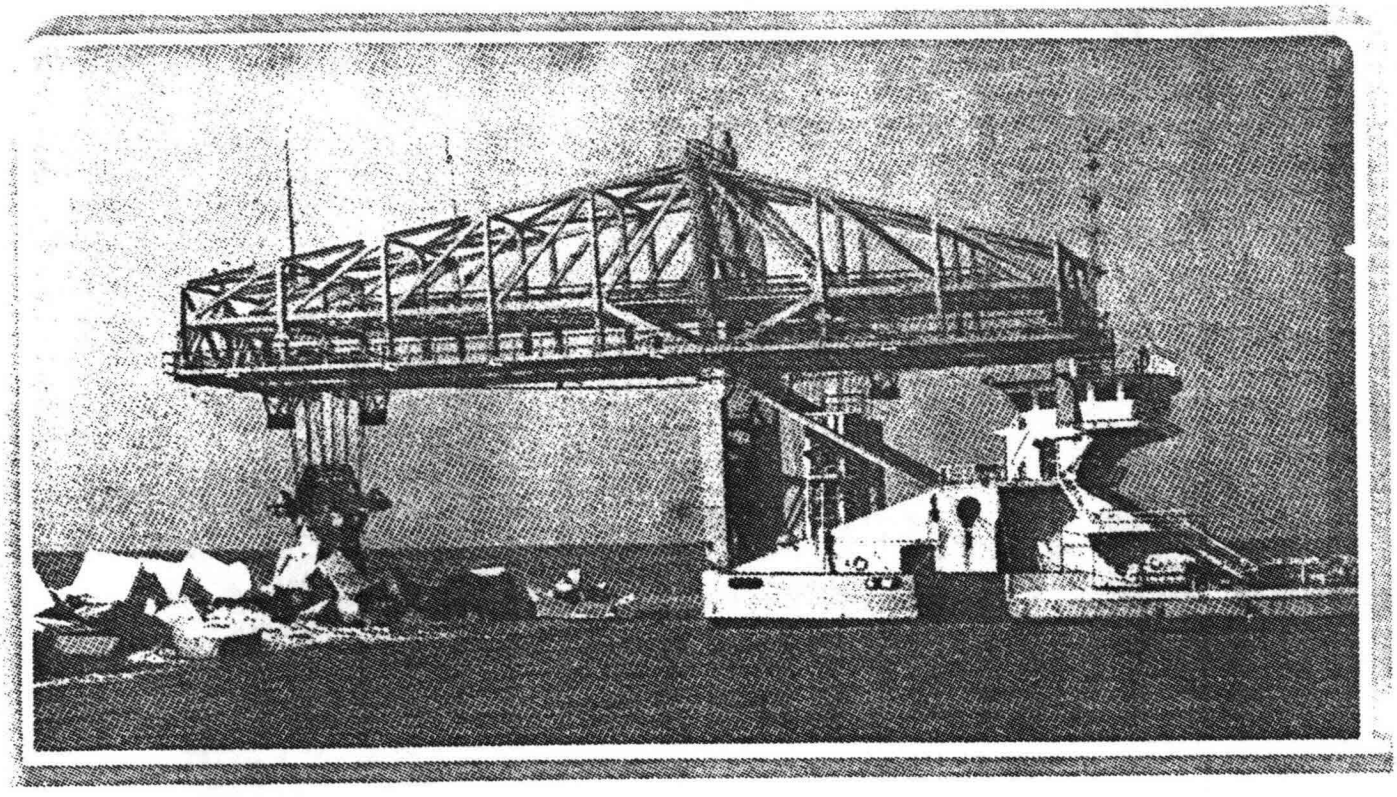
Example of breakwater construction logistic simulation

Simplified simulation of a logistic system for the breakwater construction scheme shown in Figure , with three rock size categories, a dedicated quarry at 25 km and buffer stocks at both the quarry and the site.



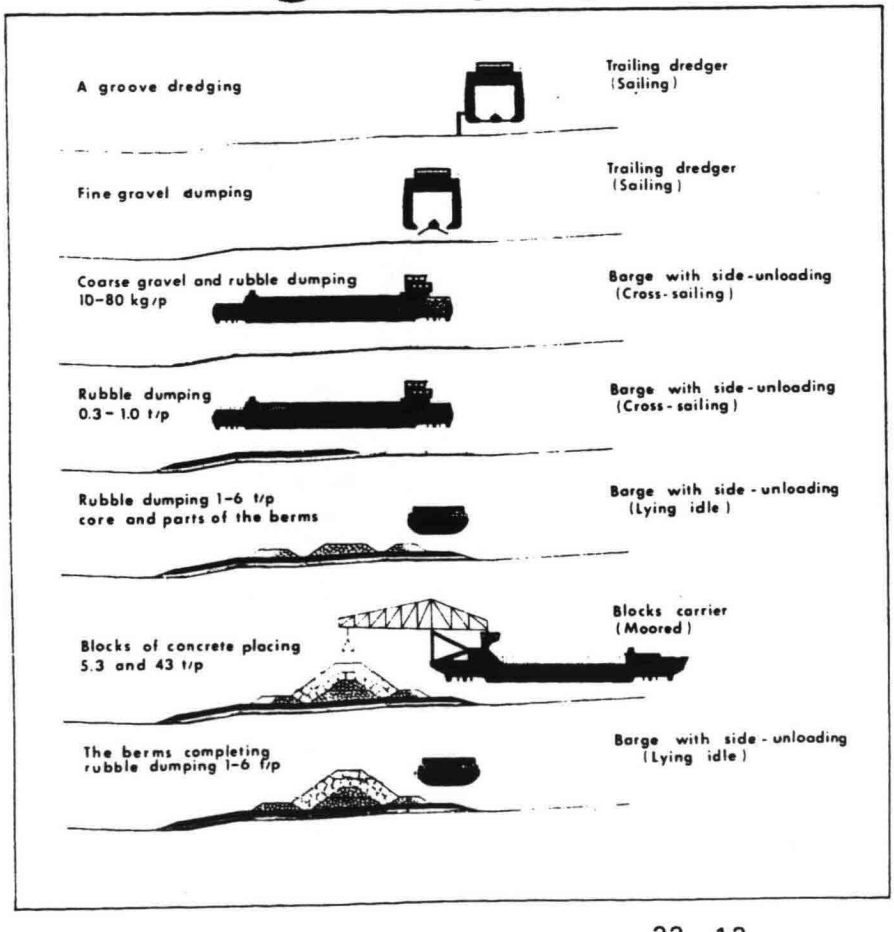
Simulation diagram for breakwater and quarry operations

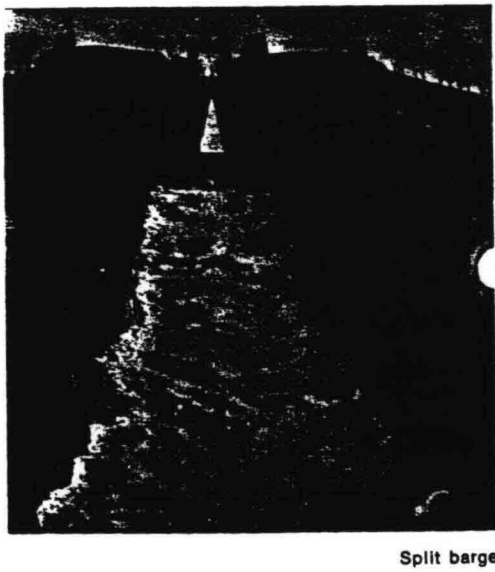
Total blast volume	=	100% *if one of site buffers* ≤ 0, *otherwise* 0%
Large-size production	=	0.2 * *total blast volume*
Medium-size production	=	0.3 * *total blast volume*
Small-size production	=	0.5 * *total blast volume*
Transport rates	=	*hauling speed/distance quarry to brw to quarry * hauling capacity, or* 0 *if side buffer* ≥ 100%
Quarry buffer	=	*quarry buffer* + Δt * (production-transport)
Lorry performance	=	100% lorry rate capacity, or 0 if site buffer medium ≤ 0 or brw 2 armour/brw core ≥ 0.9
Site buffer medium	=	site buffer medium + Δt* (transport–lorry performance)
Crane performance	=	100% of crane rate capacity, or 0 if site buffer large ≤ 0 or brw 1 armour/brw 2 armour ≥ 0.9
Site buffer large	=	site buffer large + Δt* (transport-crane performance)
Brw 1 armour	=	brw 1 armour + Δt* (crane performance)
Brw 2 armour	=	brw 2 armour + Δt* (lorry performance)
Brw core	=	brw core + Δt* (transport rate small)



Blockdumping vessels

Execution with existing equipment

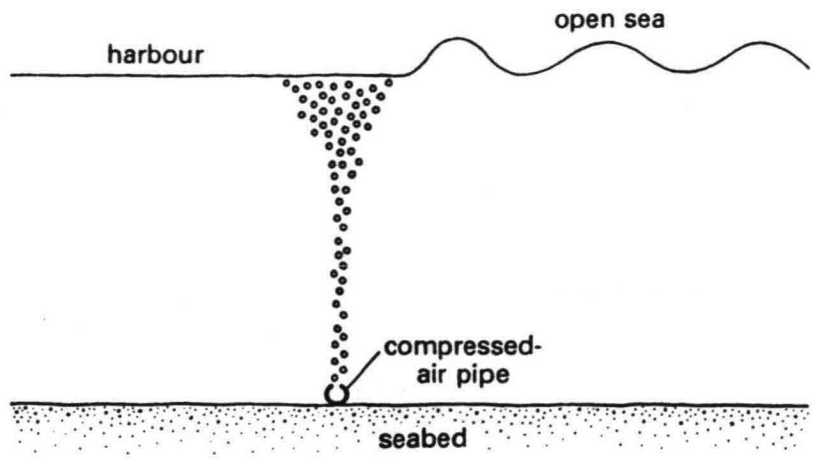




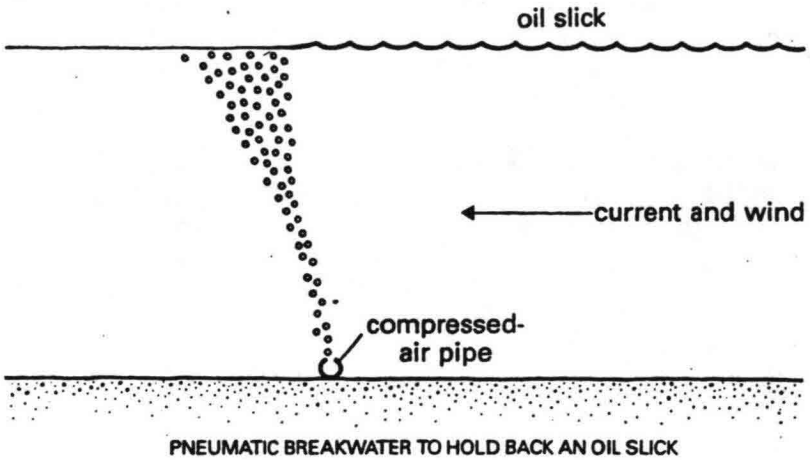
Split barge

Fig. 22.

Phases of building up breakwater



PNEUMATIC BREAKWATER AT HARBOUR ENTRANCE



PNEUMATIC BREAKWATER TO HOLD BACK AN OIL SLICK

Bubble barriers. (R. H. Waring, *Pneumatic Handbook*, Trade and Technical Press, 1982.)

22.2. Shore protection

a. **Groins (groynes)**
Groins are a series of small breakwaters (or jetties) spaced on relatively short intervals along a coast. They tend to stabilize the entire coast along which they are built by keeping the coastal sand trapped between adjacent groins. As such, they can be used to defend an eroding coast. The purpose of groins is to reduce the longshore sediment transport along a coast. **Impermeable groynes** are constructed in concrete or rock. **Permeable groynes** are constructed of wooden piles driven into the beach, with wooden planks attached between the piles.

b. **Detached breakwater (offshore breakwater)**
A segmented breakwater parallel to the coast can be used to stimulate a tombolo development. Such breakwaters reduce the longshore transport capacity. Expensive (difficult) to built, especially near the breaker line.

c. **Sea walls**
Impermeable sea wall on the beach parallel to the coast. At first sight, sea walls appear to offer a sure means of defence and protection. They are very common. They are, however, far from satisfactorily and should only be used when all other measures have been considered and discounted. Vertical sea walls will reflect virtually all wave energy. This may result in doubling the wave height and causing severe erosive action at the sea bed. Consequently, the wall foundation will be quickly undermined unless very substantial toe protection is provided.

Modern design of sea wall have a sloping face to reduce reflection and a curved wall at the top of the sea wall to deflect waves downwards, and thus dissipate reflected wave energy by turbulence.

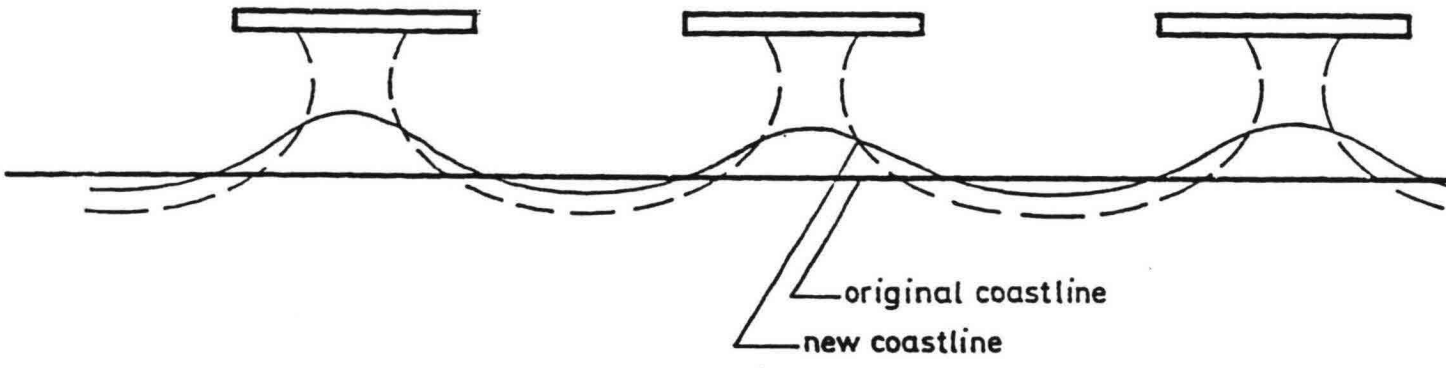


Fig. 22.9. Tombolo formation behind offshore breakwaters.

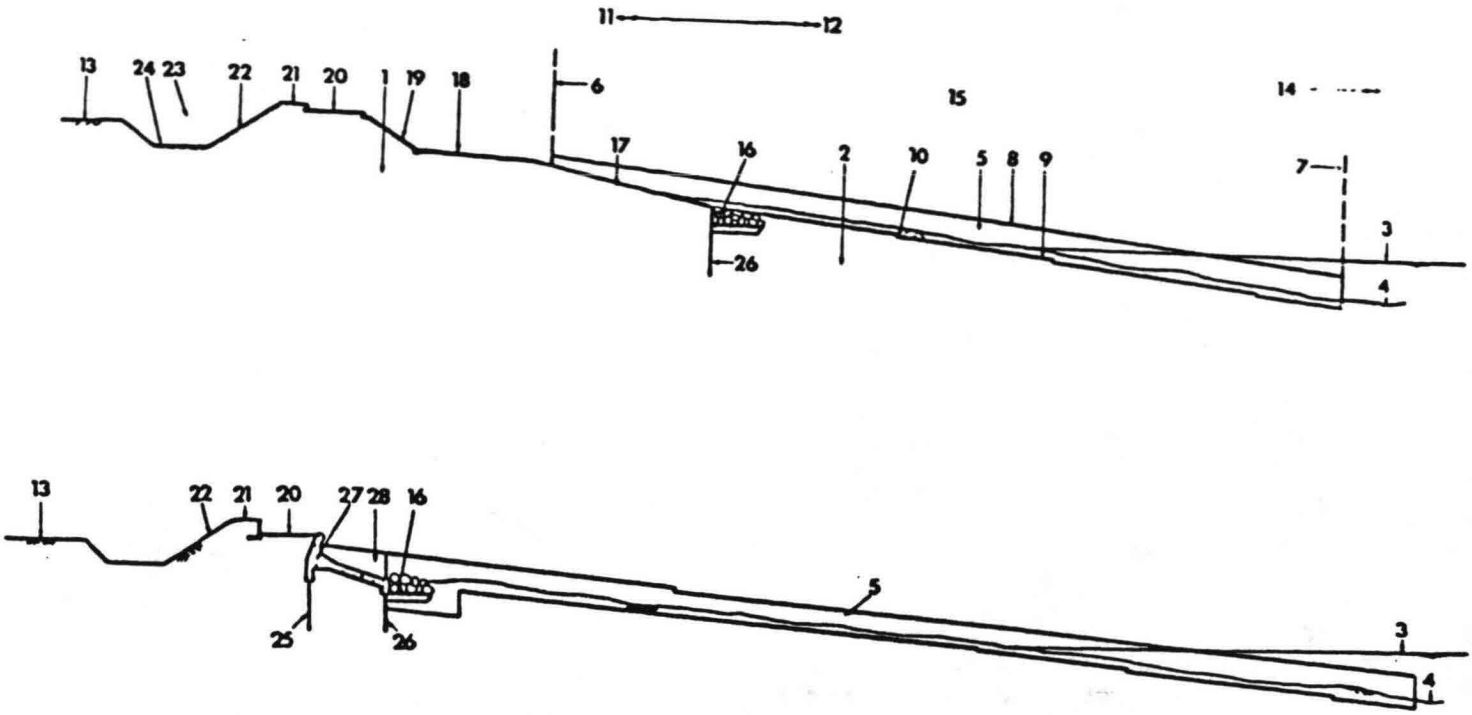


Fig. 22.10. Shore protection.

1. revetment
2. beach
3. water level
4. seabed
5. groyne (in US: groin)
6. landward limit of groyne
7. seaward limit of groyne
8. top of groyne
9. bottom groyne
10. step
11. landward
12. seaward
13. ground level
14. offshore
15. foreshore
16. toe protection
17. seaward slope (lower)
18. berm
19. seaward slope (upper)
20. road
21. crest
22. rear face
23. drainage ditch (channel)
24. channel invert
25. bearing pile
26. steel sheetpiling
27. concrete wave wall
28. groyne end.

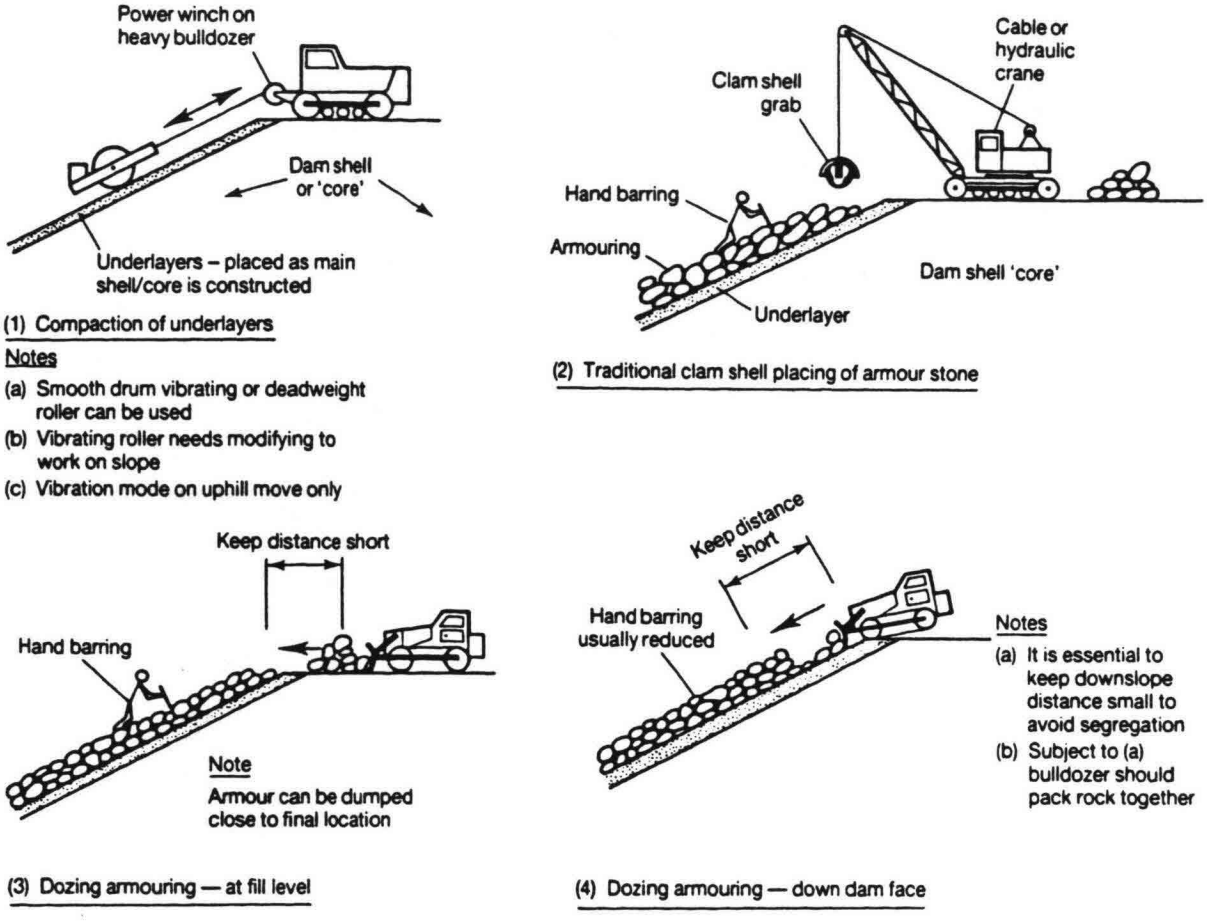


Fig. 22.11. Placing armour stone on dam faces.

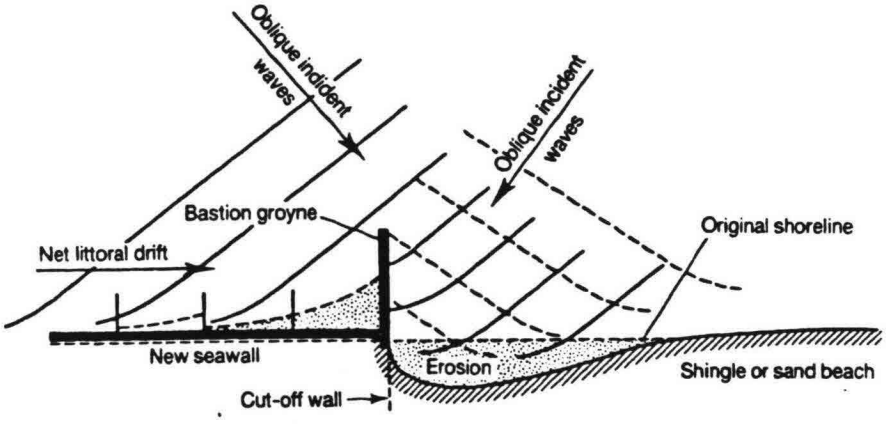


Fig. 22.12. Erosion at sewall termination.

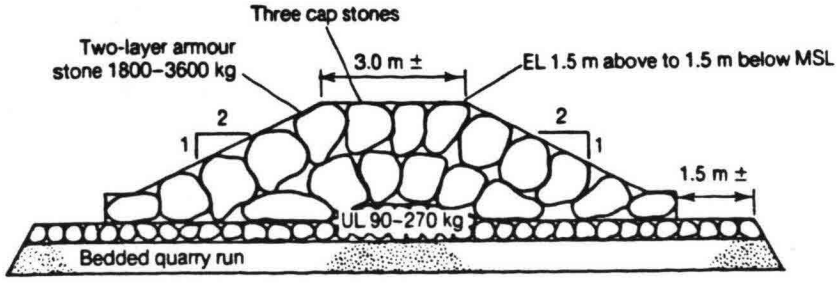


Fig. 22.13. Example of groyne cross-section from Atlantic Coast, North Carolina, USA. (this groyne is exposed to hurricanes).

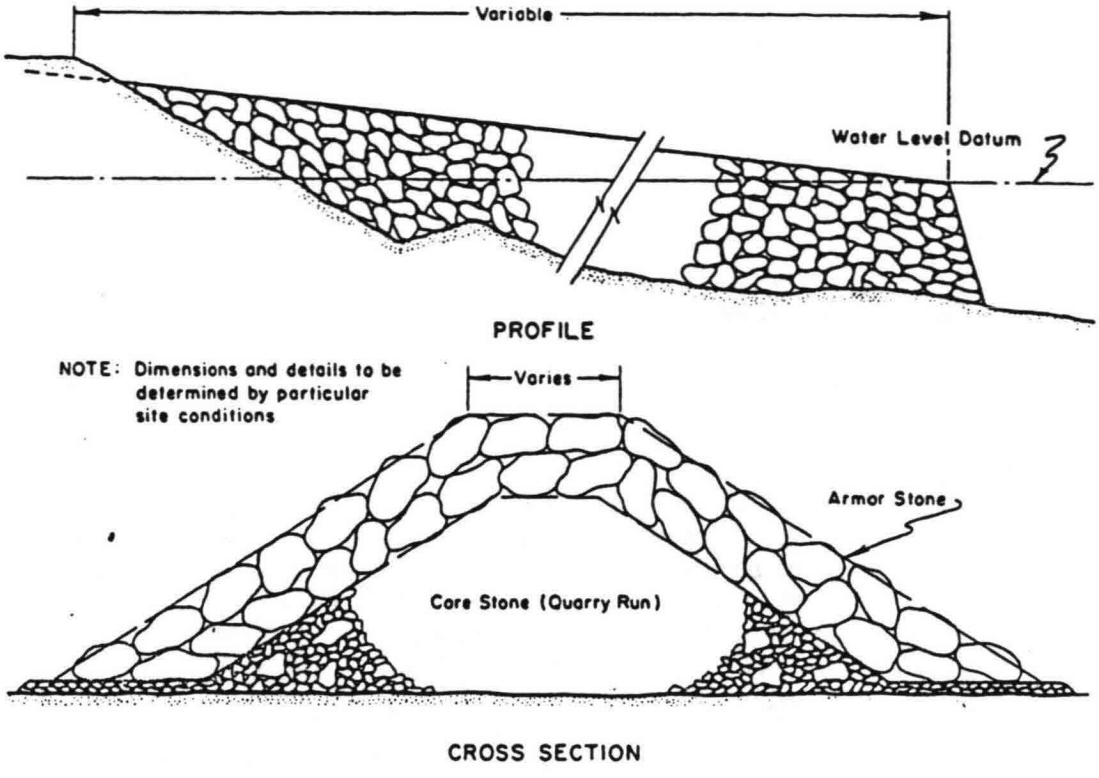


Fig. 22.14. Rubble-mound groin.

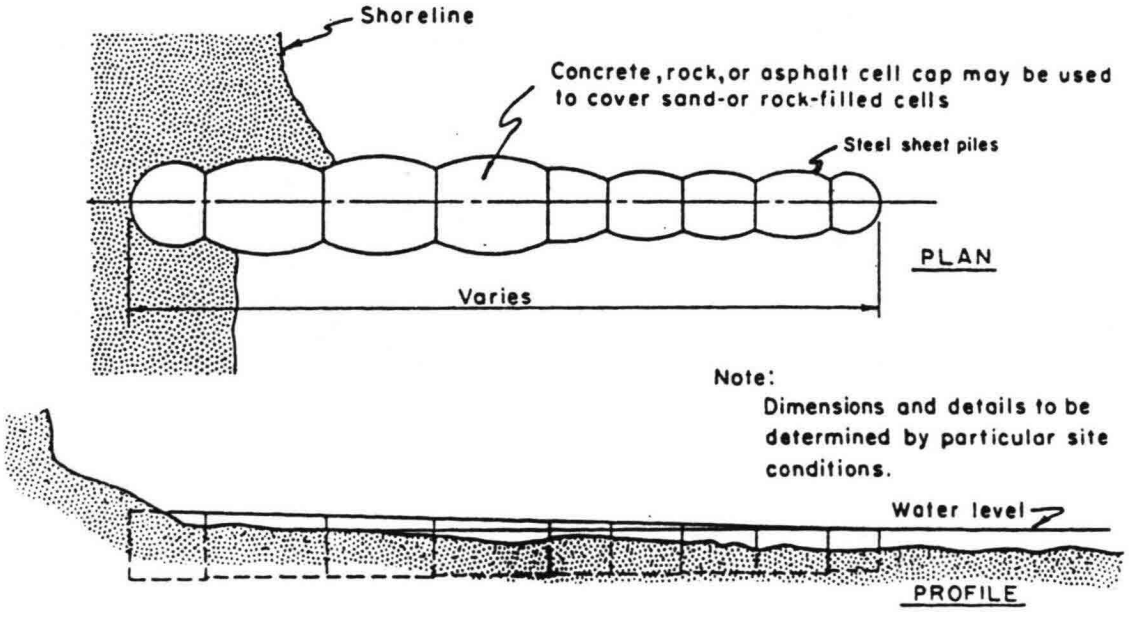
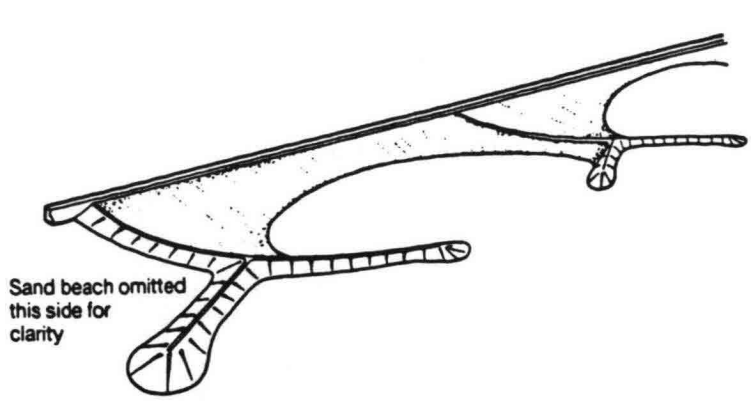


Fig. 22.15. Cellular-steel sheetpile groin.



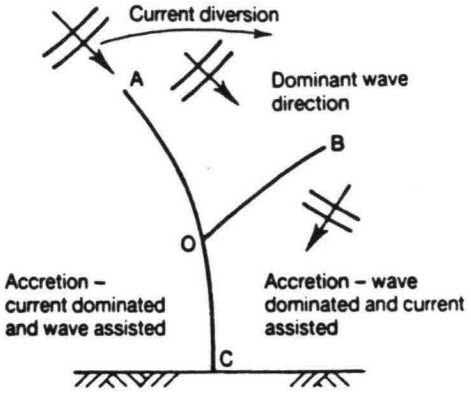


Fig. 22.16.
Fishtailed breakwaters retaining sand beach — Basic geometry of the fishtailed breakwater

Fishtailed breakwater (artificial headland)

The fundamental difference between a groyne and an artificial headland is that the latter is a more massive structure designed to eliminate problems of downdrift erosion and promote the formation of beaches. While these structures may take a number of different forms, their geometry is such that as with the offshore breakwater, wave diffraction is used to assist in holding the beach in the less of the structure. The fishtailed breakwater is a particular development of this concept, largely due to Dr P.C. Barber in the U.K.

The concept of the fishtailed breakwater is to combine the beneficial effects of the groyne, offshore breakwater and tombolo and reduce the undesirable influences of the separate structures. The basic plan shape of a fishtail breakwater is shown in fig. 22.16. The breakwater arms OA and OB act as wave energy dissipators while the arm AOC provides the interceptor to alongshore drift. Thus the updrift beach is formed by normal accretion processes associated with a groyne while the downdrift beach is formed by those associated with an offshore breakwater.

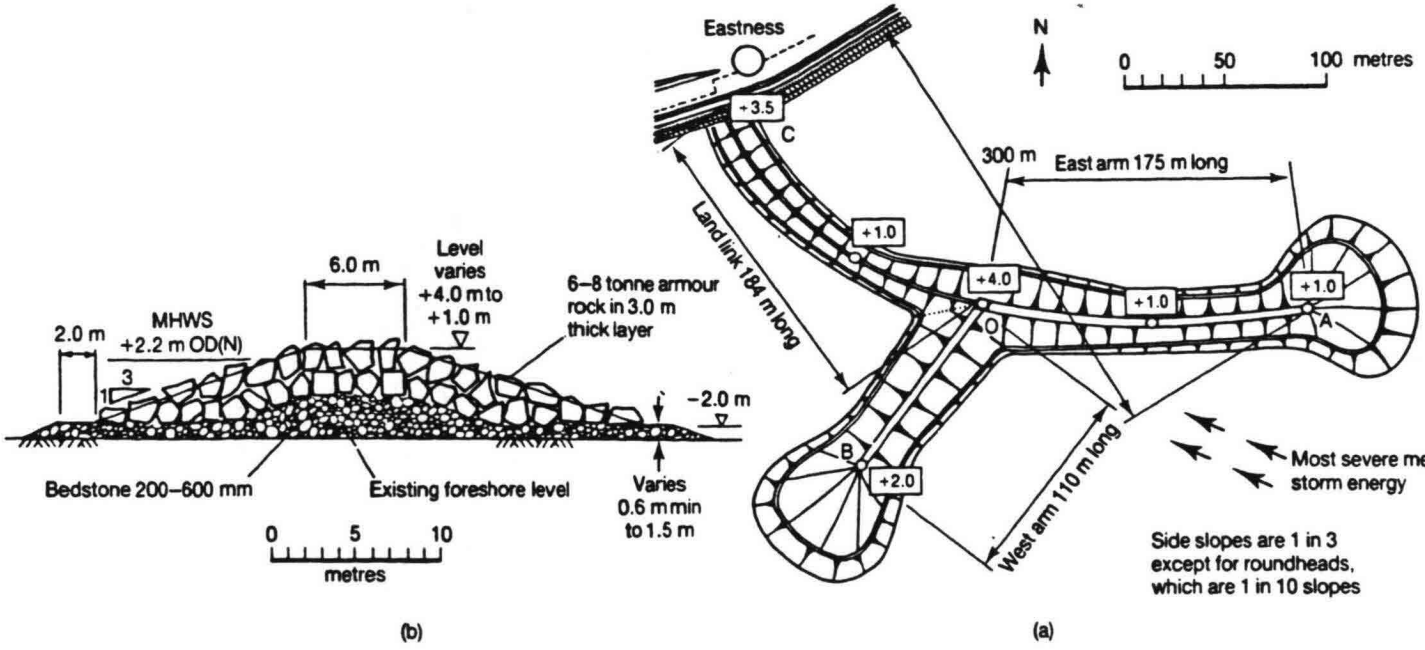
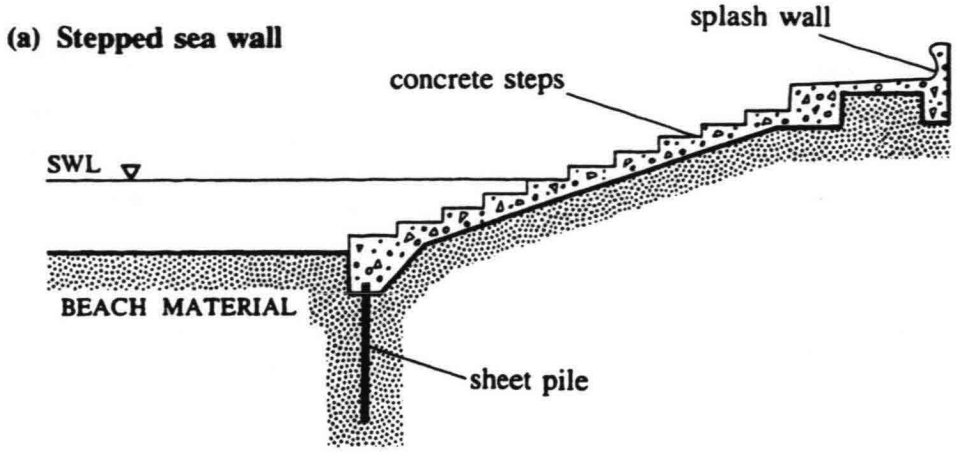
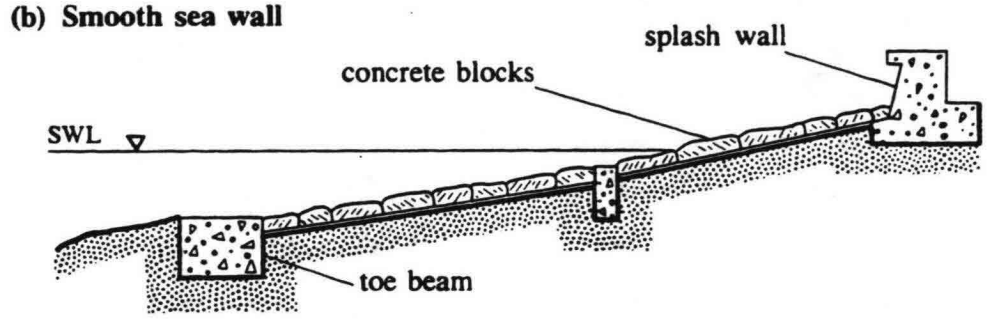
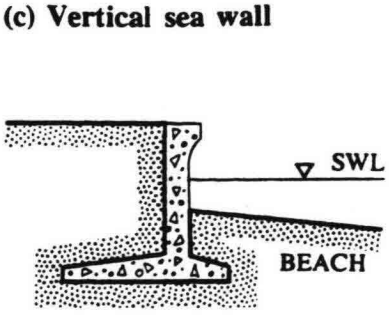
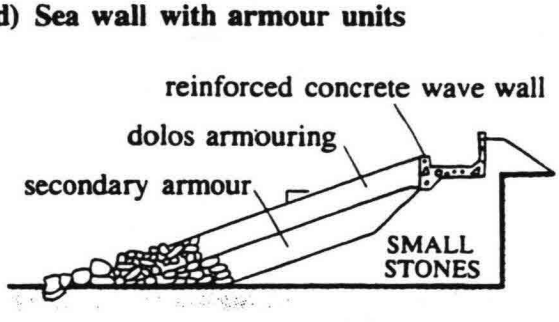


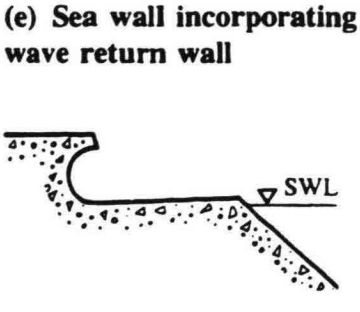
Fig. 22. 17. Eastness Breakwater, Clacton-on-Sea, UK. (a) Plan; (b) typical cross-section.
H_{max} = 2.5 m, T_2 = 6 - 8 s.











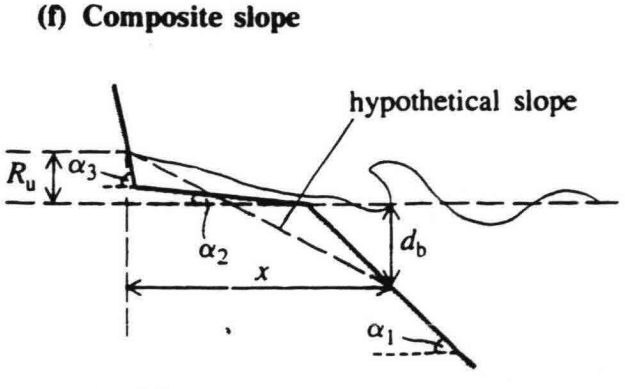
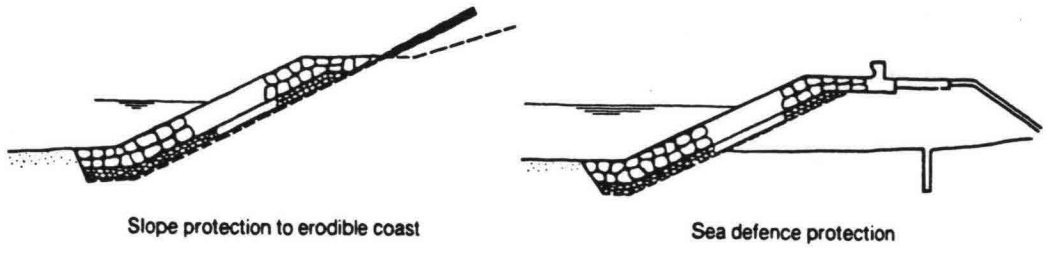
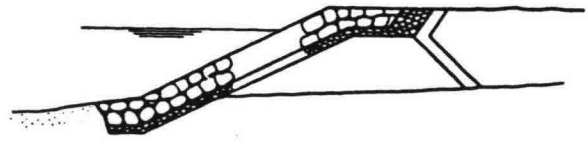


Fig. 22.18. Examples of sea wall.



Slope protection to erodible coast

Sea defence protection

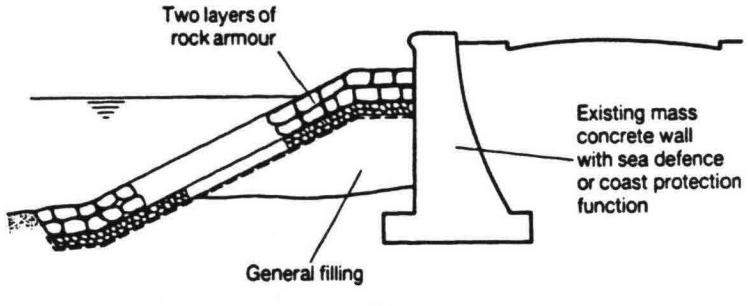


Reclamation bund/protection

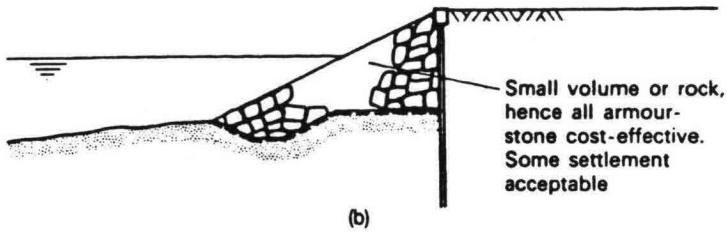
Existing vertical seawall rehabilitation mound

Anti-scour mat to existing vertical wall

Some basic seawall concepts



(a)



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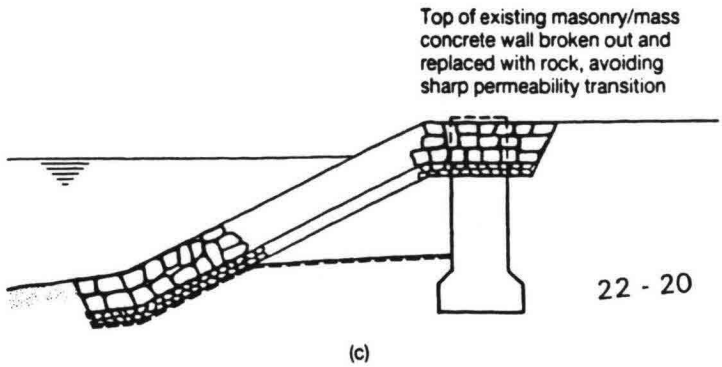


(c)

Fig. 22. 19. Typical revetment for rehabilitation of existing vertical sea walls.

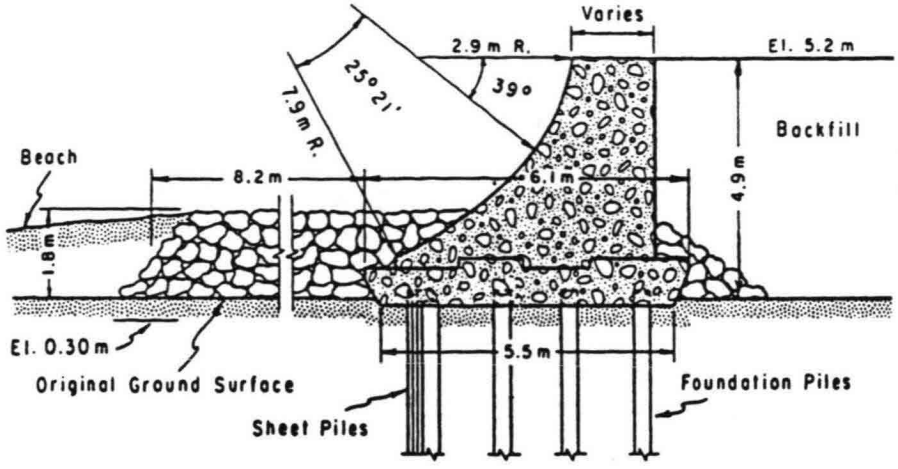


Fig. 22.20. Concrete curved-face sea wall, Galveston, Texas, USA (1971).

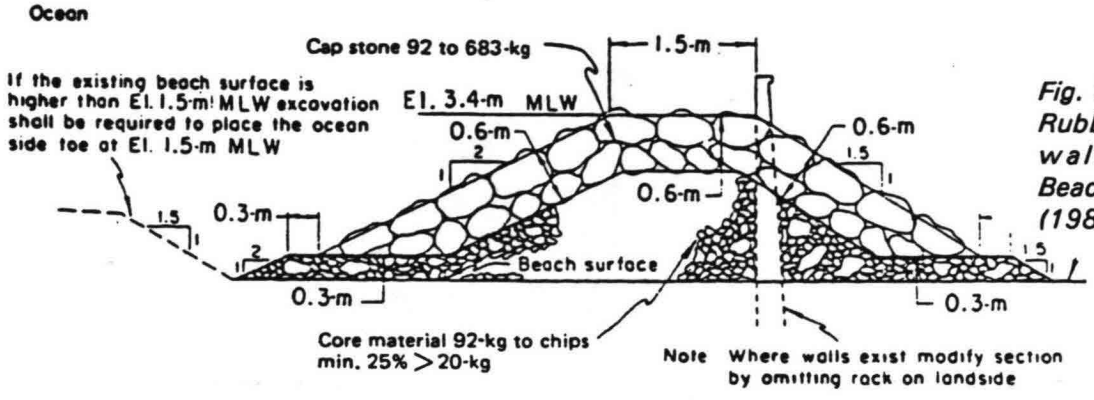


Fig. 22.21. Rubble-mound sea wall Fernandita Beach. Florida, USA (1982).

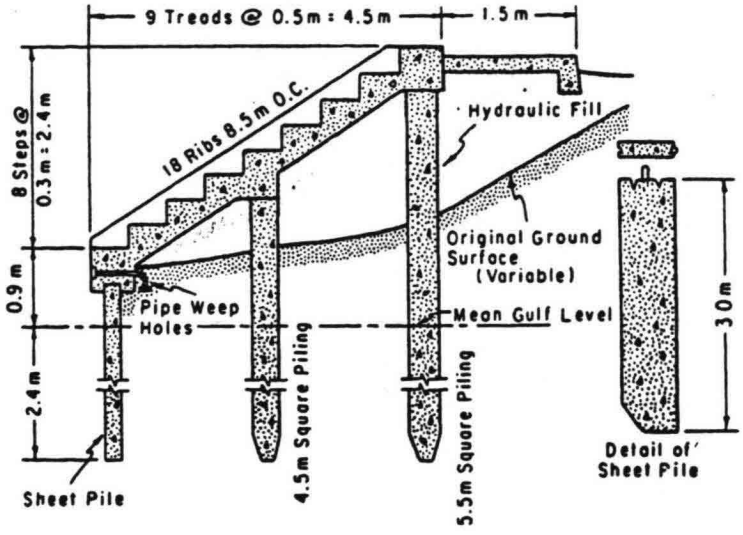


Fig. 22.22. Concrete stepped face sea wall Harrison Country, Mississippi (USA).

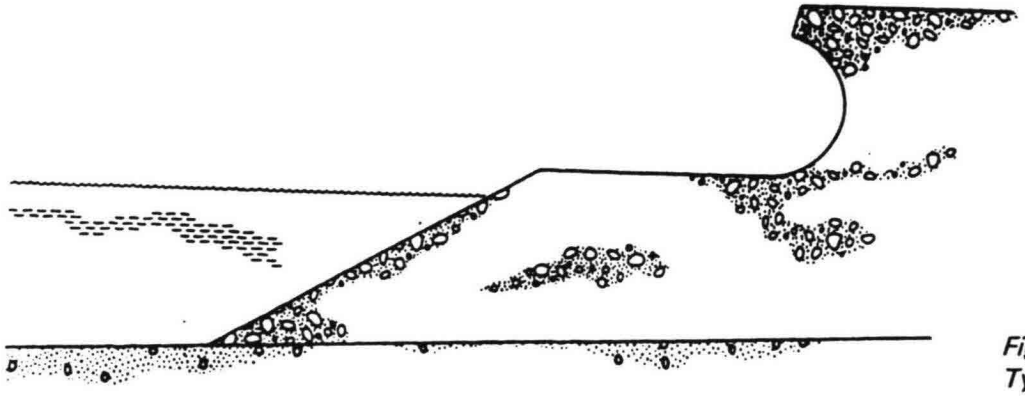


Fig. 22.23. Typical form of a modern sea wall.

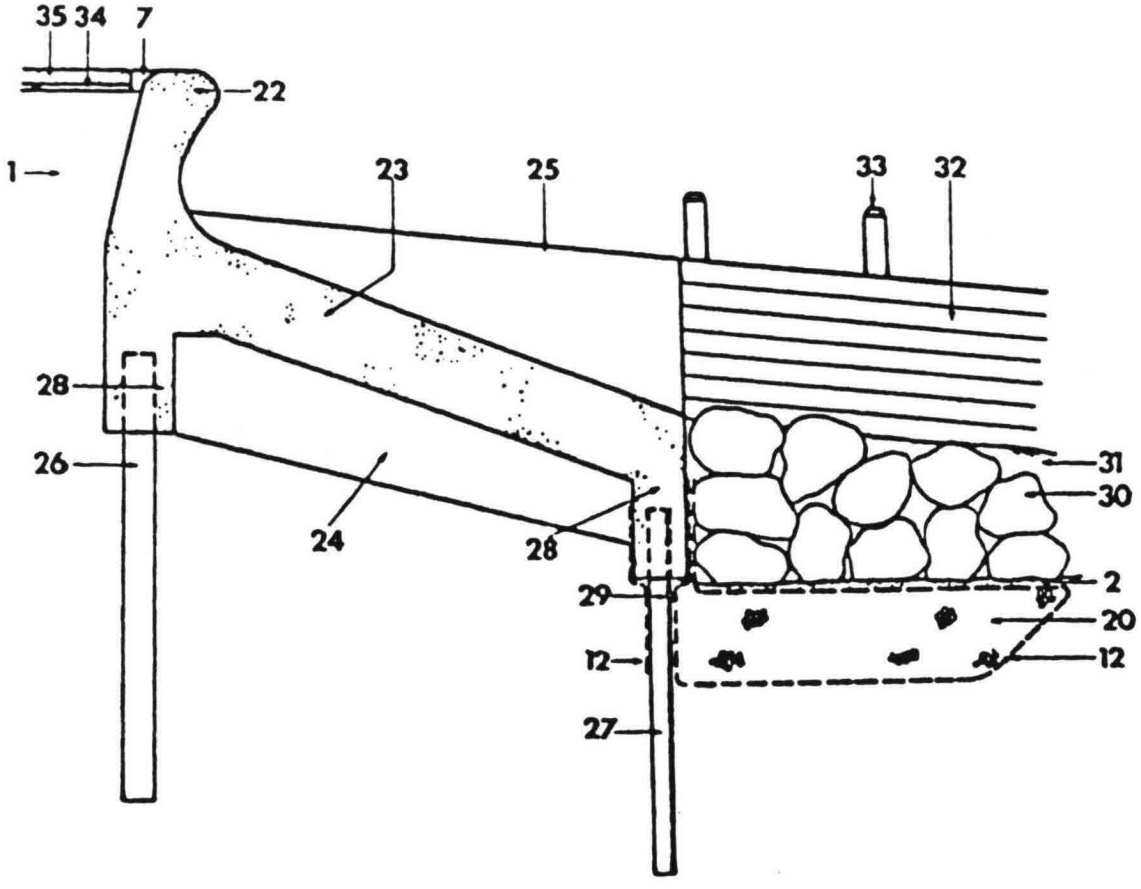


Fig. 22.24. Reinforced concrete wave wall.

1. impermeable bank
2. gravel bed
7. insitu concrete infilling
12. filter fabric (geotextile)
22. concrete wave wall
23. reinforced concrete slab
24. cross-wall
25. reinforced concrete groyne element
26. bearing pile
27. steel sheetpiling
28. concrete pile cap
29. drainage hole
30. rock armour
31. beach
32. timber groyne
33. timber pile (king pile)
34. blinding concrete
35. road slab.

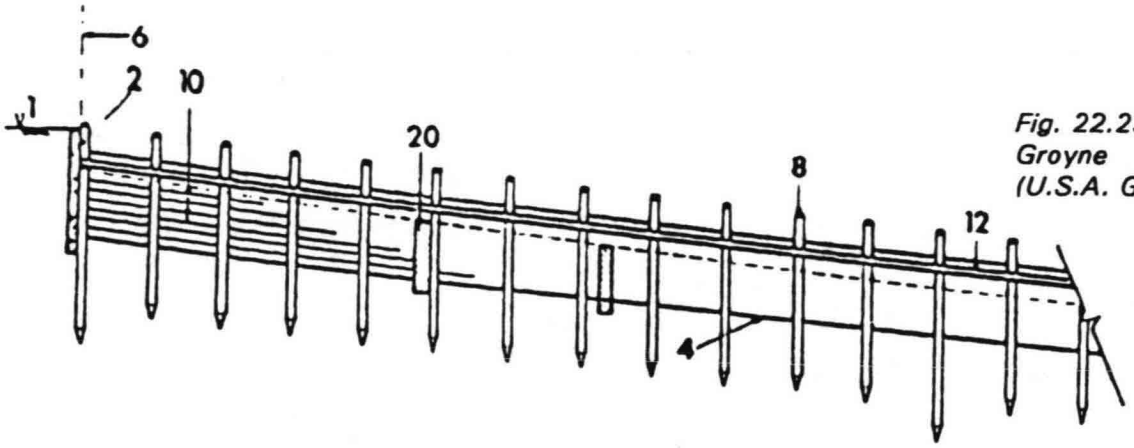
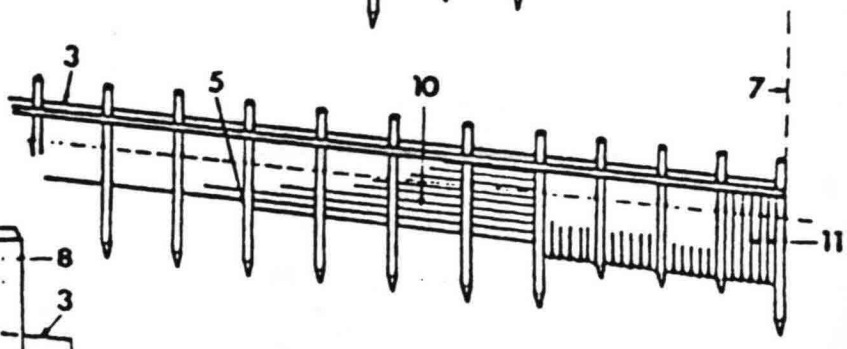
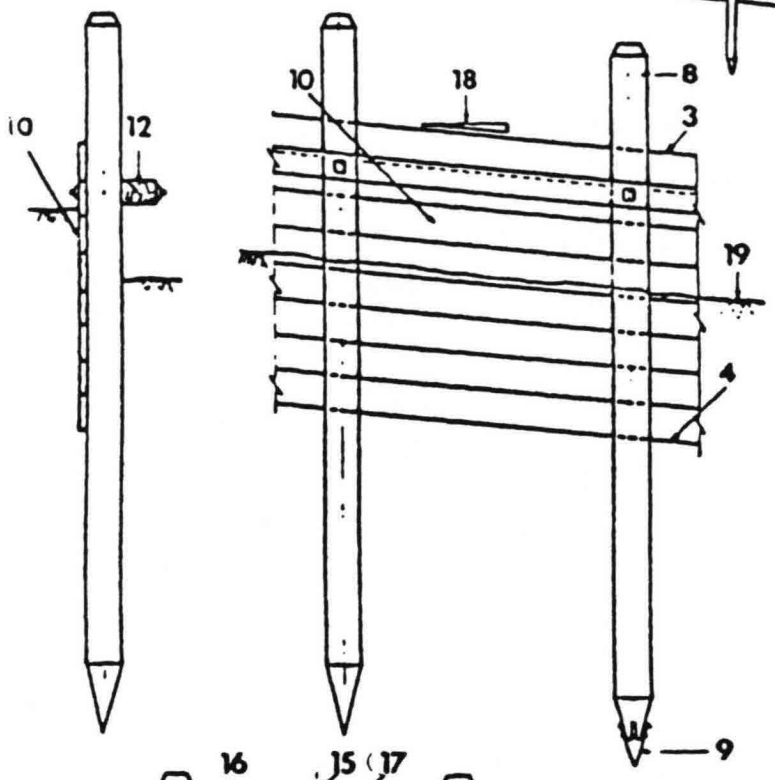
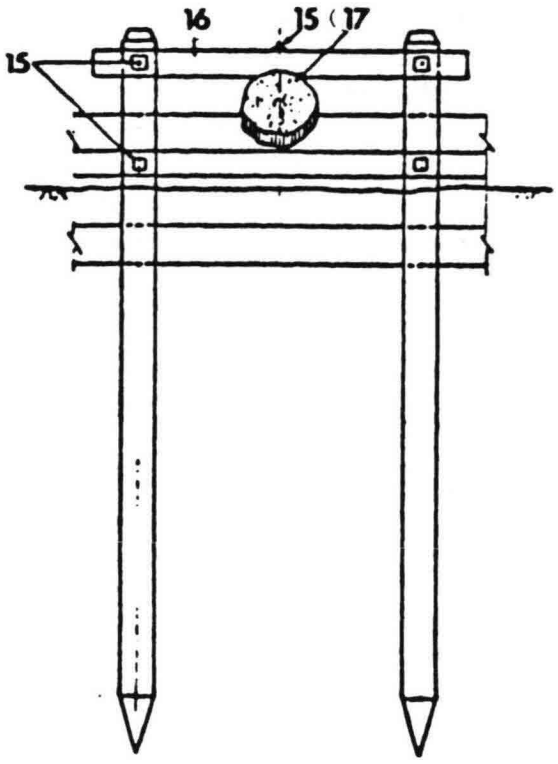


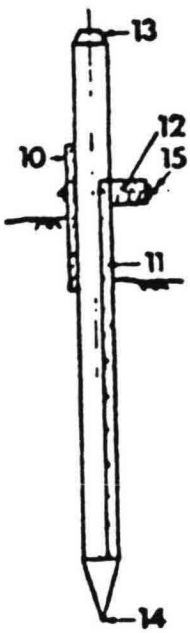
Fig. 22.25.
Groyne
(U.S.A. Groin).





1. ground level
2. head wall
3. top of groyne
4. bottom of groyne
5. step
6. landward limit of groyne
7. seaward limit
8. timber pile (king pile)
9. steel pile anchor shoe
11. vertical timber planking
12. timber walin
13. pile headg
14. pile toe
15. nut, bold and washers
16. transom
17. tie
18. slope
19. beach level
20. brushwood faggots.





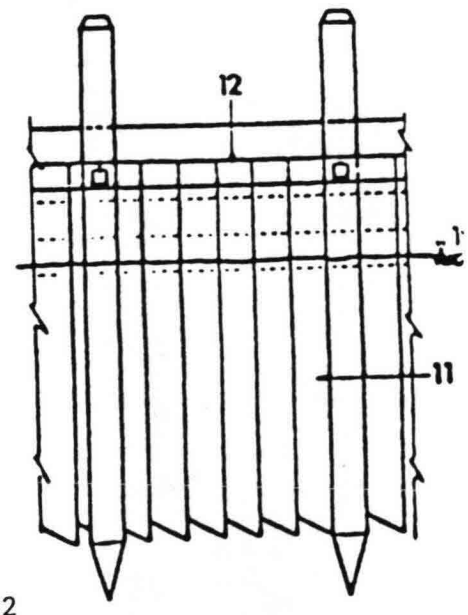


Fig. 22.26. Mass concrete sea wall at Tenby, Pembrokeshire (UK).

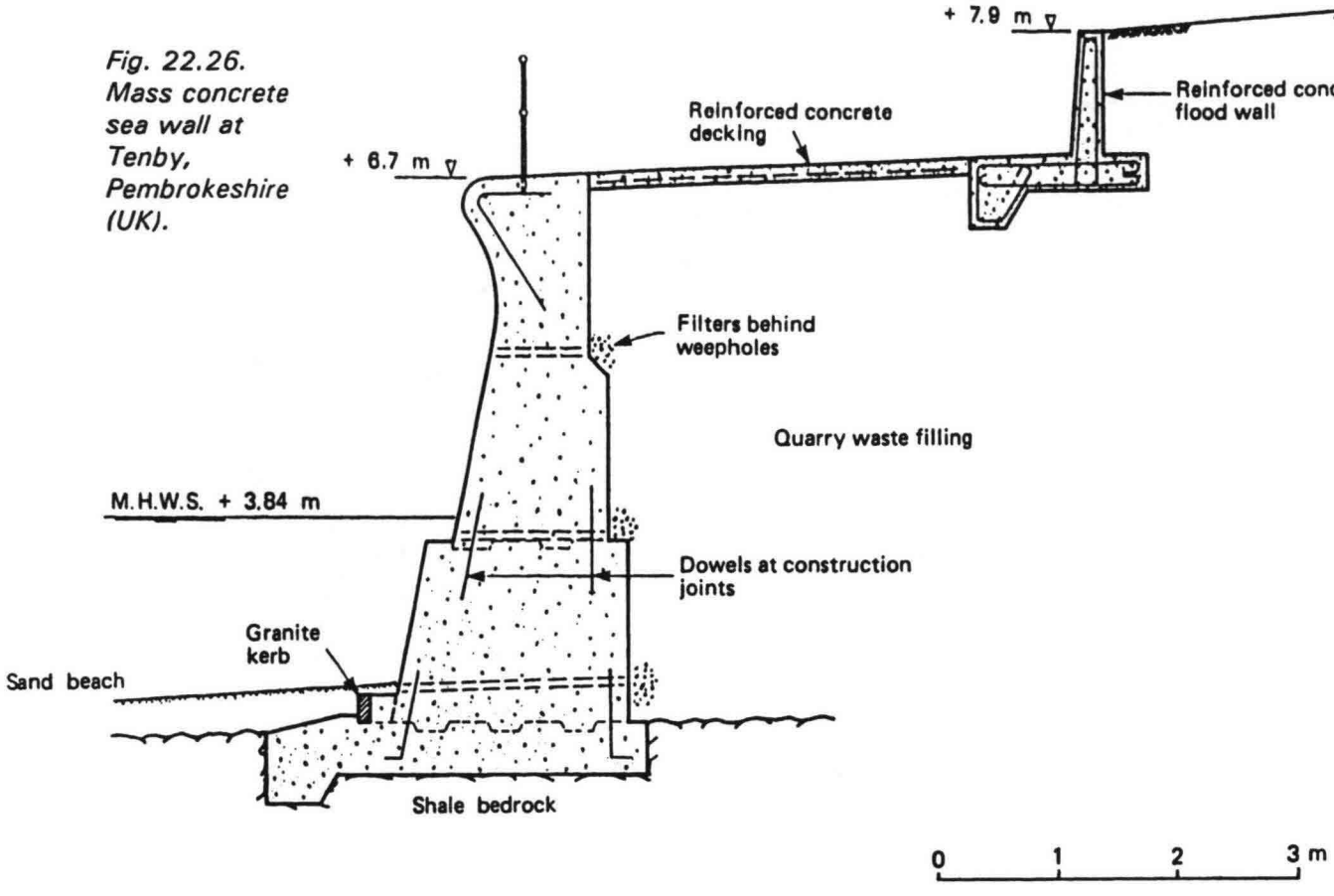
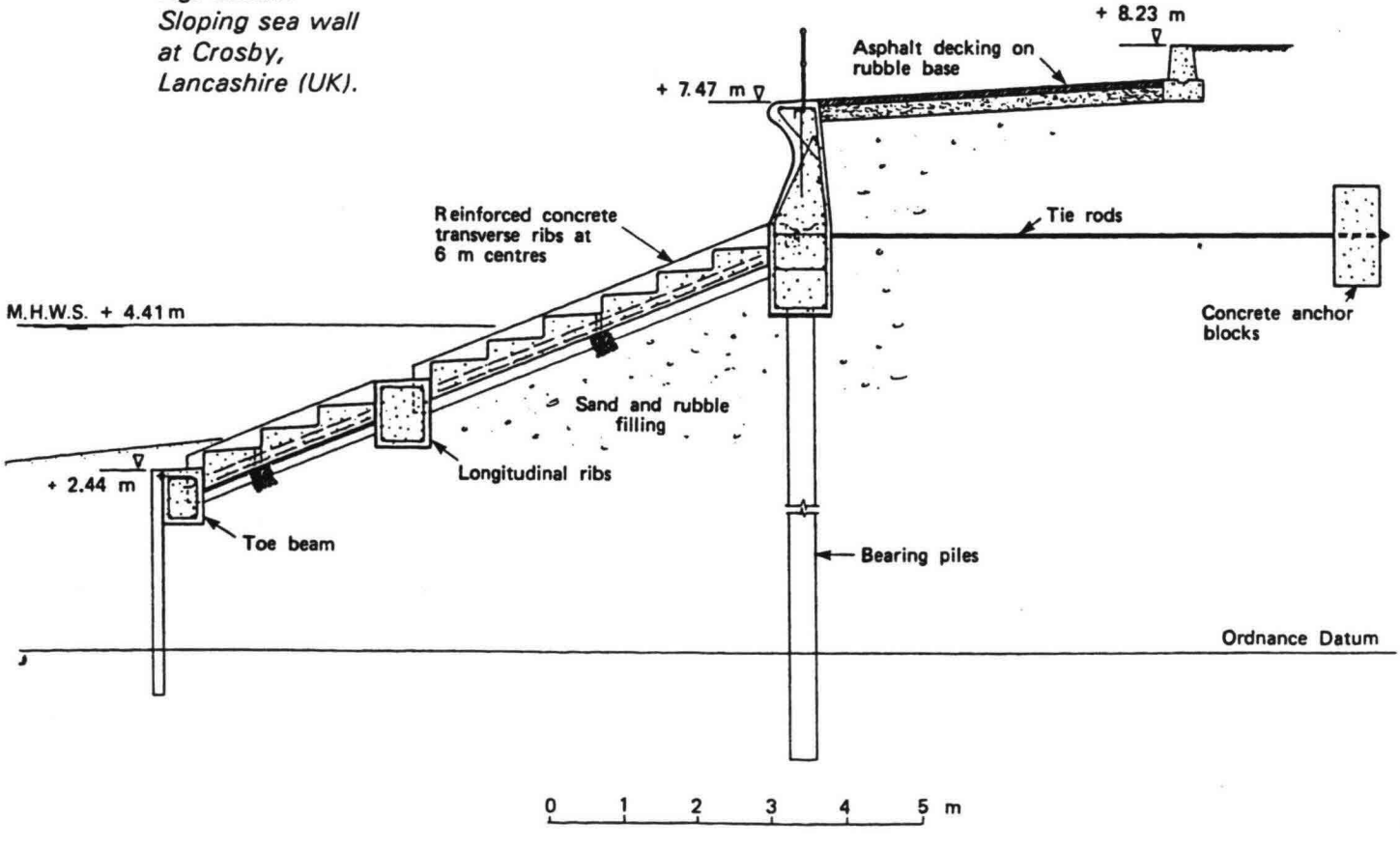


Fig. 22.27. Sloping sea wall at Crosby, Lancashire (UK).



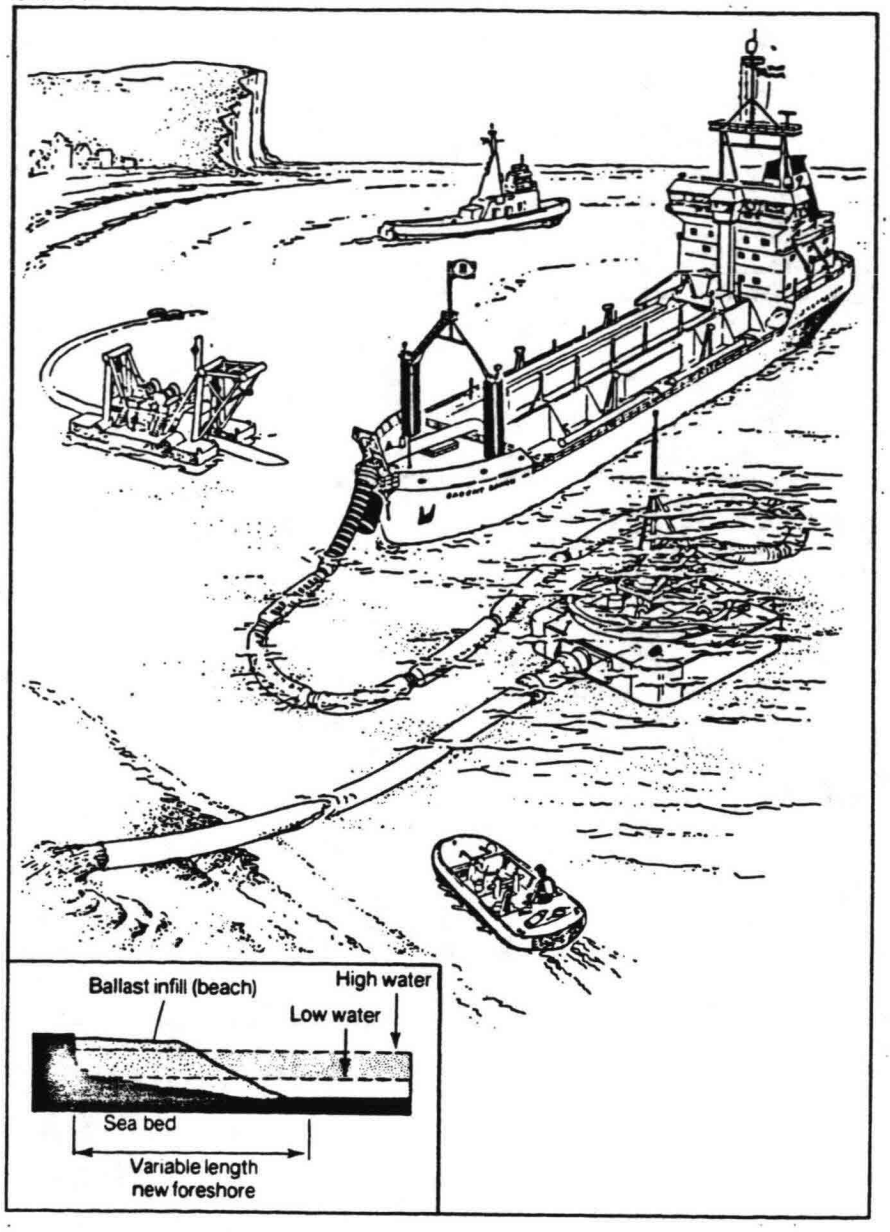


Fig. 22. 28.
Artist's impression of gravel beach construction at Seaford, UK in 1987.

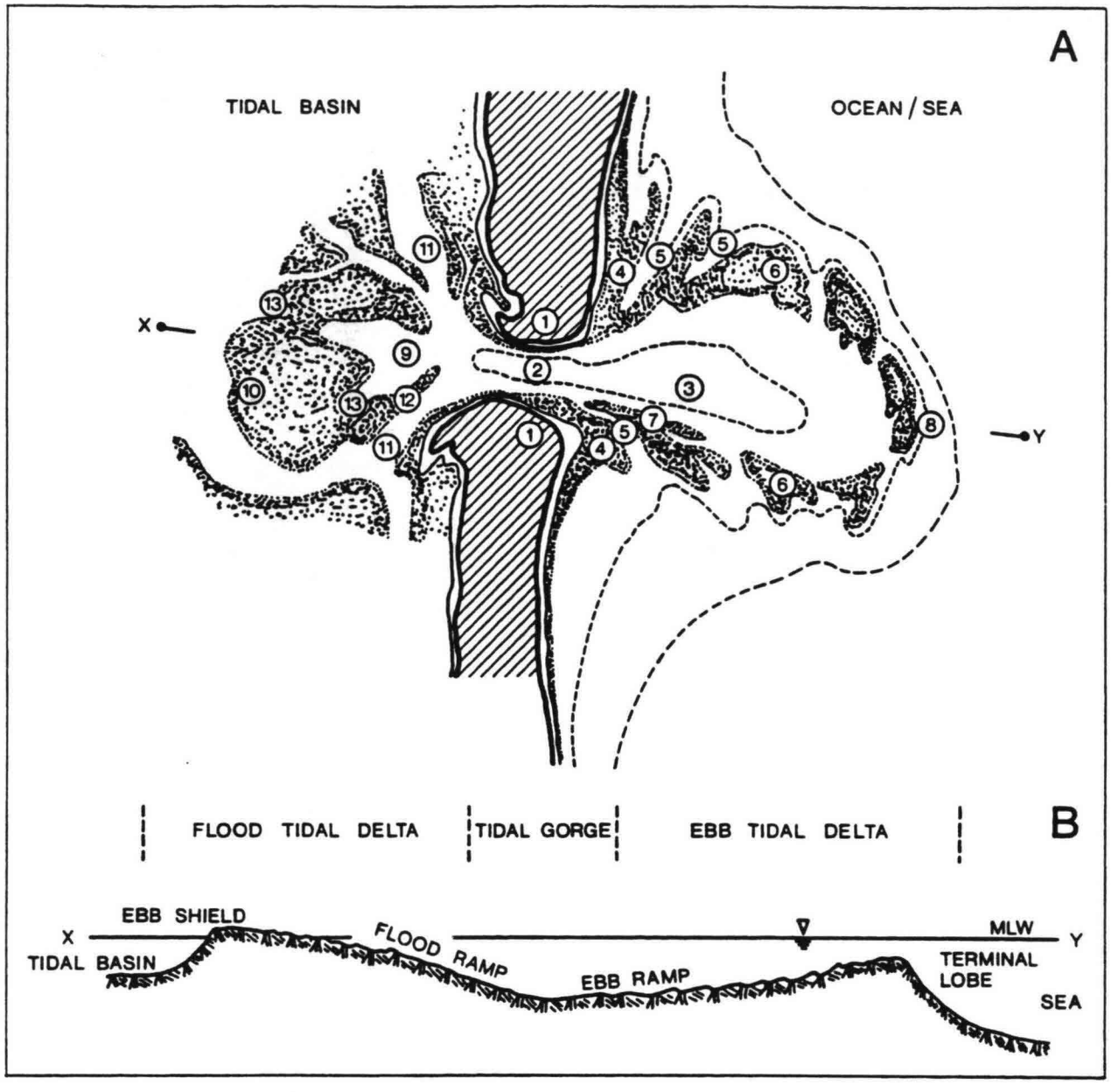


Figure A, schematic diagram illustrating the principal morphological features of a tidal inlet on a sandy coast.

1) coastal barrier or spit headland;
2) the tidal gorge;
3) the main ebb channel and ebb ramp;
4) swash platforms;
5) marginal flood channels;
6) marginal shoals;
7) ebb tidal levee;
8) ebb delta terminal lobe;
9) the flood ramp;
10) the ebb shield;
11) main ebb dominated inner channels;
12) ebb spit;
13) spill over channels.
B, cross section profile from x to y through the tidal gorge and over both flood and ebb tidal deltas.

23. Sea-water intakes and outfalls

Intake and/or outfall structures are an essential part of desalination plants, sewerage outfalls, cooling water systems for (nuclear) power plants and industrial installations. A sea intake and/or outfall is a difficult structure to design and build as it is a combination of an offshore and an onshore structure. Many alternative solutions are possible.

The choice of a certain solution depends on:
- nature of subsoil (sandy/clayey or rocky)
- slope of the foreshore (steep or gentle), determines amongst other the length of the structure
- availability of rock
- tidal range
- littoral drift
- wave climate etc.

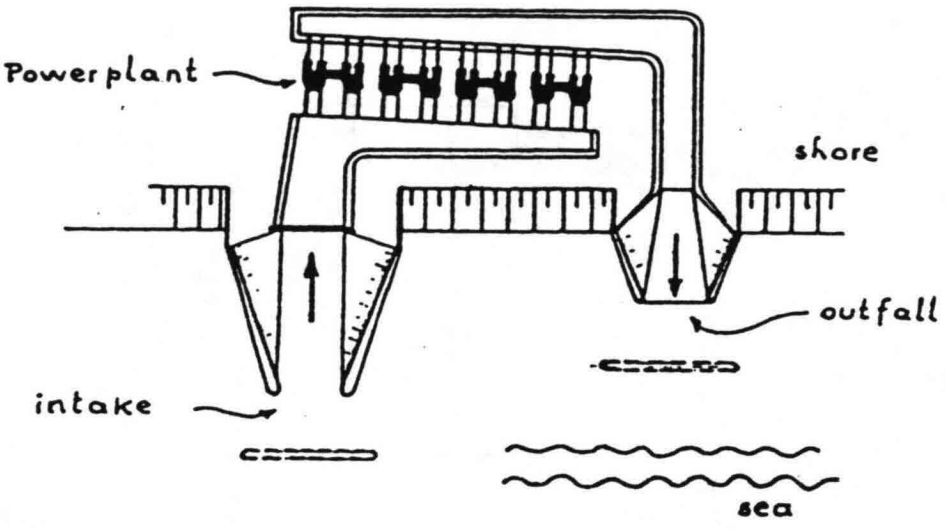


Fig. 23.1. Seawater intake and outfall for cooling of a power plant.

Type of structure	Intake	Outfall
Desalination plant	Seawater	.....
Sewer outfall	n.a.	Sewage effluent
Cooling water station	Seawater	Warm water
Firewater pumping station	Seawater	n.a.
Industrial plant	n.a.	All kind of effluent

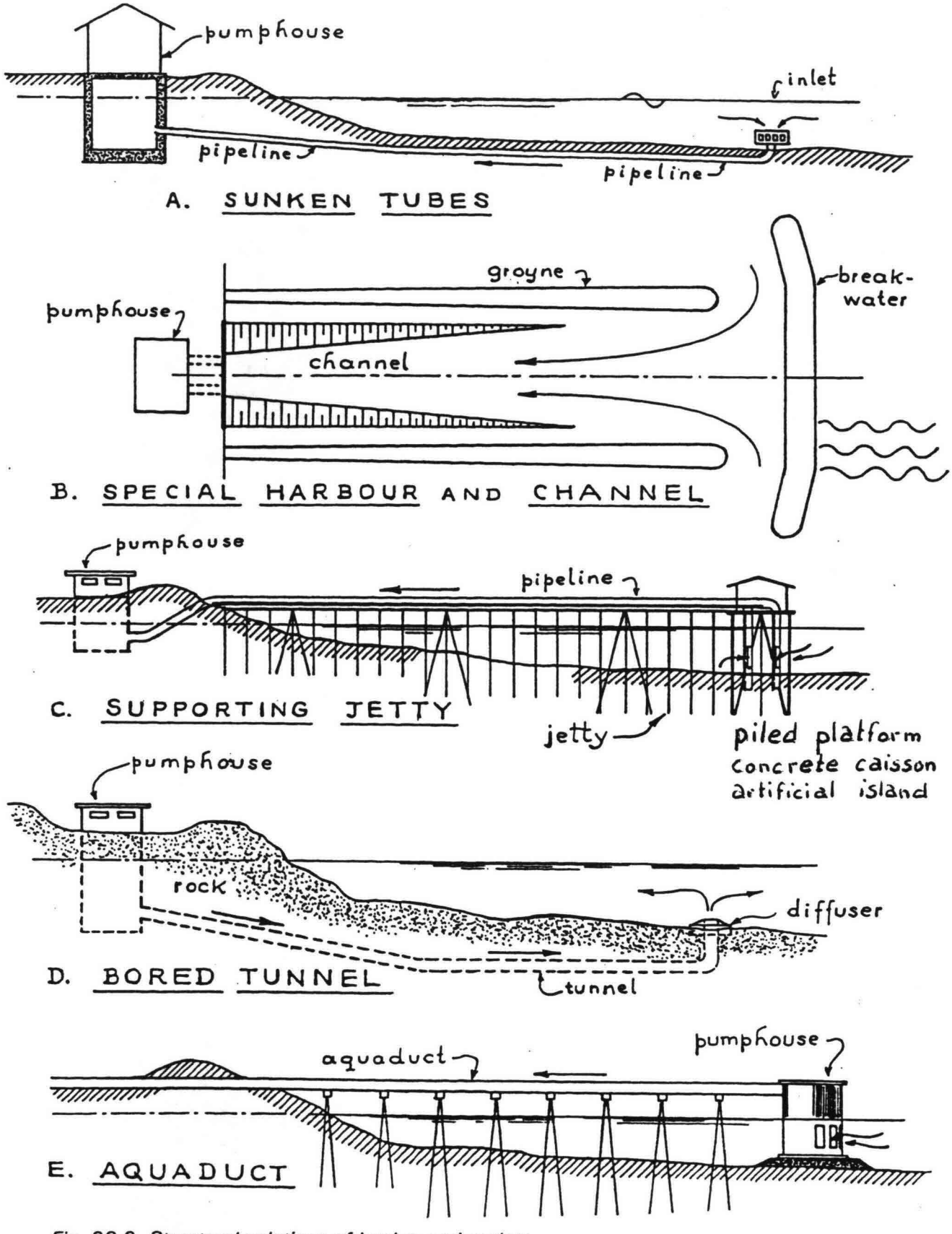


Fig. 23.2. Structural solutions of intakes and outlets.

A. Seabed pipelines to be laid or pulled - used in case of a shallow and gently sloping foreshore.

B. Groynes that act as a breakwater, applied when there is little sand transport, small tidal range etc.

C. Supporting jetty with pipeline on top.

D. Bored tunnel through rock or very hard layers (exposed coast)

E. Aqueduct.

Apart from pipes for the gas and oil, pipelines are constructed in coastal waters for the disposal and dispersion of effluents, and the abstraction of sea water for cooling purposes or industrial processes and its return to the sea. In order that effluents can be dispersed without nuisance and that relatively sediment-free water can be obtained at all times from an intake, which requires the entry port to be below the wave toughs at lowest low water and the intake pipes or culverts to be below water level up to the pump suction, works of considerable magnitude may be required. They fall into three main categories:

- jointed pipelines;
- pipelines towed or floated into position, and
- tunnels and shafts.

Jointed pipelines

These includes pipes laid and jointed by diver in a trench excavated in the sea-bed and backfilled with imported or dredged material, or partially or wholly with concrete in the case of a rocky seabed, or supported above the sea-bed on piled trestles or concrete saddles,

The most usual type for this form of construction is cast or spun iron with flexible joints, but steel, plastic, concrete and aluminium have also been used. The choice of pipe material should be made for each project, having regard to durability and method of installation; the cost of the pipe itself is usually a less important consideration.

The design of the work should take into account of methods of construction and the type of equipment to be used, which may be conventional plant working on the shore between tides, from a gantry, from floating craft, or from a jack-up spud platform. Allowances must be made for possible changes in sea-bed level to ensure that the pipe or its supports are not undermined, and if the pipes are supported above the sea-bed they must be designed to resist forces due to wave action and currents.

Pipelines towed or floated into position

Pipes installed by these methods, in which they are fabricated into long lengths on shore and towed into position by tugs or winches, have usually been of steel manufactured by either the longitudinal or spiral weld process. Protection to the pipe may be a mortar, epoxy or polyethylene lining, and externally a bitumen or coal tar impregnated glass fibre wrapping; the pipe is then encased in gunite or concrete to protect the wrapping against mechanically damage and to provide negative buoyancy at the sea-bed when sealed and full of air.

Usually the pipes are joined into long 'strings' by butt welding and, if floated and sunk into position, the strings are joined under water with flexible couplings. For lines towed into position along the sea-bed the strings are butt welded together and the lining and wrappings completed at the joints as the pipes are pulled out from the fabrication site.

In this bottom pull method the negative buoyancy of the pipe is critical and small construction tolerances can have a significant effect. A check by on-site weighing of a length of prepared pipe is advisable to determine whether or not additional detachable buoyancy needs to be added. Pulling forces vary with sea-bed condition and can range from 0.5 to 1.0 of the negative buoyancy of the pipeline. Maximum stresses in the pipe invariably occur during construction.

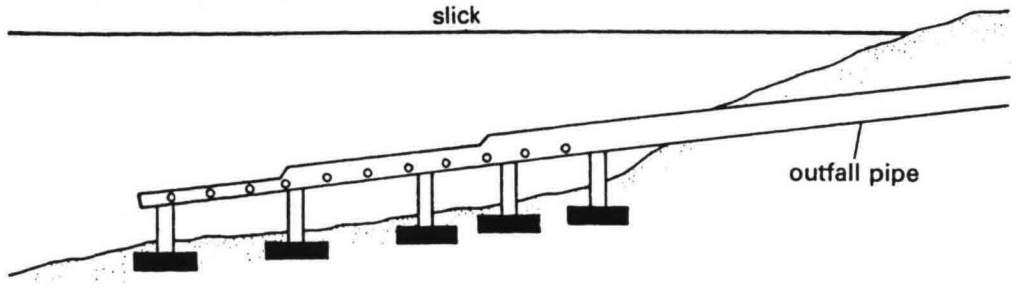
Concrete pipe has also been used for long outfalls using the techniques for submerged tube tunnels. For this operation the foundation or bedding conditions are of great importance.

In exposed situations where the bed level is liable to fluctuate it is necessary to bury the pipe. This is usually done by dredging a trench prior to launching the pipe but ploughs and jetting equipment have been used for this purpose.

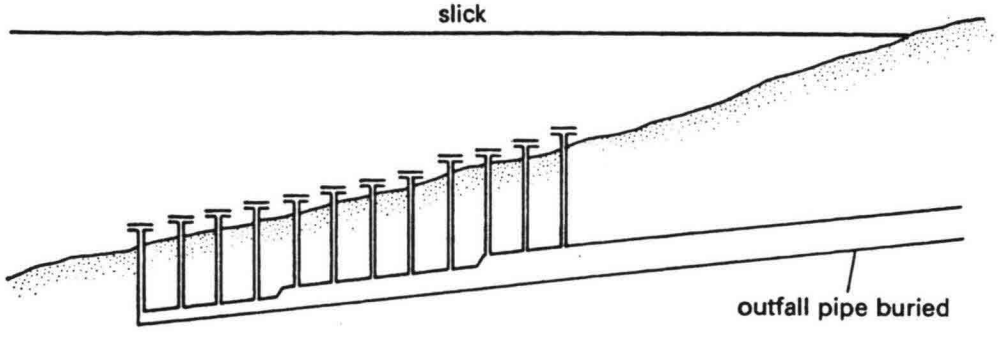
Tunnels and shafts

There have been many instances of tunnels being constructed as sea-water intakes and outfalls. In soft ground they have been confined to fairly shallow depths allowing the use of compressed air. In rock there is more latitude in the choice of depth and cover, fissured strata being treated ahead of the work by grouting.

The connection of the tunnel to the sea has in some cases been made by flooding the tunnel and breaking through with a single large round of explosive, but more usually by one or more shafts leading to some permanent structure (a tower or caisson) on the sea-bed, or drilled from floating plant or a jack-up platform.



OUTFALL PIPE CARRIED ON COLUMNS



OUTFALL PIPE IN TRENCH WITH UPWARD DIFFUSER PIPES

Fig. 23.3. Two types of diffuser at sea outfalls. A diffusor (sewage) at an outfall could be holes in the pipe to spread the sewage.

Sewer outfall. The point at which a sewer discharges into the sea or to a river. Because most effluents are warmer than the sea or lake they flow into, the effluent is usually seen on the surface as a slick, and drifts with the current. But in the summer the effluent may be cooler than the top layer of sea or lake and so may remain invisible below it. An effluent outfall to a small stream must be carefully designed not to scour the bed or opposite bank. Hard revetment of brick, stone, concrete or timber is needed. Outfall pipes may be made of steel, concrete, high-density polythene, polyvinyl chloride, glassfibre-reinforced plastic, etc. and must have a negative-buoyancy coating so that they do not float.

Negative-buoyancy coating. A concrete coating with steel mesh or polypropylene fibre reinforcement cast around a submerged pipeline to keep it safely sunk when it contains anything lighter than water.

24. Offshore pipelines

Pipelines have been used already long ago for the transportation of liquids overland. Today they are widely employed for the transportation of oil, gas, water, chemicals and slurries. The advent of offshore oil and gas has necessitated the construction of submarine pipelines on the seabed. These pipelines carry the oil or gas from the offshore production platforms to the shore.

The pipes are made of steel and are produced as either seamless or seam welded. Seamless pipes being an extruded or drawn tube, usually of relative small diameter (up to 300-400 mm). Seam welded pipes are fabricated from flat plates which are longitudinally or spirally seam welded. There is no limited in diameter.

To protect the steel pipelines against both the environment in which it is to be laid, and/or the adverse properties of the fluid mixture it is to transport, various types of coating are applied:

. external corrosion	- thin layer of bitumen + wrapped with a glass fibre tape (sometimes: epoxy coatings)
. external mechanical damage	- concrete coating
. internal protection (corrosion and erosion)	- lining with anti-abrasion and impervious materials (epoxy cement etc.).

For offshore pipelines the concrete coating also serves another very important function, in that it is a **weight coating**, to give negative buoyancy of the pipeline in order to achieve stability when on the seabed. To achieve the required weight of coating, the concrete is often impregnated with iron ore or other higher density materials.

Construction of a pipeline, whether on land or offshore is essentially the same process, that of welding short sections together to form a continuous line. **Conventional land laying** is simply the construction of a continuous length of pipe, which is then lowered into a trench using mechanical handling equipment. Burial in a trench is both for protection of the line and often environmental reasons. Sometimes the pipeline is placed on supports above the ground (Alaska pipeline).

Offshore pipeline construction requires high capital intensive equipment. The risk to the pipeline and equipment is much higher and laying is often restricted to a narrow weather window (summer period). The majority of offshore pipelines are laid using a **lay barge**. This lay barge is essentially a floating construction platform on which short lengths of pipes are joined together.

Types of pipe laying barges:

1. **Conventional barge** with a sloping ramp
   - usually 5 or 6 welding steps + non destructive testing and joint protection, layrate: around 60 meters/hour.

2. Third generation **semi-submersible** lay barges
   - less responsive to wave action
   - construction process same as conventional barge.

3. **'J' or inclined ramp** type lay barge for deep water
   - the ramp is much steeper, sometimes even vertical (no stinger and overbend)
   - construction process is limited to one of two stages.

4. **Reel barge**
   - the total length of the pipe is preconstructed and coiled onto a drum mounted on the deck of a barge (limited diameter pipelines)
   - pipelines of up to 250 mm in diameter and 4 km in length have been laid using the reel barge concept.

An alternative to the lay barge is the **long lengths approach**, where pipe strings of over a kilometre are constructed on land and towed to site for joining together on the seabed. The towing of a complete pipeline of over 20 kilometre is feasible.

Shore approaches
The interface between the seabed and the land is often the most difficult phase in the construction of an offshore pipeline (breaking waves, changing beach profiles etc.).

Two common methods:
1. Construct pipe on shore and pull it offshore using a pull barge or tow vessel (4-5 km length with diameters up to 2500 mm).
2. Locate the lay barge as close to the shore as is practicable and then pull the pipe-line onshore as it is constructed.

Stability
One of the major problems with pipelines is keeping them on the sea bottom. Ideally, they are buried deep enough over their entire length to hold them in position and protect them from ship anchors. Unfortunately, the sea bottom is not smooth; large areas are covered with irregular humps of sand similar to sand dunes, called megaripples. The stability of these megaripples is questionable. There is a good chance that under such conditions a pipeline will be buried in the crest of the megaripples and be hanging free between them.

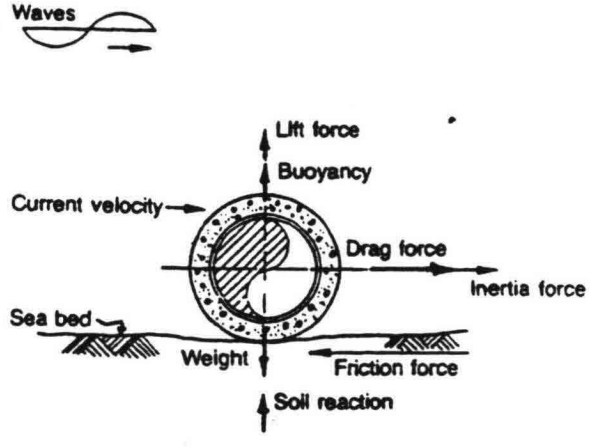


Fig. 24.1. Forces acting on submarine pipeline. Stability during installation and under operational conditions.

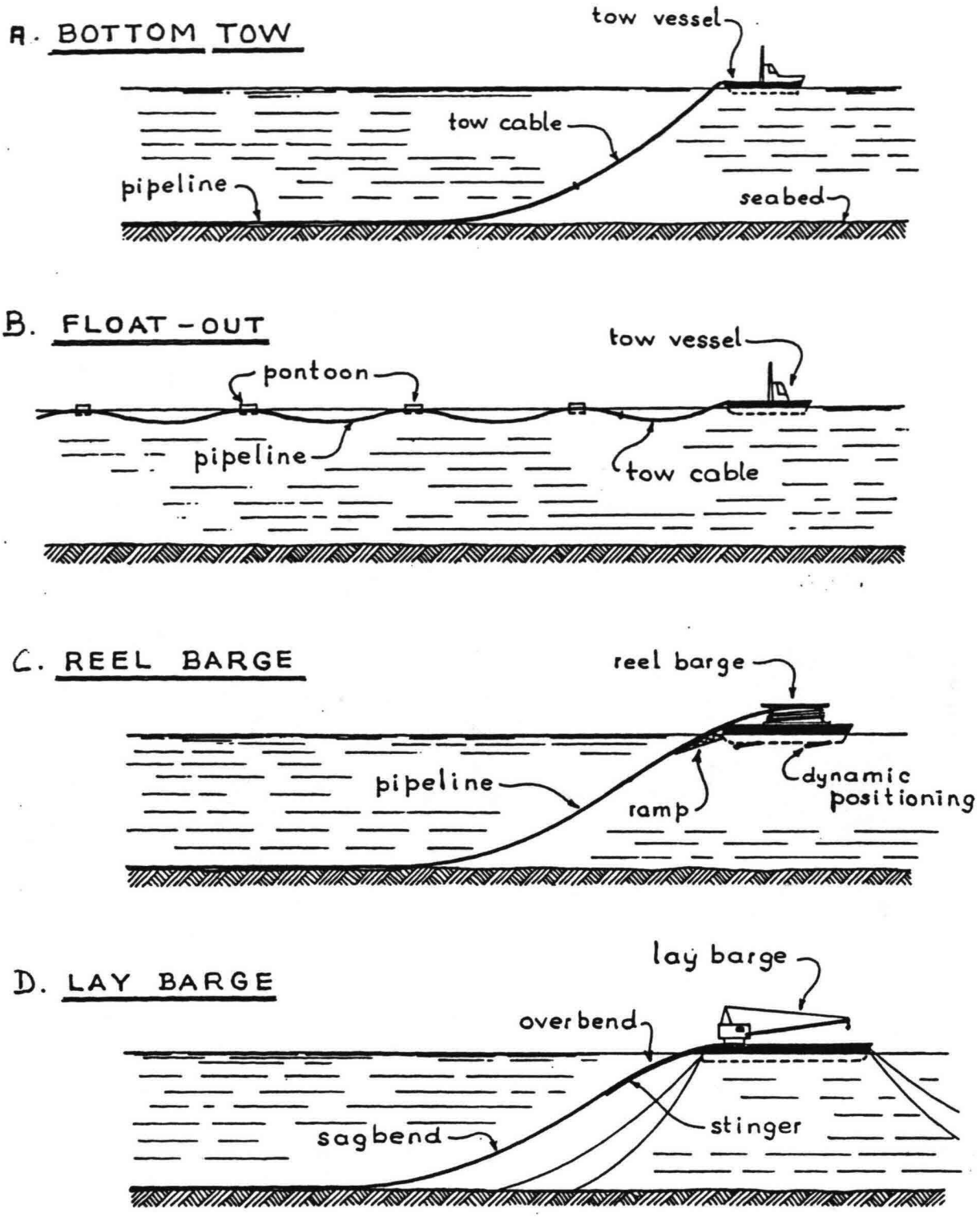


Fig. 24.2. Methods of laying pipelines.

A. The pipes are joint on land and towed under water to final location.
B. The pipes are joint on land and 'floated out'.
C. Using of laying barge with large drum (small diameter pipes).
D. Jointing pipes together onboard of the laying barge.

Protection

Why are pipelines and cables protected ? To prevent damage caused by:
- Forces on the pipeline or cable itself resulting from the action of wind, swell, waves and currents.
- Subsidence or scour of the seabed or erosion of the shore lines.
- Dropping or hooking of ship anchors.
- Hitting or hooking of fishing gear.
- Fast morphological changes of the seabed (sand- and mud waves).

Offshore pipelines are often buried in a trench. **Trenches** can be made using:
- high pressure jetting **sleds**
  - require shipboard systems of up to 40.000 hp, with pressure at the submerged jets of 2.500 psi.
  - the jet sleds use the pipeline as a monorail.
- trenching **plows**
  - a trench is ploughed beneath the pipeline; the plow being towed by a vessel from the surface.
- dredgers.

When burial/trenching of a pipeline is impracticable, another means of stabilizing the pipeline has to be used (see fig.)

Protection by means of rip rap not only resists the forces of waves and currents but also direct 'attack' by trawling gear and dragging anchors. A number of stone dumping vessels has been developed for the covering of submarine pipelines.

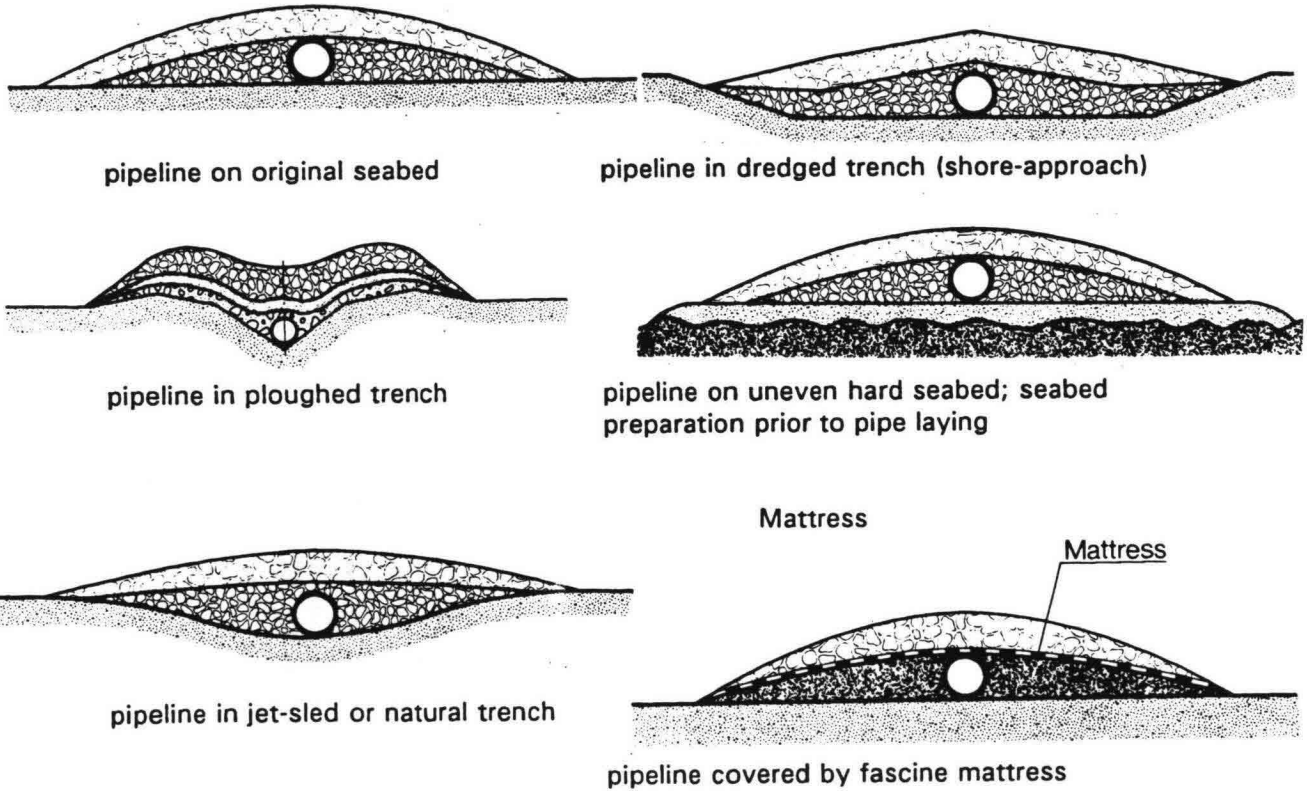
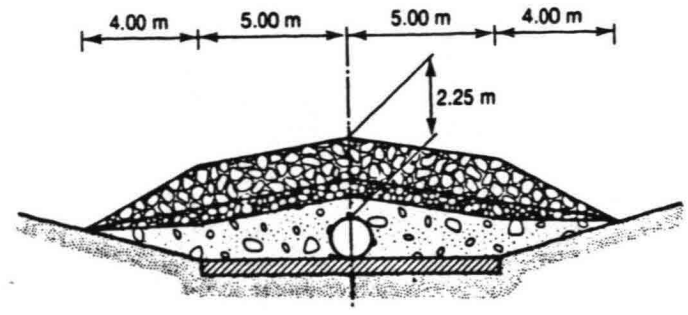
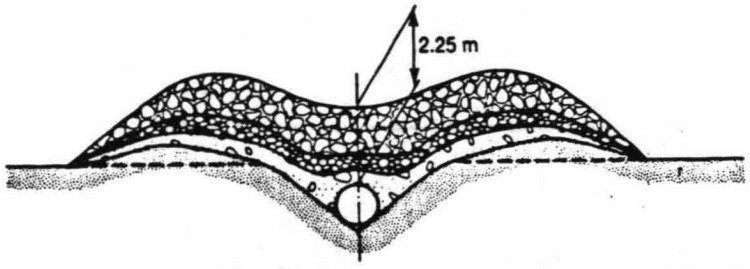


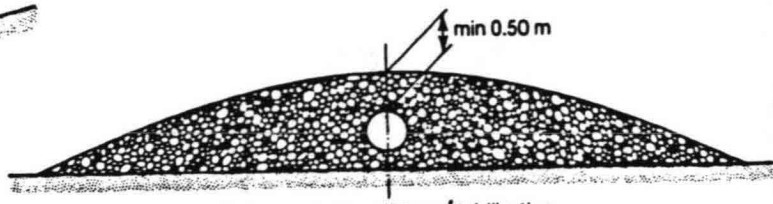
Fig. 24.3. Several alternative designs of pipeline or cable stabilization.



Cross-section dredged trench
Mermaid Sound



Cross-section ploughed or jetted trench
Mermaid Sound



Cross-section offshore stabilization
30 km — platform

= bedding layer = quarry run

= 50–200 mm = 200–600 mm

Fig. 24.4.
Examples for a pipeline rock cover protection/stabilisation.

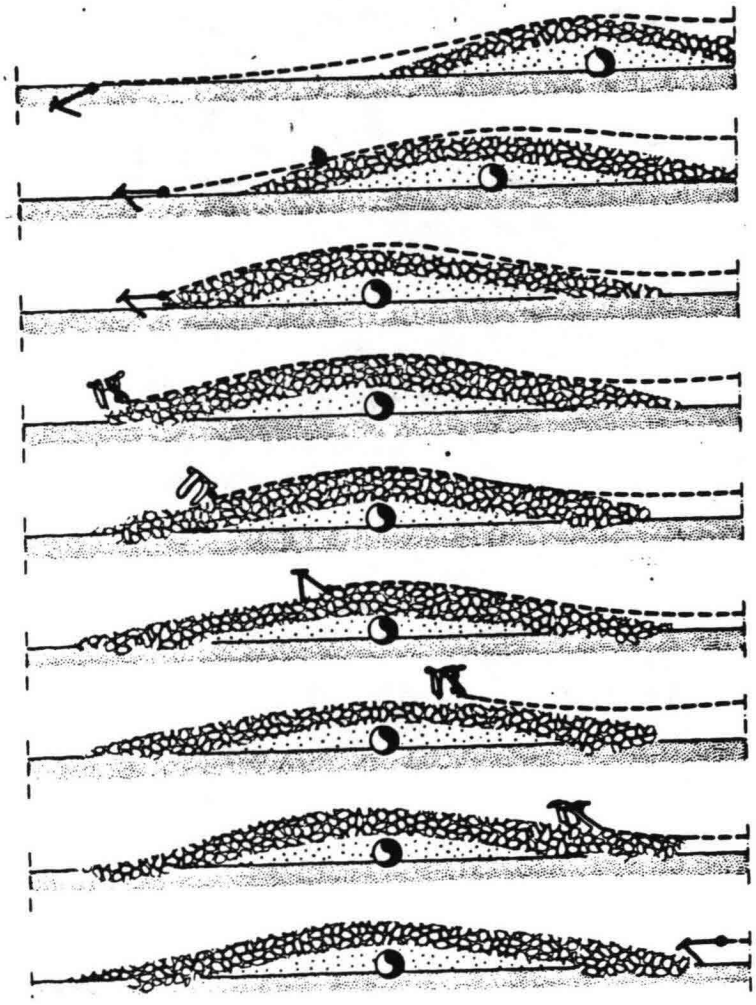


Fig. 24.5.
Anchor on rock protection lying on seabed.

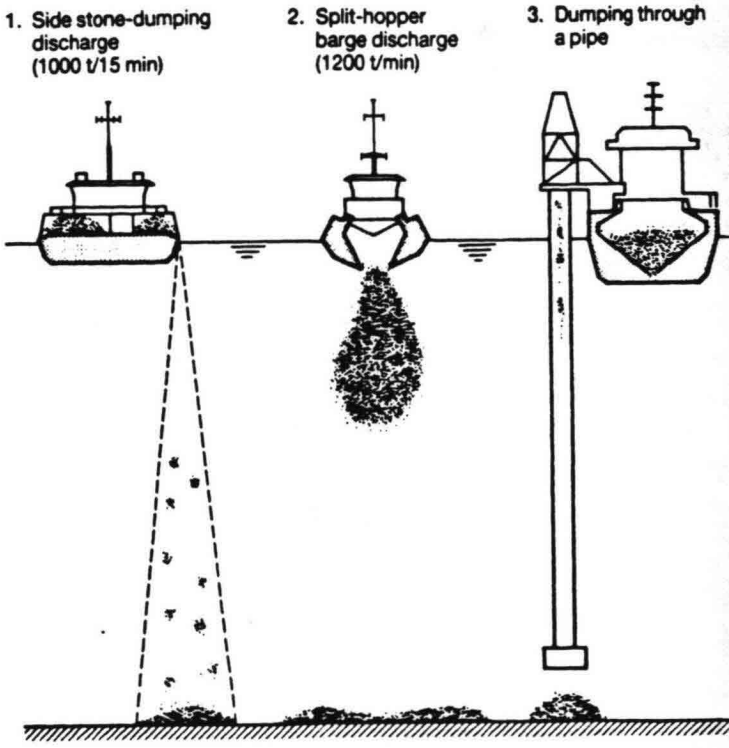


Fig. 24.6.
Common methods of dumping stone offshore.

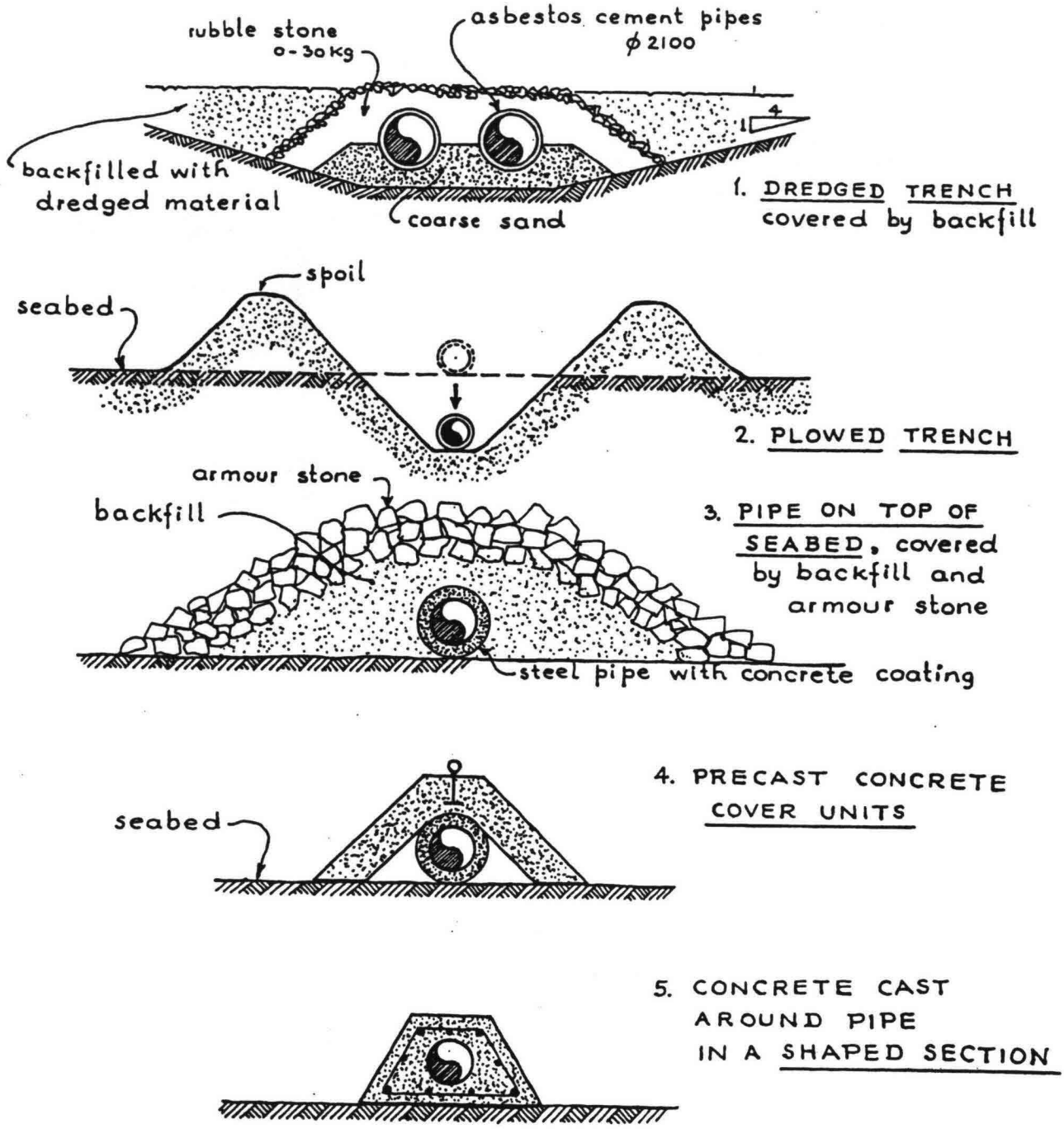


Fig. 24.7. Burial and protection of off-shore pipelines.

Notes: 1. All backfills and added concrete have to furnish the required extra weight to withstand the forces on the pipes.

2. They also give protection to the pipes against damage by anchors etc. If necessary a thicker wall or more reinforcing in the concrete cover can be applied.

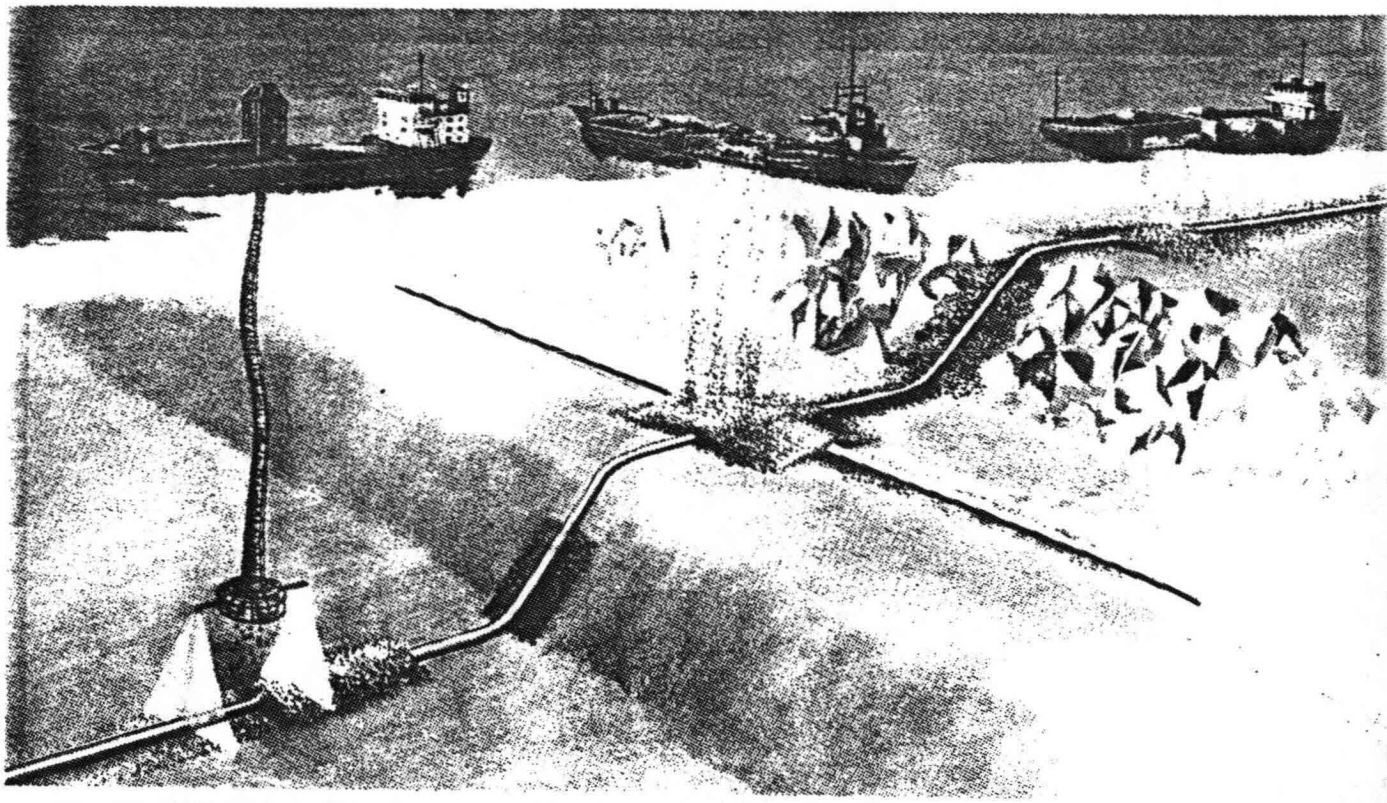


Fig. 24.8. Submarine pipeline covered by rip-rap by stone dumping vessels.

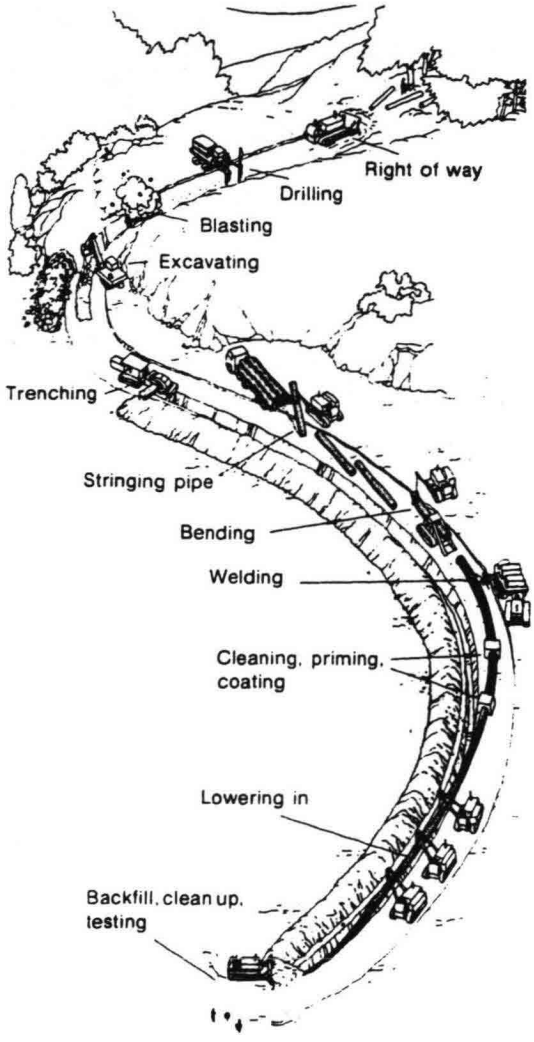


Fig. 24.9. Construction train of a conventional land pipeline.
At the top a bulldozer removes the topsoil. A machine with two (sometimes four) arms drills holes, for the blasting operation. The blasted rock is excavated, followed by a trenching machine. Meanwhile pipes are laid out next to the trench and if needed bended. The welding machine connects the pipes after coating takes place by special machines. Finally sidebooms lower the pipe into the trench and a bulldozer backfills this trench.

25. Offshore structures

Offshore engineering refers to the engineering work related to all sorts of structures located offshore. This definition includes work done by many other branches of engineering as well as civil engineering. Since many offshore structures are used by the petroleum industry, a strong relationship to mining and mechanical is obvious. Many disciplines are drawn together with civil engineering to execute offshore engineering works.

Offshore platforms consists broadly of two components:
1. the drilling and operating facilities (topsides)
2. the supporting structure and its foundation.

Offshore structures can be grouped by form roughly into three categories:
- **fixed structures**
- **anchored structures**
- **free floating structures.**

Fixed structures are most suited for water depth less than, say, 200 m and uses for which the platform must be stable. These fixed structures can again be subdivided into three groups:
- **gravity structures**
- **jacket structures**
- **jack-up structures.**

Gravity structures are the heaviest of the offshore structures and derive their stability against overturning from their weight combined with their broad base (compare gravity dam). Offshore gravity structures are built from concrete at some protected location and then floated and towed to the desired site where they are sunk and placed on the bottom. More than 20 such gravity - base structures have been built since 1970. The Staffjord platform (fig. 25.3), with a displacement of 899.000 tons became in 1981 the heaviest object ever moved by man.

A **jacket structure** is a space frame constructed from hollow tubular elements (steel). The diameter of these tubes can be up to 10 meter with a wall- thickness of 100 mm. A structure to be placed in water depth of 160 m will have a dead weight of about 30.000 tons. Jacket structures are built on land (special construction yards) and are either moved to their final location by barge or are floated and towed to position. In contrast to gravity structures, they are dependent upon pile foundations for their stability.

The third type of fixed structures, the **jack-up platform**, consists of a floating pontoon which raises itself above the sea surface by jacking itself along legs which are lowered to the sea bottom after the structure has been floated into position. Their use is restricted to shallow water locations (up to 90 m).

The first fixed structures for drilling and production out of sight of land were installed in 1947 off the coast of Louisiana (USA). These early platforms consisted of a serie of steel frames that were fabricated on shore, transported to the site by flat barges and set in place by derrick barges. They were then pinned to the sea-bottom with piles (steel template jacket structures). From 1947 until the mid- 1970's virtually all offshore oil platforms incorporated steeltemplate jackets.

Before the development of the North Sea oil fields got underway, by far the largest concentration of jacket-fabrication yards was along the U.S. Gulf of Mexico. By 1972 giant oil fields with recoverable oil reservoirs were discovered in the North Sea. The environmental conditions in the North Sea (waves up to 30 m, high winds) and the size of the reservoirs induced innovations.

Anchored structures depend upon buoyant forces acting against some form of anchor force to provide their stability and maintain their position on the working site.
Types: **ships, semi-submersibles, articulated platforms and buoys**. New developments are the **tension-leg structures** (TLS) and the **Guyed Towers**.

Semi-submersible structures are buoyant structures. Buoyancy is achieved by submerged hollow cylindrical bodies. Usually the structures is anchored on the sea bottom.

Tension-leg platforms (TLP) are not a fixed but a compliant structures: they have the ability to yield to the waves in a controlled manner. TLP's have two main structural elements: a floating drilling rig but much larger, and an array of highly tensioned vertical tethers (tension legs) at each corner. The tethers (high-tensile-strength steel tubes) pull the floating hull down so far that the tension legs never go slack. The tension legs are anchored to the sea bottom by preset foundations. This system allows a degree of lateral movement but it prevents the heave, or vertical motion. The great advantage of TLS is their relative insentivity to the increase of cost with the increase of depth, only the tension legs become longer. A second advantage is that it can be shifted to a new site (reusable).

Another advanced platform is the **Guyed Tower** that has been developed for block 280 in the Mississippi Canyon Section of the Gulf of Mexico. It is a compliant structure consisting of a slender steel tower held upright by a radial array of anchor cables.

Free floating structures are not anchored and are equipped with special propulsion systems that have been developed (dynamic positioning systems) to keep the structure at location. They are used in very deep water, where anchoring would be prohibitively expensive (up to 5000 m).

The various offshore structures can be used for:
- **navigational aids** (first navigational buoys and lightships now often fixed offshore structures, radar platforms)
- **offshore moorings**: for the oil industry such moorings provide for both ship anchoring and connection to a pipeline; mostly buoys and articulated platforms
- **oil exploration** - mostly jack-ups, dynamic positioned ships or semi- submersibles, as it is an activity of relative short duration.
- **oil production** - have a long useful life at a single site (some decades); fixed jacket or gravity platforms are used.
- **oil storage**.

Offshore structures are subjected to forces much higher than onshore buildings. The latter have to withstand lateral forces no greater than heavy winds, while offshore towers must take the impact of waves as much as 30 metres high. The topside facilities are massive superstructures (up to 40.000 tons) and must be positioned above the crest of the highest wave expected to come once in 100 years. For the North Sea that means at least 24 meter above mean sea level.

In establishing the design two wave conditions are critical:
1. Maximum wave that might come as a single event during the lifetime of the project.
2. Cumulative effect of several million waves per year - dynamic loading (fatigue lifetime).

The supporting structure must have a secure foundation. In soft sea-bottom soils it may be necessary to drive piles more than 150 m into the seabed to achieve adequate support.

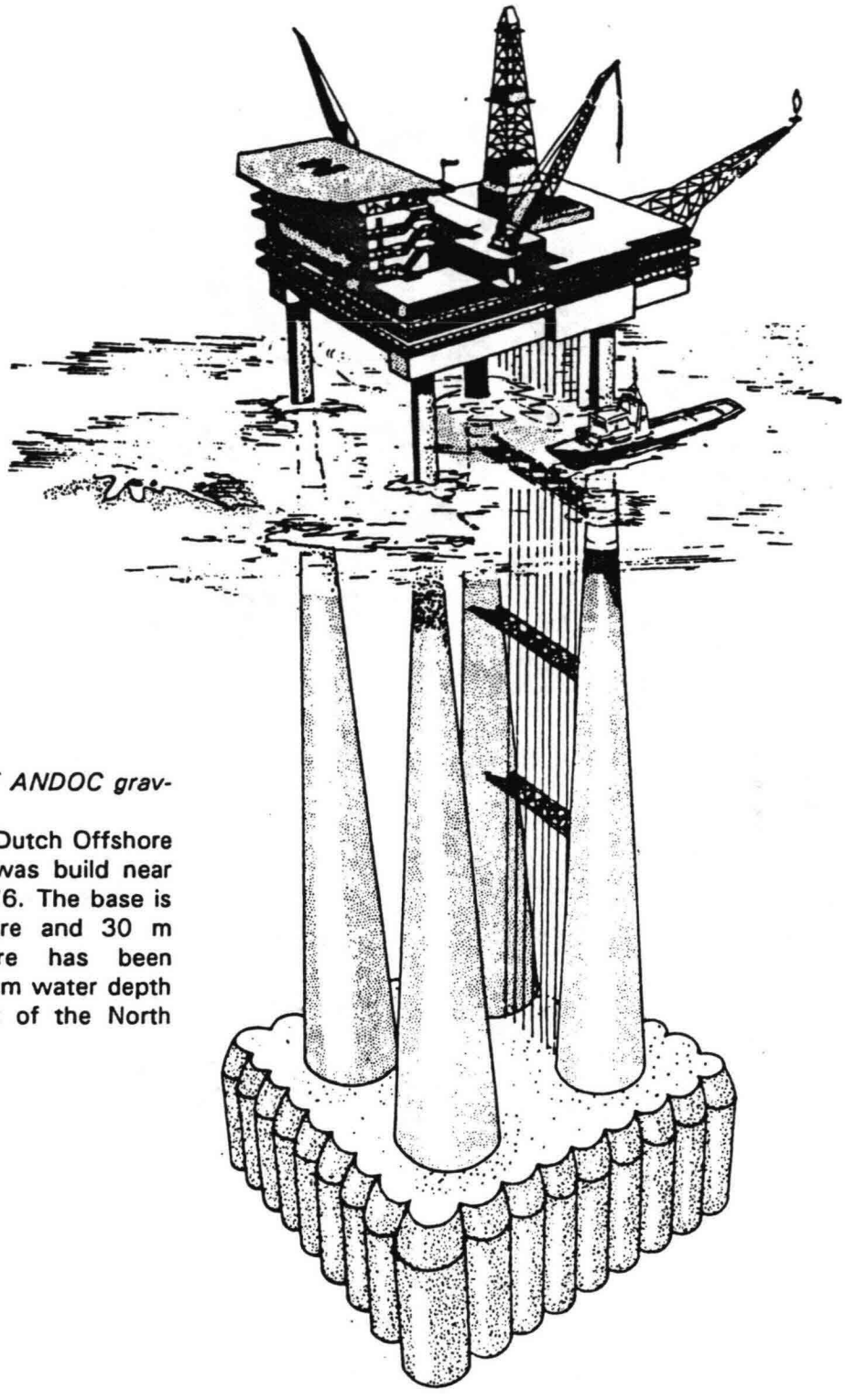


Fig. 25.1. Sketch of ANDOC gravity structure.
The ANDOC (Anglo Dutch Offshore Concrete) platform was build near Rotterdam in 1975-76. The base is about 100 m square and 30 m high. The structure has been placed in about 150 m water depth in the northern part of the North Sea.

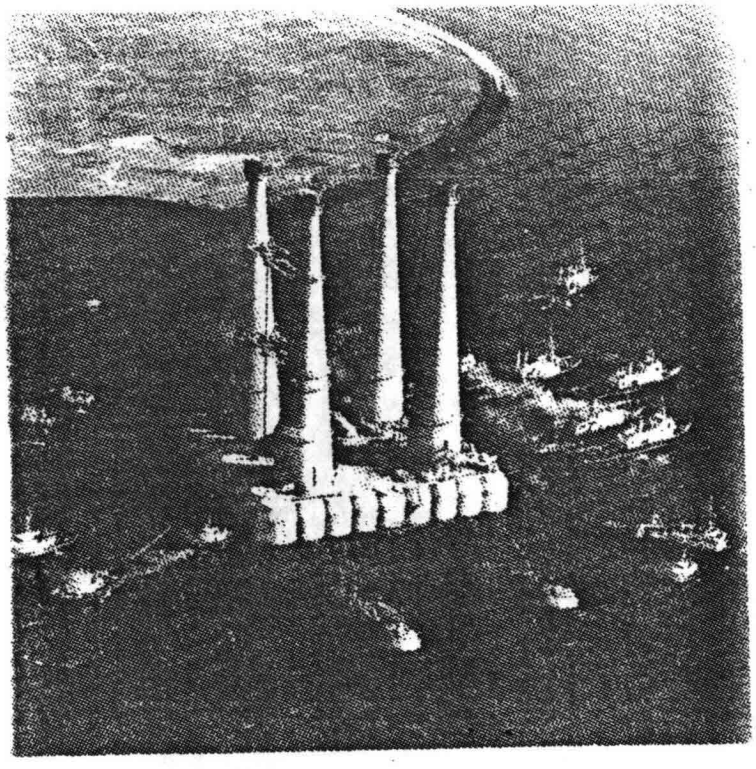
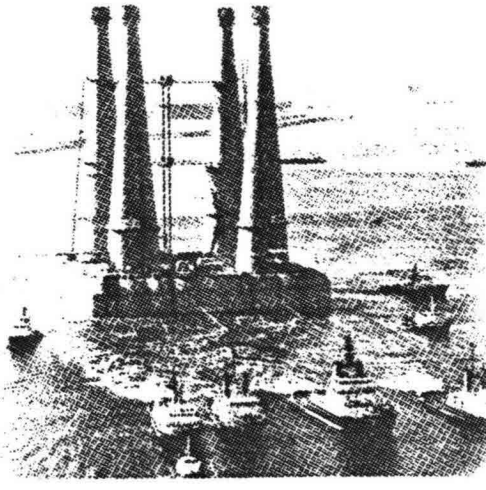


Fig. 25.3. The oil production platform, ANDOC, made largely from concrete, on its way from the construction dock in the Europort area (The Netherlands) to Norway.



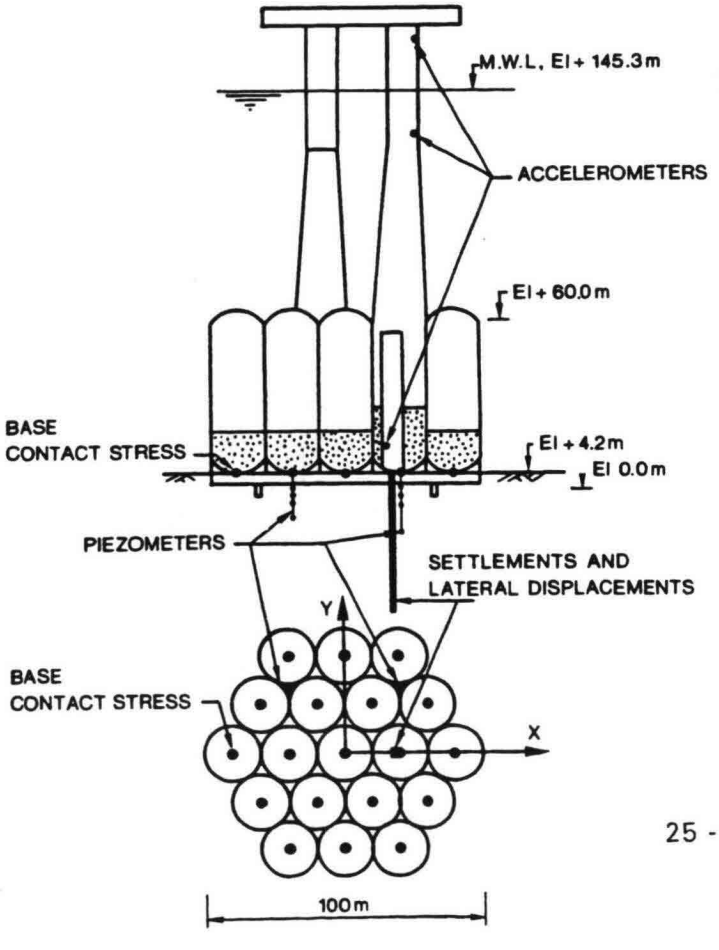


Fig. 25.4. Geometry of the Brent B- structure.

Fig. 25.5. Transport of the jacket for the production platform.

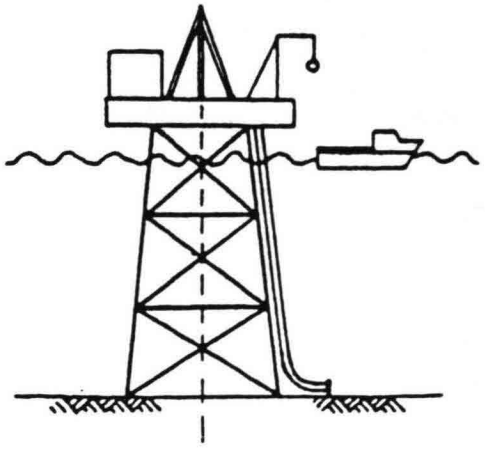


Fig. 25.6. Jacket structure.

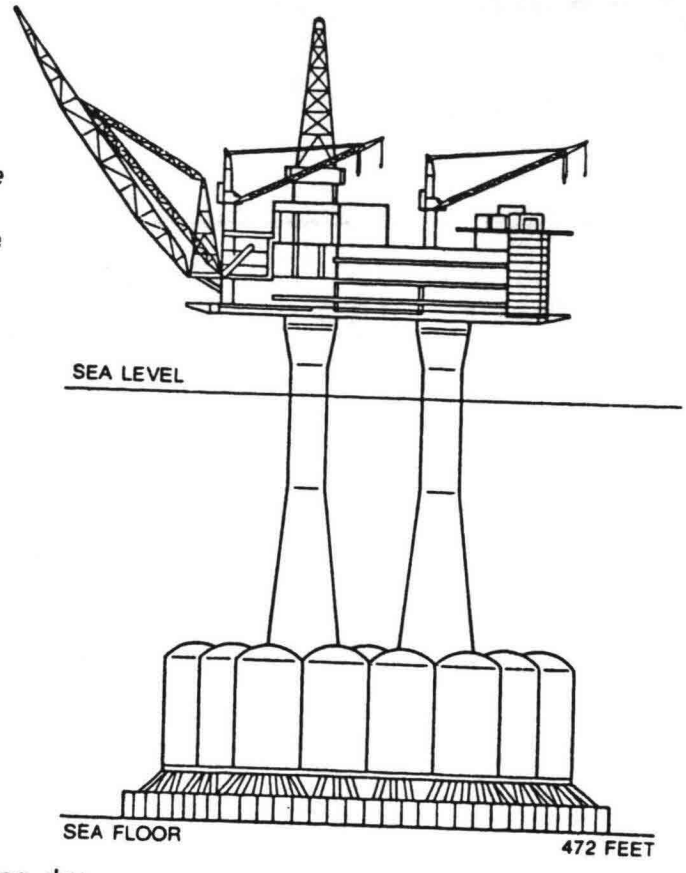


Fig. 25.7. Staffjord B concrete gravity base platform (Norway).
This platform rests on four massive concrete columns with storage tanks at the base.

Emplacement:	Aug. 1981
Overall height:	271 m
Water depth:	144 m
Location:	185 km offshore coast of Norway
Total displacement:	899.000 tons
Weight topside:	41.000 tons
Estimated cost:	US$ 1.8 milliard
Living quarters:	200
Superstructure:	8 stories
Construction:	11.000 man years
Base:	24 cells in a honey comb configuration, each cell walls of 900 mm thick.

Construction of the base was begun in a large dry dock at Stavanger (Stavangerfjord Norway). When the foundation cells were well-advanced to the point where they were buoyant and structurally competent the dry-dock area was flooded and the completed base was moved to sheltered deep water. There the job was completed with the aid of a floating concrete plant. As the construction proceeded the platform was gradually submerged in order to keep the floating level with respect to the work being done.

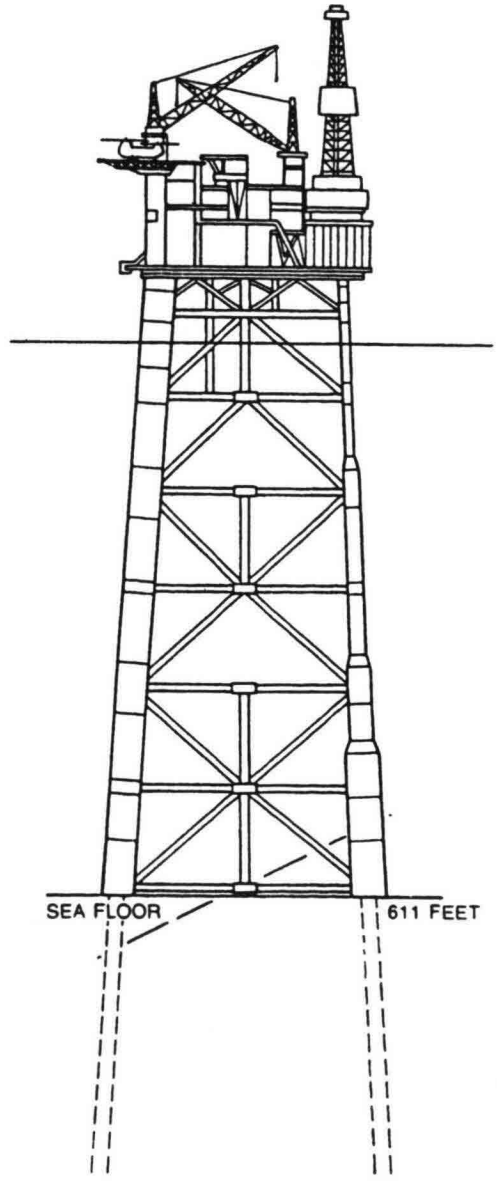


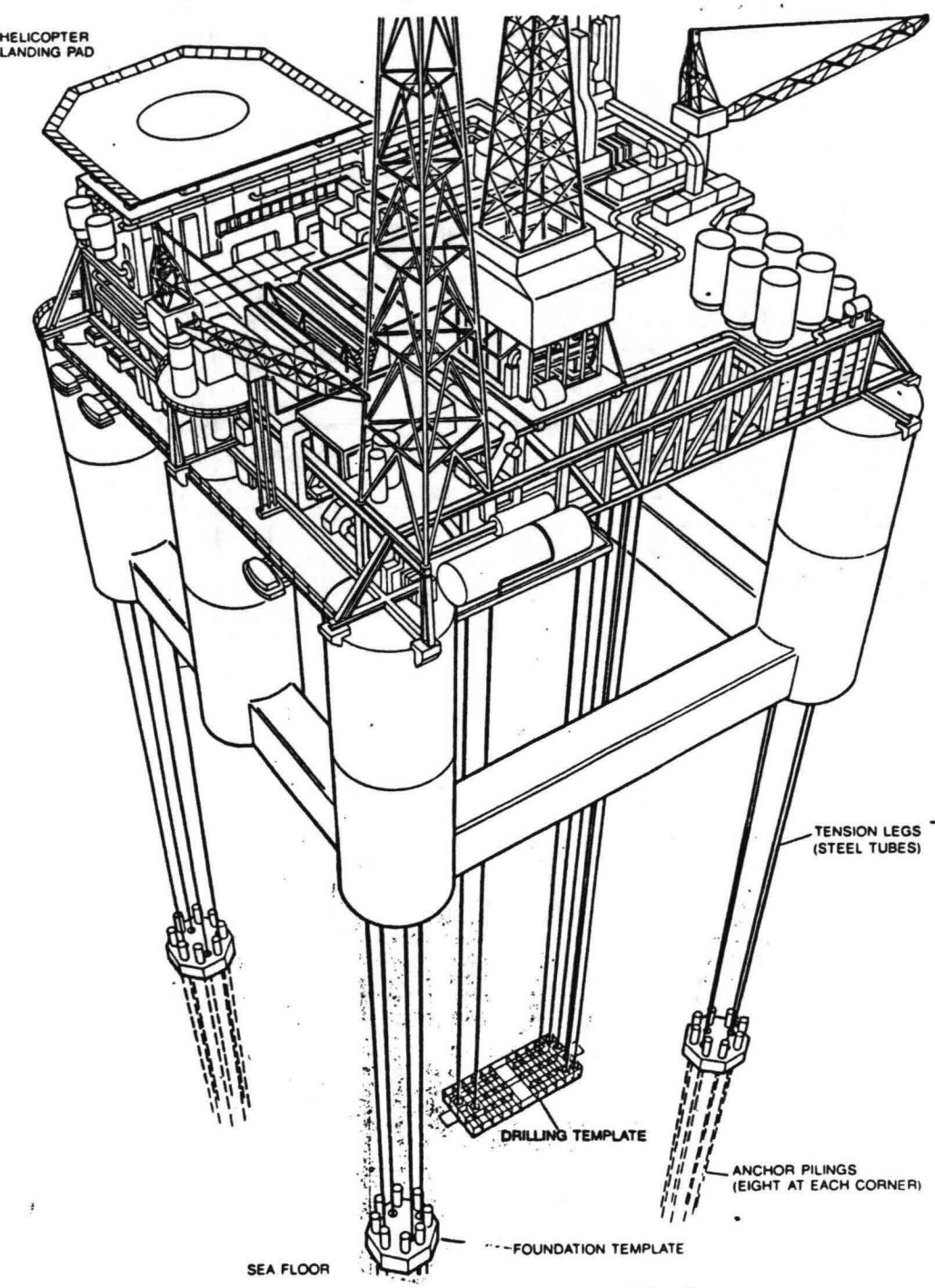
Fig. 25.8.Magnus steel - template - jacket platform (United Kingdom).

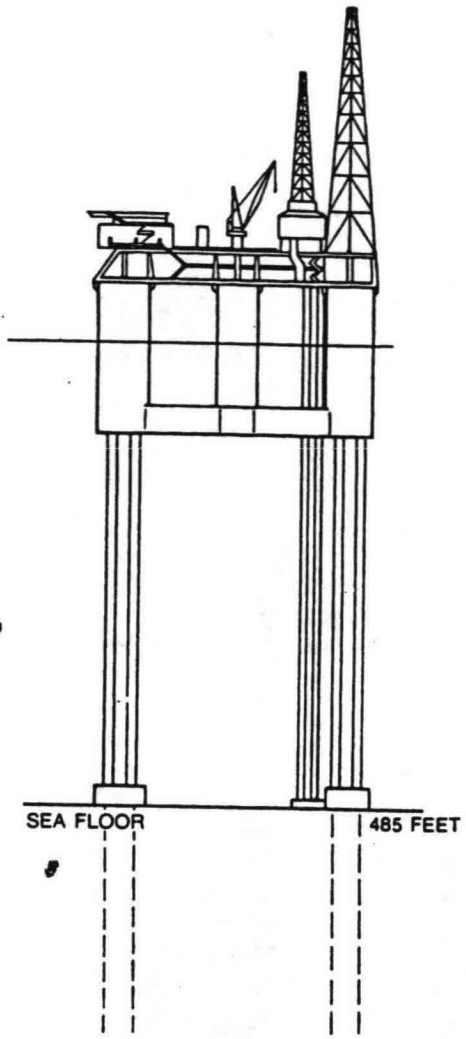
The base of the Magnus-platform (jacket structure) is the heaviest structure of its type yet fabricated: it weighs 41.00 tons(1982).Two of the platforms four legs did incorporate flotation chambers to float the structure from the construction dry dock (Nigg Bay -Scotland) to its site about 250 miles north-east of Scotland.The tower is anchored to the bottom by piles. The estimated cost is about US$ 2.6 milliard or two and a half times as much as the New York World Trade Centre Complex.

Fig. 25.9. Hutton tension-leg platform (TLP) (United Kingdom).

The Hutton structure is the first tension-leg platform (1983) and is situated in the North Sea about 15 miles south-west of Staffjord B. This structure consist of a buoyant is hull which held down by vertical steel tubes at each of its four corners. These slender steel tubes remain in tension under all weather and operating cond itions.

Water depth:	148 m
Max. horizontal motion:	24 m
Emplacement:	1983
Construction site:	Nigg Bay, Scotland.





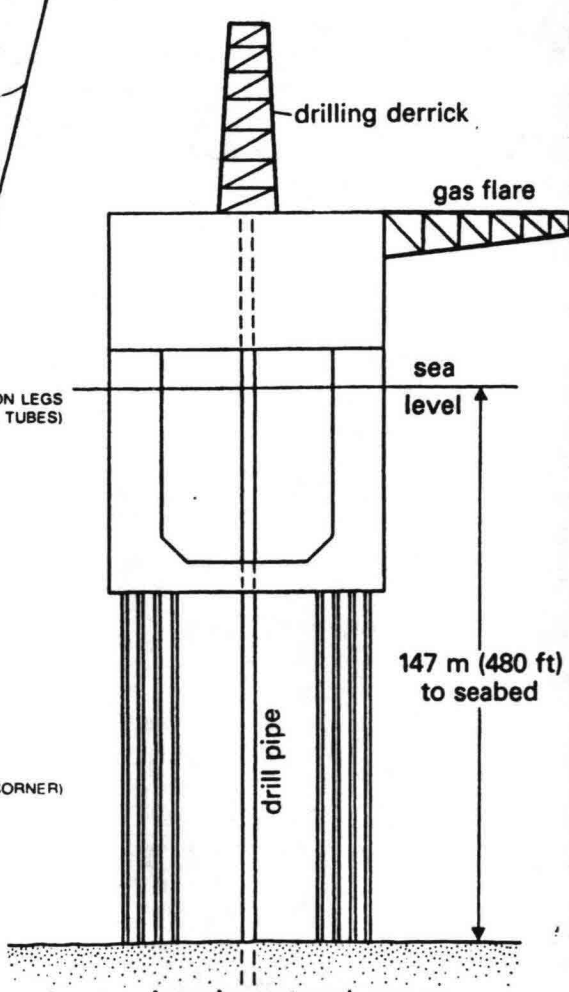


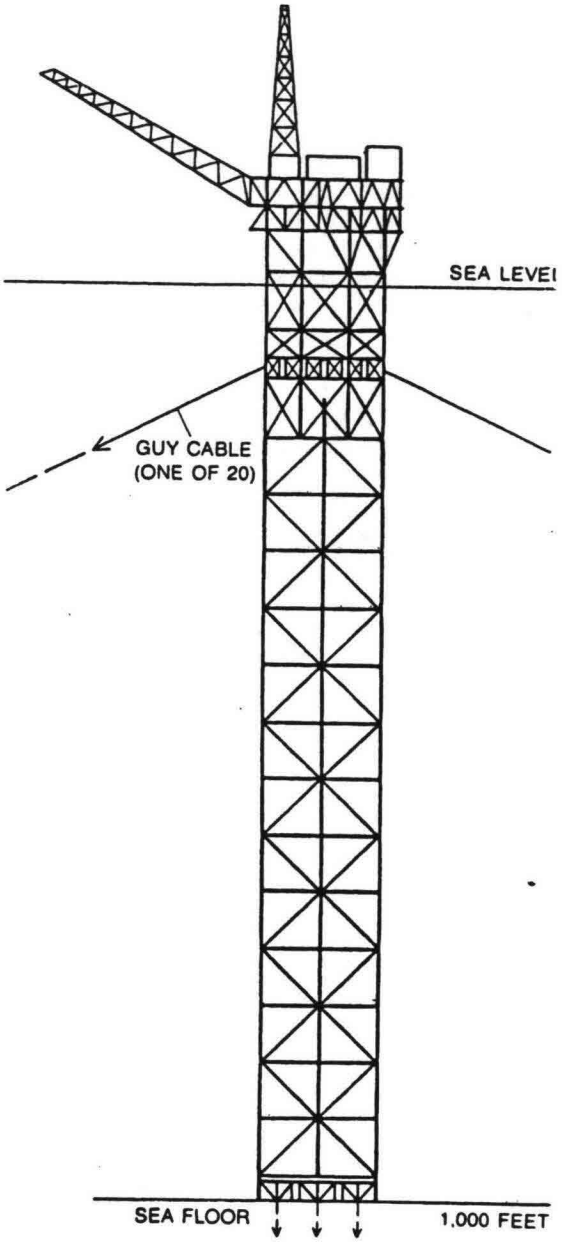
Fig. 25.10. Block 280 Guyed Tower (United States).

The block 280 Guyed Tower the first of its type is designed for service in the Gulf of Mexico in water 300 m deep. It is pinned to the sea floor by a spokelike array of 20 steel cables each one more than 1000 metres long. The tower and its guys 43.000 tons (more than four times as much as the Eiffel Tower).

Estimated cost:	US$ 800 million
Water depth:	300 metres
Steel tower:	36 x 36 m
Max. horizontal motion:	12 m

Halfway each cable a 180 tons weight is attached that under normal sea conditions will rest on the sea bottom. Under extreme storm conditions the weight will gradually lift and allow the tower to tilt. The guy wires are connected to piles (45 m deep driven in the seabed), that act as anchors.

The block 280 guyed is fabricated at Corpus Christi Texas. In 1982 the tower was barged 500 miles to a site in the Gulf of Mexico where it was launched over the side of the barge. The tower was sufficiently buoyant to float low in the water. By controlling flooding is was rotated to a vertical position and brought to rest on the sea floor. The array of 20 radiating cables were then fastened to hold the tower in place. Afterthat the foundation piles were driven and the topside facilities placed.



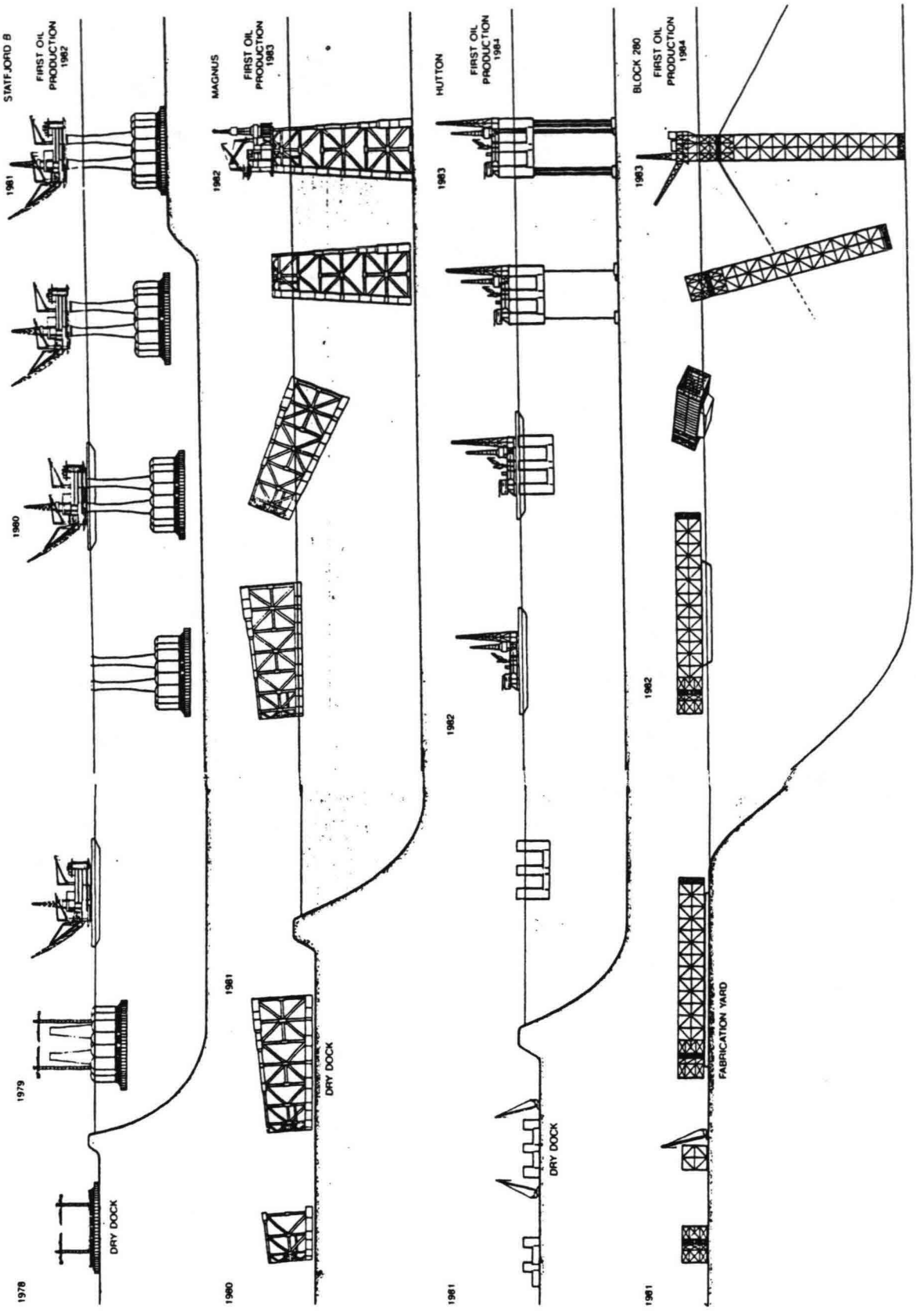


Fig. 25.11. Methods of construction and placement.

26. Man-made islands

26.1. Arctic islands

In August 1975 the first man-made island in the **Beaufort Sea,** North of Canada was constructed by a suction dredger in 7 m of water. In less than 800 hours dredging 1.75 million m^3 of sand were placed using a 500 m long floating pipeline achieving an island of 97 m across and 5 m above sea level. Leading oil companies were battling since 1973 with the problem of drilling for oil in the hostile environment of the Canadian Arctic. Conventional drill rigs are not suited to this environment where they would be crushed under the immense forces of the Arctic ice. Instead artificial islands are built in the summer season on which drill rigs can be sited and operated through the winter. On an average there are 112 days per year if it is not freezing. Between 80 to 100 days a year the sea between the north coast and the Polar ice is open.

The first artificial man-made islands were constructed from sand and gravel that could be dredged nearby. The island is reclaimed by just pumping sand until it comes above water. The long slopes (up to 1:20) of these beach islands break up the impact of the waves. The middle of the island is protected by sand-bags against the moving ice. Many islands have now been built in waterdepths ranging from six to eighteen metres, often severe conditions and even when the ice fields were forming around the dredger. The largest sunk island (Issungnak) reclaimed in 20 meteres waterdepth took three seasons to dredge and involved the use of 5,000,000 m^3 sand. The diameter of the island is 130 m and the top lies 6 m above sealevel (cost:$ 60 million).

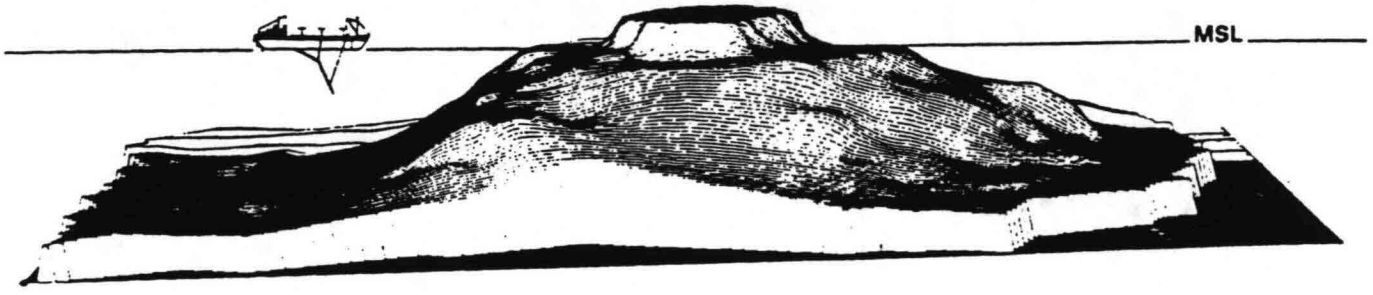


Fig. 26.1. Beach island.

In the western sector of the Beaufort Sea, where the seabed is predominantly made up of clay and silt, this type of island cannot be built without hauling enormous quantities of sand over considerable distances of more than 100 kilometres. The cost involved in doing this would be excessive.

In 1975 was a start made to develop the **Caisson retained island (CRI).** This latest design utilises a steel Caisson which is placed on a sandberm built up to eight metres below waterlevel. Once in postion on the berm the Caisson centre is then filled with sand to complete the island construction.

Advantages:
1. Considerably less sand is required
2. Less maintenance because less erosion
3. Caissons are reusable.

The 12,000 tonnes steel caissons includes eight separate sections each 43 m long, 13 m thick at the base, 8 m thick at the top and 17 m high. These separate sections are held together by a series of 76 mm thick cables. When pieced together the segments form an octagonal structure more than 100 m in diameter.

The **Tarsint island**, waterdepth 21 m, was built using 4 concrete caissons (80 x 15 x 1m) placed on an underwater berm 7 m below the waterline. Only 1,000,000 m^3 sand had to be dredged, but on a distance of 150 km (slope 1:5). The island was completed in one session. The sand could not be dumped direct via the bottom doors of the trailing suction hopper dredgers as this caused a deep hole in the bottom but had to be placed carefully with a dumping pontoon with a vertical discharge pipe.

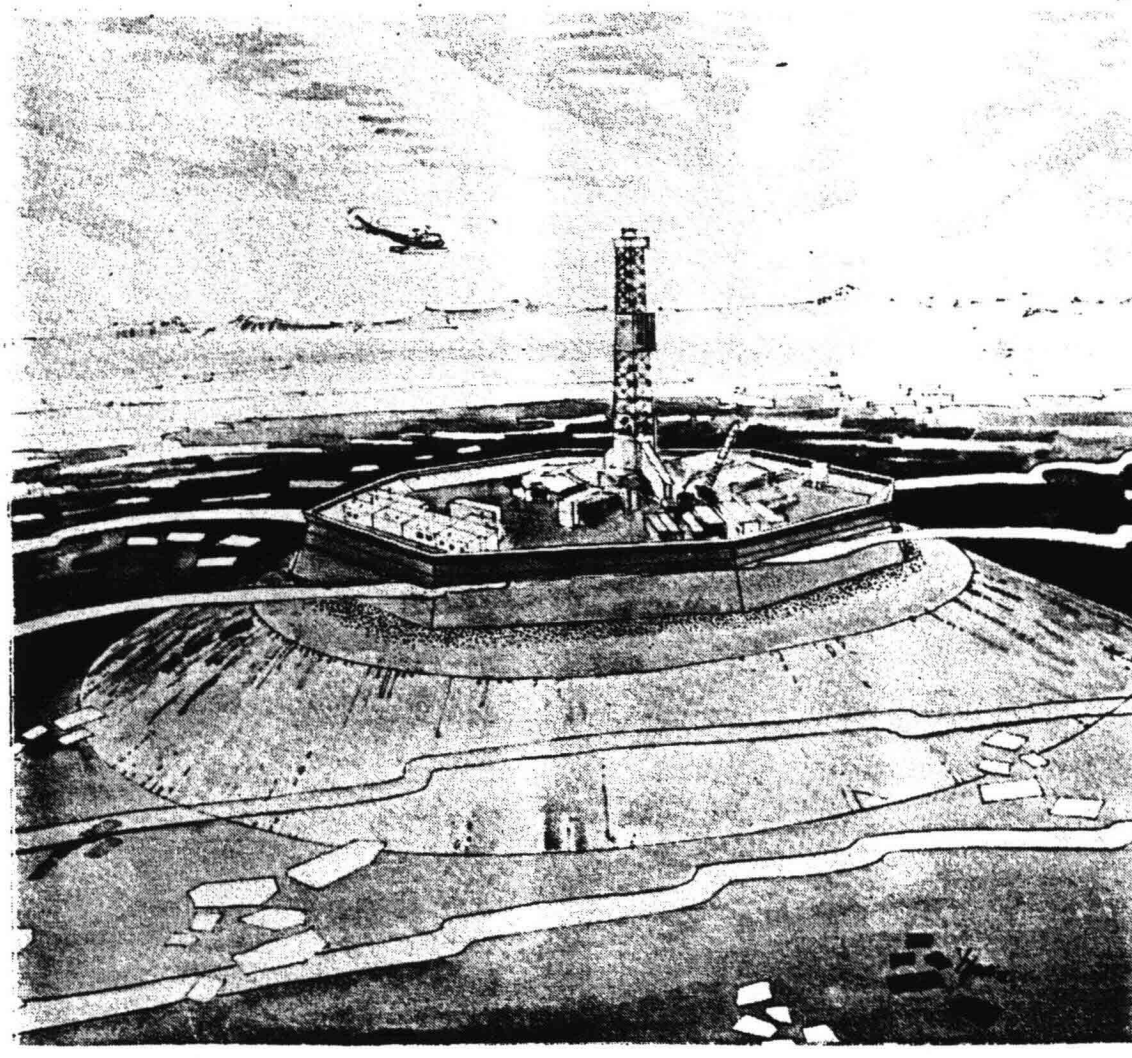


Fig. 26.2. Caisson retained island.

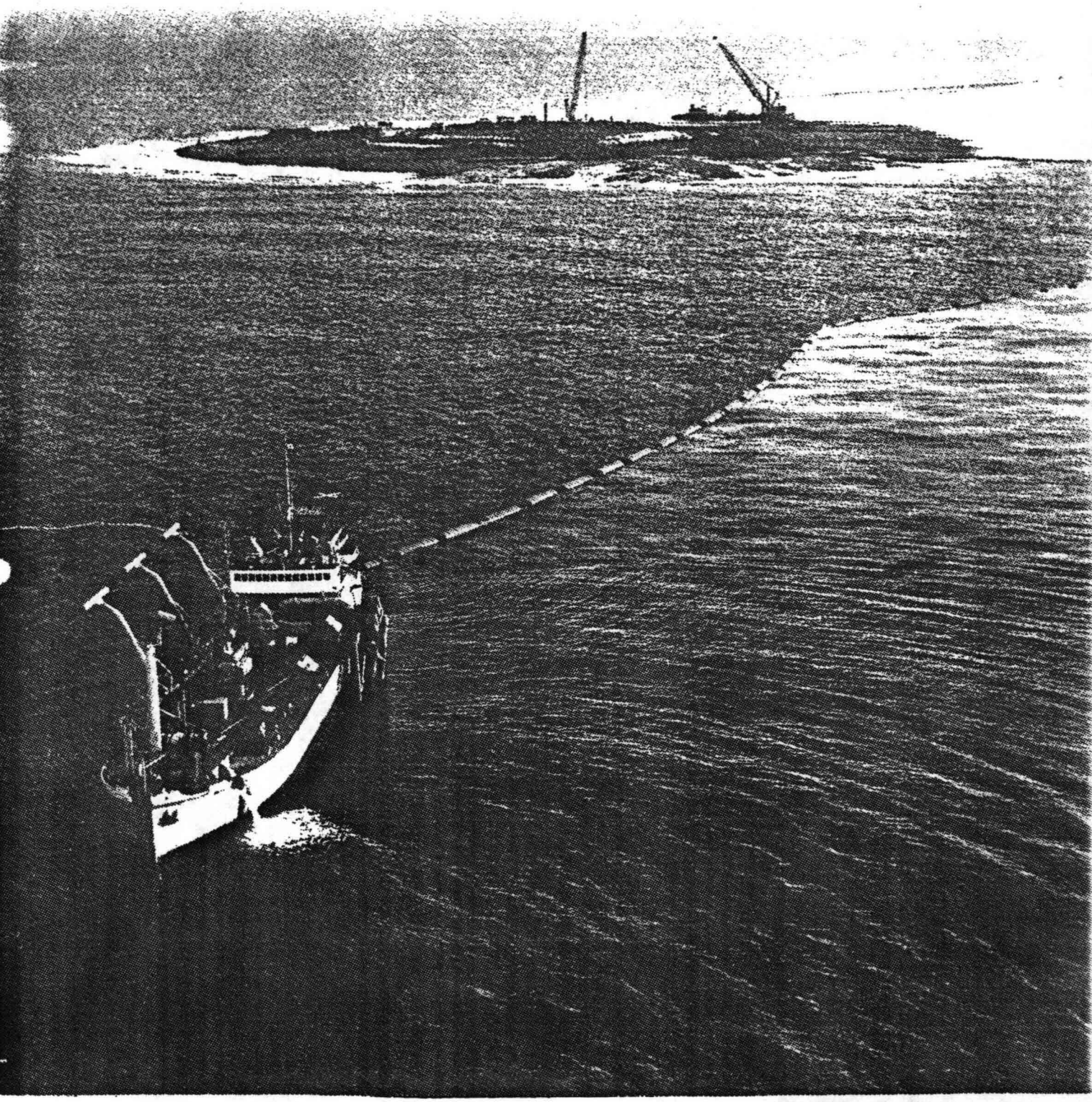


Fig. 26.3.
Reclamation of artificial island in the Beaufort Sea, North Canada
by the 'Beaver Mackenzie' of Royal Boskalis Westminster Dredging Company.

26. 2. Other islands

Middle East Island

In 1982 the first two islands were built in the Arabian Gulf for the Abu Dhabi Oil Company. The islands are reclaimed by dredging in very shallow water. These locations could not be reached by floating drill rigs (water too shallow), and on the other hand it is too deep to site landbased equipment. The answer was to create artificial islands by dredging and reclamation, and to provide deep water access channels for the transportation of the drilling plant. The islands were reclaimed by the sand dredged in these channels.

The islands measure 380 m by 190 m, are 4 m above low waterlevel, and have a work harbour of 200 m by 80 m. The slopes (1:3) are protected by heavy duty synthetic filter fabric, with sandbags (3,3 x 1,4) on top.

Industrial islands - Singapore

To provide more land for industrial use, the Singapore authorities have embarked upon a programme of land reclamation which will see a number of low lying reefs and small islands raised out of the sea to create valuable dry land.

Pulau Busing was reclaimed in 1983 (50 ha - oil storage terminal). Pulau Sakra and Pulau Bahan were reclaimed to one island of 155 ha. The third project awarded by the Jurong Town Corporation was Terumin Peseh (30 ha) (holding station for imported pigs, capacity 25.000 pigs). The island slopes (generally 1:4) are protected by polypropylene filter cloth with stone armouring.

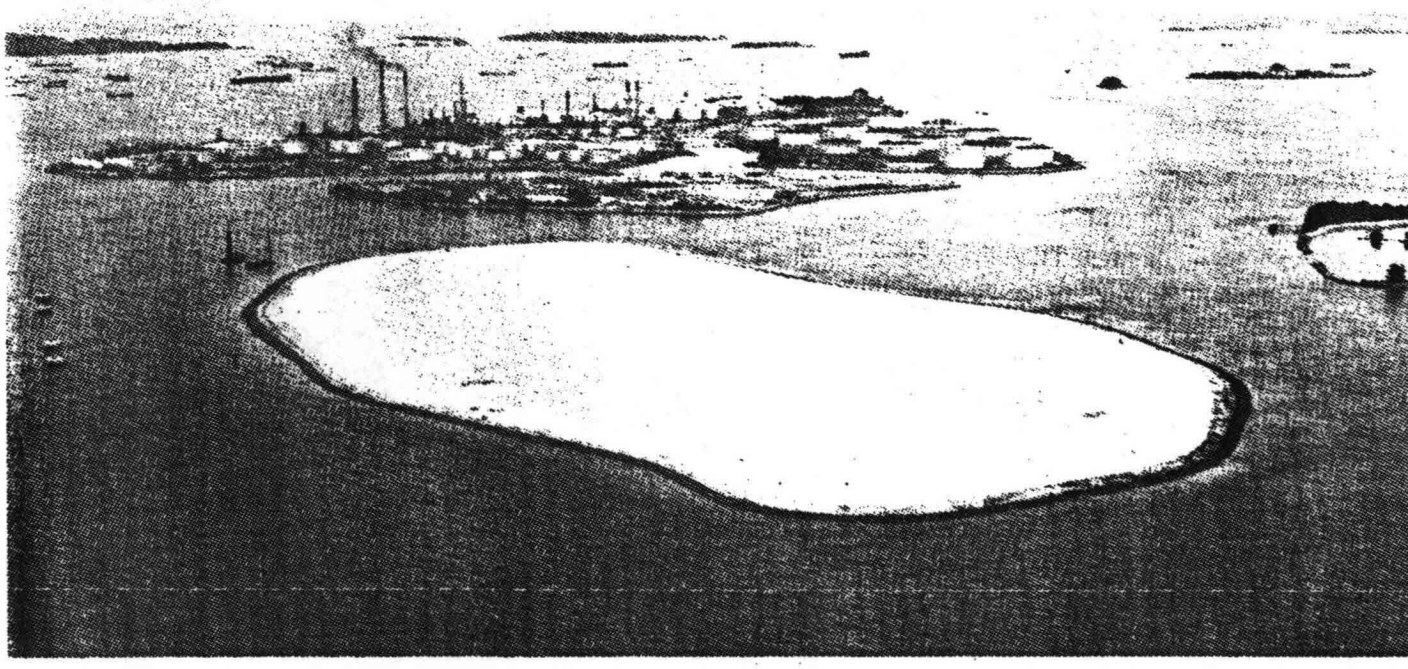


Fig. 26.4. Industrial islands in Singapore.
Reclamation of Pulau Busing, 1983-84, 3,200,000 m^3, stone revetment around the island 130,000 tons. Reclamation of Pulau Sakra and Pulau Bakau, 1984-85, 8,000,000 m^3, stone revetment around the island 250,000 tons.(Jurong Town Corporation).

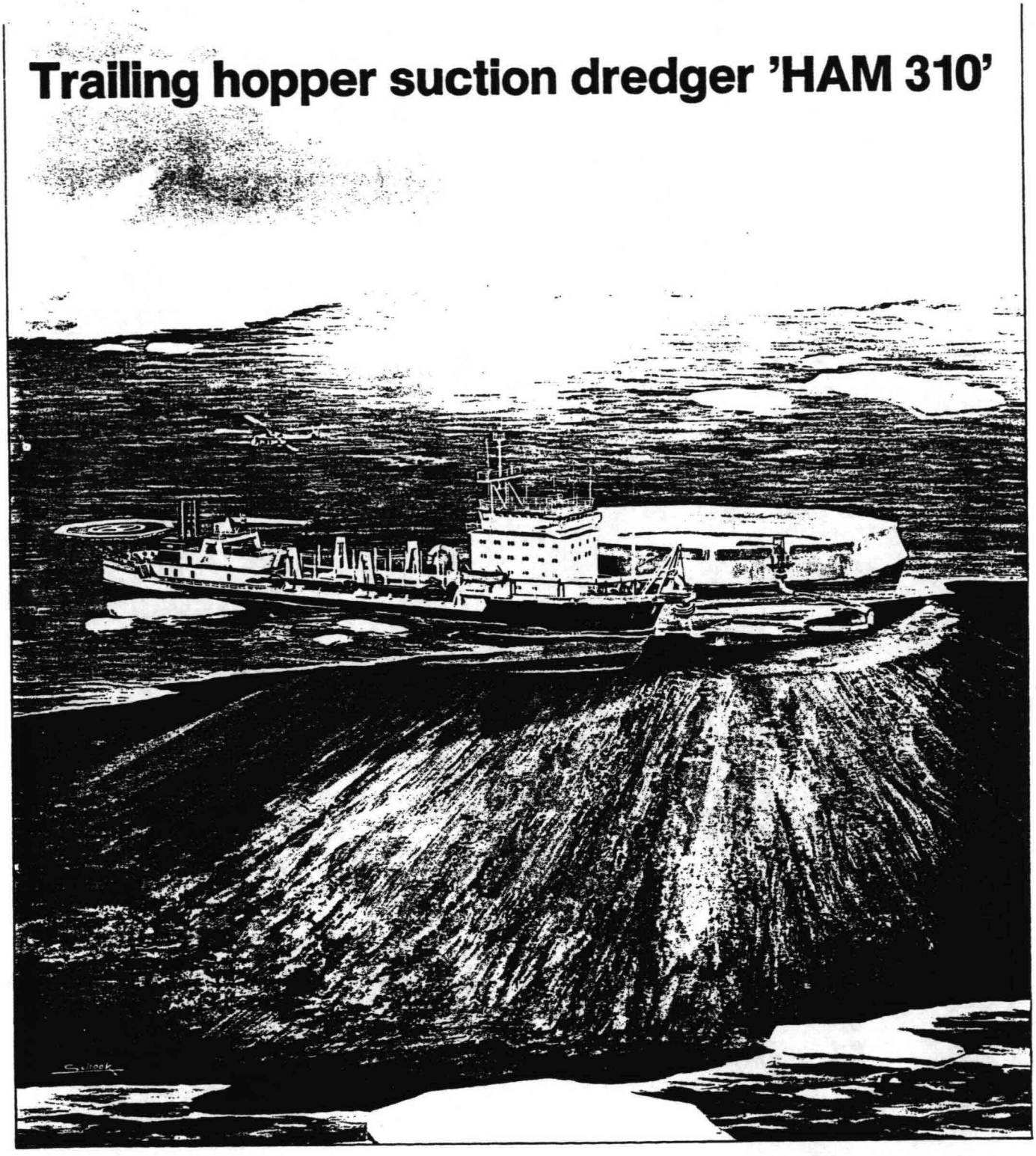


Fig. 26.5. Reclamation of aritificial island by a "Trailing Hopper Suction Dredger" pumping its load (dredged somewhere else) ashore via bow and a floating pipeline.

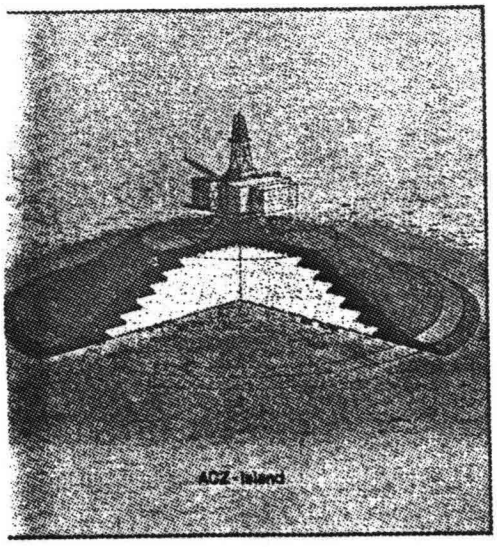
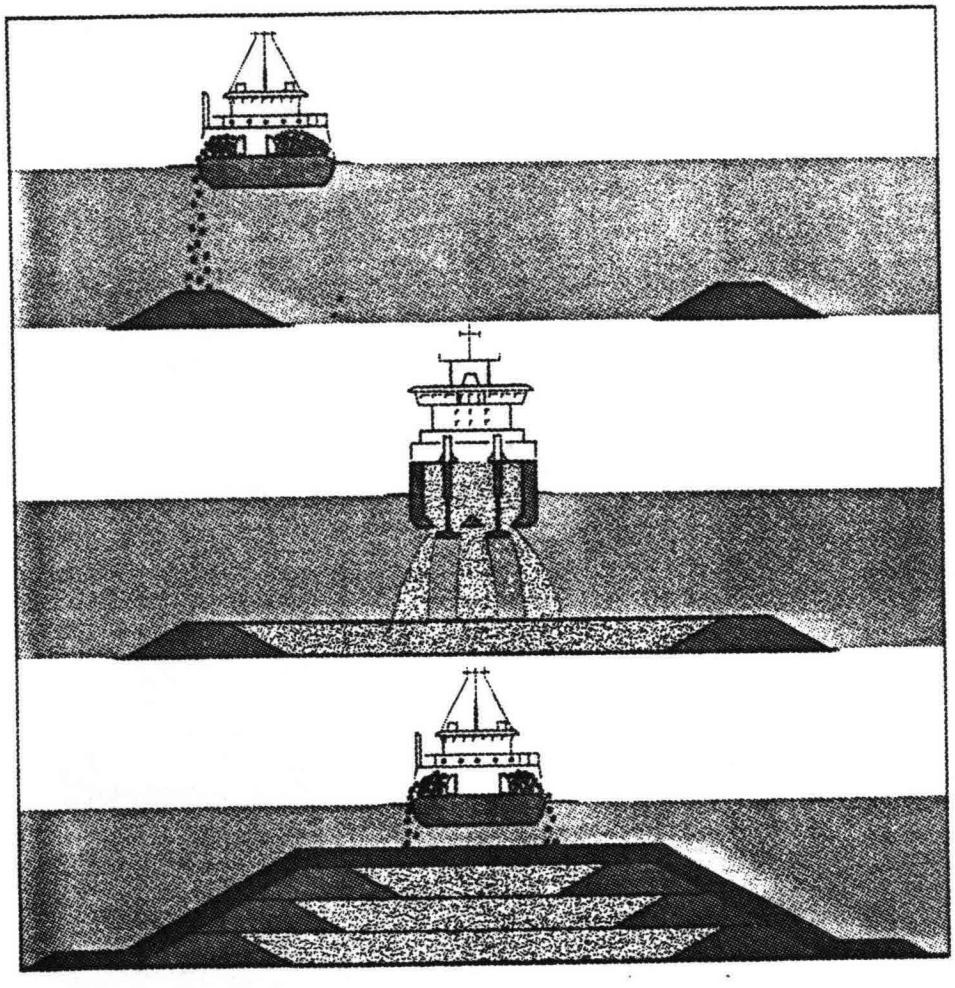


Fig. 26.6. Artificial island concept for Arctic offshore developments (ACZ-concept).



Island construction. The steep island concept.

If constructed solely from hydraulic sandfill artificial islands require enormous volumes of fill. These volumes can be reduced by peripheral bund walls from stone or gravel which have a much steeper slope. The side stone dumping vessel first places a bottom protection mattress or granular filter layers to stabilize the seabed. Then follows pherical berm or toe and bund wall construction. At the end armour layers are installed.

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