waterloopkundig laboratorium
delft hydraulics laboratory

guidance for hydrographic and hydrometric surveys

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Presentation

This guidance is directed towards a practical instruction with respect to preparation, performance and processing of hydrographic measurements in tidal and non-tidal areas. The following kinds of measurements are considered (Chapter 3)

- geodetic measurements;
- position fixing;
- determination of water levels;
- bathymetric measurements;
- velocity measurements;
- discharge measurements;
- sediment transport measurements; and
- salinity measurements.

Attention is paid to a description of current types of instruments (Chapter 2) and to an analysis of sediment samples (Chapter 4). Moreover in Chapter 1 some general consideration are given regarding preparation and organization of surveys in which special attention is paid to the purpose and the required accuracy of the measurements.

1 Organization of survey

1.1 General

In order to carry out a field survey certain necessary preparations have to be made to ascertain a succesful course of the survey and the following is essential:

- An evaluation has to be made of the data required for the project on hand, the desired accuracy and the circumstances under which the survey has to be carried out in order to decide on sort and number of instruments to be used.

- If possible a reconnaissance has to be made of the area involved, to select locations for the measurements and related to this to decide on the number of vessels, manpower, auxiliary equipment and materials, like staff gauges and triangulation beacons and bench marks.
  If available aerial photographs of recent data should be studied, to obtain an overall picture of the survey area and its surroundings.

- In tidal areas data of the vertical tide of the area or nearby area have to be obtained to set up a measuring program related to vertical tide predictions.In non-tidal areas hydrological- and waterlevel data and/or Q-h relation curves of previous years to be collected to set up a measuring program.

- An assesment has to be made of all materials which will be required during the survey, like ballpoints, pencils, wire, nails, generators and batteries etc.. Although among these there are small items of minor costs, they are indispensable and most probably can not be purchased in the field.

- If frequency bound equipment is planned, like walkie talkies, electronic positioning systems, or wave measurement equipment, information about allowed frequencies should be collected and required permits be obtained.

- Measuring forms have to be prepared for all sorts of measurements. To prevent loss of data, all measurement information must be recorded at least in duplicate.

- The logistic support of the survey team must be well organized to prevent discontinuity in measurements due to lack of spare parts, food or fuel.

- All instruments which are selected for the survey, must be thouroughly checked and provided with spare parts.

1.2 Purpose and accuracy of measurements

In the following list a brief review is given of the sort of measurements that will be considered in this manual and their purposes.

Sort of measurement	Purpose of measurements
Geodetic measurement	- to establish a network of beacon- and bench-marks - to relate bench-marks to a reference plane by levelling
Positioning	- to determine the position of the survey vessel of floats during the measurements
Waterlevel measurement	- to obtain waterlevel information in non-tidal and tidal areas - to obtain data for tidal analysis to be used for tidal predictions - to obtain MSL and reference plane - to obtain correlation with current velocities
Bathymetry	- to select and determine cross-sections for discharge measurements - to make a hydrographic chart of the area for navigational purposes but also as boundary conditions for models - to determine dredged quantities - to determine siltation and degradation
Velocity measurements	- to obtain sediment transport data - to obtain the velocity and the direction of the current for navigation purposes - to obtain flow patterns in an area

<u>Discharge measurements</u>	- to obtain rating curves - to determine tidal volumes
<u>Salinity measurements</u>	- to determine salinity and temperature distribution for irrigation purposes - to determine salinity intrusion
<u>Sediment measurements</u>	- to obtain data about sediment transport - to determine bottom composition
<u>Wave measurements</u>	- to determine period and height of waves

Measurements have to be considered as an estimation of the measured quantity while the required accuracy depends on the purpose of the measurement, the absolute precision that can be obtained depends as well on the technical possibilities and limitations as on the phenomena to be measured itself.

As far as the required accuracy concerns no general indications can be given except that, in order to develop mathematical descriptions of certain phenomena (e.g. water movement or sediment transport) the maximum overall technical accuracy is required. This overall technical accuracy depends on (not considering the influence of the phenomenon itself):

a) the instrument and its calibration;
b) experience and quality of personnel;
c) organization and schematization of the measurements (e.g. number of verticals, sampling time etc.)
d) influence of the measuring device on the phenomenon itself.

In general terms it can be said that experience and quality of personnel (item b) plays an important role in all sorts of measurements and items a and c in most of them. The influence of the measuring device on the phenomenon itself only is of any importance when measuring bed load.

Summarizing it has to be expected from a hydrographer (as far as accuracy considerations are considered) that he has:

- a good insight in purpose and use of the measurements;
- a good physical insight in the phenomena to be measured; and
- a good knowledge of the above mentioned technical and personnel possibilities and limitations.

Only when giving all three aspects their due considerations a good, sound and useful survey can be made.

1.3 Measuring program

As a rule the following sequence can be followed:

- Water-level recorders and staff gauges to be installed to obtain water-level information from the very start of the survey which can be essential in the further planning and adjustment of the program.
- Bench marks, triangulation beacons and cross-section marks to be established.
- Measuring and determination of the locations of triangulation beacons, marks en bench marks and the determination of heights of bench marks and their relation to staff gauges and water-level recorders.
- A bathymetric survey to be carried out and/or cross-sectional soundings to determine the bottom configuration of the area and to select locations for the various measurements.
- In tidal areas, current, salinity and sediment transport measurements to be planned during spring and neap tides in the wet and dry season and all other measurements to be planned in between.
  In non-tidal areas the same measurements to be planned in the dry and wet season related to waterlevels.

When the measuring program and the time-schedule is ready a script must be made for the personnel involved in the measurements and a script for the vessels in which instructions are given regarding dates of measuring, kind of measurement, times of departures, times of anchoring in cross-sections and the duration of each measurement.
Each captain of a vessel should be supplied with a copy of the script.
In case the program is changed or times have to be altered due to circumstances, the personnel should be notified in time.

The observers must be divided in groups and clear-cut instructions have to be given regarding their tasks and obligations.
It is most important to give them an outline of the purposes of the project on hand because nothing is more frustrating then to do a measurement without knowing the purpose.

1.4 Logistic support

One of the most essential parts of the whole survey is the logistic support, if this support fails the whole survey can become a failure and all costs and time for preparation have been in vain.
The logistic support consists of the following parts:
- food supply
- fuel supply for the vessels and speed boats
- medical supply
- supply of spare parts for instruments and auxiliary equipment like outboard motors, generators etc.
- distribution of letters from home.

Before commencement of the survey this support should have been fully arranged and checked so that no delay in or discontinuation of the measurements will occur.

1.5 Measuring forms

In order to note the measured data for each type of measurement special measuring forms must be prepared.
These forms should be made in booklets in which each second page is perforated.
While noting down the data a carbon paper is laid in between the perforated page and the non-perforated page so that all data are copied.
The perforated sheets are then taken out of the booklet after the calculation has been done and stored in a different location than the data still in the booklet. This is done to prevent loss of all data due to fire or any other mishap.
For the same reason the booklet and the perforated sheets should never be taken from the field to the office by one and the same person.

1.6 Preparation of instruments

Before commencement of each survey, instruments must be checked and should be overhauled and calibrated. More detailed information is given in Chapter 2.

1.7 Vessels

When vessels are to be selected for the measurements the following points should be taken into account:

- When vessels are used in rivers the draught should not exceed 1.50 m.
- Vessels must have sufficient working space preferably on the fore deck.
- If Ott Current-meters are used a strong davit has to be installed with sufficient height to attach the largest Ott winch to it and with sufficient span so that the Ott Current-meter is at least 1.5 meter free of the side of the vessels when the davit is swung out.
- Each vessel must be provided with a heavy anchor with a chain anchor line (rope lines to be condemned as vessels will drag their anchors during the measurements in currents exceeding 0.75 m/sec).
- Cooking facilities on board.
- Sufficient accomodation so that the observers can take a rest in turns.
- Safety equipment to be on board, like life-jackets and life-buoys in sufficient numbers.
- All vessels to be provided with day signals: 2 red balls and 1 white diamond shaped signal each with a diameter of 0.6 m (see Figure 1.1.1 a and b) and night signals: 2 red and 1 white light which should be visible all around.
- Sufficient lights to be on board for night observations either battery powered or generator-powered, if this can not be supplied the vessels must be provided with high pressure oil-lamps (Colemann).

Figure 1.1.1a Survey vessel with day signals

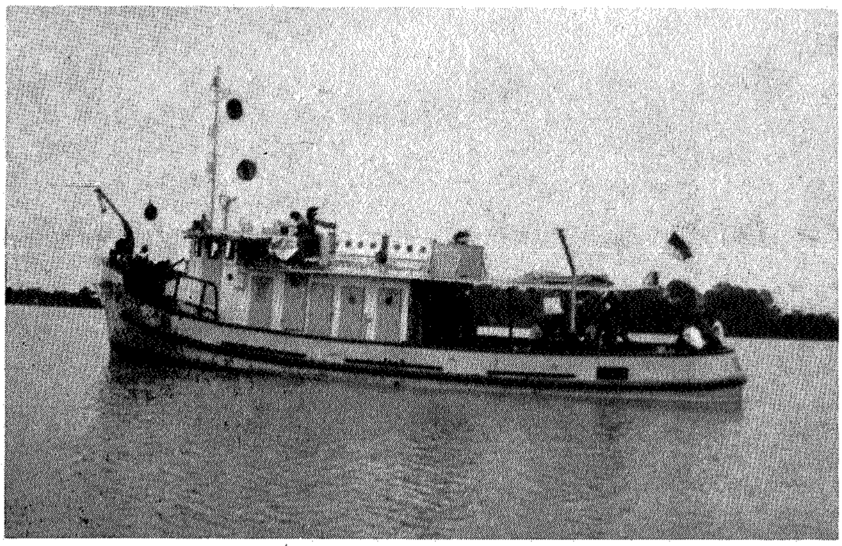


Figure 1.1.1b Survey vessel with day signals

1.8 Safety

In case that measurements are to be carried out in navigation channels, local authorities like the Harbourmaster or Chief Pilot, should be informed in time about the locations and positions where survey vessels will be anchored to carry out their measurements during the surveying period.
This has to be done in order to prevent collision between passing ocean-going vessels and the survey vessels, which consequently will result in loss of live and equipment.
The Harbourmaster should be requested to order his pilots to pass the survey vessels with utmost caution and at low speed.
If feasible, radio-communication should be maintained between the survey vessels and between the survey vessels and their base camp or office.

2 Instruments

2.1 General

For hydrographic and hydrometric surveys various kinds of instruments are needed. The instruments to be used for a certain survey have to be selected, depending on:
- sort of measurement
- purpose of measurements
- circumstances under which measurements have to be carried out
- type of vessels available for the survey.

The sort of measurement determines the kind of instrument like
- geodetic-equipment for triangulation, tachymetry, levelling
- current-meters for local current velocity and direction measurements, discharge measurements and velocity distribution determination
- water-level recorders for vertical tide measurements
- echosounders for bathymetric survey
- salino-meters for salinity or conductivity measurement
- wave-height meters for wave measurement
- position-fixing instruments for establishing the vessels position during soundings and current-velocity measurements
- sampling equipment for bottom sampling, water sampling or sediment transport
- miscellaneous

The purpose of measurements, the sphere of interest, the required accuracy, the local circumstances, the funds and the type of boats available determine which type of instrument has to be used:
- geodetic-equipment: theodolites, tachymetric instruments, infra-red distance meters, levelling instruments, measuring tapes
- current-meters: floats, pendulum meters, propeller current meters

- water-level recorders: pneumatic, float-well type, pressure transducer type, staff gauges
- bathymetry: 200 kc echosounder, 30 kc echo-sounder, hand lead
- salino-meters: inductive type, resistor type (2-4-7 electrodes type), water-sampler
- wave-height meters: wave measurement buoy, wave pole
- position-fixing: sextant, theodolite, range-finder, electronic instruments
- sampling-equipment: suspended-load samplers, bed-load samplers, bottom-samplers, water-samplers
- miscellaneous: tools, stop-watches, binocular, winches, beacon material etc.

It is necessary therefore to know of each instrument, its principle, advantages and disadvantages, applications and preparation for field use.
The following chapters will give the above-mentioned information.

2.2 Current meters

2.2.1 Propeller current meters

Principle. The propeller current-meters work on a principle that a propeller of certain pitch is turned by the water particles passing along. Each revolution of the propeller corresponds with a travelling distance of a water particle equal to the pitch of the propeller.
The number of revolutions in a certain time-lapse gives then the velocity of the current in a unit of time, according to the calibration formula of the propeller.
There are instruments with the propeller axis in the direction of the current (Ott-propeller instruments) and instruments with the axis perpendicular to the current (Price-cup current meter).
The direction (and also the magnitude) of the current fluctuates due to turbulence.
Instruments with a vertical axis (perpendicular to the direction of the current) are not critical to the direction of the stream.
For instruments with horizontal axis, however, propellers have been developed which measure within a range of about 40^{o} (or even 60^{o}) the component of the velocity perpendicular to the cross-section.

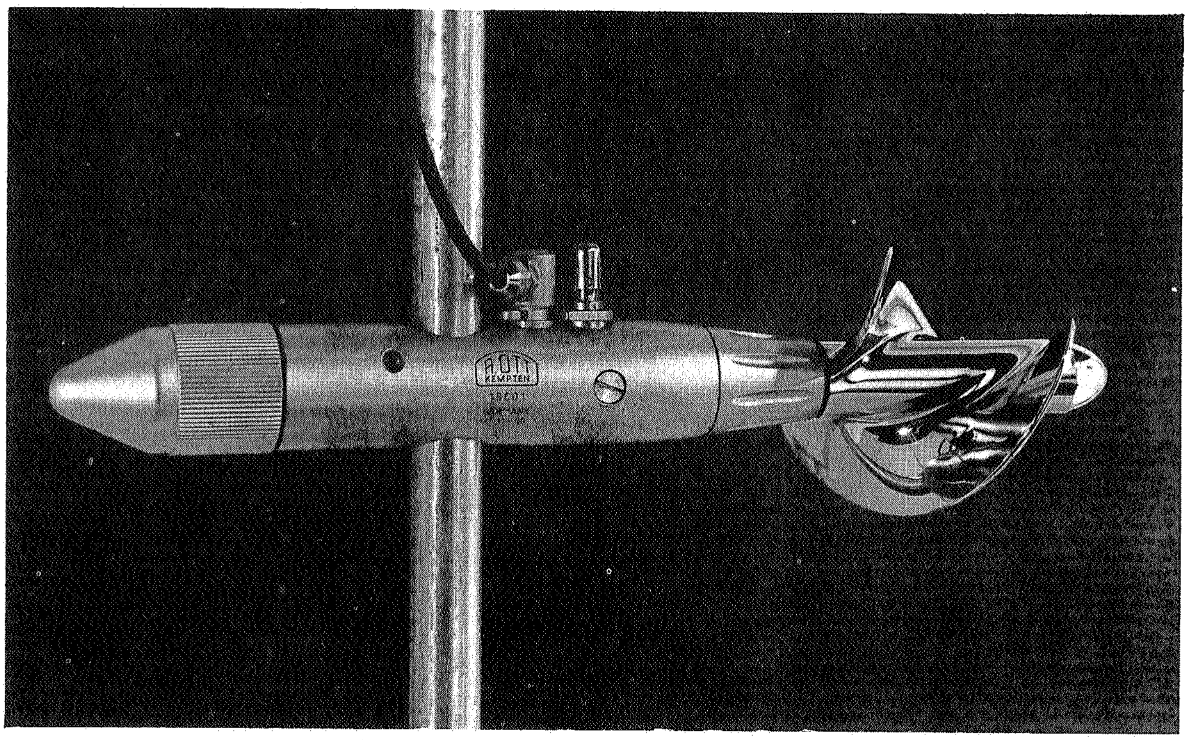


Figure 2.2.1 Ott-C31 propeller instrument

Advantages : Propeller current-meters are excellent tools for fast and accurate work, provided careful maintenance is done and great care is exercised when using the instrument and if the calibration of the propellers is checked regularly. Calibration formula can easily be transformed in a velocity graph, which makes the elaboration more easy.

Disadvantages : No current-direction can be obtained.
The current-meter will be influenced by the pitching and rolling of the survey vessel, measuring other components than the current velocity due to river flow. The components increase or decrease the number of revolutions of the propeller so that no true velocity is obtained. Frequent calibration tests have to be made.

Application : The Ott-C31 current-meter can be used either suspended from a wire with a weight of 25 or 50 kg or connected to a rod. In the first version, it can be used for measurements in estuaries and rivers.
In the second version, it can be used for measurements in small canals either by wading or from a small craft or a bridge.

Preparation : Propellers should be calibrated, this can be carried out by the Ott-factory or can be done in a towing tank of hydraulic laboratories.
Calibration graphs to be prepared.
Sufficient spare parts like ball-bearings, packing rings, magnetic switches etc., must be provided and Ott-propeller oil should be supplied so that each current meter can be replenished every 6 hours of observations.
This period of 6 hours may well be shortened to 4 hours in case velocities exceed 1 m/sec and the water is full of sediment.
Be sure that original Ott oil is used in the field and no substitute is allowed as Ott propeller oil is stable of viscosity under all circumstances and temperatures and guarantees smooth running of the propeller, while other kinds of oil may damage the ball-bearings.

2.2.2 Pendulum meter "Planeta"

Principle. This instrument is based on the principle that a resistance body, suspended by a thin nylon wire from a measuring device, causes a deflection of the wire from its vertical position due to the current velocity.
The angle of deviation is read on the cupola and is a measure of the current velocity. After applying the corrections for wire bending the velocity can be read from a calibration graph.
In the same manner the horizontal angle in regards to the longitudinal axis of the vessel can be determined and in combination with the ships compass, the azimuth of the current is obtained.

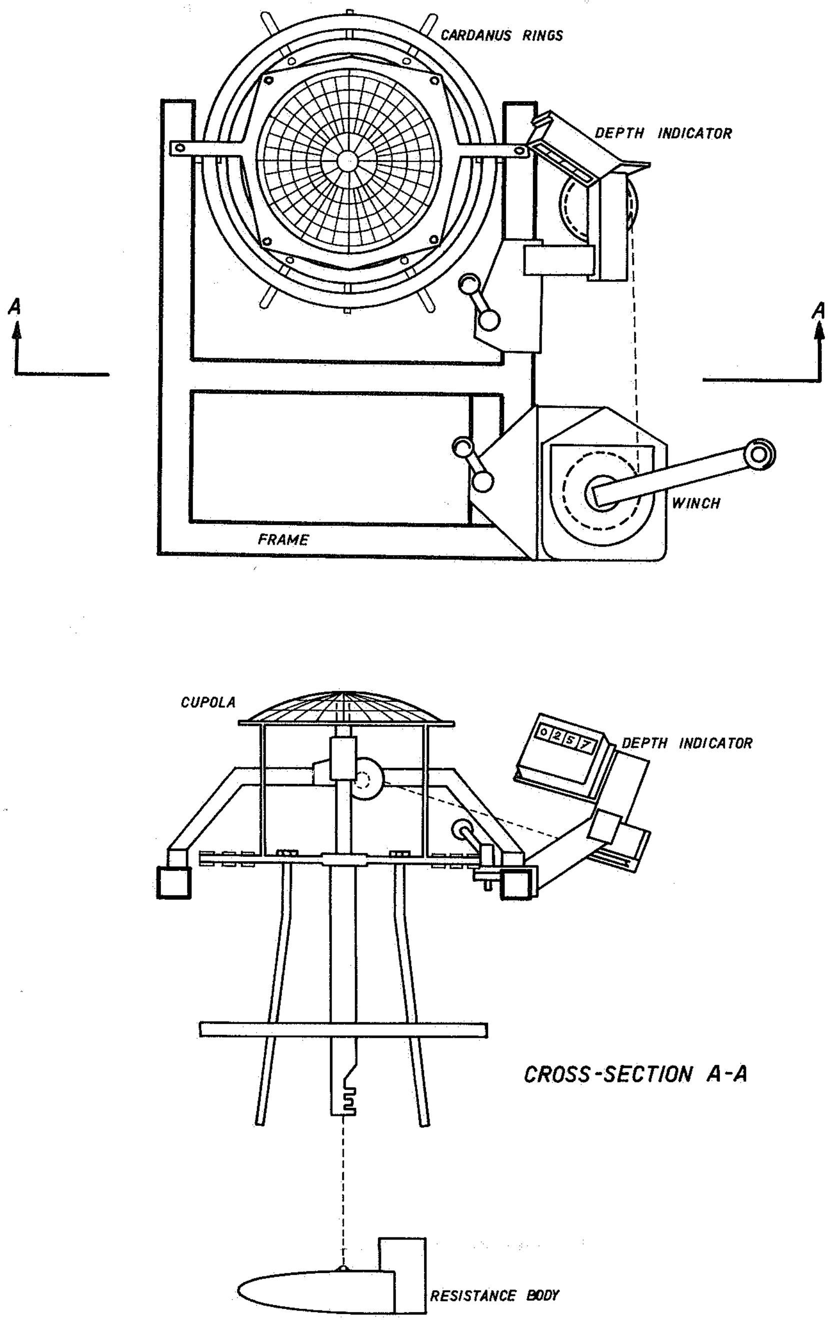


Figure 2.2.2 Planeta

<u>Advantages</u> : With this instrument velocity and direction of the current can be obtained and in tidal areas density currents can be detected.

The instrument is completely mechanical and sturdy, small repairs can be made in the field.

<u>Disadvantages</u> : The correction for wire-bending makes the calculations to obtain velocities rather cumbersome, although graphs are supplied. An experienced observer will have no difficulties.

With wave action the vessel may roll and pitch and sway, causing difficulties while reading the angle deviations on the cupola.
Due to wave action and floating debris etc. in the river, resistance bodies can be lost.

Application : The Planeta can be used for measurements in rivers and estuaries and can be utilised from any craft.
Velocities between 0.10 m/sec up to 4.2 m/sec can be measured.

2.2.3 Hand pendulum meter "K.L.M."

Principle. The hand pendulum "K.L.M." is based on the same principle as the Planeta but no current-direction can be obtained due to the difference in the measuring device.

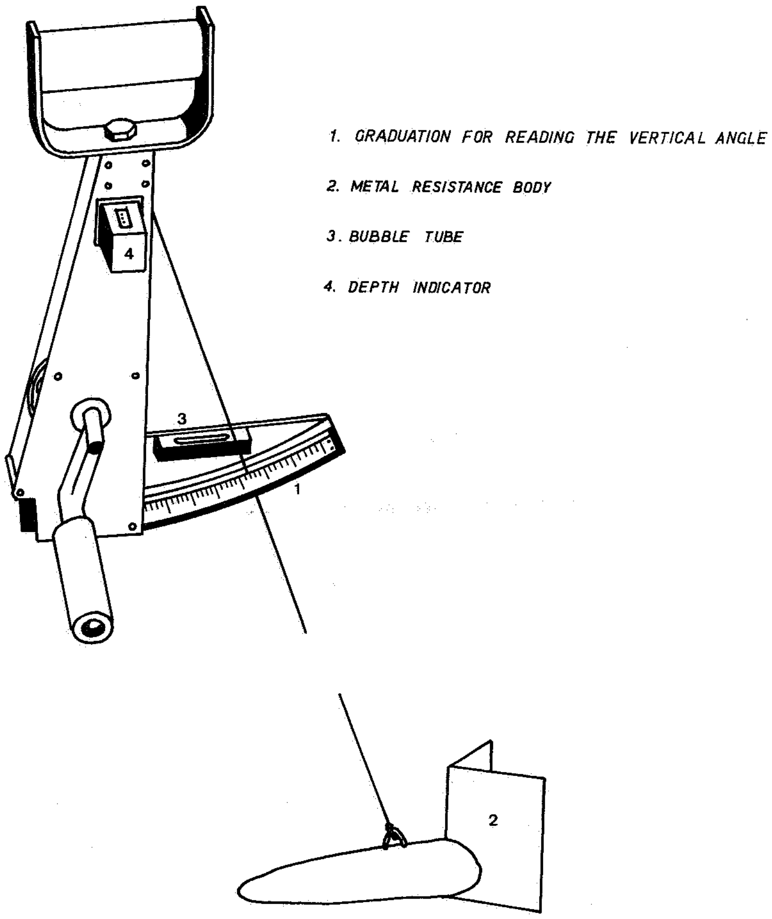


Figure 2.2.3 Hand pendulum

Advantages : The instrument is small, light weight and extremely useful for reconnaissance surveys.

It can be used manually from all sorts of crafts and from bridges e.g..

Disadvantages : Calculations of velocity are rather cumbersome just like the Planeta.

Application : The instrument can measure velocities from 0.10 m/sec up to 2 m/sec. It can be used in rivers and small canals and is useful as a spare current-meter to be utilised when one of the larger current meters breaks down.

Preparation : The nylon wire of the pendulum meters should be checked on abrasion and wear, and if in doubt, new wire must be put on the instruments or the affected part should be cut off; spare nylon wire to be supplied.
All moving parts to be oiled.
Resistance bodies to be checked on damages and provided with shackles.

2.2.4 Electro-magnetic water current-meter

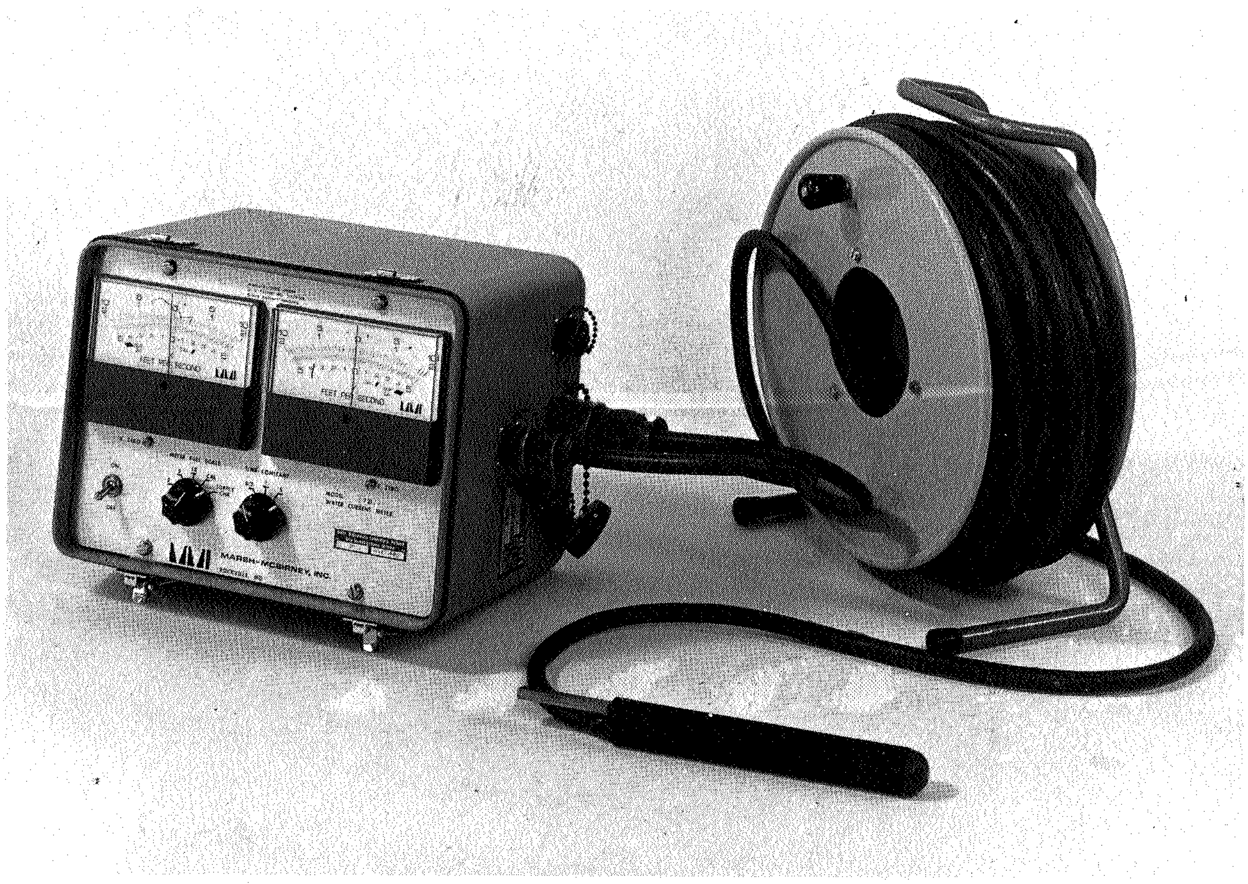


Figure 2.2.4 Marsh-Mc. Birnie electro-magnetic current meter

Principle of operation:

The operation of these current-meters is based on the Faraday principle of electro-magnetic induction.
Simply formulated this principle states that "a conductor" (the water) moving in a magnetic field (generated from within the current-meter probe) produces a voltage that is proportional to the velocity of the flowing water.

The magnetic field, created by an AC electro-magnet within the probe, is produced parallel to the longitudinal axis of the cylinder.
Electrodes placed in the wall of the cylinder detect the voltages caused by water flowing past the probe in a plane perpendicular to the flow probes axis. If two pairs of electrodes are used, the velocity vector can be resolved into an X and a Y component.
From this the magnitude and direction of the flow can be reconstructed.

Marsh - Mc Birney model 721 electro-magnetic water current meter

This instrument consists of a probe with cable and a signal processor display unit, housed in a portable case.

Range of the instrument: 0 - 10 ft (in 3 steps)
Accuracy : $\pm$ 1 cm/sec and $\pm$ 2% of reading
Power : internal rechargeable batteries for 40 hours continuous use, battery charger built-in.

Advantages :
- no moving parts
- minimum of maintenance
- small probe
- battery operated
- two axes
- not sensitive for polluted water.

Disadvantages :
- to obtain magnitude and direction of flow, the probe has to be fixed under water
- X and Y readings still have to be converted to magnitude and direction.

Note:

Nowadays, portable instruments are also available fitted with a built-in electronic compass, and a one axis probe, as to give a direct read-out of flow magnitude and direction with respect to the magnitude north.

Application : Electro-magnetic current-meters can be used for discharge measurements and combined with a telemetring system for autonomic use, that is, that the instrument can be installed attached to a structure or in a frame and can telemeter the information about direction and current-velocity to a receiving station.

2.2.5 Floats

Principle. A floating body is moved by the current and the velocity is derived from the travelling distance divided by the time lapse.

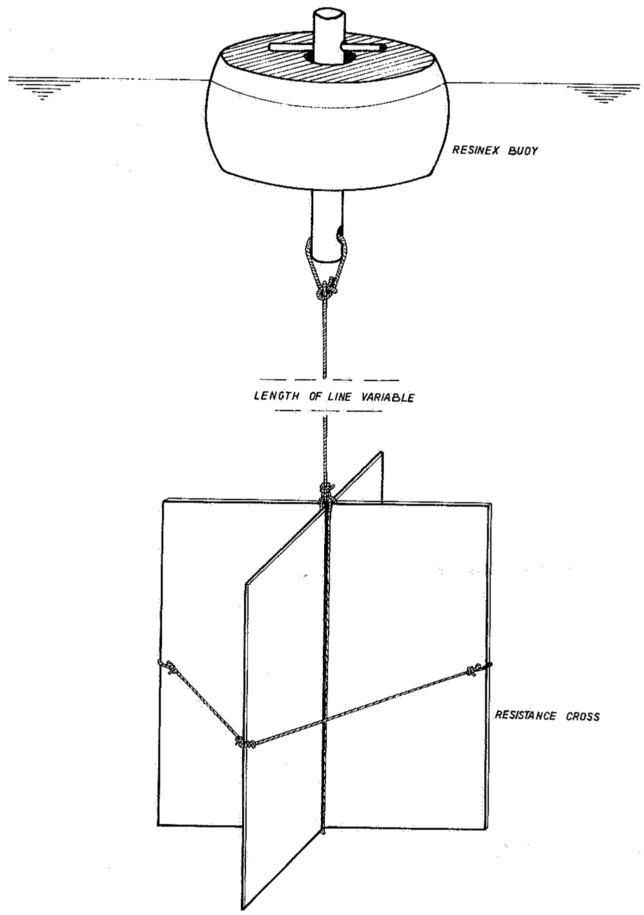


Figure 2.2.5 Float

Advantages : Floats can be made locally in various designs, each design for a different purpose.

Disadvantages : The instrument is only suitable for rough estimations. The velocity distribution should be known from other measurements to relate the measured velocity to the mean velocity to evaluate the discharge.
By strong winds and waves, the float-path can be affected, and during float measurements,wind observations should therefore be carried out, in order to calculate the wind drift.

Application : Floats can be used for emergency purposes if a current-meter breaks down and has to be repaired.
But in some cases it may be necessary to use floats, as no other instruments can be used (wave action, current paths).
With floats with a resistance-cross set at a certain depth, the average velocity and direction at this depth can be obtained by taking the time and position of dropping and recovery.
With staff floats the average velocity and direction can be obtained for a water column equal to the submerged part of the float.

Preparation : Floats should be prepared before commencement of a survey and resistance crosses should be not less than 50 cm wide and high. The float should be such that only a small part is protruding above the water in order to lessen the wind-influence.

2.3 Water-level recorders

2.3.1 Ott XX water-level recorder

Principle. A float attached to a wire with at the other end a counter-weight is led over a pulley of the recorder. The float is located in a float-well and the float follows the vertical movement of the water. This vertical movement is transformed into a horizontal movement of the writing-stylus by the pulley. The writing-stylus records this movement on a registration paper which moves perpendicular to the movement of the pen, drawing a tidal curve.

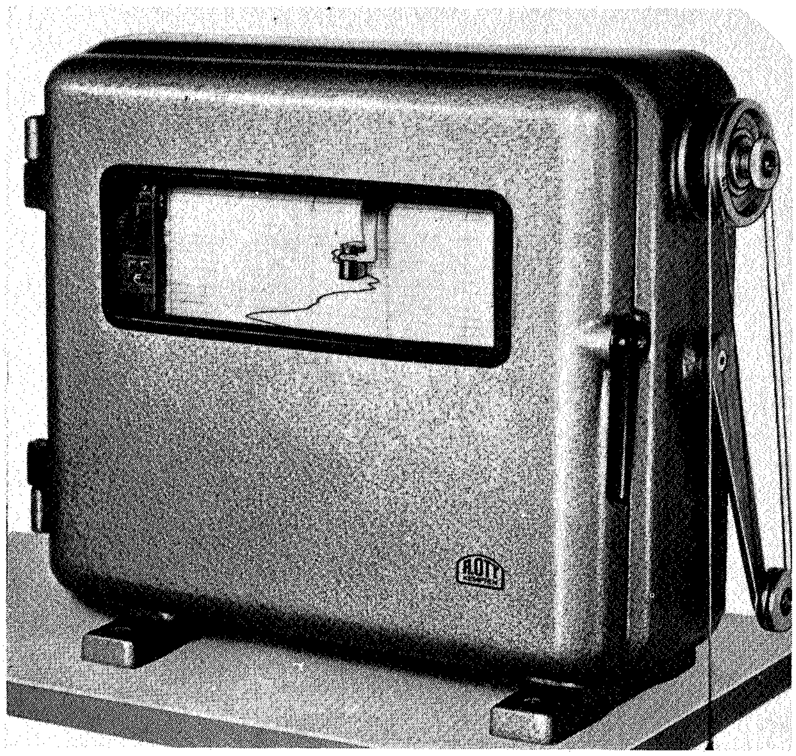


Figure 2.3.1 Ott XX water-level recorder

Advantages : Due to the possibility to adjust the vertical and horizontal scale a very accurate picture of the tidal movement can be obtained.

Due to the system of supply-roll and take-up spool a continuous registration can be obtained, depending on the horizontal scale an operating time of maximum 4 months is possible.

And by means of the reverse spindle, the width of the recording paper is no restriction in recording peek high waters or low waters.

Disadvantages : An expensive structure is required for the recorder and float-well, the location should always be in deep water in order to have sufficient water in the well at Lowest Low Water.

Application : In case of permanent installation in tidal and non-tidal areas, the Ott XX water-level recorder can be utilised to measure the vertical tide in order to obtain data for a tidal analysis, calculation of MSL and to determine a datum plane, and to measure flash flood water-level fluctuation.

Preparation : Before leaving for the field this instrument should be assembled in the office, care should be taken that the correct pulley is attached corresponding with the required vertical scale.
Sufficient rolls of recording paper, spare pens and ink to be supplied.

2.3.2 Pneumatic water-level recorder "Van Essen"

Principle. An air-filled balloon connected to the recorder with a thin nylon tube is set under water and follows the pressure fluctuation in proportion to the water-height above the water. The pressure fluctuations are measured by a bellow-spring system in the recorder. The displacement of the bellow is transferred to a writing-stylus by a lever system.
The pen with ample ink stock provides an undisturbed registration of the tidal curve on the registration paper which will last for 24 days or 48 days depending on the paper speed.

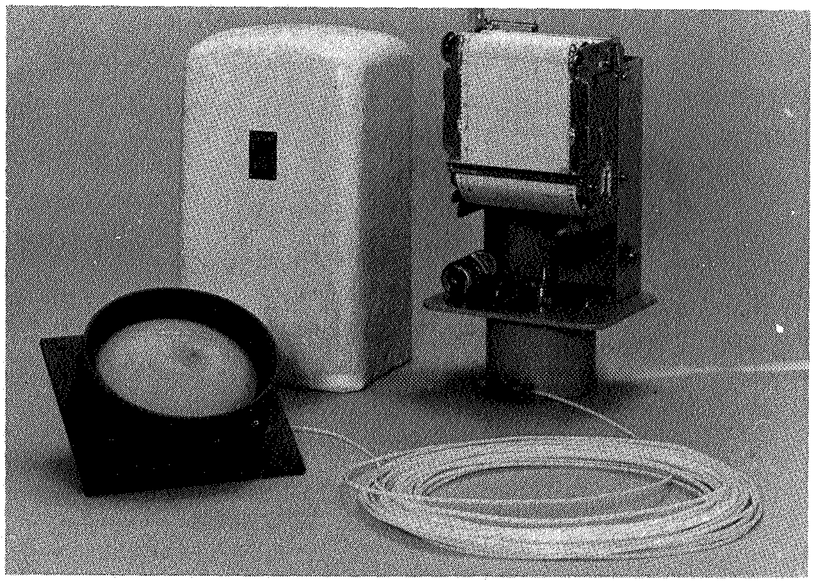


Figure 2.3.2 "Van Essen" pneumatic water-level recorder

Advantages : No expensive structures have to be made as the balloon can be put on the bottom of the sea or river-bed weighed down or attached to a small pipe.
This instrument is especially useful for short time survey at places where dry-falling flats exist, as up to 100 m nylon tube can be used.

The recorder can be placed anywhere on the shore even attached to a tree.

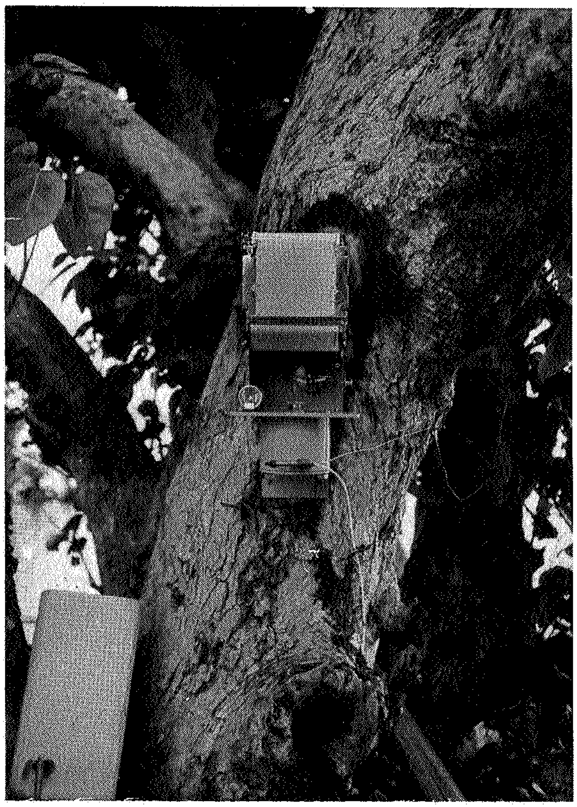


Figure 2.3.3 Pneumatic waterlevel recorder attached to a tree

Disadvantages : Although the instrument can be purchased for tidal ranges of 2 m, 5 m and 10 m, the paper width will remain the same. The readings of the recording for a 10 m range will therefore be less accurate.

Meticulous care must be taken in connecting the nylon tube to the balloon and the recorder as very small leaks cause non-reliable registrations; therefore frequent checks have to be made and recordings have to be checked by actual water-level readings of a staff gauge.

In case air has escaped from the system, a bicycle pump is provided to fill up the system with a few strokes.

Application : To obtain data of water-levels in areas where no permanent installation is required.

Preparation : All moving joints should be lightly oiled. Balloons must be checked on leaks.

Check to be made that all couplings are available in sufficient numbers, sufficient length of nylon tubing, batteries, registration paper and recording pens to be supplied.

2.3.3 Pressure-transducer - water-level recorder

Principle. A pressure-transducer mounted in a frame which is lowered to the seabed floor measures the water pressure continuously.
The information of the pressure-transducer is conveyed to the buoy by a signal/mooring cable.
Waves with a period shorter than for instance 5 minutes will be attentuated by means of an electronic filter.
After the filtering procedure only the tidal information is converted into a digital frequency-code, which is transmitted every 60 seconds to a shore based/ship based receiving station.
In the buoy a set of drycell batteries provides power to the transducer, the electronics and the transmitter, one set of batteries will last for approximately six weeks.

At the receiving station the incoming frequency code after filtering is demodulated into a digital code which is checked on number of bits, bit-length and bit-interval.
Every word should be preceded by a digital 0 and a digital 1.
If the incoming data is valid it will be stored in the memory and will be converted into an analogue signal; if the incoming data is not valid only the last valid data will be used.
The analogue signal will be corrected for density of the water and the barometric pressure variations.
If the density of the water is known a correction factor can be pre-set and will be processed automatically.
The barometric pressure fluctuations will be measured by a pressure-transducer in the receiving station and will be applied automatically.

Advantages : This instrument can be used located on the seabottom with a surface buoy to house the batteries and transmitter but can also be attached to an existing structure at sea with the batteries and transmitter on a platform and can therefore be utilised anywhere without building expensive structures.
Ashore a daily check on the proper functioning of the instrument is guaranteed at the reveiver end, while the barometric

pressure fluctuations are automatically filtered out and the density of the water can be pre-set.
The instrument can be located in deep water up to 50 mtr to obtain tidal data for a tidal analysis, for reduction purposes without the necessity to correct for shallow water-influences.

Disadvantages : None, only the correlation of MSL or Datum to a reference plane ashore will be more difficult.

Preparation : Pressure-transducer should be calibrated before send to the field. New batteries to be inserted in the buoy, transmitter to be checked receiver to be checked and frequency permit to be obtained in the country where instrument is to be used.

2.4 Echosounder

2.4.1 Echosounder Elac 30 kc

Principle: The working of the echosounder is based upon the following principles:

1. Water is a good medium to propagate sound waves with a speed of ± 1435 m/sec.
2. Sound waves are very well reflected by the sea- or river bottom.

In the bottom of the vessel or hanging over the side an oscillator (transducer) is located in which electrical vibrations are transformed into sound vibrations and transmitted to the sea bottom.
The reflected vibrations are received by the same or an other oscillator and transformed into an alternating current. This alternating current is amplified and led to the registration unit.
The time-lapse between transmission and reception of the vibration is a measure for the depth of water and is recorded as such on the registration paper.

Advantages : With a recording echosounder a continuous registration of the bottom profile can be obtained.
With the 30 kc echosounder, the differences between soft mud, clay and sand can be defined and the thickness of a mud layer can therefore be estimated.

Figure 2.4.1 Elac echosounder "Castor"

Disadvantages : Due to the rather heavy weight of the transducer it is not possible to attach the transducer to every small craft.
The change in water temperature and density during a sounding day affects the accuracy of soundings.
Therefore frequent test-bar checks have to be made to make corrections.
To read the water-depth on the recording paper a separate scale is required.

Application : The instrument can be used to make a bathymetric chart of the area, to determine cross-section profiles, to determine siltation rates in dredged channels.

Preparation : Echosounder to be checked according to instructions in Manual. Sufficient writing stylusses and recording paper to be supplied.

Caution: The echosounder operates on 24 volt DC and when connecting to batteries make true that the (+) and (-) connections are placed in their corresponding poles on the batteries. Misconnection will damage the echosounder.

2.4.2 Raytheon DE 719 B

Principle: The same is applicable as for the Elac echosounder, but the instrument operates in a frequency of 208 kc.

Figure 2.4.2 Echosounder Raytheon DE 719 B

Advantages : The recorder gives a continuous registration of the bottom profile, while the recording paper is provided with its own scale reading so that no separate reading scale is required.

The speed of sound of the sound-pulse in the water can be read on the recording paper as soon as with a bar-check the echosounder is calibrated.

This makes it possible to detect any discrepancy in sound

velocity during the soundings.
The instrument operates on a 12 volt DC battery.

Disadvantages : The instrument gives no penetration in the bottom and only the top bottom layer is recorded.

Application : This instrument can be used for any bathymetric survey as long as no penetration in the bottom is required.

Preparation : Echosounder to be checked according to instructions in manual.
Sufficient writing stylusses and recording paper to be supplied.
All connecting joints of the extension tubes of the transducer to be cleaned and greased.

Caution: Echosounder operates on 12 volt DC and when connecting to battery make sure that the (+) and (-) connections are placed on their corresponding poles on the battery. Misconnection will damage the echosounder.

2.4.3 Furuno 200 Mark III

Principle. The same is applicable as for the Elac 30 kc echosounder.

Advantages : Due to its size and light weight, easy to handle and to be carried and to be installed on small crafts.

Disadvantages : Only one paper-speed available and paper-width small.
Therefore only suitable for reconnaissance survey.
Frequency of 200 kc gives no penetration and therefore only the top of the bottom is sounded even when this is a soft-mud layer.
To read the recording paper a separate scale is required.

Application : To be used for reconnaissance survey only to select cross-sections for discharge measurements, to take soundings in small canals.

Preparation : Check echosounder according to instructions Manual.
Supply spare 1½ volt batteries and sufficient rolls recording paper and writing stylusses.

Calibration of echosounders see Chapter "Bathymetric".

2.5 Conductivity meters

2.5.1 Salinometer Beckman RS-5-3

Principle. The operation of this instrument is based upon the direct proportionality between the magnitude of an induced electric current and the electrical conductivity of the medium in which it is induced.

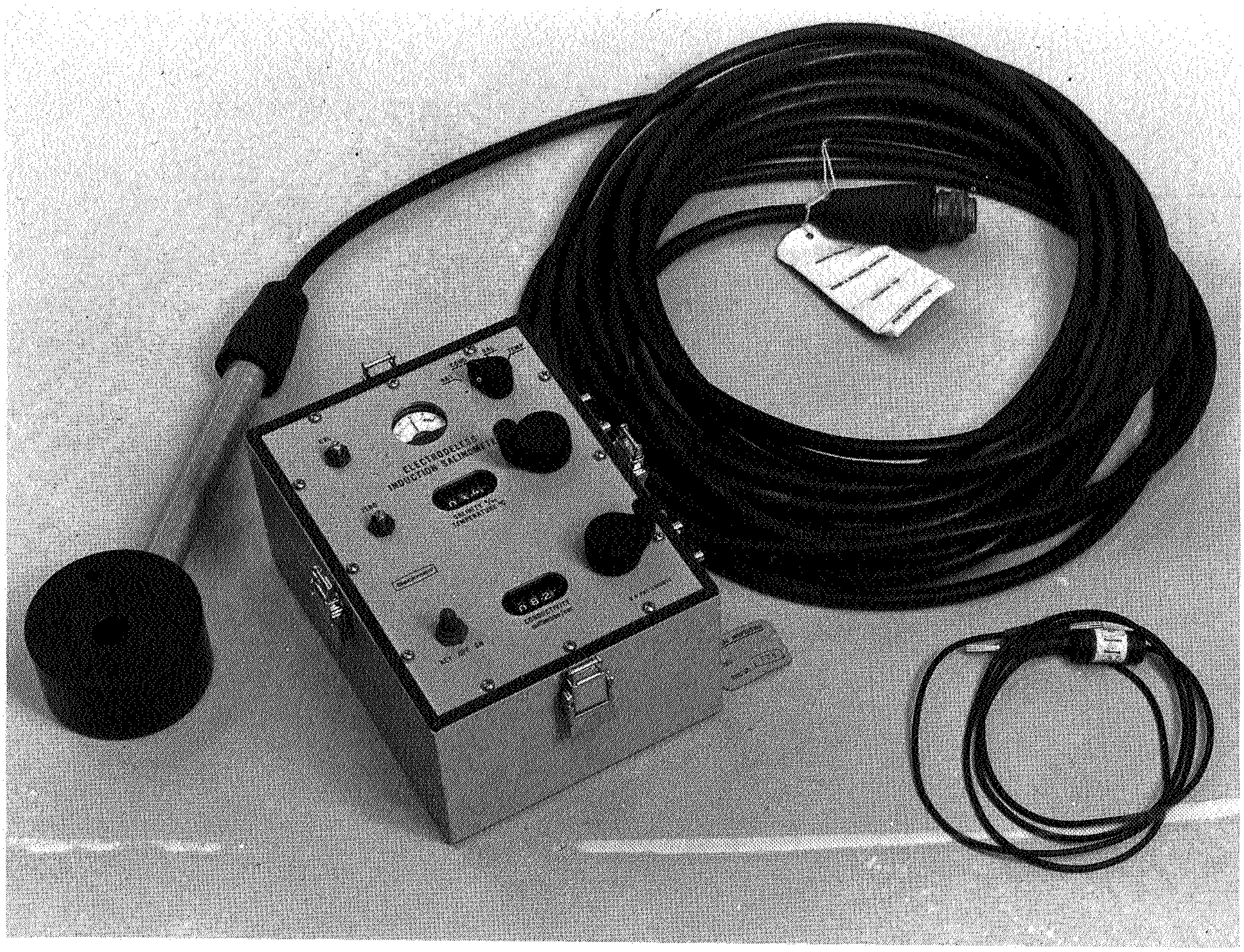


Figure 2.5.1 Beckman salinometer

Advantages : With this instrument salinity, conductivity and temperature measurement can be carried out in the field without taking any water-samples.
The instrument is direct reading so that all information can be elaborated in the field.
By using a calibration resistor, the calibration of the instrument can be checked in the field without any complicated tests with standard seawater.

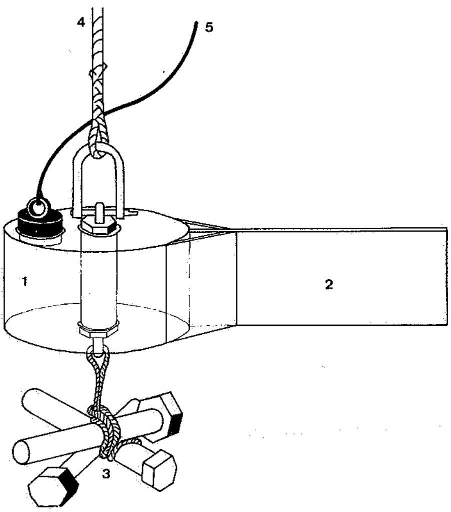
Disadvantages : Due to the rather thick suspension cable of the measuring cell, the instrument has to be lowered from a drifting boat to measure the parameters in a vertical when the current exceeds 0.75 m/sec, even when the cell is provided with a heavy weight.

Application : The instrument can be used in estuaries, rivers and canals to determine salinity distribution and the extent of salt intrusion.

2.6 Sediment sampling

2.6.1 Water-sampler

Principle. The water-sampler is lowered into the water with the hole closed with a plug. At the required depth, the plug is pulled out and the water flows in the sampler.



1. TRANSPARENT PERSPEX BODY
2. TAIL, AGAINST KNOTTING OF THE SUSPENSION-LINE AND THE STOPPER-LINE IN CURRENTS
3. IMPROVISED WEIGHT
4. SUSPENSION-LINE
5. STOPPER-LINE

Figure 2.6.1 Water-sampler (light-weight)

Advantages : The perspex water-sampler is light-weight and suitable for reconnaissance surveys.

Disadvantages : Due to its light-weight it will be difficult, even with a weight under the sampler, to lower the sampler vertically in currents exceeding 0,75 m/sec.

Application : Sampler can be used to obtain water-samples to determine sediment content, or quality of water in general.

Another type of water-sampler consists of a heavy body in which a ½ liter milk bottle serves as sample bottle. The cork is pulled out at the required depth to take a sample.
Same advantages, disadvantages and applications are valid, with the exception that in velocities up to 1.25 m/sec the sampler can still be used without much deviation from the vertical.

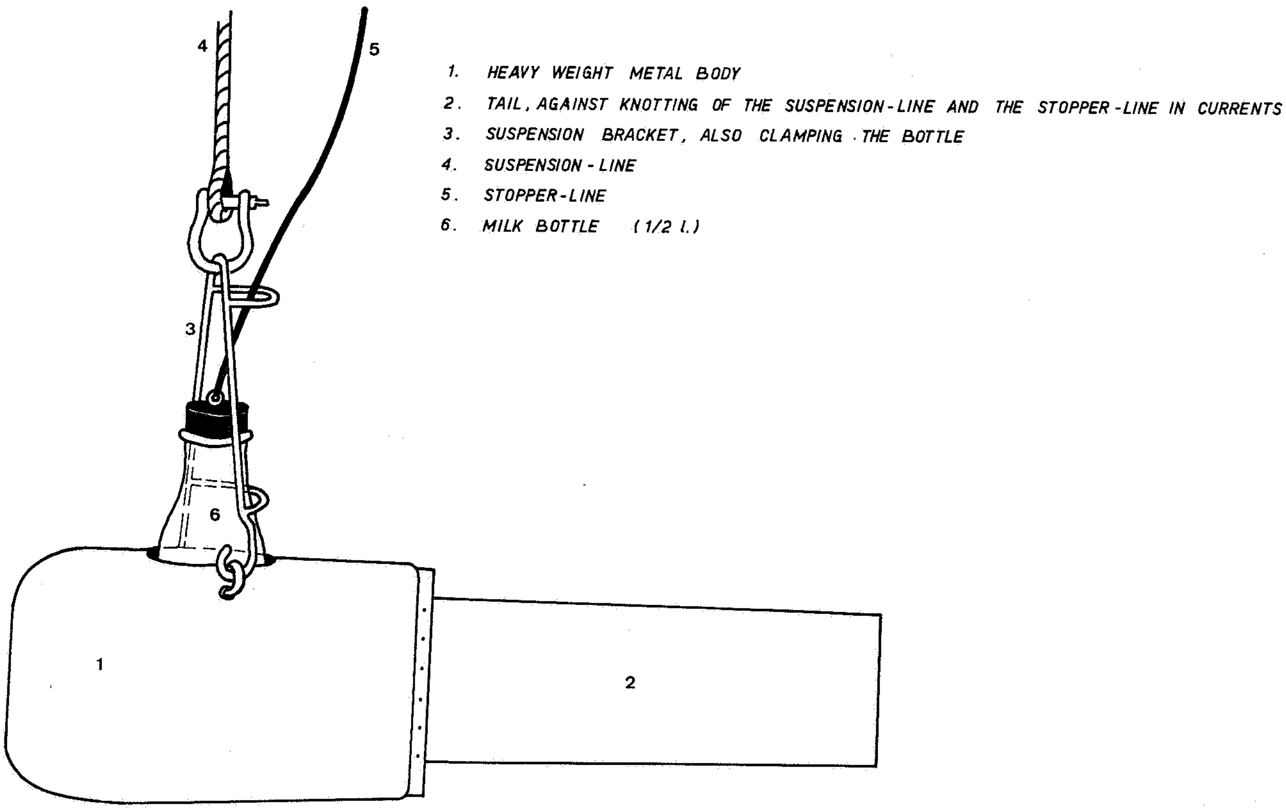


Figure 2.6.2 Water-sampler (heavy-weight)

2.6.2 Delft Bottle

The purpose of the suspended sediment sampler is to obtain a sample that is representative of the suspended load[1)] of the stream in the vicinity of the sample.
Several requirements for the design of the sampler should be fulfilled such as:

1. entry velocity of the water into the sampler the same as the surrounding stream velocity

---
[1)] Roughly particles > 50 μm

2. samples close to the river-bed possible
3. at least as possible disturbance of flow pattern at the intake of the sampler
4. deposition of small fractions of suspended load (no wash load)
5. simple, robust, easy to handle and standard volume.

The principle of most suspended load samplers is based on the principle that the hydraulic coefficient is equal to (or approximates) 1, that means no disturbance at the intake. This can be achieved by the following hydraulic measures in the design:

1. positive head of the flow at the entrance
2. negative head at the end of the air-outlet tube
3. hydrostatic pressure difference due to difference in elevation between entrance and outlet.

One of the widely used suspended load samplers, is the Delft Bottle (DF). The straight nozzles are used with the DF bottle suspended on a wire while the bent nozzles are used with the DF bottle on a frame to measure from the bottom up to 0.50 mtr above the bottom.
For measuring in moderate and high velocities the nozzles with the small internal diameter of 1.55 cm (area 1.9 cm^2) are used in moderate and high velocities, the nozzles with an internal diameter of 2.2 cm (area 3.8 cm^2), are used in small velocities.
It is possible with the DF to measure suspended load with water velocities up to 2.5 m/s , although the correction factor increases considerably for these high velocities. The grain size of the sediment caught exceeds 50 µm, the grains with smaller diameter flow through the instrument.

Principle. The sediment containing water flows through a bottle shaped sampler. The form of the sampling body induces a low pressure at the rear face in such a way, that the water enters the nozzle of the sampler with almost the same velocity as the undisturbed flow.
The sharp decrease of the velocity in the wide sampling chambers causes the sediment to settle there. This settled material can be taken out and measured volumetrically after the DF is hoisted on board of the survey vessel.
The sampling body is divided into three chambers; a diffusor-cone as an extension of the nozzle protrudes in the central chamber.

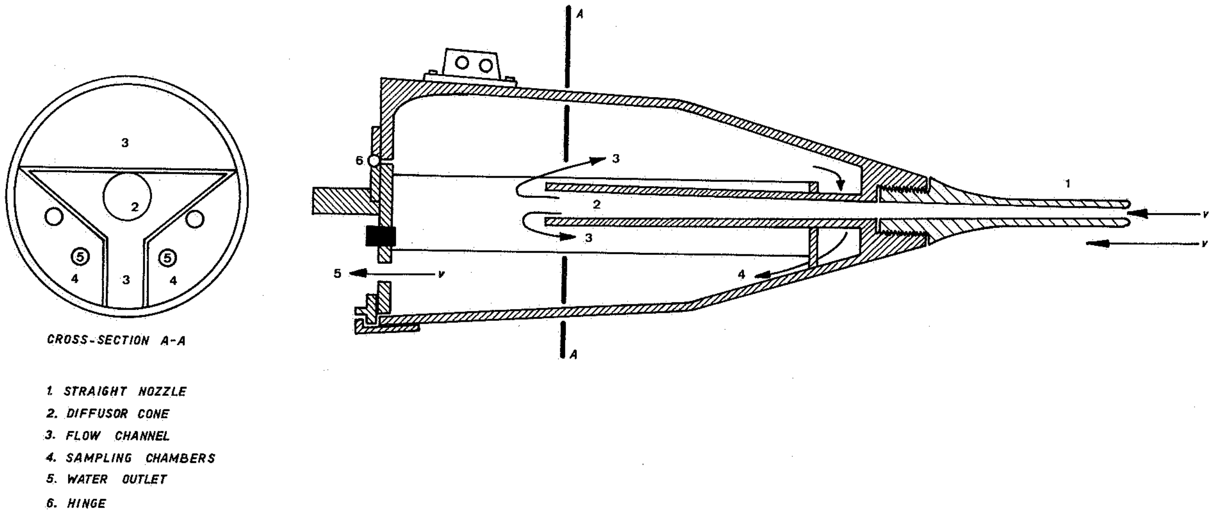


Figure 2.6.3 Delft Bottle

Advantages : - because of the flow-through principle a large volume of water is sampled; and the sediment sampling is a direct transport measurement
- mechanically simple
- sturdy
- can be used at any depth.

Disadvantages : - handling requires a davit with depth counter and winch due to the weight of the instrument
- working-out procedure requires simultaneous measurement of velocities for application of the calibration coefficients
- particle sizes < 50 μ can not be catched.

Application : The instrument can be used in two ways:

1. Suspended on a cable for all depths from surface till 0.5 m from the bottom (Figure 2.6.4). A tail-fin keeps the nozzle in up-stream direction.
2. Standing in a frame on the bottom (Figure 2.6.5) for distances of 10 - 20 - 30 - 40 and 50 cm from the bottom. The inclination of the body requires the use of bent nozzles.

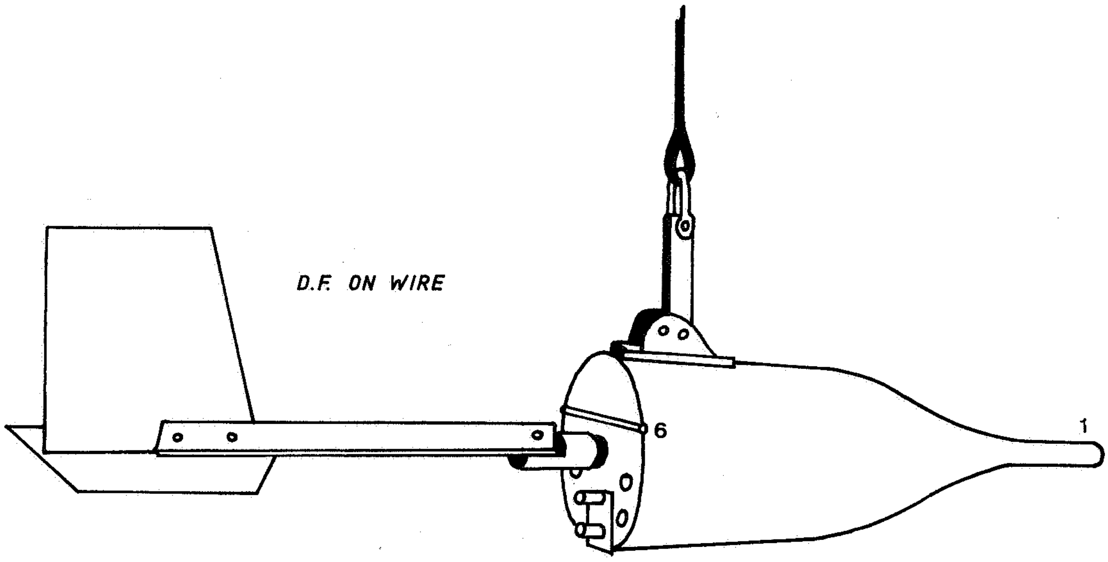


Figure 2.6.4 Delft Bottle, arrangement for sampling from water-surface till 0.5 m from river-bed

In the arrangement of Figure 2.6.5 so close to the bottom the catch will also contain grains, which belong to the bed-load transport.
In computation procedures with the catched sediment volume, correction factors should be applied for evaluating the actual sediment transport. The correction factor is the ratio of the loss coefficient and the hydraulic coefficient.

- The loss coefficient is the ratio of the total sand volume that enters the nozzle and the part that settles down in the DF.
- the hydraulic coefficient in the ratio of the discharge through the nozzle and the discharge through the same imaginary orifice after removal of the instrument.

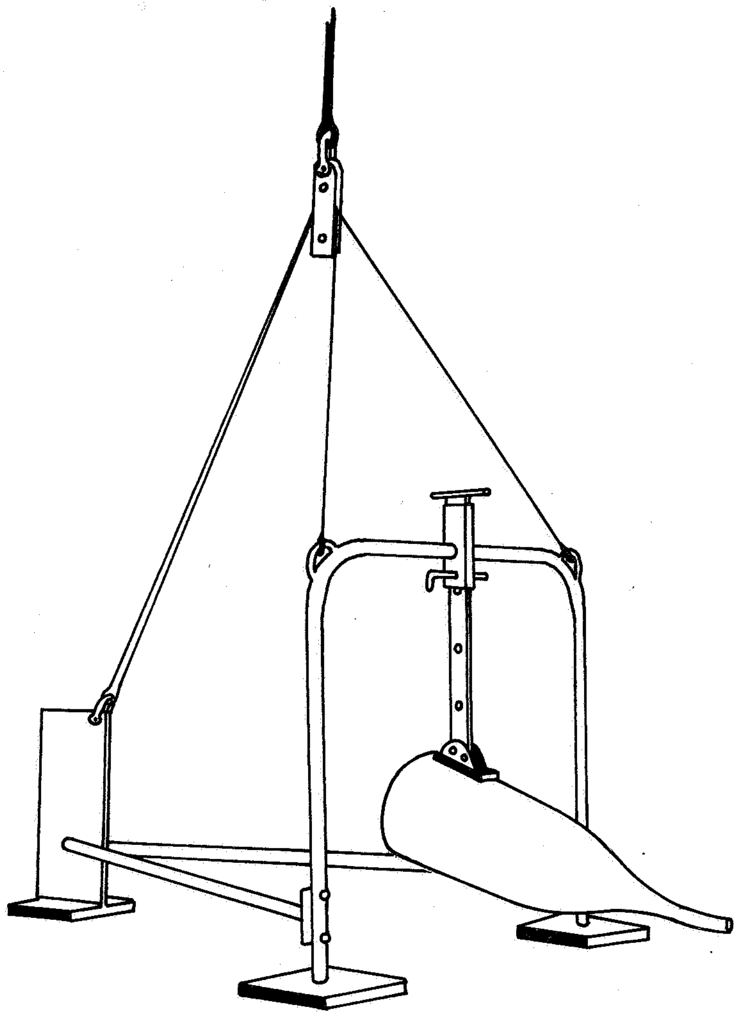


Figure 2.6.5 Delft Bottle, arrangement for sampling close to the river-bed

2.6.3 Point-integrating sampler US-P 61

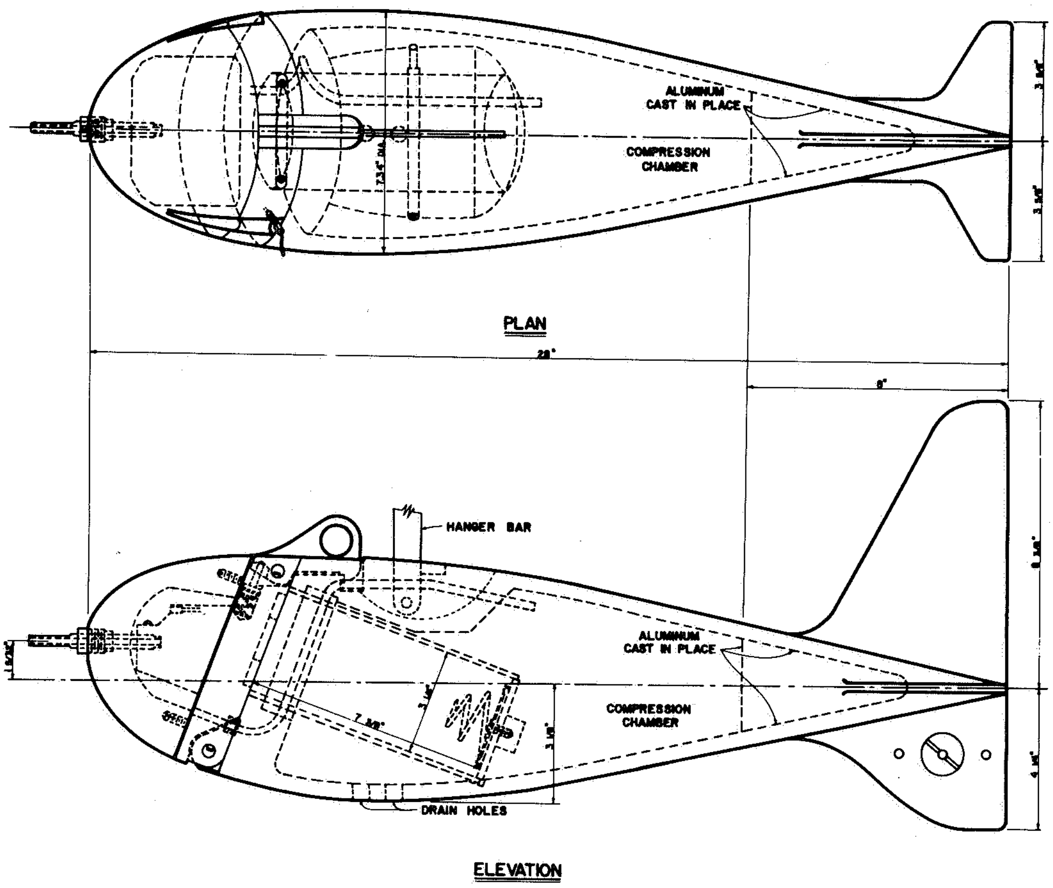


Figure 2.6.6 Point-integrating sampler US-P 61

The US-P 61 is a 50 kg sampler having an electrically operated valve for collection of a suspended sediment sampler at any point in a stream cross-section or to take a depth-integrated sample over a range of depth.

Like the US-D 49 the intake nozzle in the head points directly into the approaching flow. The sample head is hinged to provide access to the milk bottle sample container. An exhaust port pointing downstream on the side of the sampler being collected. Valve mechanisme enclosed in the head of the sampler is electrically activated to start and stop the sampling process. The valve operating switch energized by 48 volts is located at the observers station.

To eliminate sudden inrush at a selected sampling point below the water surface, the diving bell principle is used to balance the air pressure in the bottle with the hydrostatic pressure at the nozzle prior to opening the valve at the start of sampling. This is accomplished through a body cavity which is connected by ports through the valve system to the surrounding stream and to the sample bottle.

2.6.4 Depth-integrated sampler US-D 49

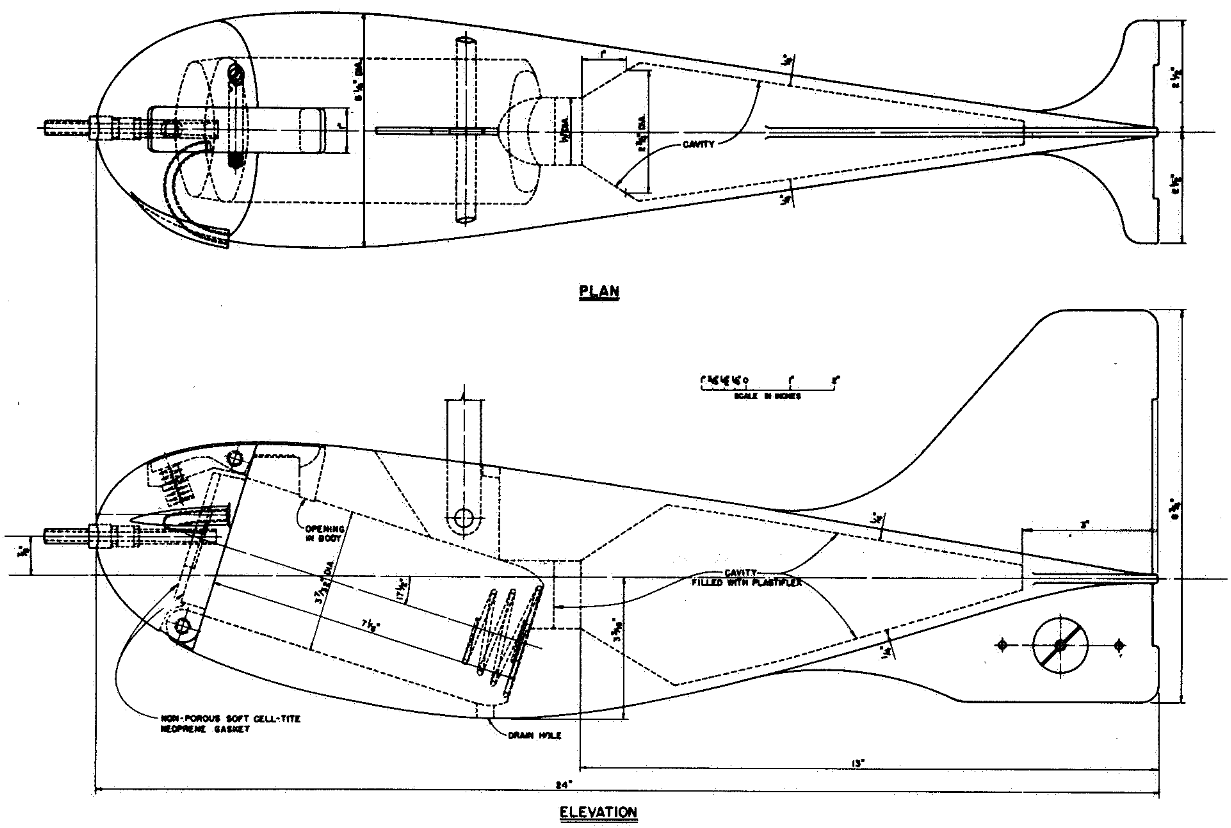


Figure 2.6.7 Depth-integrated sampler US-D 49

The US-D 49 is a 30 kg sampler to be used suspended on a cable in streams not deeper than 5 ½ mtr.

It consists of a cast bronze stream-lined body of 24 inches in which a round ¼ ltr milk bottle can be enclosed. The head of the sampler is hinged to permit access to the sample bottle. To orientate the instrument in the stream flow tail-vanes are provided. The head of the sampler is drilled and tapped to receive intake nozzles with the following orifice diameters

1 nozzle 0.63 cm Ø 1/4"

1 nozzle 0.48 cm Ø 3/16"

1 nozzle 0.32 cm Ø 1/8".

A hole which points downstream is provided on the site of the samplers head, from which air escapes as it is displaced by the sample being collected in the milk bottle.

2.6.5 Bed-Load Transport Meter Arnhem (BTMA)

Principle. A sampler mounted in a frame is pressed on the river bottom by a leaf spring. Behind the mouth of the sampler, opening $0.085 * 0.05\ m^2$, a basket of fine wire mesh is fixed.

The form of the basket causes a low pressure behind the instrument so that water and transported material enter the mouth with the same velocity as in undisturbed flow. The bed material particles which are too coarse to pass the meshing are caught. This bed-load sampler catches material coarser than 300 μ and finer than 5 mm. The lower boundary is due to the openings of the fine wire meshing, the upper boundary to the opening of the entrance of the BTMA.

Advantages : The instrument is of simple and sturdy construction and can easily be repaired and maintained in the field.

Disadvantages : Because of its weight and dimensions a davit and winch are required for handling it. The current range in which it can be used is limited to 2.5 m/s due to the construction of the basket.

Application : The BTMA is an instrument to measure the bed load of coarse sand and fine gravel just above the river bottom.

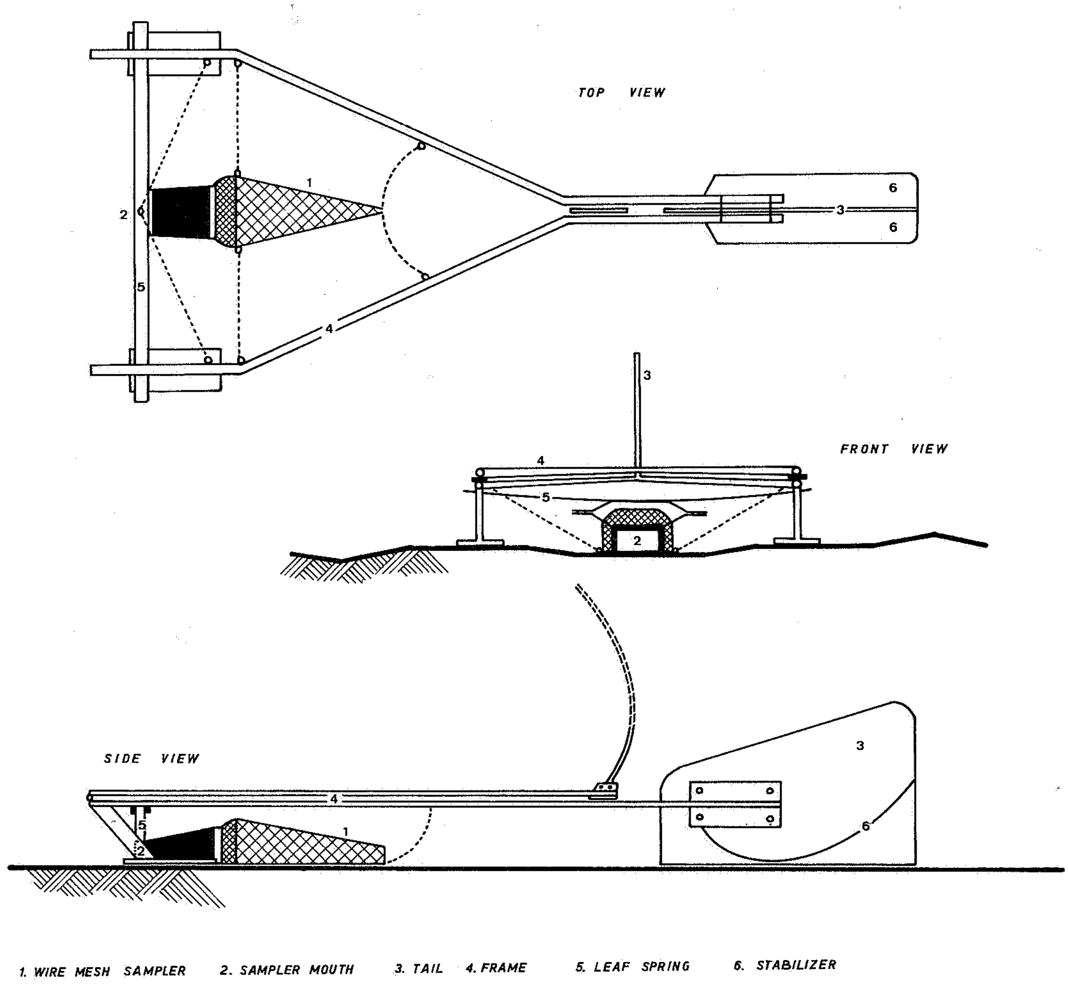


Figure 2.6.8 Bed Load Transport Meter Arnhem

2.6.6 Bottom grab

The "Van Veen" bottom grab works on a clam-shell principle; by a lever both clams of the grab are kept open, when lowered into the water and when touching the bottom, the lever will be released. While pulling the cord to raise the instrument both clams will close taking in this process a part of the toplayer of the bottom.

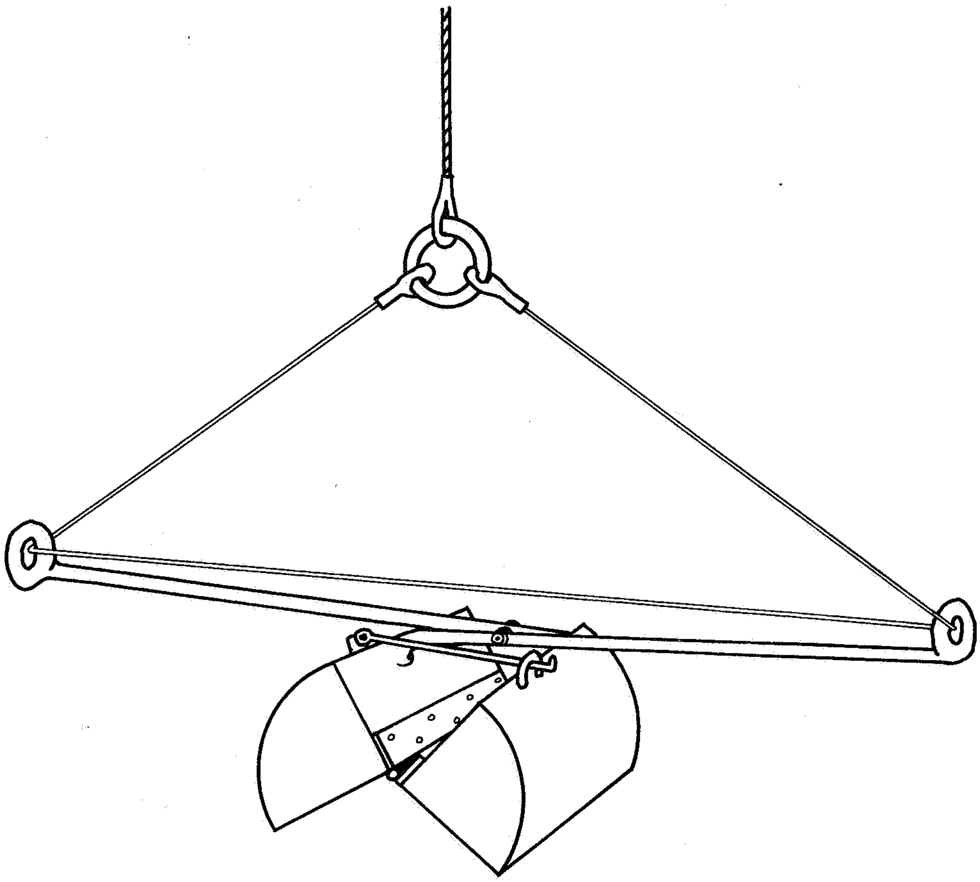


Figure 2.6.9 "Van Veen" grab

Advantages : A sturdy and simple device which can be used manually from any craft.

Disadvantages : If the bottom consists of gravel or small pebbles, the grab may remain open if a pebble or piece of gravel remains between the clams edges, causing the sample to be lost.

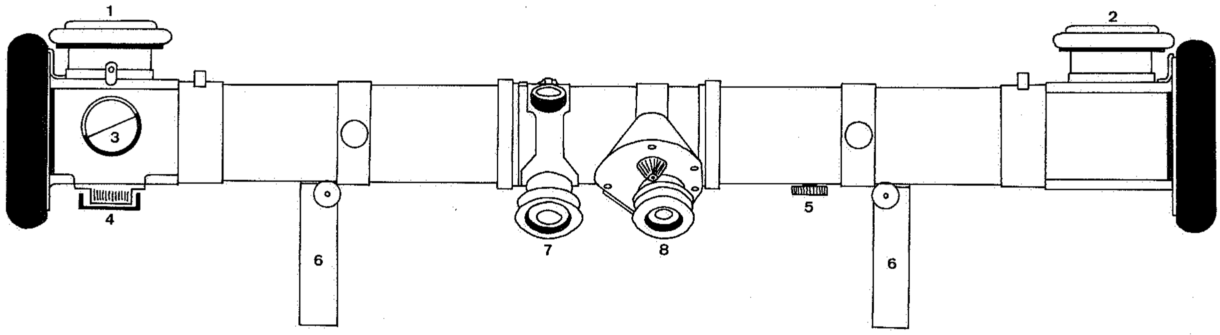
Application : The bottom samples can be used in estuaries, rivers and canals to obtain an insight in the bottom composition of the toplayer.

2.7 Position fixing

2.7.1 Range-finder

Principle. This instrument which determines the distance from the observer to a certain object directly, is based on the principle of the human eyes,

only the base distance between the two telescopes (eyes) being 80 cm in stead of a few centimeters.
Two telescopes, one with line of sight fixed under 90^{o} to the base and the other turnable give two images of an object, by turning the movable telescope both images are brought in coincidence and the distance is read directly.



1. LEFTHAND TELESCOPE 2. RIGHTHAND TELESCOPE 3. CO-INCIDENCE ADJUSTMENT 4. HALVING ADJUSTMENT 5. RANGE WORKING HEAD 6. SUPPORTING HANDLES 7. LEFT EYEPIECE (SCALE) 8. RIGHT EYEPIECE (FIELD OF VIEW)

Figure 2.7.1 Range-finder

<u>Advantages</u> : The instrument gives a direct reading of the distance between observer and a certain object, can be used without tripod, and therefore very useful to determine the vessel position in cross-sections from a moving boat.

<u>Disadvantages</u> : Because the system is based on the principle of the human eyes it is clear that the accuracy is only fair to a certain distance and lessens with increasing distance.
The error varies proportionally with the square of the distance.

<u>Accuracy</u> :

at 50 mtr	0.05 mtr
at 100 mtr	0.15 mtr
at 150 mtr	0.3 mtr
at 200 mtr	0.5 mtr
at 250 mtr	0.8 mtr
at 500 mtr	3.2 mtr
at 1000 mtr	13 mtr
at 2000 mtr	52 mtr
at 3000 mtr	118 mtr
at 4000 mtr	208 mtr
at 5000 mtr	320 mtr.

Application : The range-finder can be used for measuring the river-width, for position fixing while sounding a river cross-section and in combination with sextant and theodolite for locating the position of floats from ashore. The distance scale starts at 50 m, but with the aid of a small board as beacon, the distance between 0 and 50 m can also be determined. The board should be 80 cm long, divided in 5 parts of 16 cm. This board should be fixed to a beacon from which smaller distances than 50 m are to be known.

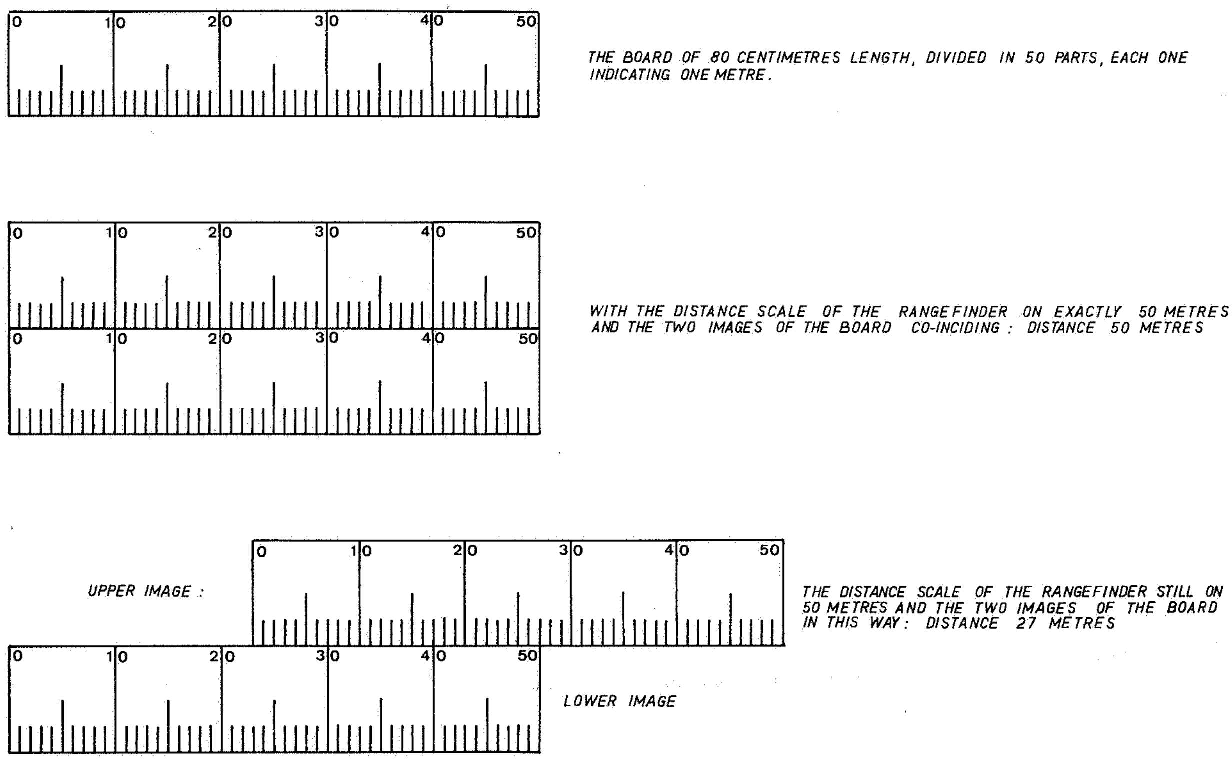


Figure 2.7.2 Auxiliary distance board

Preparation : The range-finder must be calibrated and this can be done in the following way:

- In the night a star is observed, the distance scale is set on infinite. The astigmatiser is switched on and if the two images of the star coincide the range-finder is properly adjusted. If both images do not coincide, the calibration knob is turned until coincidence is obtained.
- A distance of 100 and 200 meters is set-out by tape mea-

suring and stakes are erected on the two positions. Distances are measured by range-finder with the scale on respectively 100 and 200 meters. If both images coincide, the range-finder is properly calibrated. If both images do not coincide the calibration knob is turned until coincidence is obtained.

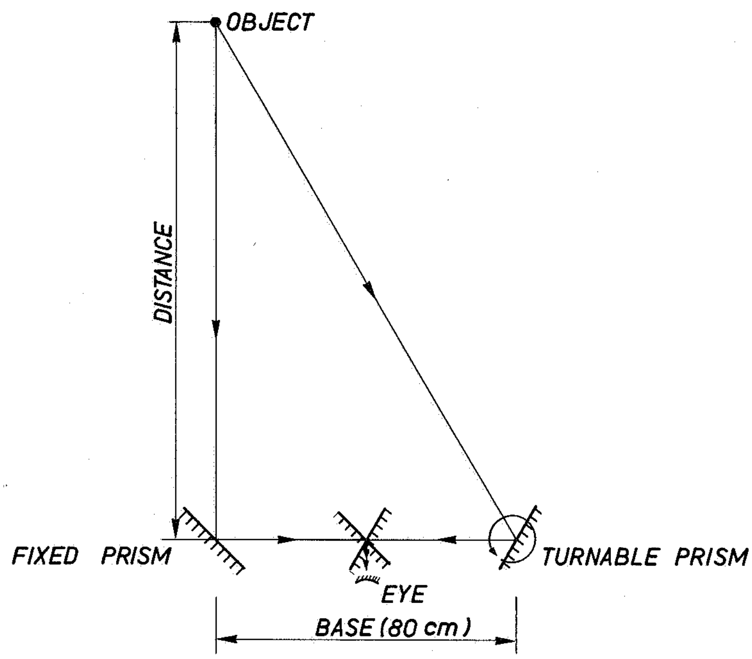


Figure 2.7.3 Principle of range-finder

2.7.2 Sextant

Principle. Two beacons, of which the angle between them is to be measured are brought in coincidence. The left hand beacon is seen directly through the telescope through the clear glass on top of the small mirror, while the right beacon is double reflected by the main mirror on the alhidade, and the small mirror in front of the telescope. The angle can then be read in degrees and minutes on the alhidade and microscrew.

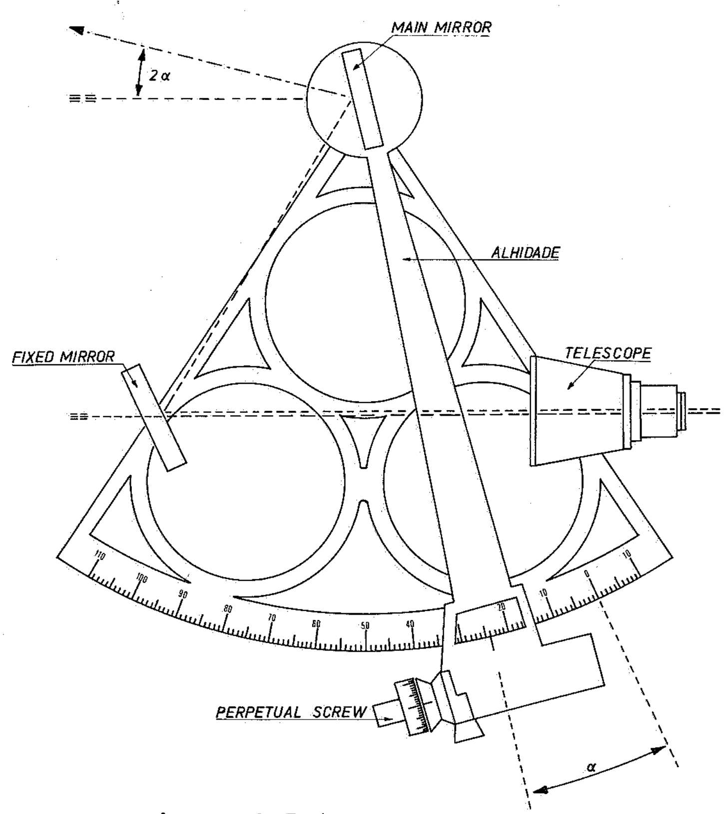


Figure 2.7.4 Sextant

Advantages : This instrument, held in the hand, is very useful to measure angles from a moving boat, where a theodolite on its tripod can not be used.
In minor triangulations in the bush or river-banks or shore line, the sextant can be utilized for measuring angles from difficult places like trees and masts.

Disadvantages : The accuracy of a sextant is small compared with the theodolite, but for the position fixing during measurements and for sounding purposes the accuracy is sufficient taking in consideration that the accuracy of plotting the fixes depends on the scale of the chart.

Application : The sextant, which is actually meant for taking vertical angles between celestial bodies and the horizon, can be used for position finding by taking horizontal angles from beacons. Therefore the instrument is very well suited to obtain position fixes of a sounding vessel, if two angles are measured simultaneously by two observers.

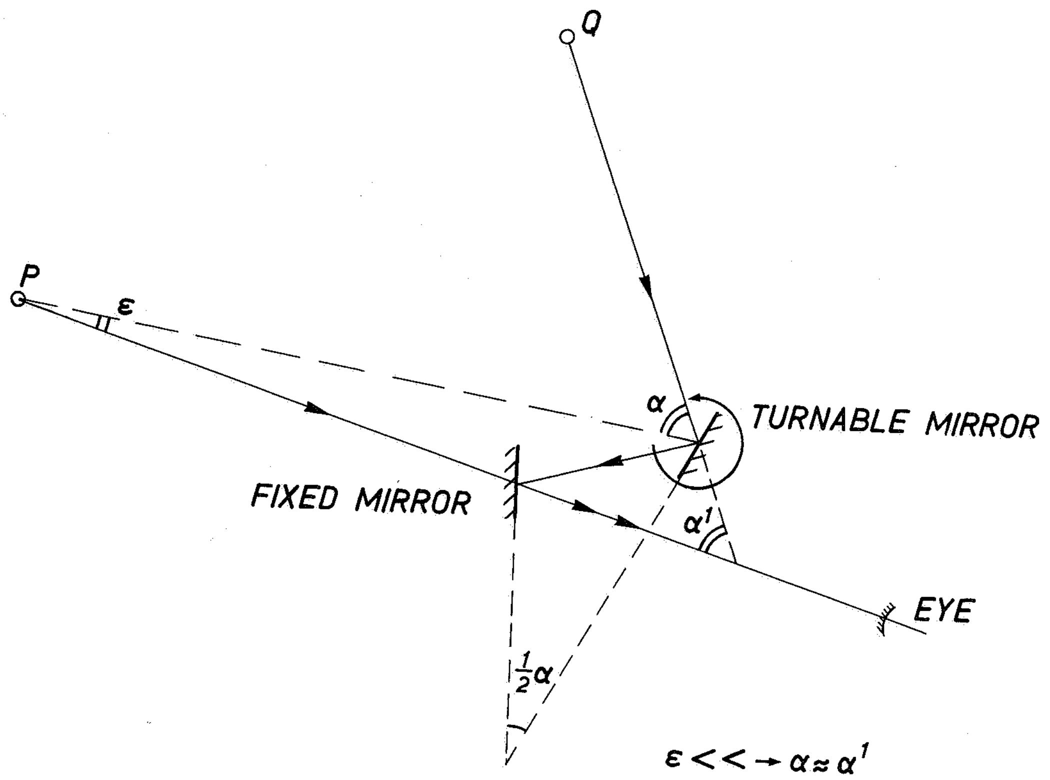


Figure 2.7.5 Principle of sextant

Determination of the errors and adjustments:

Errors:

a) index mirror and large mirror are not perpendicular to the plane of the graduated arc.

b) the optical axis of the telescope is not parallel to the plane of the graduated arc.

c) at a reading zero on the graduated arc, the index mirror and the large mirror are not parallel.

a) to check whether the large mirror is perpendicular to the plane of the graduated arc the alhidade is set in the centre of the arc, holding the sextant horizontal, look pass the main mirror to the arc and check whether the reflected image of the arc is in the same line as the direct viewed arc. If this is not the case the mirror position is adjusted with the adjusting screw until the image and the direct viewed arc are in one line. After adjustment of the main mirror the fixed mirror is checked by holding the sextant horizontal and setting the alhidade at zero.

Sight a far-away object if possible the horizon, the direct viewed horizon should be in the same line as the double reflected image, if this is not the case the fixed mirror must be adjusted by the upper adjusting screw.

b) Is only valid for astronomical telescopes.

c) Set the alhidade at zero, hold the sextant vertical and check whether the reflected image of the horizon is in one line with the direct viewed. If this is not the case, adjustment should be applied with the lower screw on the fixed mirror. Sometimes this will not suffice and a small error will remain. The horizon is then brought in one line by turning the micro-screw, the index correction is then found on the microscrew. The index correction is positive (negative) if the zero of the vernier is right (left) of the zero of the graduated arc (see Figure 2.7.6).

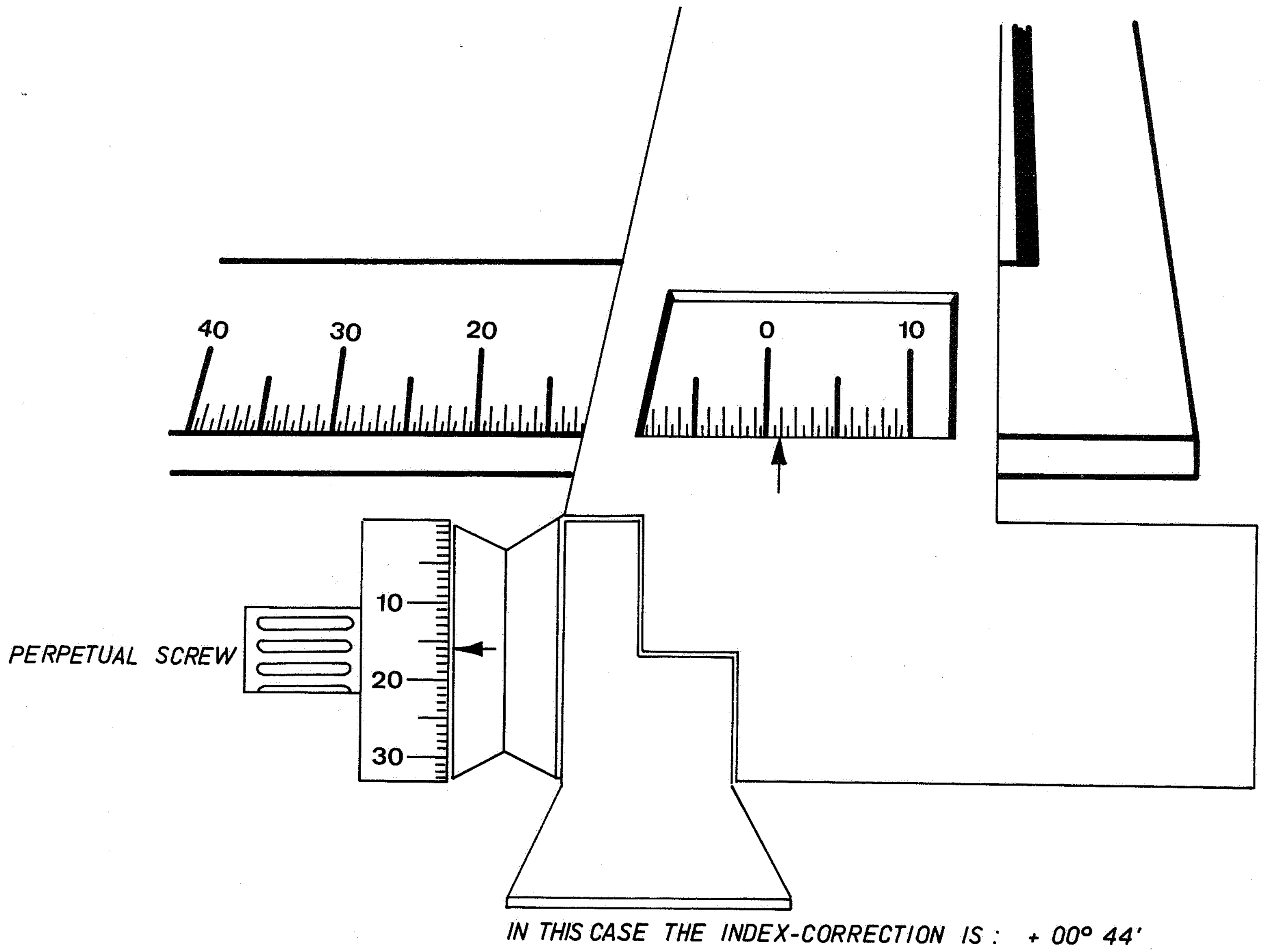


Figure 2.7.6 Illustration of index-correction

2.7.3 Decca Trisponder

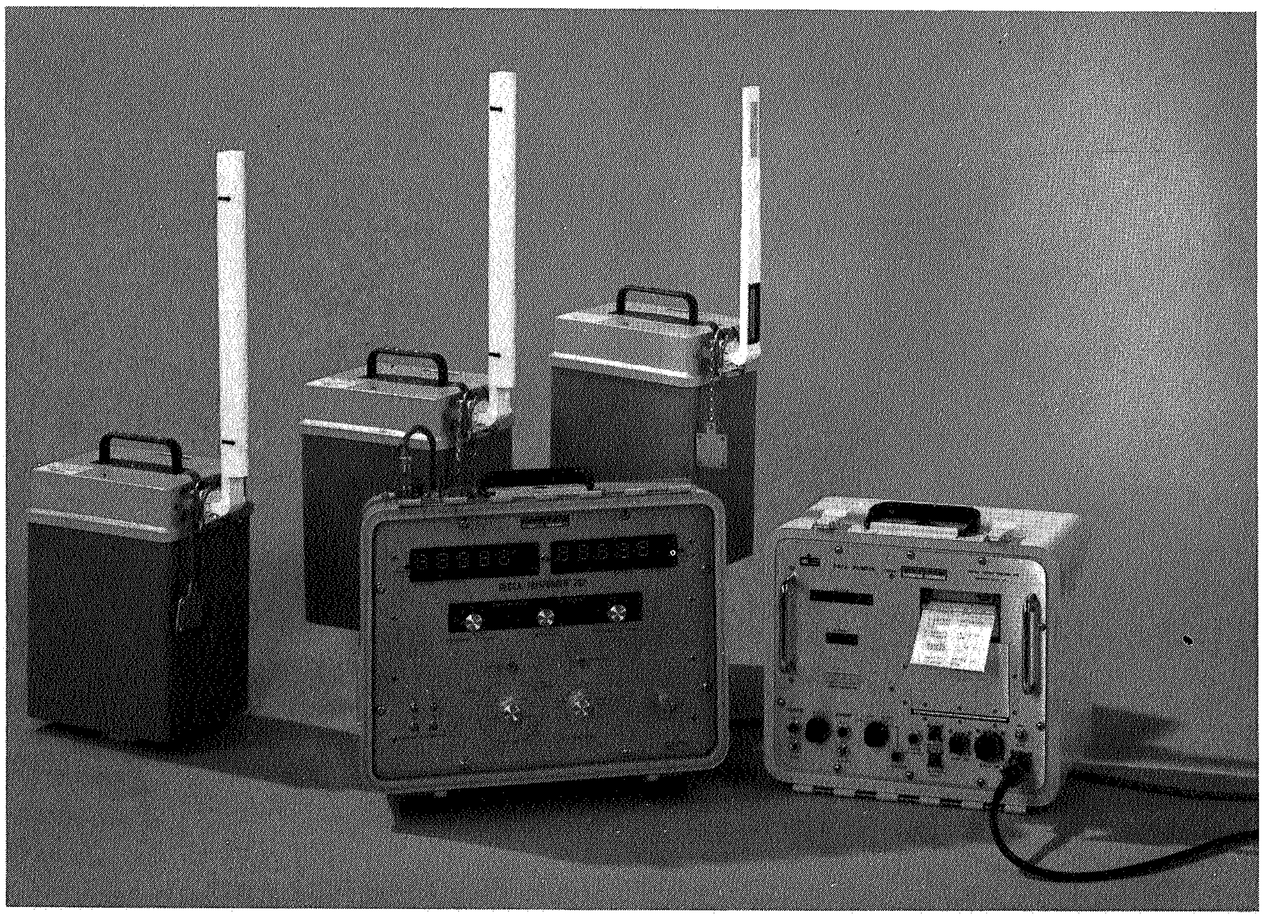


Figure 2.7.7 Decca Trisponder units

Principle. The working of the Decca Trisponder is based upon the following principles:

1. Straight line propagation path
2. Constant propagation velocity.

The distances are measured by using the relatively constant velocity characteristics of X-band (9400 mc/s) energy. A coded series of pulses is transmitted from the Master unit; this signal is received and decoded by all remote stations. Only one station possesses the right code for a specific coded series of pulses. If the remote station finds the proper code transmitted by the master unit, it returns a like series of pulses to the master unit, and thus establishes a RF-link. The time taken by the RF-signal to make the round

trip (from master to remote and back to master) less any (electronic) delays is converted into distance.
The distances from 2 remotes can be displayed simultaneously on board of the ship (master unit) when the position of the two remote stations is known, the position of the ship (master unit) can be plotted or calculated by means of trilateration.

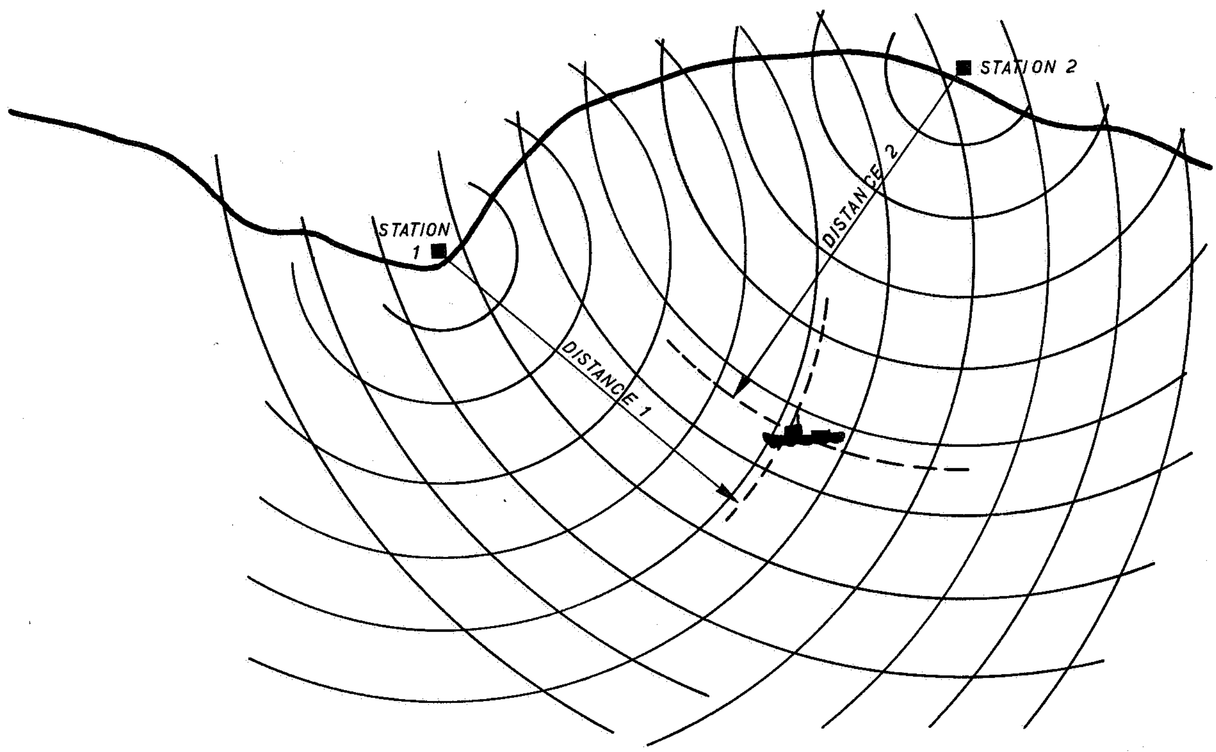


Figure 2.7.8 Ranging patterns of Trisponder system

Specifications:
name : Trisponder
nature of lines of position: circular
range : 90 km

On board of the survey vessel the Master transponder is located and the Distance Measuring Unit (D.M.U.).
On the D.M.U. two ranges to transponders are simultaneously displayed in meters, which is printed also on a digital-printer.
The digital-printer prints, besides the distances, the time of measuring, and the number of the fix (event).

The printer can be set to print every 10 sec, 1 minute or 10 minutes, but can also print on manual request.
Connected to an echosounder the fixes will automatically be produced on the echosounder recording. If a digital output of the echosounder is available, the printer will print-out the digital value of the sounding at a fix.

All units operate on 24 volt DC.
Maximum range 80 km, minimum distance 30 meters.
Accuracy $\pm$ 3 m.

Advantages : Only 2 locations to 3 locations are required for the remote transponders, which reduces the cost on beacon material and the time required to install a chain of beacons ashore.
Possibility to work beyond the range of the human eye, and no lost days due to bad visibility caused by rain, drizzle, fog or haze.
The accuracy of $\pm$ 3 meters is sufficient for all hydrometric and hydrographic purposes.
The instruments are light-weight, water-tight and weather-proof and all units operate on 24 volt DC power so that they can be installed at any place or boat.

Disadvantages : Line of sight is a necessity, but will give no problems on an open coast. If other vessels pass between the remote and the surveying vessel, signal may be affected for seconds. Instruments will, however, give the correct distance again as soon as the object is out of the path of propagation.
In order to charge the batteries in remote areas a charging generator is required.

Application : This instrument is very suitable for positioning the sounding vessel, to follow float tracks etc..

2.7.4 LORAN (Long Range Navigation)

Basic principles time difference measuring systems:
Radio signals consisting of short pulses are broadcasted from a pair of special shore-based transmitting stations.
These signals are received aboard ship on a specially designed radio receiver.
The difference in time or arrival of the signals from the two radio stations

is measured on a special indicator.
This measured time difference is utilized to determine directly from special tables or charts a line of position, on the charts surface.
Two lines of position, determined from two pairs of transmitting stations, are crossed to obtain a Loran position flux.

Specifications:

name	: LORAN-A
nature of lines of position	: hyperbolic
range	: 1500 km
accuracy	: under excellent conditions: 400 m under normal conditions, on maximum range: 9 km
frequency	: 2 mc/s
remarks	: navigation only, multi-user.

2.7.5 Artemis

Short description of the system:

The "Artemis" position fixing system consists of two antenna-following systems, coupled to each other by means of a micro-wave transmission path.
By the automatic following system the transmission path will be maintained, even if the station "Mobile" is moving with respect to the station "Fix", (the antennae will keep looking to each other).
In a situation as shown in Figure 3.2.12 the position of the "Mobile" will be measured with respect to the position of "Fix", in polar coordinates.
The distance between the "Mobile" and the "Fix" is measured with a distance measuring system, using the micro-wave transmission path (radar-principle).
The angle between the "Mobile" and a certain reference direction, as seen from the "Fix", is being derived by means of an angle-encoder from the position of the antennae-axis of the "Fix". Via the transmission-path this angle information is sent to the "Mobile".
The polar coordinates data, angle θ and distance R, will be available at the "Mobile", to be used for further processing, display and/or storage.

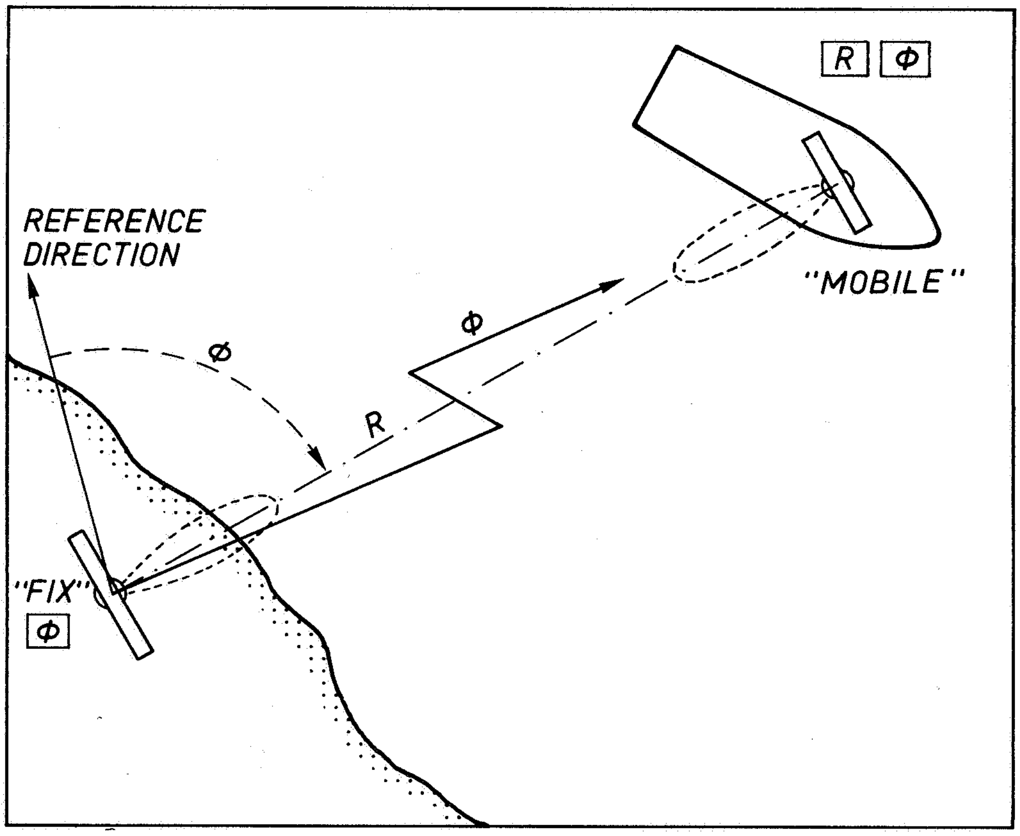


Figure 2.7.9 Artemis, distance-azimuth position fixing system

Specification:

name	: Artemis
nature of lines of position	: polar coordinates
range	: 50 m - 30 km
accuracy	: ± 1.5 m, independent of distance
frequency	: 9200 mc/s
signal type	: pulsed
remarks	: survey only, single user, line of sight required.

2.7.6 Distomat DI 3

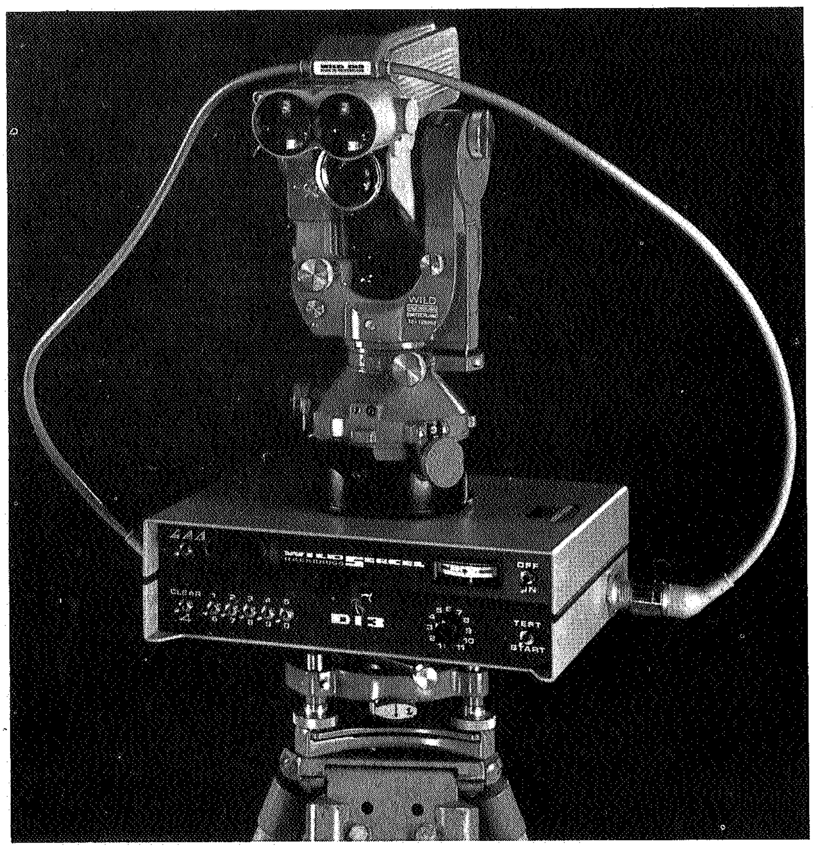


Figure 2.7.10 Distomat and T-2 theodolite

Principle. The invisible infra-red radiation of a Ga-As diode is used as the carrier wave, this is amplitude modulated by varying the supply current. Glass prisma reflectors return the infra-red beam parallel but laterally displaced by a small amount after total reflection.
The phase difference is measured digitally by counting the impulses of a quartz oscillator between the outgoing and returning waves (reflected waves). The instrument measures the distance in metres but has an inbuilt convertor to change metres to feet if so required.
Standard deviation = $\pm$ 5 mm + (5 $*$ 10^{-6} $*$ D).

Instrument can be mounted on a T2 Wild theodolite and angle- as wel as linear measurements can be carried out.
Distances to reflectors which are on a different elevation than the instrument can be converted by the built-in computer to straight horizontal distances while the differences in elevation can be displayed as well.

Advantages : Fast way of measuring distance and in combination with the T2 Wild theodolite, trilateration, triangulation and travers-

ing are carried out in less time.
Instrument can be carried by one person, has nickel cadmium rechargeable batteries, on one battery about 500 measurements can be done. Instrument is rain-proof.

Disadvantages : Line of sight is a necessity, even trees can disturb the measurements if they are in the path of the infra-red signal. Range only 1000 meters with a nine-prism reflector.

Application : To be used for any geodetic survey with sides of triangles and traverses not longer than 1000 meters.

2.8 Wave height measurements

2.8.1 Datawell "Waverider" buoy

The "waverider" is a buoy, containing an inertial accelerometer, for wave-height measurements. The buoy has been designed for moored and free floating operation.

Principle. From the vertical displacement at sea the acceleration is measured. (The accelerometer is pendulous suspended in a special damping fluid.) This acceleration is in the buoy transformed into a frequency modulated signal on a subcarrier of 259 Hz. The signal is then fed into the transmitter, which is operating in a 27 MHz. band channel.
The complete waverider system consists of stainless steel buoy, a $\frac{1}{4}$ λ polyester glassfiber whipantenna, fastened into the detachable roof-hatch, and of a mooring line of polypropylene or nylon-covered steel rope, with 15 meters of a special rubber cable, with stainless steel terminals and swivels.

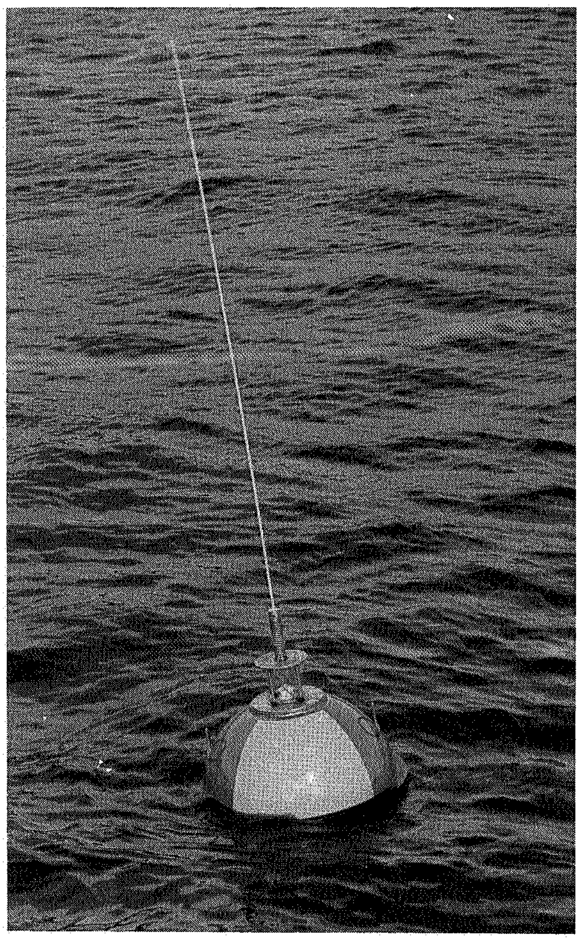


Figure 2.8.1 Datawell-buoy

<u>Advantages</u> : The waverider can be moored in shallow water (buoy dry at low tide) as well as in deeper water, can be used to hold at current up to 4 m/sec combined with a wave height of more than 20 meters.

Paper recording is given in the Warep receiver ashore, so that information is direct available and daily control of good functioning of the buoy is obtained.

If receiver is connected to a taperecorder the signal can be stored on magnetic tape for further computer processing via an A/D convertor.

Waveriders transmit contineously on a battery pack lasting 9 months while at the receiving station the receiver can be programmed to record the wave information at certain schedules or continuously.

<u>Disadvantages</u> : Buoy can be run over by passing vessels, or local fishermen may drag the anchoring cable by their nets and in process to save their nets may cut the anchoring cable and let the buoy drift away.

<u>Application</u> : Very useful instrument to obtain long term and short term data about wave period and height for coast protection studies, harbour designs etc..

3 Measurements

3.1 Geodetic network

3.1.1 General

Any hydrometric survey, regardless its size, needs a basic network of beacons in order to position the survey vessels.
Depending on the kind of measurements, the network can be extensive or simple, it can be a local network or connected to an existing network expressed in grid coordinates.
Whatever is required, the basic principles for a triangulation remain the same. The only difference will be the accuracy of angle-and distance measurements, depending on the type of instruments and methods which are used, and whether locations of beacons have been calculated or are obtained by a simple semi-graphical method.

In order to locate each beacon in its triangulation-scheme angles and distances have to be measured.
This implies that beacons must be set up in certain patterns to obtain polygons. To facilitate calculations and to obtain maximum accuracy, strict procedures should be established for the set up of the geodetic network.
Due to the circumstances in the field it may be necessary once in a while to deviate from the general procedures.

3.1.2 Geometrical figures used in triangulation

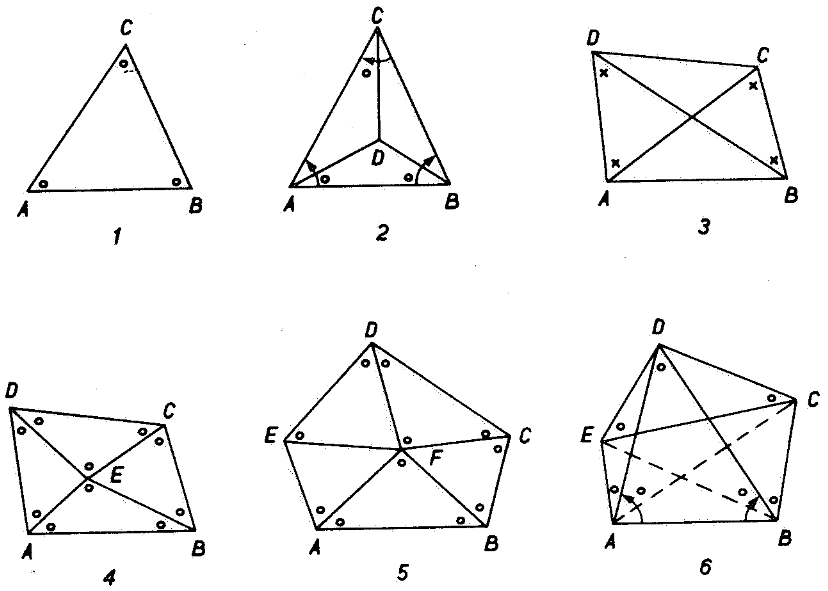
The main geometrical figures used in the triangulation work are:
a. Triangles
b. Quadrilaterals

If a triangulation is being carried out by the mathematical method its accuracy will depend very much on the shape and nature of the figures which are used. There are three basic rules to remember:
a) It must be possible to work through the triangulation by two seperate routes in order to obtain a check on the observations and calculations.
b) The angle opposite the known side in any triangle must never be too small, this must be applied regardless whether the triangulation is computed

forwards, from start to finish, or backwards.

c) Provided the above mentioned conditions are met, a simple triangulation will be better than a complicated one.

These rules do not apply if semi-graphic methods are being used.



o - Angle should not be less than about 40° or greater than 140°

x - Angle should not be less than about 35°

Figure 3.1.1 Geometrical figures

1 - The single triangle (see Figure 3.1.1)

This is the simplest figure which may be employed, and it covers the ground well. However, errors in one triangle will be automatically transferred to all the triangles which follow it. No receiving angle in a triangle should be less than about 40°, unless the length of one of the sides containing it can be measured.

Triangles may sometimes have to be used for turning a corner, or triangulating under difficult circumstances (such as in meandering rivers or creeks); they will accumulate least error if the unknown side is never allowed to lie opposite a small angle.

2 - Triangle with a central station (see Figure 3.1.1)

The addition of a central station D does not strengthen the original triangle ABC. It is, however, useful sometimes for breaking down a long side, AB or CB in the figure, into a shorter one CD or BD; D may be anywhere within the triangle.

3 - The braced quadrilateral (see Figure 3.1.1)

The quadrilateral in which both diagonals have been observed is the strongest of all figures available for triangulation. It should be used as much as possible.

If, the triangulation is being carried from AB to DC or vice versa, the angles marked with a cross should not be less than about 35°. If in exceptional circumstances this can not be achieved the quadrilateral may be strengthened by measuring the small angle carefully, or better still measuring the length of one of the sides (preferably the diagonal) containing it.

4 - The quadrilateral with central station (see Figure 3.1.1)

A quadrilateral with a central station is not such a strong figure as a braced quadrilateral, but it is easier to observe, since there are no diagonals. If, the triangulation is being carried out from AB to DC or vice versa, the angles marked with a small circle should not be less than about 40° or greater than 140°. This condition is far more stringent than that applicable to a braced quadrilateral, but is necessary since in this case the computation can only make use of individual angles, whereas with a braced quadrilateral double angles can be used. Note that in the braced quadrilateral (no. 3), D can be reached from AB in one triangle, whereas (in no. 4) two triangles are needed.

In exceptional cases the figure may be strengthened by measuring small (or very large) angles very carefully, or better still by measuring the length of one of the sides containing them.

5 - The polygon with central station (see Figure 3.1.1)

A regular pentagon is probably the best type of this class. Hexagons are reasonable, but do not cover the ground well. Figures with more than six sides get weaker as the number of sides increases, and are difficult to adjust; they should be avoided if possible. The five or six-sided polygon is weaker than a braced quadrilateral, but easier to observe, since no long diagonals are involved. If, in the figure, the triangulation is being carried from AB to DC or vice versa, the angles marked with a small circle should not be less than about 40° or greater than about 140°. Weak angles can be strengthened in a similar way to that used for the quadrilateral with a central station.

6 - The polygon without a central station (see Figure 3.1.1)

This type of figure is not recommended, since it is not very strong unless four diagonals can be observed, when it degenerates into two overlapping braced quadrilaterals, which are not recommended either.

7 - Overlapping figures

Figure 3.1.2 shows a part of an imaginary triangulation scheme. It consists of two overlapping quadrilaterals, ABCD and BECD (the triangle BCD is common to both), a five-sided polygon BCFGH with a central station E, which again overlaps the second quadrilateral in the triangle BEC.
Such an arrangement is, in theory, very strong, since the common triangle ties the figures together very firmly. It would be very strong in practice provided the system was adjusted to mathematical consistency as a whole. However, such an adjustment would be so laborious and complicated that it is not a practical proposition for the hydrographic surveyor.
Overlapping figures should therefore generally be avoided in practice.

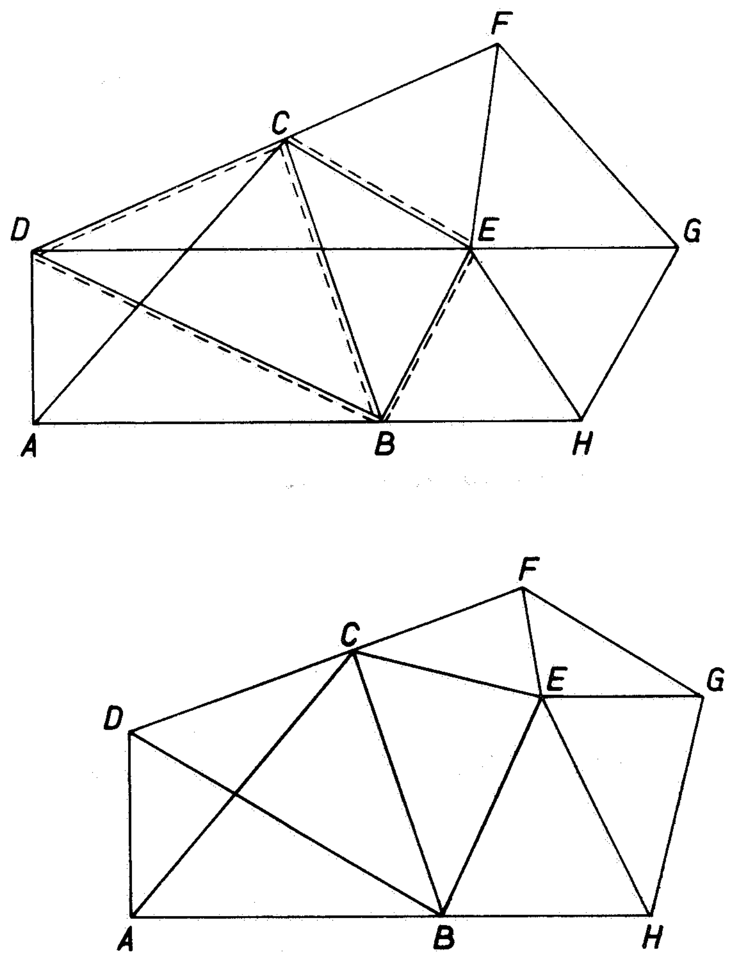


Figure 3.1.2 Triangulation-scheme

3.1.3 Triangulation figures in practice

The hydrographic surveyor may well find on occasions that the sites available for main stations make it impossible to follow the general rules. In this case he should give serious consideration to strenghtening his figures by observing distances.

3.1.4 Figure Adjustment

3.1.4.1 The triangle

Before the grid angles of a single triangle are used in the sine formula, they must be made to add up to exactly 180°. If the angles are assumed to have been equally well observed, the excess or deficiency should be distributed amongst them equally; if they have not been well observed equally, the excess or deficiency should be distributed in inverse proportion to the estimated accuracy of each angle. If triangles form part of a more complicated figure the adjustment of their angles is carried out as a part of the adjustment of the whole figure.

When only two angles of a triangle have been observed, the third must be assumed to be the difference between their sum and 180°.

3.1.4.2 The Braced Quadrilateral

a) General Principles

In Figure 3.1.3 ABCD is a quadrilateral in which all angles 1 up to 8 have been observed

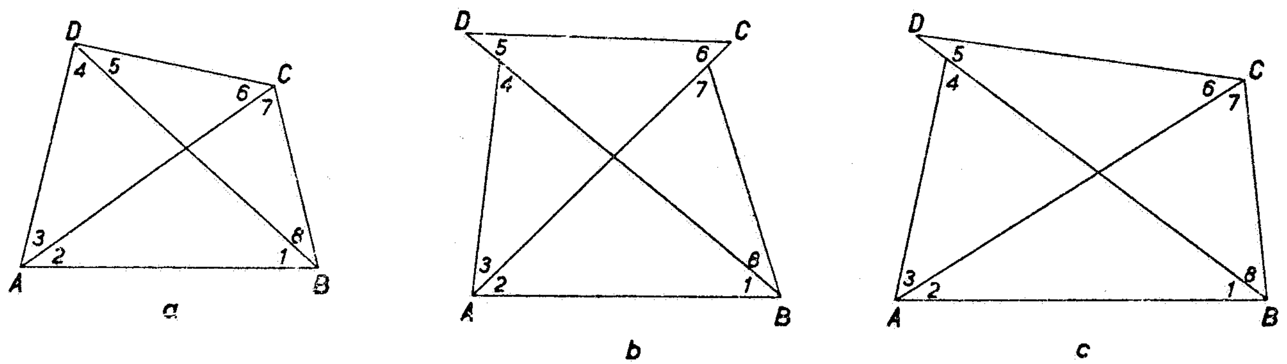


Figure 3.1.3 Quadrilaterals

By the law of geometry:

(i) The sum of the grid angles 1 - 8 should be exactly 360°, the correction of the angles to satisfy this equation is the quadrilateral adjustment.

(ii) The grid angles of each of the four triangles making up the quadrilateral should sum to 180° exactly; the correction of the angles to satisfy this equation is the triangle adjustment.

Both the quadrilateral equation and the triangle equation will be satisfied if the 8 angles are adjusted to add up to 360° exactly, in such a way that

$$1 + 2 = 5 + 6 \quad \text{and} \quad 3 + 4 = 7 + 8$$

At first sight it might appear that provided the two conditions above have been satisfied, the figure would be geometrically consistent. But this is not necessarily so. Figures b and c show this point. It is evident that the length of the sides must be taken into account, as well as the angles; this requirements is known as the side equation.

$$DC = \frac{DA \sin 3}{\sin 6} = \frac{AB \sin 1 \sin 3}{\sin 4 \sin 6}$$

and by the second route

$$DC = \frac{CB \sin 8}{\sin 5} = \frac{AB \sin 2 \sin 8}{\sin 7 \sin 5}$$

Note that if these two formula were applied (in error) to Figure b and c they will give two different values for DC; they will only give the same value for DC if the figure is in fact a true quadrilateral. They may therefore be combined to bring this about.

If the figure is to be true quadrilateral,

$$DC = \frac{AB \sin 1 \sin 3}{\sin 4 \sin 6} = \frac{AB \sin 2 \sin 8}{\sin 7 \sin 5}$$

and removing DC and AB,

$$\sin 1 \sin 3 \sin 5 \sin 7 = \sin 2 \sin 4 \sin 6 \sin 8$$

or if logarithmen are used

$$\log \sin 1 + \log \sin 3 + \ldots = \log \sin 2 + \log \sin 4 + \ldots$$

b) Adjusting a quadrilateral

In Figure 3.1.4 EFGH is a quadrilateral in which the angles numbered 1 to 8 have been observed. The adjustment is carried out as follows:

(i) Tabulate the grid angles of the four component triangles, and sum them, in order that the triangular misclosures may be seen.

Triangle	Angles	Grid Angles	Triangle	Angles	Grid Angles
EHG	1 2 + 3 4	57° 54' 20" 72° 40' 00" 49° 25' 42"	HGF	3 4 + 5 6	24° 48' 05" 118° 51' 14" 36° 20' 47"
		180° 00' 02"			180° 00' 06"
GFE	5 6 + 7 8	69° 25' 32" 73° 31' 57" 37° 02' 32"	HEF	7 8 + 1 2	37° 11' 10" 94° 56' 52" 47° 51' 55"
		180° 00' 01"			179° 59' 57"

Sometimes it is possible to get some idea as to which angles are least reliable, but it is not wise to speculate, and the angles should be considered to be of equal value unless there is definite evidence to the contrary. In this case all the angles are assumed to be equally reliable.

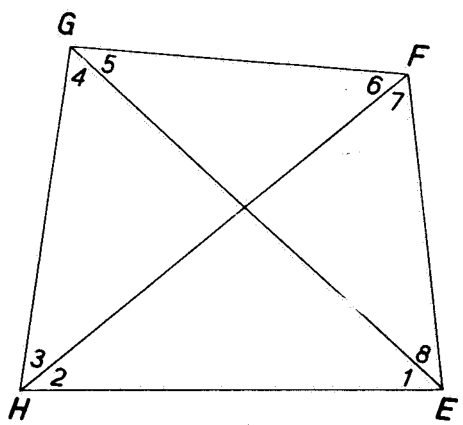


Figure 3.1.4 Quadrilateral

(ii) The next step is to apply the quadrilateral and triangle equations. The angles should be tabulated as shown in columns 1 and 2 below; note that they are arranged in odd and even groups.

1 Angles	2 Observed Grid Angles	3 Closing Adjustment	4 Closed Angles	5 Corrn. for Double Angles	6 Adjusted Angles
1	57° 54' 20"	- 0.4	57° 54' 19".6	+ 1.0	57° 54' 20".6
3	24° 48' 05"	- 0.3	24° 48' 04".7	- 1.2	24° 48' 03".5
5	69° 25' 32"	- 0.4	69° 25' 31".6	- 1.1	69° 25' 30".5
7	37° 11' 10"	- 0.4	37° 11' 09".6	+ 1.3	37° 11' 10".9
2	47° 51' 55"	- 0.4	47° 51' 54".6	+ 1.0	47° 51' 55".6
4	49° 25' 42"	- 0.4	49° 25' 41".6	- 1.3	49° 25' 40".3
6	36° 20' 47"	- 0.3	36° 20' 46".7	- 1.0	36° 20' 45".7
8	37° 02' 32"	- 0.4	37° 02' 31".6	+ 1.3	37° 02' 32".9
	360° 00' 03"	- 3.0	360° 00' 00".0		360° 00' 00".0

Double angles from column 4

1 + 2	105° 46' 14".2	3 + 4	74° 13' 46".3
5 + 6	105° 46' 18".3	7 + 8	74° 13' 41".2
Diff.	4.1		5.1
$\frac{1}{4}$ Diff.	1.0		1.3

Then,

(a) Sum the angles in column 2. The excess or deficiency from 360° is the amount required for the quadrilateral correction; here it is 3". It must be distributed arbitrarily among the eight angles and it should be evenly divided unless there are good reasons for not doing so. There is no point in tabulating to hundredths of a second, so the discrepancy is divided up as shown in column 3. The smallest correction is applied to the smallest angles.

(b) Tabulate the closed angles in column 4, and check that they sum to 360°. It is now necessary to apply the triangle equation; this is done by making the angles (1 + 2) equal to angles (5 + 6) and angles (3 + 4) equal to angles (7 + 8). The corrections are calculated in the lower part of the form, the quantity of each angle being $\frac{1}{4}$ of the difference between each of the pairs. Strictly these corrections ought to be 1".025 and 1".275, but they have been applied to the nearest 0".1, making certain that the total correction in column 5 sum arithmetically to 4.1 and 5.1 respectively.

Tabulate the adjusted angles in column 6, and check once more that they sum to 360°.

This completes the quadrilateral and triangle adjustments.

(iii) The method applying the side equation depends on whether logarithms or natural functions are being used. Using logarithms, tabulate the log sines of the adjusted angles from column 6 above, at the same time noting the "Diffs for 1 sec". These differences are given in any logarithm table. It is important to note that an angle greater than 90° has a negative difference since the log sine is decreasing.

	6	7	8	9	10	11
Angles	Adjusted Angles	Log Sines	Diff. for 1"	Final Angles	Final Log Sines	Log Sin Corrn.
1	57° 54' 20".6	9.927 9731	+ 13.20	57° 54' 23".2	9.927 9765	+ 34
3	24° 48' 03".5	9.622 6983	+ 45.55	24° 48' 06".1	9.622 7101	+ 118
5	69° 25' 30".5	9.971 3751	+ 7.90	69° 25' 33".1	9.971 3772	+ 21
7	37° 11' 10".9	9.781 3313	+ 27.75	37° 11' 13".5	9.781 3385	+ 72
	A' =	9.303 3778			9.303 4023	+ 245
2	47° 51' 55".6	9.870 1529	+ 19.05	47° 51' 53".0	9.870 1480	- 49
4	49° 25' 40".3	9.880 5779	+ 18.03	49° 25' 37".7	9.880 5732	- 47
6	36° 20' 45".7	9.772 8059	+ 28.62	36° 20' 43".1	9.772 7985	- 74
8	37° 02' 32".9	9.779 8899	+ 27.90	37° 02' 30".3	9.779 8827	- 72
	B' =	9.303 4266	C = 188.00	360° 00' 00".0	9.303 4024	- 242
	A' =	9.303 3778				
	A'- B'=	488	Corr = $\frac{A-B}{C} = \frac{488}{188.00} = \pm 2''.6$			

Then,

(a) Sum the log sines of each set of angles to find quantities A and B; if the side equations were already satisfied these two quantities would be equal. In this case they differ by 488 in the last three figures, so they must be made to agree by applying equal corrections to the angles in each set; these corrections will in this case be positive to the odd angles (to make the log sines greater) and negative at the even angles.

(b) The amount of the correction "e" is found by dividing A - B by C, the algebraic sum of the Diffs for 1", and in this case it comes to $\pm$ 2".6 . Very often it will not work out conveniently to exactly 0".1, there is little point in working to 0".01, and unless a round 0".05 is involved (in which case it may be applied to pairs of angles as 0".0 and 0".1) it should be rounded off to the nearest 0".1. As will be seen below, this rounding off does not affect the accuracy of the final log sines. The final angles are tabulated in column 9 and a check should be made to see that they still sum to 360° exactly.

(c) To find the final log sines proceeds as follows:
Set the quantity "e" (2".60 in this case) on a slide rule, and multiply it by the difference for 1", and in this case, for angle 1,
$2.60 \times 13.20 = 34.$
Tabulate these corrections in column 11; their sign should be evident from the sense of the corrections already applied to the angles; but note that if an angle is more than 90° the correction in column 11 will be in the opposite sense to the rest of the group. The algebraic sum of all corrections in column 11 should be zero; usually it will not be quite this, due to errors in the last figure, but there is no need to worry about a discrepancy of a few digits in the last place. Apply the corrections in column 11 to the log sines in column 7, and tabulate the results in column 10; in this case they will be positive to the odd group of angles (to make the log sines greater) and negative to the even group. As a check, obtain new values for A and B; they should now agree within the last two digits.
The adjustment of the quadrilateral is now complete.
The quantity "e" is a measure of the distortion introduced by this method of adjustment. For any given quadrilateral it can be changed by alternating the original quadrilateral corrections applied to the angles in column 2. If "e" is large (greater than about 5"), it is a sign that the method is breaking down; it should not be more than 5" if the observations have been good, unless several overlapping figures have been adjusted previously.

3.1.5 Traverses

Traversing must not be adopted generally as an alternative to triangulation and trilateration on land. Its great asset is that it can be used to provide control in areas where triangulation or trilateration would either be impossible, or extremely difficult and expensive.
In an area along a coast, where no possible room is available to carry out

a normal triangulation due to forest or heavy scrub along the beach, traversing may be the only way to position the required beacons for a hydrographic survey.

Types of traverses

Two main types of traverses will be described.

a. Accurate traverse

When it is uneconomical or impossible to carry out triangulation or trilateration, an accurate traverse may be employed to fix intermediate stations between two existing triangulation benchmarks.
To achieve an accuracy equivalent to that of a triangulation with a limit of misclosure in any one triangle of 10" of arc, the legs should be roughly the overall length of the sides of the triangulation, and should be measured with the greatest possible degree of accuracy either electronically (Tellurometer, Distomat etc.) or by tape.
Slope correction must be applied before reducing the lengths via the geoide and chosen spheroid to the plane of the projection and grid in use.
The grid bearings of the legs are obtained by measuring the angles at the turning points by the theodolite and reducing them to the plane of the projection and grid in use by applying arc to chord corrections.
Normally an initial grid bearing can be obtained at the starting point of the traverse from a previous triangulation. If not, a geodetic azimuth must be obtained at the starting point, which is then converted to grid bearing; the remaining calculations are then carried out in grid terms.

b. Minor traverse

Minor traverses may be run between two known points which are not too far apart, and plotted graphically. They can be most useful for coastlining, when it is not possible to obtain a sextant fix.
The criterion to be applied is that there should be no plottable error in the adjusted traverse points, on the scale of the survey.
The length of the legs may be measured by any suitable method (electronic distance-meter, invar baserod, tape etc.) and the measurement may be used directly in the plot, although it will generally be better to apply some of the corrections which are necessary in more precise work.
Bearings may be measured directly with a compass, or indirectly with a theodolite, level or sextant. Arc to chord corrections may safely be neglected.

In most cases of a hydrometric and hydrographic survey a minor traverse is sufficient accurate and shall therefore be described.

3.1.6 General rules for traversing

a. Open and closed traverses

An open traverse is one that starts from a known point, where a known grid bearing is obtainable and proceeds into a unsurveyed area.
A closed traverse runs between two stations which have been triangulated on the same geodetic point of origin, in this way a check is provided both on the correctness of the linear measurement and on the bearing carried through the turning points of the traverse.

b. Shape of a traverse

In Figure 3.1.5 (a) an ideal traverse is shown between two known points A en B. The lengths of the various legs Aa, Ab etc. are equal, and the grid bearing of the legs are the same, the traverse is, in fact, a straight line on grid connecting the two points A en B.
In the absence of errors the traverse illustrated in Figure 3.1.5 (b) would be as good as the one above. However, in practice errors will always be present and the more the traverse deviates from the straight line joining two known points, and the greater the variation in lengths of the legs, the less reliable will the traverse be.
In Figure 3.1.5 (c) the traverse starts and finishes at the same point. In this case although there is a check on the angular observations there is none whatever on the linear measurement. Even if the traverse closes perfectly on the starting point, the intermediate turning points a, b, c etc. will only be in their correct position if there is no constant error of scale in the instrument used to measure distances.
The firm line indicates the traverse plotted correctly, whilst the pecked line shows the displacement of the turning point caused by a large constant scale error.

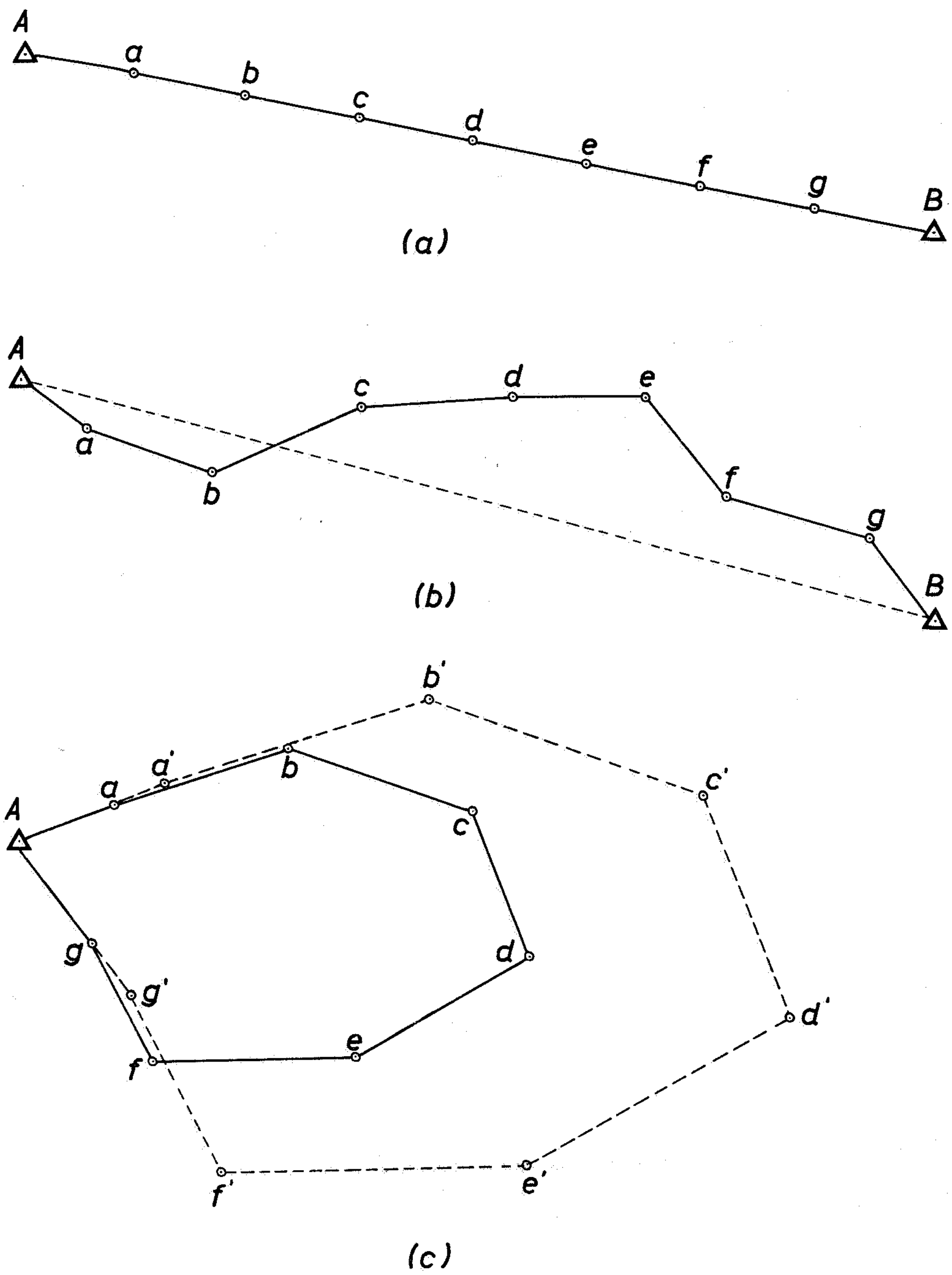


Figure 3.1.5 Shapes of traverses

In the above-mentioned figures and descriptions known points are introduced, with known coordinates from former triangulations, but in most cases that a hydrometric survey or hydrographic survey has to be carried out, the area is often in unsurveyed regions.

For the hydrographer and engineers who have to carry out their survey, it is

essential to be provided with shore beacons in order to establish their position at sea for the current measurements, bathymetry etc..
A traverse could therefore be made along the stretch of coast by starting from an arbitrary point with coordinates zero or a round figure, the azimuth bearing to the next point in the traverse can be obtained by compass bearing (taking into considerations the magnetic influence like variation in the area and the deviation of the compass) or by an astronomical azimuth on the sun. The traverse to the next points should be done by theodolite for the angle measurements and by electronic distance-meter for the linear measurements. All points can then be plotted graphically or calculated in arbitrary coordinates. On a later date or if more time is available the start or the end point can be connected to an existing triangulation benchmark.
The survey can start as soon as the positions of the beacons are known in respect to each other.

3.1.7 Elaboration of traverses

3.1.7.1 Angle adjustment

The sum of all the measured angles, added to the initial bearing, must give the closing bearing. Because of unavoidable and accidental errors, which will occur in all survey work, there will always be a discrepancy which, however, must always be kept within certain allowable limits, depending upon the accuracy requirements for the particular type of survey that is being made.
For low order traverse, with short sides, this limit is sometimes determined by the formula

$$f(\alpha) = 1.5' \cdot \sqrt{n}.$$

where: $f(\alpha)$ is misclosure factor
n is the number of angles measured.

Any misclosure which is found by comparison with the known closing bearing is distributed equally amongst the angles.

3.1.7.2 Calculations of bearings

The bearing of each traverse side is obtained by adding the adjusted angle to the bearing of the previous side, less 180^{o} (results coming to more than 360^{o} must be decreased by this amount of course).

For example: (see Figure 3.1.6).

Initial bearing between point A en B	$= 340^{o} - 30' - 15.3''$
	$= 230^{o} - 15' - 17.0''$
total	$570^{o} - 45' - 32.3''$
	$- 180^{o} - 00' - 00.0''$
	$390^{o} - 45' - 32.3''$
	$- 360^{o}$
bearing B-C	$30^{o} - 45' - 32.3''$

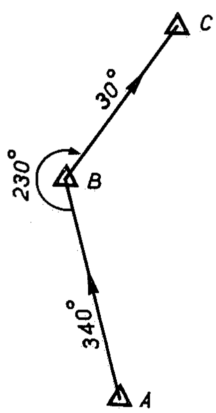


Figure 3.1.6 Illustration of applying angle to bearing

3.1.7.3 Calculations of coordinates

The traverse length (distance) multiplied by sin t and cos t, respectively, gives the values Δx and Δy, for each side (see Figure 3.1.7a).
Particular attention must be given to the changing significance of the sin- and the cos-functions if the various quadrants and their effects on the signs of Δx and Δy. The sum of the Δx and the Δy and the initial coordinates of the preceding point, gives the coordinates of the successive traverse points.
In Figure 3.1.7b a sketch of a open traverse is given, while in Figure 3.1.7a the calculation of coordinates is elaborated with an initial bearing from a known point to point A of $20^{o} - 14' - 19.1''$.

Station	Angle α	Bearing t	Distance d	sin t	cos t	Δx sin t * d		Δy cos t * d		coordinates X	Y
	° ' "	° ' "	m			+ m	− m	+ m	− m	m	m
		20-14-19.1									
A	251-41-56.7									586.602.55	393.283.40
		91-56-09.8	1803.76	+0.999429	-0.033784	1802.73			60.94	588.405.28	393.222.46
B	139-15-41.6										
		51-11-51.4	1595.03	+0.779312	+0.626636	1243.03		999.51		589.648.31	394.221.97
C	139-39-39.7										
		10-50-51.1	1269.52	+0.188196	+0.982131	238.92		1246.84		589.887.23	395.468.80
D	210-04-36.6										
		40-55-27.7	1609.51	+0.655062	+0.755575	1054.33		1216.11		590.941.56	396.684.91
E	139-05-02.1										
		00-00-29.8	1126.08	+0.000144	+0.999999	0.16		1126.08		590.941.72	397.810.99

Figure 3.1.7a Example of calculation

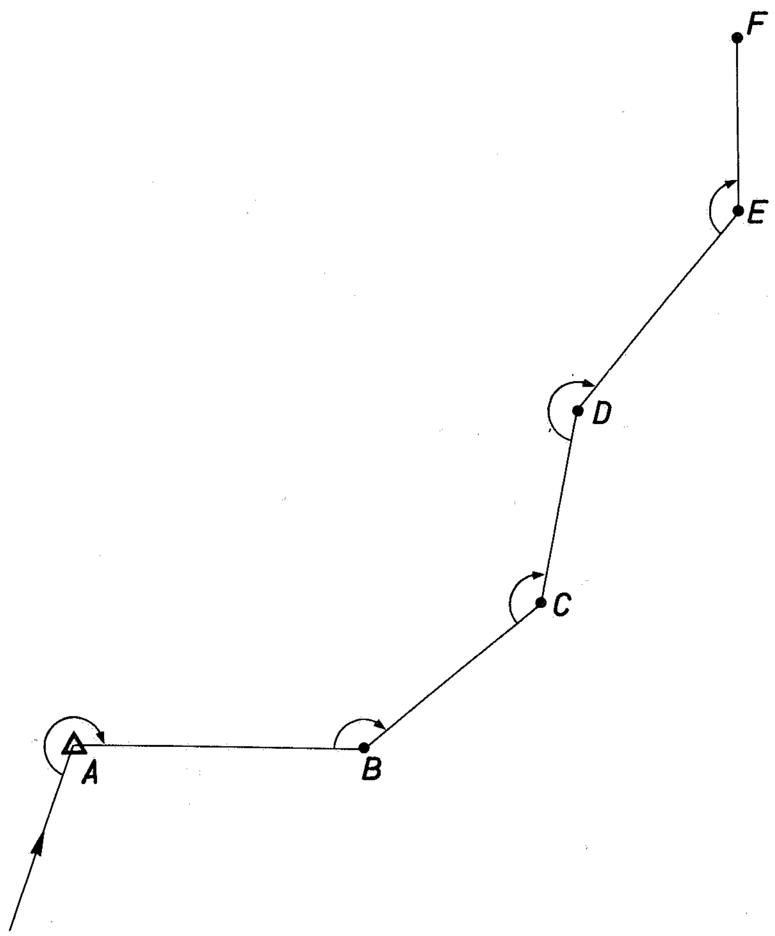


Figure 3.1.7b Sketch of open traverse

3.1.7.4 Adjustment of coordinates

Any misclosure with the final coordinate value of the last point (if coordinates are known from former triangulations) compared with new triangulated coordinates is distributed among the individual Δx and Δy values. The usual method being to make each adjustment in direct proportion to the length of the sides.

A permissible misclosure for low order traversing would be

$$f(D) = 0.01 \cdot \sqrt{D}$$

where: f(D) is misclosure factor

D is the total length, in meters.

The vector misclosure f(D) is made up of the misclosure of the coordinates f(x) = x(is) - x (should) and f(y) = y(is) - y(should) giving

$$f(D) = \sqrt{(f(x)^2 + f(y)^2)}$$

To adjust the calculated coordinates the method described below belonging to Bowditch can be used.
This method depends on the assumption that the standard errors in length and bearing produce equal displacements at the end of a leg and that standard errors of linear measurement are proportional to the square roots of the lengths of the legs. According to this theory this method is suitable to the adjustment of compass rather than theodolite traverses, but its simplicity has brought about its use in nearly all traverse adjustments, except those intended for primary geodetic purposes. It may, however, safely be used in all hydrographic work.

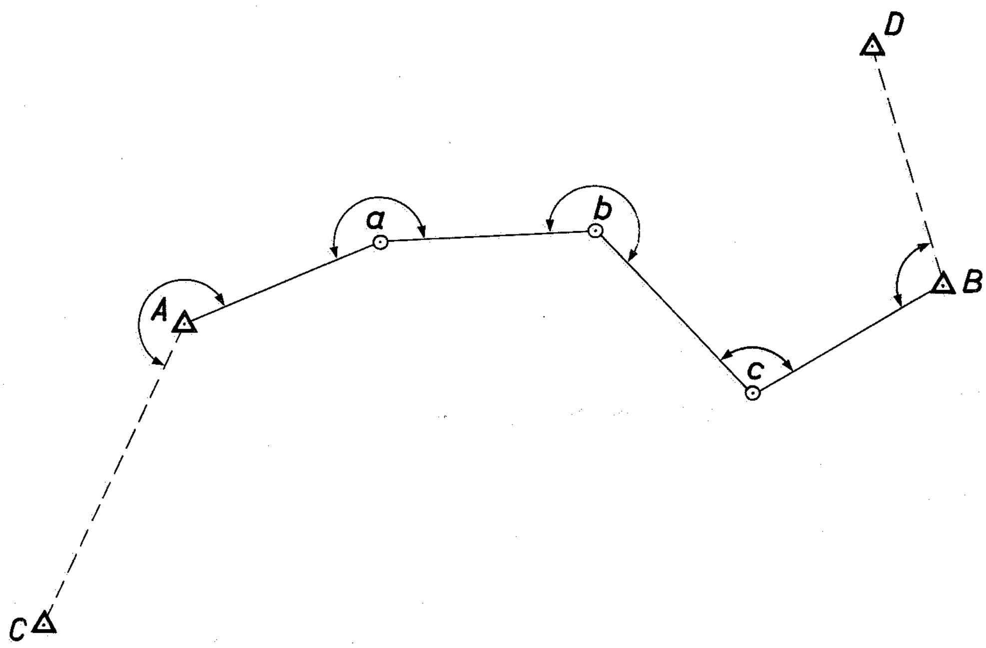


Figure 3.1.8 Adjustment of traverse

Referring to Figure 3.1.8, it is assumed that the coordinates of the points a, b, c and B have been calculated from grid lengths and adjusted or unadjusted grid bearings.
The coordinates of B, calculated through the traverse, will not usually agree with its known coordinates.

The adjustment consists of apportioning this discrepancy in a regular way throughout the traverse.

The procedure is as follows:

- Obtain the difference of Eastings (X) between the value of B computed through the traverse and its known value.

  Divide this difference by the total length of the traverse i.e. the sum of the distances Aa, ab, bc; call this quantity (m).
- Then the correction to the Easting of each turning point is the product of (m) and the total traversed distance to the turning point. In this case the correction will be:

  to the Easting of a: m * Aa

  to the Easting of b: m * (Aa + ab)

  to the Easting of c: m * (Aa + ab + bc) etc..

  This last value is of course the original discrepancy in Easting at B.
- Repeat the procedure for the Northing.

3.1.8 False Stations

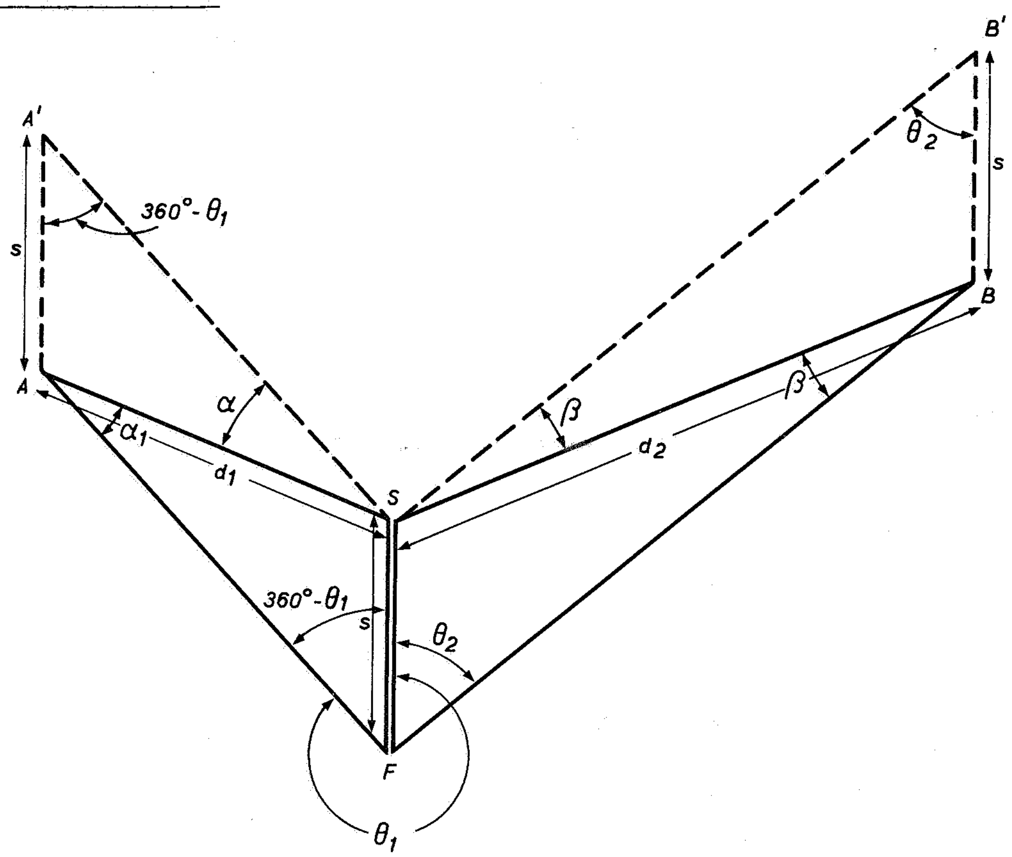


Figure 3.1.9 False station correction

It is not always possible, when measuring horizontal angles, to centre the theodolite or other instrument directly over the observing station. The horizontal circle readings obtained at the False (or satellite) Station must therefore be corrected to what they would have been had they been measured at the true station. It is important to note that the false station corrections are always applied to horizontal circle readings, and not to angles; if the angles between various stations have been measured with a sextant, they should always be converted to circle readings (based on one reference object) before working out and applying false station corrections.

In Figure 3.1.9 horizontal circle readings have been observed at the false station F to A and B. It is required to calculate what these circle readings would have been had they been observed at the true station S. The theodolite has been zeroed on A, and circle readings obtained to B and the true station S; the small distance s between the false station and the true station has been measured in the field and the distances d_1 and d_2, between S and A and B respectively, have been measured off a rough plot.

Take the circle reading to A first. Since A is the zero it is 360° 00' 00". The circle reading to the true station S has been measured, and from the diagram it is $360^\circ - \theta_1$; θ_1 is the difference between the circle reading to the true station S (note that in this case θ_1 is greater than 180°).

Imagine the theodolite at F with rays extending from it to S and A; now move the theodolite to S making certain that the original ray to S still lies along FS produced; the original ray to A will now pass through A', where FS = AA' = s. At S therefore the original circle reading to A will be too great by the small angle α, and from the sine formula:

$$\frac{s}{\sin\alpha} = \frac{d_1}{\sin(360^\circ - \theta_1)} = -\frac{d_1}{\sin\theta_1}$$

by re-arranging this

$$\sin\alpha = -\frac{s.\sin\theta_1}{d_1} = -\frac{\text{False station distance} * \sin\theta_1}{\text{Distance from true station to observed object}}$$

Note that from the diagram α must obviously be subtracted from the circle reading at F to obtain the corrected circle reading at S, and from the formula it also comes out as a minus quantity.

Now consider the ray into station B. The false station correction is β, and from the figure it must be added to the circle reading obtained at F to B.

θ_2 is the difference between the circle readings at F to B, and the true station S (note that it is now less than 180^{o}).
From the sine formula:

$$\sin\beta = \frac{s.\sin\theta_2}{d_2}$$

and β is therefore positive.

3.1.8.1 The general formula for false stations

Putting the arguments above into general terms, the false station formula becomes:

$$\sin\begin{pmatrix}\text{Correction} \\ \text{circle reading}\end{pmatrix} = \frac{\text{False station distance} \times \sin\theta}{\text{Distance from true station to observed object}}$$

where θ is the difference between the circle reading to the observed object and the circle reading to the true station, both measured at the false station.
The correction must be added to the circle reading obtained at the false station if θ is less than 180^{o}, and subtracted if θ is more than 180^{o}.

3.1.8.2 Errors introduced by false stations

Error can be introduced into a false station correction by the method used for calculation, but this is more than likely to be swamped by errors made in the field measurements.
To maintain an accuracy of 0".2 in the correction:

(i) θ should be known to the nearest 1' of arc.

(ii) The distance from the true station to the station observed to must be known to within 1 part in (10× the correction in seconds), e.g. if the correction is about 15" the distance must be known to within 1 part in 150. It may generally be taken off a rough plot.

(iii) The false station distance must be known to within 1 part in about 2,000,000 of the distance from the true station to the station observed to; e.g. if the distance between stations is 3 km, the false station distance must be measured to within 1,5 cm.

To maintain an accuracy of 1" these allowances may be increased five times. It can be seen that the measurement of the false station distance is the critical factor. If the measurement cannot be made along the level, particular care must be taken to correct it for slope. If the false station distance is more than 1/1000 of the distance between the true and observed stations (this may give a correction of about 3½') the correction is almost bound to introduce error, however, much care is taken over the measurements. Four basic rules can be laid down for dealing with false stations:

(i) Avoid using them if possible.

(ii) If they have to be used, make them as small as possible.

(iii) Measure the false station distance as accurately as possible with a reliable tape.

(iv) Except in exceptional circumstances do not apply a false station correction of more than 3' to a main angle.

Calculating false station corrections

Several methods may be used, depending on the accuracy required. Provided the conditions above have been fulfilled:

(i) If the correction is greater than 40' and is required accurate to 0".1 it should be calculated by the full formula:

$$\sin \text{Correction} = \frac{\text{False station distance} \times \sin\theta}{\text{Distance from true station to observed object}}$$

The distances should be in the same units.

(ii) If the correction is required accurate to 0".1 and is not greater than 40', or if it is required accurate to 1" and is not greater than 80', it may be calculated by the following simplified formula:

$$\text{Correction in seconds} = \frac{\text{False station distance} \times \sin\theta \times 206{,}265\dagger}{\text{Distance from true station to observed object}}$$

At any one false station the term:

False station distance $\times$ 206,265 (figure 206,265 = cosec 1")

will be a constant, and for machine working the formula then becomes:

$$\text{Correction in seconds} = \frac{\text{Constant} \times \sin\theta}{\text{Distance from true station to observed object}}$$

3.1.9 Determination of Sun's azimuth

3.1.9.1 General

In order to determine the direction between two triangulation or traverse markers in relation to the True North, the horizontal angle between this direction and the direction of the Sun has to be measured simultaneously with the altitude of the Sun above the horizon.
The azimuth of the Sun can then be calculated and the horizontal angle added to this azimuth will give the direction between the two points in relation to True North.

The following instruments are required for this measurement
- one theodolite with tripod
- one Roelofs prism
- one chronometer
- one thermometer
- one barometer.

3.1.9.2 Principle

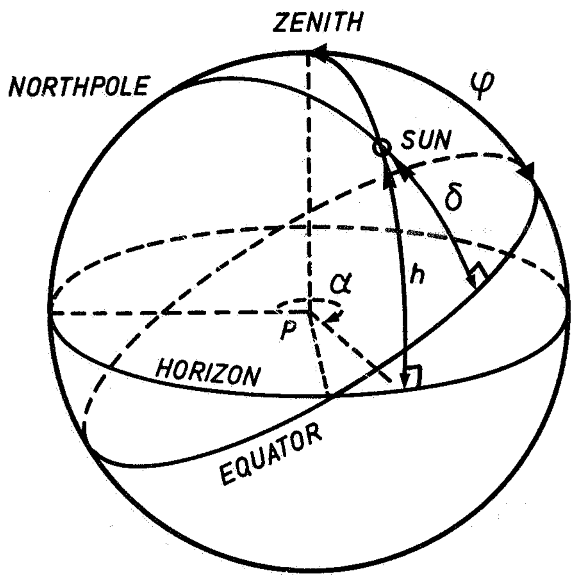


Figure 3.1.10 Sphere

The azimuth of the Sun is calculated in the spherical triangle Northpole-Zenith-Sun, out of three parameters viz.

- the measured altitude of the Sun (h)
- the declination of the Sun (δ)
- the geographical latitude of location of observation (ϕ).

The declination of the Sun can be derived from the Nautical Almanac on the date of observation as a function of Greenwich Mean Time (GMT).
The latitude of observation can be derived from the hydrographic chart in the nearest minutes.

The calculation can be carried out using the formula:

$$\cos a = \frac{\sin\delta - \sin\phi \sin h}{\cos\phi \cosh}$$

In Figure 3.9.11 only the spherical triangle is illustrated for clearness sake.

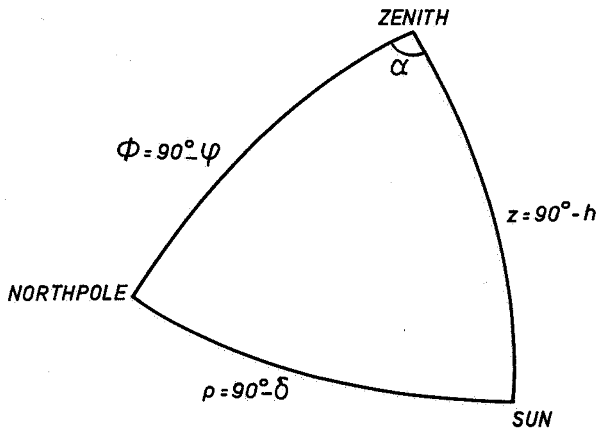


Figure 3.1.11 Spherical triangle

3.1.9.3 Corrections on measured altitude and direction

There are two corrections which has to be applied caused by
- astronomical refraction
- parallax.

3.1.9.4 Astronomical refraction

The atmosphere refraction plays an important role during observations on celestial bodies as the rays enter denser airlayers which causes a refraction to the normal, see Figure 3.1.12.

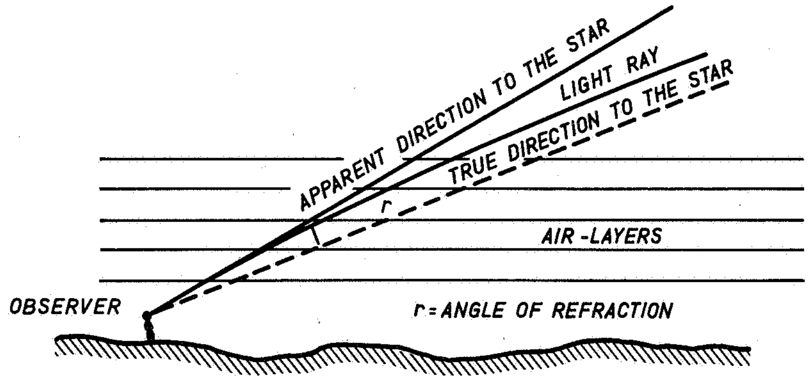


Figure 3.1.12 Refraction

The correction to the measured altitude of the celestial body is a function of the altitude (h) and the barometric pressure (p) and the temperature (t) at the place of observation.

The refraction can be found from the tabel Figure 3.1.13 which has been developed by Prof. Roelofs.

Refraction $z = \frac{h}{270+t}$ p in millimetres, t in centigrades, h = uncorrected altitude of Sun.

$r = \frac{p}{270 + t} R$			p in millimeters			t in Centigrades		
h'	R	10^4 vers. per 1'	h'	R	10^4 vers. per 1'	h'	R	10^4 vers. per 1'
15°	78.35	- 828	40°	25.40	- 147	65°	9.95	- 75
16	73.38	- 740	41	24.52	- 140	66	9.50	- 73
17	68.94	- 662	42	23.68	- 137	67	9.06	- 73
18	64.97	- 598	43	22.86	- 130	68	8.62	- 72
19	61.38	- 542	44	22.08	- 125	69	8.19	- 70
20	58.13	- 493	45	21.33	- 122	70	7.77	- 70
21	55.17	- 452	46	20.60	- 118	71	7.35	- 68
22	52.46	- 415	47	19.89	- 113	72	6.94	- 68
23	49.97	- 383	48	19.21	- 112	73	6.53	- 68
24	47.67	- 355	49	18.54	- 107	74	6.12	- 67
25	45.54	- 330	50	17.90	- 103	75	5.72	- 67
26	43.56	- 308	51	17.28	- 102	76	5.32	- 65
27	41.71	- 287	52	16.67	- 98	77	4.93	- 65
28	39.99	- 270	53	16.08	- 97	78	4.54	- 65
29	38.37	- 253	54	15.50	- 93	79	4.15	- 65
30	36.85	- 238	55	14.94	- 92	80	3.76	- 63
31	35.42	- 227	56	14.39	- 88	81	3.38	- 63
32	34.06	- 213	57	13.86	- 87	82	3.00	- 63
33	32.78	- 202	58	13.34	- 87	83	2.62	- 63
34	31.57	- 192	59	12.82	- 83	84	2.24	- 63
35	30.42	- 183	60	12.32	- 82	85	1.87	- 63
36	29.32	- 175	61	11.83	- 80	86	1.49	- 60
37	28.27	- 167	62	11.35	- 78	86.5	1.31	- 63
38	27.27	- 158	63	10.88	- 78	87	1.12	- 63
39	26.32	- 153	64	10.41	- 77	87.5	0.93	- 60
40	25.40		65	9.95		88	0.75	- 63
						88.5	0.56	- 63
						89	0.37	- 60
						89.5	0.19	

For example:

$h' = 46°11'$

$p = 754.3$ mm $R = 20.47$

$t = 17°.6$ C

$$r = \frac{754.3}{287.6} * 20.47 = 53.7''$$

Figure 3.1.13 Table to calculate refraction

3.1.9.5 Parallax

The second correction which must be applied to the measured altitude, because the place of observation is not in the centre of the earth but at its surface, is the parallax.

This correction is p = 8".8 cosh

if h = 0^o - 10^o - 20^o - 30^o - 40" - 50"

p = 0" - 9" - 8" - 8" - 7" - 6".

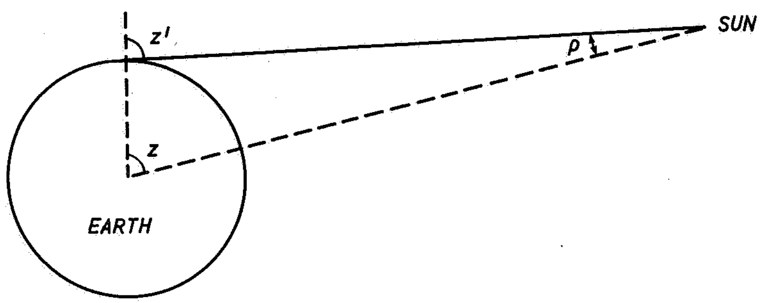


Figure 3.1.14 Parallax

The sign of the corrections are apparently negative for the refraction and positive for the parallax

h = h' - refraction + parallax

z = z' + refraction + parallax.

3.1.9.6 Roelofs prism

The Roelofs prism is a device to be attached to the theodolite's object-lens in order to measure the altitude and direction of the Sun. Contrary to the normal coloured filters, no meridian corrections have to be made and the measurement can be simply accomplished.

The vertical and horizontal axis of the Sun's rosette is made to coincide with the vertical and horizontal reticle lines.

With the horizontal drive-screw the vertical axis of the Sun's rosette is kept on the vertical reticle line until the horizontal axis of the Sun's rosette coincides with the horizontal reticle line.

The time of coinciding is then noted down.

In Figure 3.1.15 the principle of the Roelofs prism is illustrated.

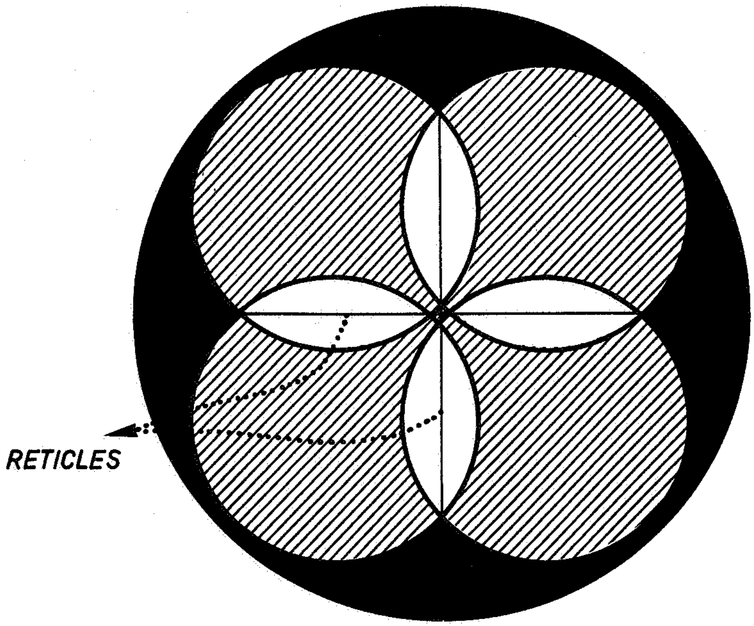


Figure 3.1.15 Principle of Roelofs prism

3.1.9.7 Measuring procedure

1st serie
1. Set-up the theodolite accurately, and centre the circular bubble level and the plate level
2. Sight the terrestial object and read the horizontal circle.
3. Sight the celestial body after coinciding the vertical index level, read the vertical scale and horizontal circle and note down the time
4. Repeat reading 3 and note down the time
5. Repeat reading 2

2nd serie
6. Transits the telescope in Face Right position
7. Repeat reading 2, 3, 4, 5, noting down the time at each Sun's observation

3rd serie
8. Change the horizontal circle setting 45^{o} and repeat readings 2, 3, 4, 5, noting down the time at each Sun's observation

4th serie
9. Transits the telescope in Face Left position
10. Repeat reading, as indicated in 2, 3, 4, 5, noting down the time at each Sun's observation.

Depending on the required accuracy, the number of series could be extended, changing the circle setting 45^{o} and transiting the telescope as indicated above.

On account for the uncertainty in the determination of the refraction factor the measured altitude (h) of the Sun can not be too small, therefore h = 15^o to 20^o viz early in the morning or late in the afternoon.
To eliminate systematic errors in ϕ and h, observations should be taken twice, once in the morning and once in the afternoon.

3.1.9.8 Calculation

Calculate the azimuths of the Sun after applying the corrections for parallax and astronomical refraction, add the corresponding horizontal angle to the terrestial object.
The results from the Face Left and Face Right observations can then be summed up and divided by the number of observations to obtain the average azimuth.

Example of calculation form:

Chronometer reading		=
Difference to GMT	$\pm$	=
GMT		

or:

Local actual time	$-$ $+$	=
longitude in time East/West		=
Greenw. actual time (estimate)		
"e"		
Greenw. Mean Time (estimate)		

δ (declination)		=
ϕ (latitude)		=
h (measured)		=
refraction	$\pm$	=
h.o		=
parallax	+	=
h (corrected)		
$\sin \delta$		=
$\sin \phi \sin h$	$-$	=
I		=
$\cos \phi$		=
$\cos h$	x	=
II		
$\cos a = \frac{I}{II}$		=
azimuth to sun (a)		=
angle to terrestial object	$\pm$	=
azimuth terrestial object		=

Note: if no chronometer is available then the actual local time can be taken.
"e" is equation of time to be derived from almanac.

3.2 Position fixing

3.2.1 General

A distinction has to be made between the manner of position fixing in rivers and in estuaries or along open coast in view of the different purpose of the various measurements and the restricted area in rivers in comparison with the more wider areas in estuaries and along coasts.

In rivers, the bathymetric survey consist mainly of cross-sectional soundings and soundings along the talway. Soundings can be taken in connection with current velocity measurements in order to determine the river discharge or total sediment transport while the measuring vessels have to be positioned in one line across the river on pre-selected locations. While in estuaries and along open coasts the bathymetric survey is carried out along tracks as much as possible perpendicular to the depth contour-lines.
Possible current velocity measurements are generally not with the purpose to calculate discharges but to obtain an insight into the velocity and direction of the current this in view of sediment transport, navigation purposes and design of channel allignments.

This implies that in rivers in many occasions means have to be made in order to facilitate the crossing of a river by the survey vessel on a line perpendicular to the river flow. In general this is done by a second beacon at a certain distance behind the existing triangulation marker thus creating leading-lines across the river.
The existing triangulation marker can be any conspicuous object like painted trees, flags or other marking facilities.
In coastal areas, estuaries and wide rivers alignment normally does not take place with two beacons and the vessels movement in relatively free. The shore beacons should be high and of such dimension as to facilitate sighting from a far distance, also the colour of the top marks should be such that it is in contrast with the back-ground.
The beacons should be established preferably on one line or the centre of a serie of three beacons should be re-entrant (see Figure 3.2.1).
Position fixes can be carried out with optical methods or electronic methods.

3.2.2 Optical methods

When position-fixing is carried out by optical means the following instruments can be used:
- sextants
- theodolites
- range-finders.

3.2.2.1 Two sextants (three beacons)

Two angles between three beacons are measured simultaneously by two observers. The position of the boat from which the angles are observed, is established during the soundings, either by:
1. setting the angles on a station-pointer, obtaining a graphical solution
2. plotting in a chart of arcs
3. calculating the grid coordinates of the unknown position directly from the coordinates of the beacons.

Care should be taken that the angles are not smaller than 15 degrees and not larger than 90 degrees. In some cases when the cutting is still reasonable, smaller angles and larger angles than as given above are allowed.
The described method is called the Snellius method of resection. It is the most common one to locate the position of a moving boat during soundings.

If this method is used, attention must be paid on the position of the consecutive beacons which are to be used.
In case that the centre-beacon out of three consecutive beacons is entrant-wise, the cuttings are not optimal and even an area of no cuttings exists around the circumscribed circle of the three beacons.
The best cuttings and more accurate positioning are obtained when the centre beacon is re-entrant wise, there never can be an area of the circumscribed circle (see Figure 3.2.1).

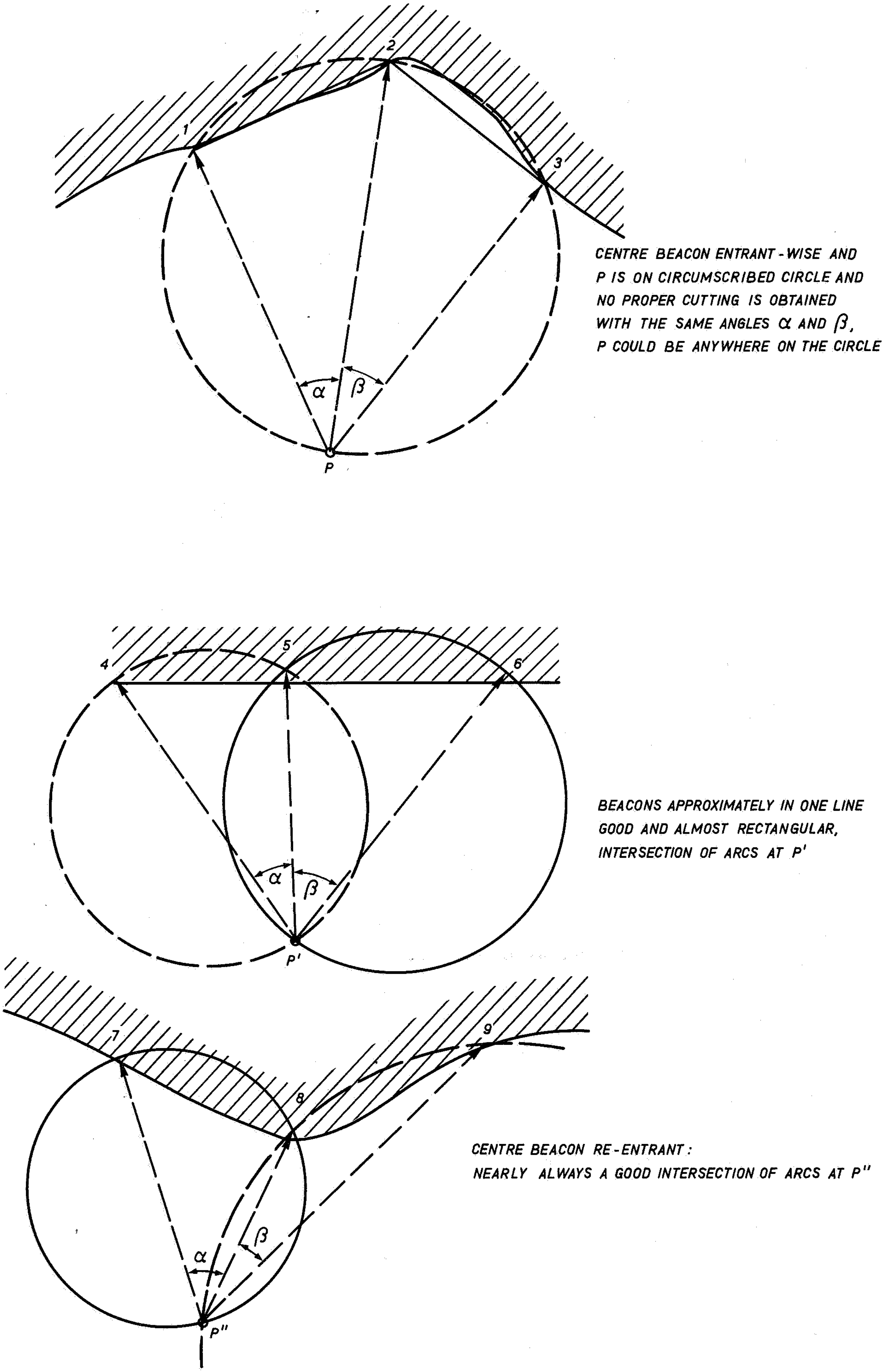


Figure 3.2.1 Beacons on shore

3.2.2.2 One sextant and leading line

The observer is in line with two beacons, and measures the vertical angle between two markers, one on the top and one on the foot of one of those beacons to obtain his distance to this beacon or measures the horizontal angle between two other beacons to obtain a cutting with the leading-line.

3.2.2.3 Two theodolites

In this method, the position of the vessel is observed from the shore side. Two observers, each with a theodolite, in constant communication with each other and the vessel measure the direction to the vessel simultaneously in respect to a reference direction.
The two theodolite locations should be known in coordinates.
The position of the vessel can either be plotted or calculated.

3.2.2.4 One range-finder

The observer is in line with two beacons and measures the distance to one of the beacons.

3.2.2.5 Angular method (Figure 3.2.2)

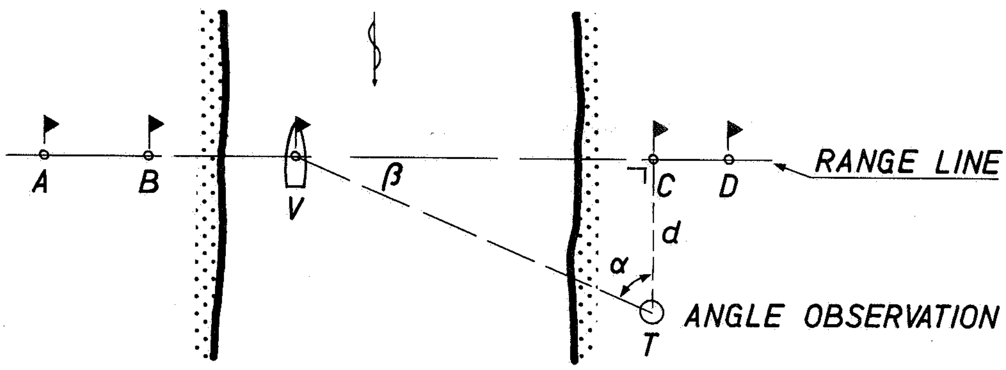


Figure 3.2.2 Angular method $VC = CT\ tg\alpha$

A theodolite is set-up on one of the banks and angular measurements are taken to the boat. It is also possible to take angles with a sextant from the vessel to flags in the range line and to the reference point T.

This method is suitable for fixing the boat's position taking soundings.

3.2.2.6 Linear measurement (Figure 3.2.3)

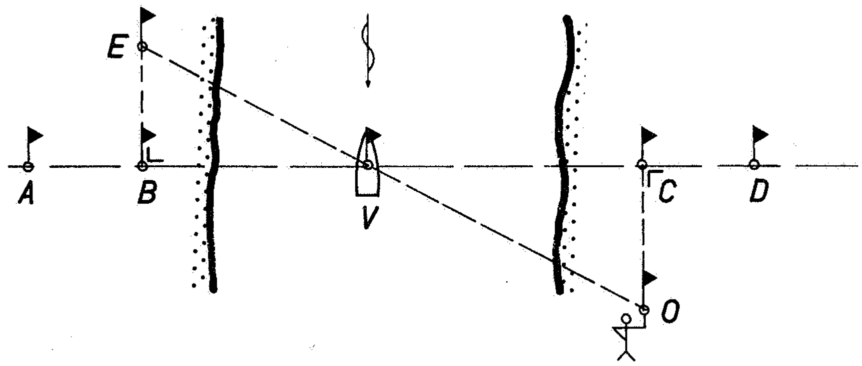


Figure 3.2.3 Linear measurement $VC = \frac{BC \times CO}{CO + BE}$

The flags A, B, C, D are fixed in the range line. Flag E is fixed along a line perpendicular to AD and with a known distance from B. The observer O moves along a line also perpendicular to AD until the flag E and flag V on the vessel and the observer's flag O are in one line. Then the distance CO is determined and VC can be computed.
If the river is very wide so that the flags on the opposite bank are not clearly visible, the boat's position can be fixed at the method given in Figure 3.2.4 , where flag E is fixed on the same bank as the observer.

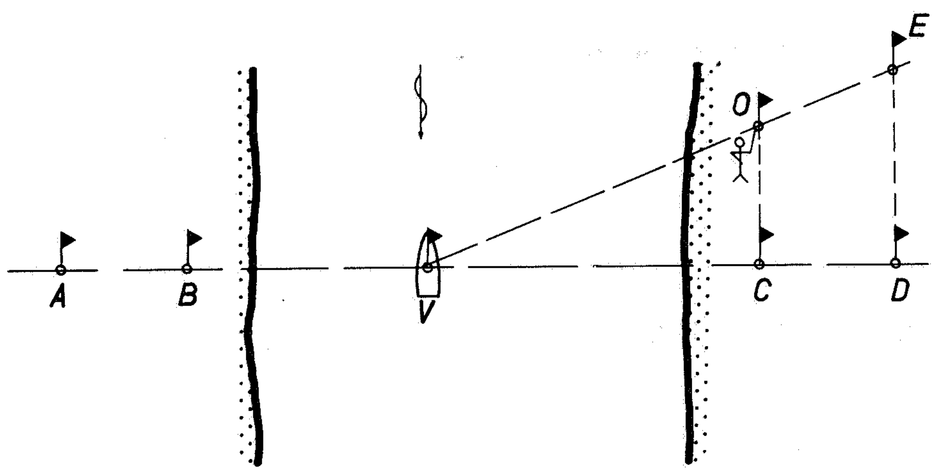


Figure 3.2.4 Linear measurement $VD = \frac{DE \times CD}{DE - CO}$

3.2.2.7 Pivot-point method (Figure 3.2.5)

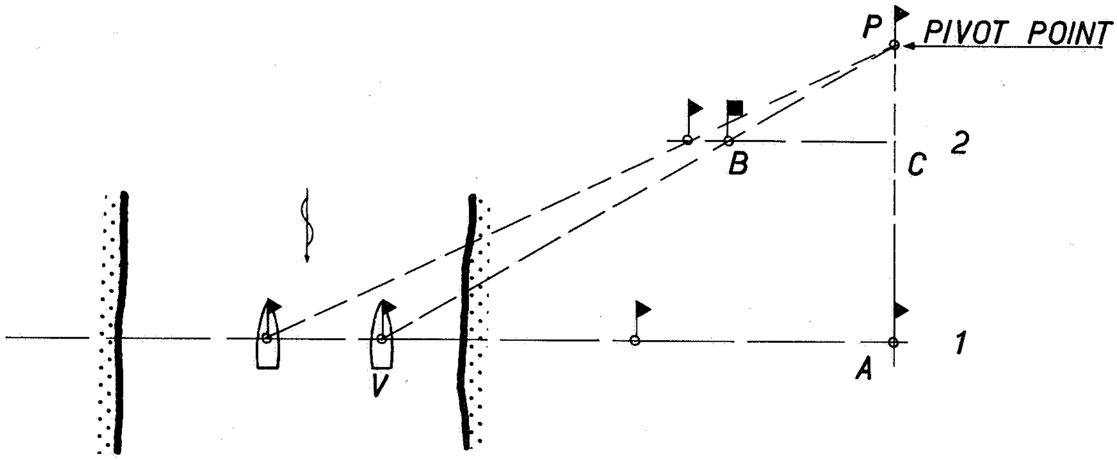


Figure 3.2.5 Pivot-point method $VA = \frac{BC \times PA}{PC}$

If the river is wide and the banks of the river are flat the so called pivot-point method can be used. The distance PA is more or less the same as the width of the river. The distance between range line 2 and pivot-point P is about 20% of PA. On range line 2 characteristic beacons are fixed at intervals depending on the distance between the selected verticals in the river. The vessel moving in the cross-section can be fixed by lining up the selected beacon and the pivot-point. A second set of pivot-points on the other bank can be used if required e.g. because of the width of the river.

3.2.3 Electronic Positioning Methods

3.2.3.1 General

First, two groups of systems have to be distinguished:

1. Electronic Positioning Systems for Navigation.
2. Electronic Positioning Systems for Hydrography or Surveys.

The first group has to provide ships with the means of obtaining their position fix at any time and under any conditions, with sufficient accuracy to permit following a selected course or maintaining a reasonable distance from dangers.

The second group is designed to provide a position to survey ships, with the best possible accuracy. In the case of hydrographic work this accuracy must obviously be greater than that required for navigation.

Therefore, a navigational system usually does not have the necessary qualities to allow accurate surveys to be carried out, whereas a hydrographic system, in principle, may always be used in navigation, although supplying information of superfluous accuracy.

In this short description of a few systems, only the hydrographic system will be dealt with. This will give an insight into the navigational system as well, since it is operating on the same principles, differing only in accuracy and range.

The various systems can be characterised with respect to their transmissions, by the wave frequency which they use, their power and signal type.

Basically, all systems transmit electromagnetic waves, which are received on board of the ship.
Some systems measure the time, elapsed between the moment of sending the signal, and receiving it: this will give, when the propagation speed of the wave is known, the distance from each transmitter to the receiver, and thus the position of the ship, when the location of the transmitters is known.
Other systems measure phase differences between continuously transmitted waves from different transmitters, this is more complicated, and will be dealt with later on.

The systems can also be distinguished in:

1. system using "range-range" patterns for position fixing. This means that distances are measured from the ship to two shore based stations. This can be done by measuring time difference or phase difference. In these systems there can only be one user at the time, although some systems can be operated on a time-sharing basis by up to 4 users (see Figure 3.2.6)
2. systems using "hyperbolic" patterns, where phase differences are measured between transmissions from 3 shore based stations. Since the "master" station in these systems is shore based, these are the multi-user systems (see Figure 3.2.7).

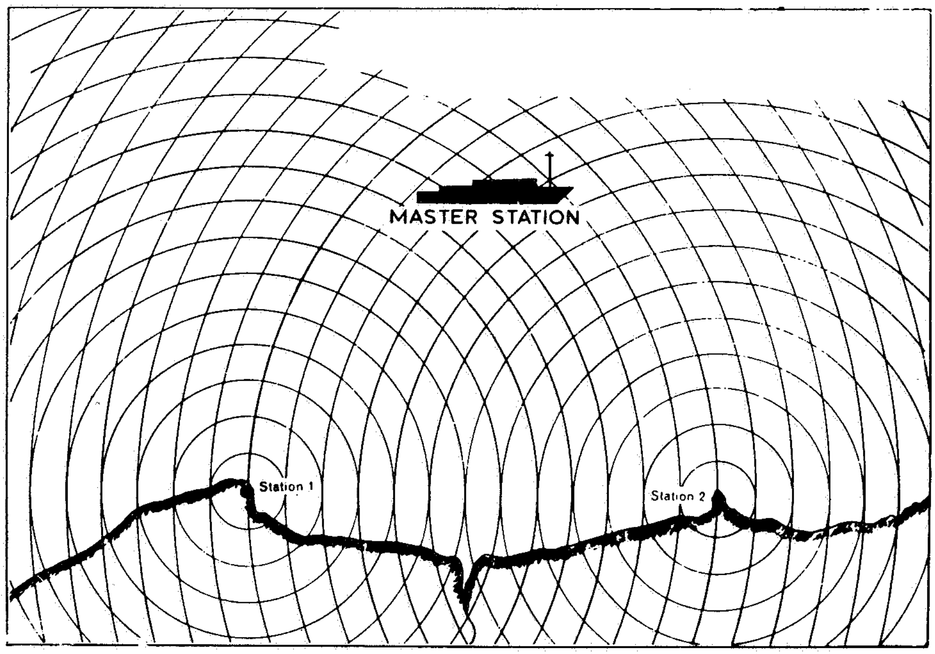


Figure 3.2.6 Ranging patterns

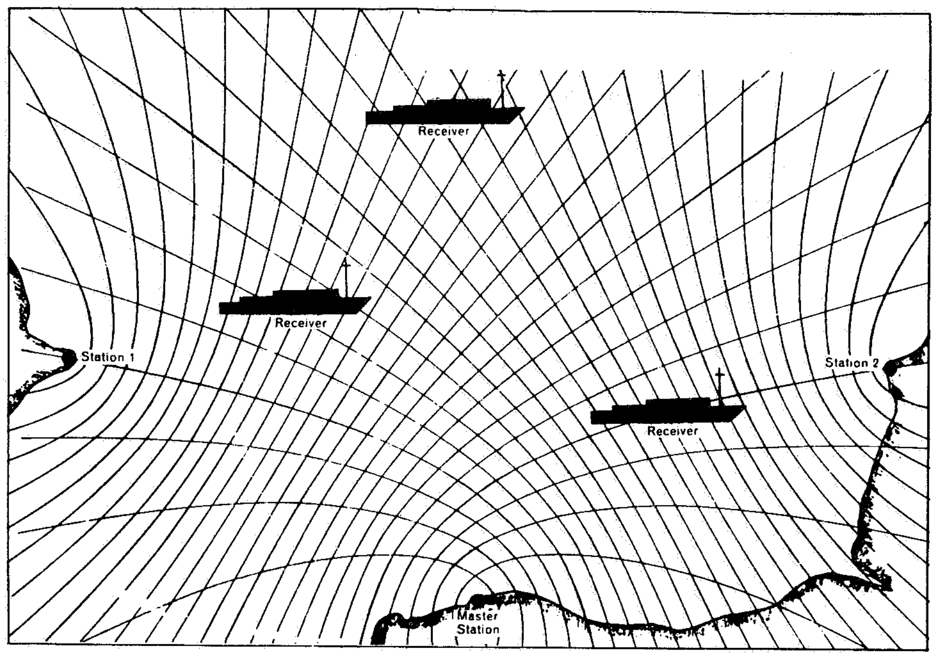
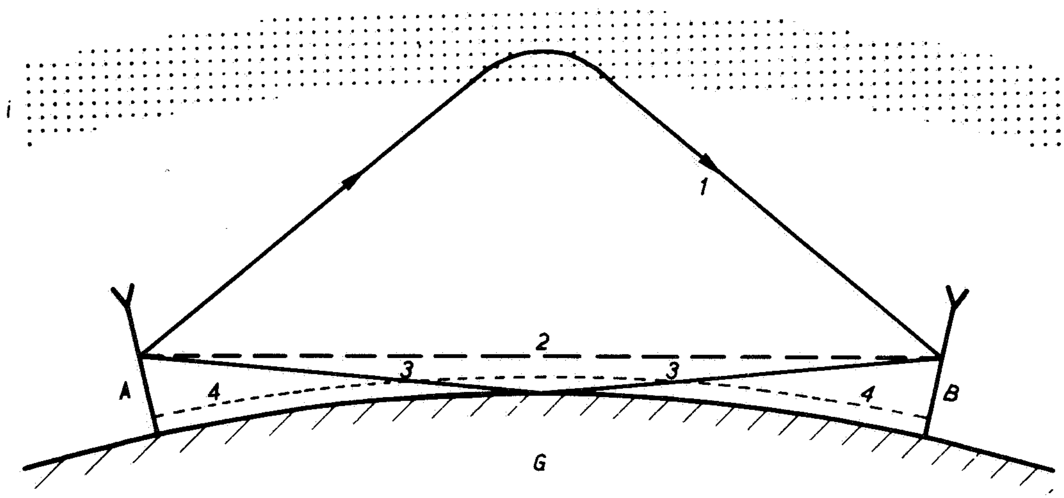


Figure 3.2.7 Hyperbolic patterns

3.2.3.2 Paths of radio-electric micro waves

Radio-electric waves radiated by a transmitting aerial can reach the receiving aerial by various paths.
Figure 3.2.8 shows the paths of these waves between transmitting and receiving aerial units.
Long distance communication is provided by the sky wave. Owing to the curvature of the earth, the direct and ground reflected waves only enter into consideration when the transmitting and/or receiving aerial are of sufficient height in relation to the path to be covered. This is the type of transmission used in ultra-short wave, micro wave and radar communication. Surface waves are considered mainly in the medium and long wave field, for short range radio communication and radio broadcasting purposes.



G = Ground	1 = Sky wave
A = Transmitting aerial	2 = Direct wave
B = Receiving aerial	3 = Ground reflected wave
i = Ionosphere	4 = Surface wave

Figure 3.2.8 Paths of radio-electric and micro waves

SYSTEM	RANGE KM.	ACCURACY	FREQUENCY	USERS	NAVIGATION	SURVEY	NOTES
HYPERBOLIC SYSTEMS							
LORAN A	1500	0.4 - 90 km	2 mc/s	multi	x		
LORAN C	2200	15 - 360 m	100 kc/s	multi	x		
DECCA NAV.	450	0.4 - 4 km	100 kc/s	multi	x		
DECCA SURV.	400	7.5 - 96 m	100 kc/s	multi	x	x	
OMEGA	8334	1.8 - 18 km	10 kc/s	multi	x		
HI-FIX	70	7.5 - 45 m	2 mc/s	multi	x	x	
RANGING SYSTEMS							
SHORAN	30	9 - 15 m	300 mc/s	6		x	
RAYDIST	200	3.6 - 30 m	3.2 mc/s	2		x	
HYDRODIST	25	3.6 - 30 m	3000 mc/s	1		x	
HI-FIX	30	3.6 - 30 m	2 mc/s	2*		x	portable
TRISPONDER	50	3 m	9400 mc/s	4*		x	portable, quickly installed
MINIRANGER	20	3 m	5200 mc/s	4*		x	portable, quickly installed
ARTEMIS	30	1.5 m	9200 mc/s	1		1	

*with time sharing only, without time sharing single user.

Figure 3.2.9 Electronic positioning systems

3.2.3.3 The hyperbolic pattern (Decca Navigator, Decca Survey systems)

From each shore based station continuously unmodulated waves are radiated in all directions. The loci of points of equal phase around one station at a certain time will then be spheres having the transmitting aerial as their centre.

A receiver placed in the service area of two transmitters records the phases of the waves at that place, and its position in the field follows from the phase difference of the two received signals. Position lines then become hyperbolic. The origin of the hyperbolae is explained as follows: assuming that two transmitting stations M (Master) and S (Slave) separated by a known distance b (base-line) radiate continuous unmodulated waves, and further that:
At every moment the phase of the transmitted wave at M is equal to that at S, and
the stations M and S operate on the same frequency (or wave length being immature).

Figure 3.2.10 gives the situation under the above conditions for a specific time. The phases of the emitted waves at M and S are both 0° in this case. The length of one wave is taken as λ cm, and from the diagram it follows that circles represent the loci of points where the phases are zero.

Obviously when circles from M intersect those of S, the differences in phase will be zero as well. The curves connecting intersection points as shown in Figure 3.2.10 will be homophocal hyperbolae as they have the geometric property that on each point thereof the difference in distance from the transmitters remains constant. An example is given for field positions F_1 and F_2. At the time M and S are at phase 0° the phase at F_1 from the M-signal is 120° (+ 6 x 360°) and from S-signal will be 120° (+ 13 x 360°), so the phase difference is 0° (the whole numbers of cycles are omitted, being dealt with later on). Similar considerations show that the phase difference at F_2 will be 0°, as the phases from M and S-signal at F_2 are both 240°.

The speed of propagation of electromagnetic waves is somewhere near 3 x 10^8 m/sec, so a fraction of a second later the concentric circles of Figure 3.2.10 are further away from their origin. This will, however, not effect the position of the hyperbolae - provided M and S remain in phase.

At a fixed position in the field the phases received from M and S will both be higher to the same amount so the phase difference at the observers place will be constant and unvariable with time.

In Figure 3.2.10 only a few 0^{o}-hyperbolae were drawn so as not to overload the drawing. The length of a wave in comparison to the length of the base-line has been made too long.

The actual number of 0^{o}-hyperbolae will be different for each pattern and will depend on the frequency of the transmissions and the length of the base-line.

The area between successive 0^{o}-hyperbolae is called a lane; phase differences measured therein will increase when proceeding towards a Slave side.

A phase difference (Master minus Slave) is measured upto a hundredth of a lane, as the phase-measuring equipment is sensitive to 3.6 arc-degrees ($0.01\ L = 3.6^{o}$). Loci of points with identical fractional lane value will also be homophocal hyperbolae. They can be calculated and drawn on the lattice maps when required.

Three shore based stations will of course give two hyperbolic patterns, crossing each other. The position of a receiver can now be determined by taking the appropriate crossing of the lines of the two patterns (see Figure 3.2.11). Lane identification can also be effectuated by means of introducing sub-frequencies and sub-patterns.

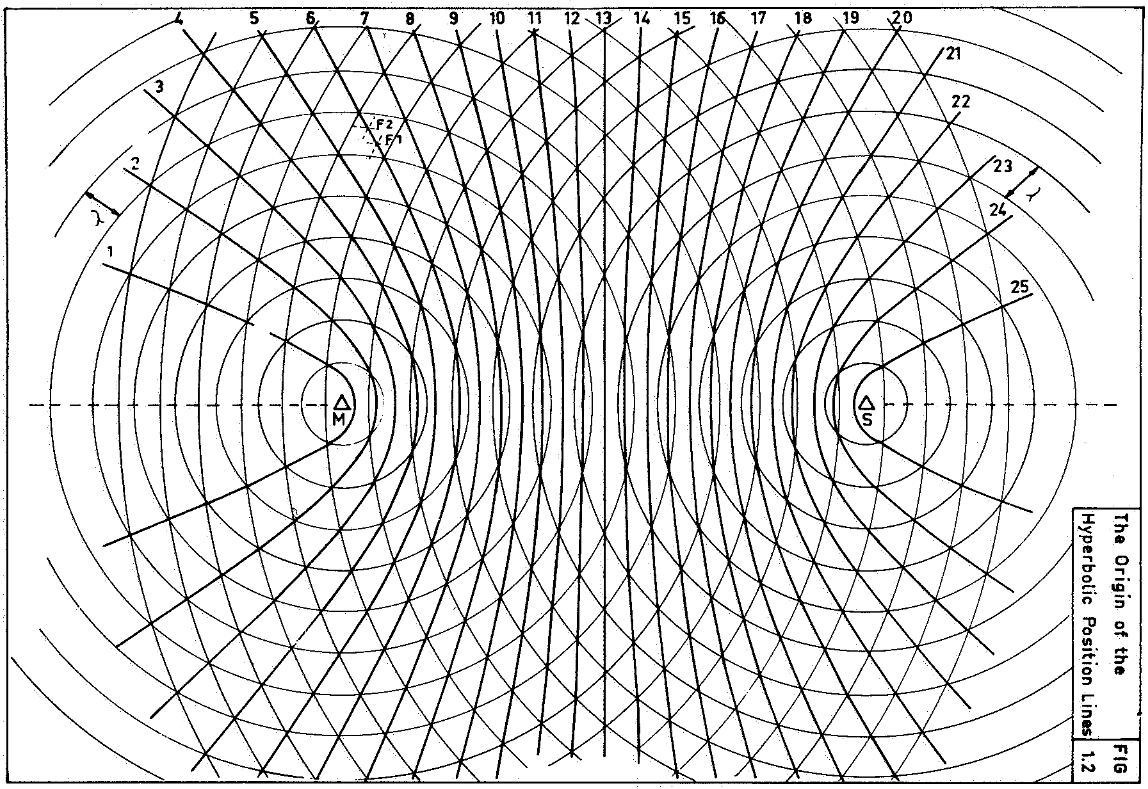


Figure 3.2.10 The origin of the hyperbolic position lines

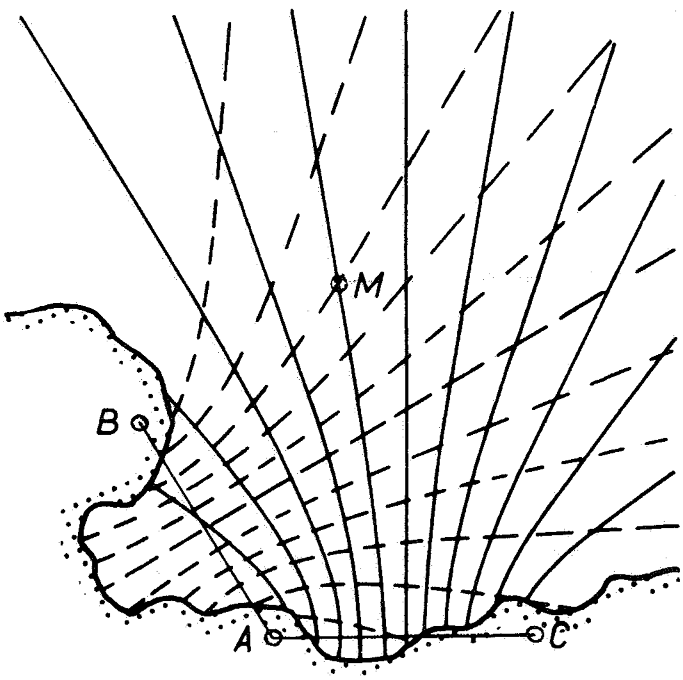


Figure 3.2.11 Hyperbolic Patterns

3.3 Water-levels

3.3.1 General

In non-tidal rivers water-level measurements are essential for the following purposes.

Navigation. The knowledge of water-levels for a full hydrological year are required to obtain information about the least available depth at shallow spots in the river, so that the design of river craft can be adapted to the circumstances or a dredging scheme is set-up in order to create the necessary water-depth for existing river crafts.

Hydrology. In order to obtain stage-discharge relation curves, the slope of the water-level etc.; to make duration curves, frequency of exceedance curves etc., water-level information for a long period are required.

Design. For design purposes of bridges in connection with height above water of vessels to enable them to pass even with the highest flood, - intakes of cooling water inlets correlated to least available water-level, - irrigation channels etc. water-level information is essential.

In tidal-rivers and estuaries water-level measurements are essential to obtain an insight into the propagation of sea-tide and for several other purposes.

Navigation. From the water-level information of at least 29 consecutive days, harmonic constituents can be derived by tidal analysis. A tidal prediction can then be made and this is of importance to give information to the sea-farer about available water-depth at the bar or shallow spots in a river or estuary. Current velocities and directions are often correlated with the vertical tide, which will give information about current conditions at various phases of the tide.

Hydrography. By means of the tidal curve, sounded water-depths can be corrected and reduced to a certain reference plane.

Design. From the tidal analysis the Mean Sea Level can be obtained, furthermore maximum and minimum water-levels are known and essential for design purposes of maritime structures etc..

In non-tidal rivers the water-levels are influenced and determined by the amount of precipitation and the base-flow (ground water).
In tidal-rivers the water-levels are influenced by the discharge of the rivers (upland flow) and the sea-tide while in estuaries and along the coast the fluctuations in water-level are mainly influenced by the tide.

To obtain information about the water-level fluctuation in non-tidal and in tidal-rivers, water-level recorders and/or staff-gauges have to be established at various locations, which have to be selected with care to obtain reliable data.

The location selected for a water-level observation should comply, as far as possible, with the following requirements:

- the site must be accessible at all times
- sufficient water must be available so that the water-level recorder/gauge can not fall dry during the lowest water-level
- the gauge should be long enough to record high floods
- the water-level recorders/gauges should never be installed too close to confluences and bifurcations in tidal areas. In non-tidal rivers they can be installed directly downstream of confluences or bifurcations
- the water flow should not be restricted by banks and shoals.

In case water-level recorders are established a staff-gauge must be provided in order to relate the recording to the zero of the staff-gauge.

If the water-level recorders and staff-gauges are erected, benckmarks must be established in the vicinity of each gauge. The gauge should be connected to the benchmark by means of levelling. Benchmarks to be installed at such places and in such a manner that no settling can occur. Even when the benchmarks are not connected to each other, it is advisable to have the gauge already connected to its benchmark in order to obtain a manner to check the settling of the gauge. Furthermore if a staff-gauge is washed away, a new staff-gauge can be installed and the relation between the old and new gauge can easily be obtained via the benchmark.

3.3.2 Vertical tide

The forces which generate the tides are the attraction by the moon and by the sun. Although the mass of the sun is far greater than the mass of the moon, the effect of the moon is more than double the sun's influence. The tide raising forces are direct proportional with the mass of the celestial body but inversely proportional with the third power of the distance between this celestial body and the centre of the earth.

3.3.2.1 Vertical tidal motion

Both tide generating forces causes the water-level to raise and fall, and when the vertical motion is assumed to be harmonic, a sinoide will be the result. In Figure 3.3.1 PA rotates around the point P in a time cycle between two consecutive high waters. This cycle is called the tidal period. For the M_2-tide, which notation is used to indicate the tidal component generated by the moon and having two cycles each day, this period is 12 h 25 min.

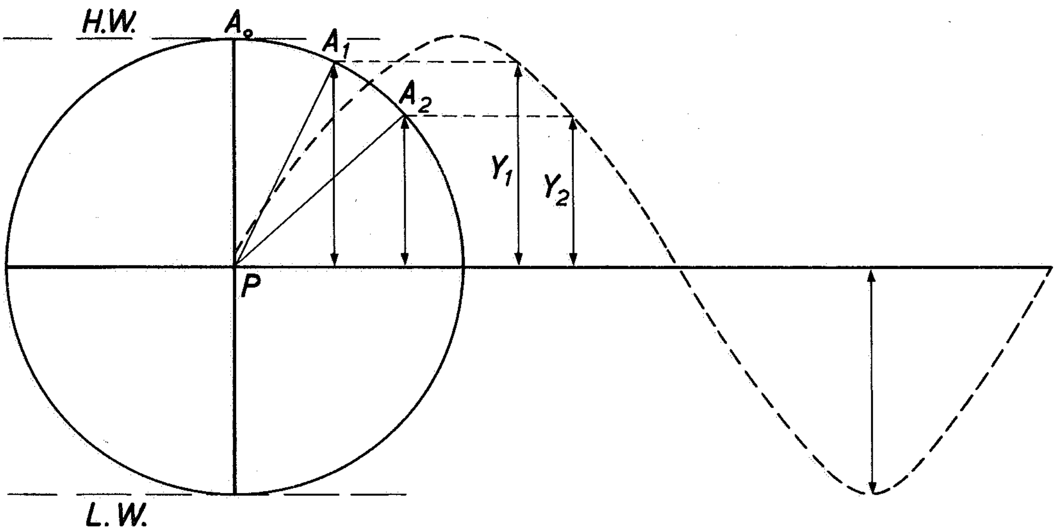


Figure 3.3.1 Tidal motion

At HW the arm is in position PA_o and at LW in a position which differs 180^o. In a certain time lap the arm PA_o has moved to a position PA_1, the angle between both positions is called the phase of the tide (E) at that particular time, and is expressed in degrees whereas one degree is 1/360 of the cycle. The phase-change per hour is called the speed of the constituent, which is for the M_2 tide $28,9841^o$/hour for calculation purpose 29^o.
P lies in the plane of mean level and the length of the arm PA_o is equal to the height of the water-level above mean level at HW.
One hour and a half after HW the arm is in position PA_1, two hours after HW at PA_2 etc..
The height of A_1 and A_2 above mean level gives the height of the water-level respectively $1\frac{1}{2}$ and 2 hours after HW. If Y_1 is the height of A_1 above mean level, then $Y_1 = PA_1 * \cos A_oPA_1$, whereby the angle A_oPA_1 is the phase of the tide $1\frac{1}{2}$ hour after HW.
The height of the water-level above mean level at HW is called the amplitude F of the tide and as $PA_o = PA_1 = PA_2$ = amplitude F, it follows that:

Water-level = amplitude (F) * cosine phase (E)

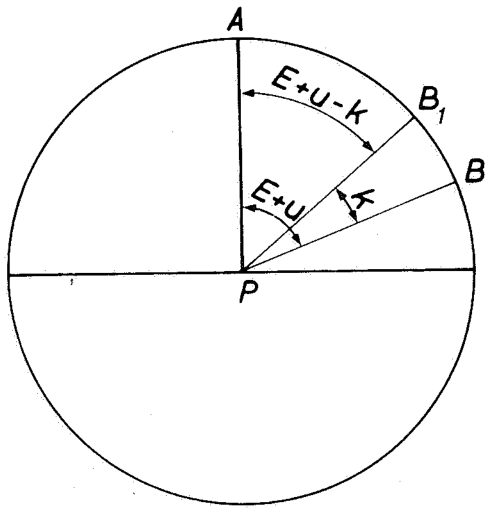
3.3.2.2 Astronomical argument and phase lag

Due to the fact that the plane of the moon's orbit rotates slowly, returning to its original position in space after 19 years, the magnitude and phase of each constituent (F and E respectively) vary slowly on either side of the values they would have if the moon's orbit were fixed. This variation could be allowed for by additional constituents, but it has been found more convenient to allow for it by introducing a factor f and a phase correction u, which vary slightly about their mean values of 1.000 and 0 degrees respectively with a period of about 19 years.

For any period of less than a year it suffices to take f and u as constant at the values they have at mid-point of the period concerned. These nodal corrections f and u differ for different constituents at the same year and date but are the same for a particular constituent, year and date all over the world. Therefore F and E must be replaced by fF and (E+u) so that the tide raising force becomes: fF cosine (E+u).

(E+u) is called the astronomical argument.

Any particular constituent of the actual tide as produced in the ocean usually lags behind the constituent of the tide raising force which produces it; that is, the phase of any constituent of the tide differs from the phase (E+u) of the constituents of the tide raising force which has the same speed. This is due to the time of travel through intervening seas, and up the gulf or estuary connecting the place of observation to the ocean, so that a further lag is introduced in the response of the water at a particular place, to the tide raising constituent which produces it. This lag is called Kappanumber (see Figure 3.3.2)



Angle APB = Phase of tide for the meridian of Greenwich according to the theory of equilibrium.

Angle APB_1 = Phase of actual tide.

Angle BPB_1 = Phase lag K (kappanumber) which is a constant if both tides have the same speed.

(E+u) = "Astronomical argument" sometimes called "initial argument".

Figure 3.3.2 Illustration of phase lag

Phase lag g consists of:

kappanumber K + geographical longitude of location - standard time in hours

$\times$ speed of constituent $g = (K + LL - nS)$

where:

LL = local longitude (WL + ; EL -)

n = speed of constituent

S = longitude of standard meridian in hours (WL + ; EL -)

In the Admiralty Tide Tables tidal angles and factors and the astronomical argument (E+u) are given for every day. The factor f is given for every month for the middle of the month.

For a number of standard ports and secondary ports, the values of g and H for the M_2, S_2, K_1 and O_1 tides (harmonic constants) are given (see paragraph 3.3.2.4).

The water-level of a constituent in relation to mean level is obtained by the following formula:

$$Y = fH \text{ cosine } (E + u + nt - g)$$

in which n = speed of constituent

t = standard time in hours

The lag of the tidal constituent at a given place behind the corresponding tide raising force constituent at Greenwich is a constant for the place and constituent and is equal to the difference between the meridian of Greenwich and the meridian of that particular place and should be added to the kappa-number.

This total lag is denoted as g and is measured in degrees. Hence the phase of the tidal constituent at a particular place is:

phase (E+u) minus g. The lag g is a constant angle for a given place and constituent, but varies from place to place and can only be established by <u>tidal observations</u> in that particular place.

Theory indicates and observation verifies that the amplitude of a particular tidal constituent at every place in the world varies in direct proportion to the f of that constituent. The amplitude of a particular tidal constituent on a date when its f is unity, is a constant for the constituent at that place and is denoted by H.

The amplitude of the same constituent on any other date at the same place will be fH.

<u>H can only be found by direct tidal observations.</u>

3.3.2.3 Declination tides

In the previous theory it is assumed that the moon moves with a uniform motion with an unaltered distance to the earth in the plane of the equator. However, in reality this is not so, the plane of the moon's orbit makes an angle with the plane of the equator. This angle is the moon's declination, and varies from approximately N $28\frac{1}{2}^{\circ}$ to S $28\frac{1}{2}^{\circ}$. The same applies for the sun.

The change in declination of the moon causes to adopt 3 more tidal constituents:
the K_2 tide with a period of $11^h\ 58^m$
the K_1 tide with a period of $23^h\ 56^m$
the O_1 tide with a period of $25^h\ 49^m$
The K_2 tide gives HW twice a day and also two times LW and is just like the M_2 and S_2 tides a semi-diurnal tide.
The O_1 and K_1 tides, however, give only HW once a day and one time LW and are therefore diurnal tides.
The change in declination of the sun gives for the same reason the necessity to adopt 3 more tidal constituents:
the K_2 tide with a period of $11^h\ 58^m$
the K_1 tide with a period of $23^h\ 56^m$
the P_1 tide with a period of $24^h\ 04^m$.
The K_1 and K_2 tides of the moon and the sun together form one K_1 and one K_2 tide.

3.3.2.4 Moon elliptical tide

The orbital plane is elliptical so that the distance of the moon to the earth varies according to its position in this orbit. This causes the various changes in the tide raising forces which gives inequality in the form of the water-level curve.
Due to the variable distances of the moon periodical changes in amplitude of the moon-tide occur, which can be regarded as being caused by series of tides of which the principal is the semi-diurnal elliptical moon tide N_2, with a period of $12^h\ 40^m$.
The changes in distance between the sun and the earth are generally disregarded.
Each of the above mentioned tidal constituents have their own amplitude, speed, astronomical argument and g.

The water-level for every tidal constituent is: amplitude * cosine phase.

The tidal movement depends mainly on 7 tidal constituents:

tidal constituent	speed	period
M_2 tide, mean lunar tide	$28^{\circ}.98410$	$12^{h}\ 25^{m}$
S_2 tide, mean solar tide	30°	12^{h}
N_2 tide, Moon elliptical tide	$28^{\circ}.43973$	$12^{h}\ 40^{m}$
K_2 tide, Sun/Moon declination tide	$30^{\circ}.08214$	$11^{h}\ 58^{m}$
K_1 tide, Sun/Moon declination tide	$15^{\circ}.04107$	$23^{h}\ 56^{m}$
O_1 tide, Moon declination tide	$13^{\circ}.94304$	$25^{h}\ 49^{m}$
P_1 tide, Sun declination tide	$14^{\circ}.95893$	$24^{h}\ 04^{m}$

Secondary tides. In shallow waters the harmonic smoothness of a tide may be distorted by secondary tides which are much weaker than the main tidal constituents but must be taken into consideration in order to avoid large errors in tidal predictions. The amplitudes of the secondary tides are smaller than those of the main tides and decrease with decreasing period. The following tides are secondary (shallow water tides):

the S_4 tide, speed is 2 * S_2 tide
the M_4 tide, speed is 2 * M_2 tide
the MS_4 tide, speed is sum of speeds of M_2 and S_2 tides
2MS tide, speed is difference of speeds of M_4 and S_2 tides
S_4, M_4 and MS_4 give HW four times a day, and four times LW.
2MS is a semi-diurnal tide.

Mixed tide. The characteristics of a mixed tide differ depending whether the semi-diurnal constituents or the diurnal constituents are predominant. With the pure semi-diurnal and the pure diurnal tides the deflection with reference to the mean level at HW and at LW spring-tide till neap-tide and reverse, increases and decreases regularly. With the mixed tide this is not so. The inequality of the water-level at successive HW's or LW's is called the daily inequality in height between two HW's.

3.3.2.5 Type of tides

Semi-diurnal tide.

This tide depends mainly on the M_2 and S_2 tide, there properties are mainly:

- twice a day HW and twice a day LW
- spring-tide every $14\frac{3}{4}$ days
- two consecutive HW's are approximately equal in height, and LW lies approximately halfway between two HW's.

Diurnal tide.

This tide depends on the K_1 and 0_1 tide, its properties are mainly:

- one time per day HW and once a day LW
- the period between the two spring-tides equals the time lapse between two moments of the moon's maximum declination = $13^{2/3}$ days.

3.3.2.6 Combination of tides

Spring-tide.

On the day that the S_2 and M_2 or the K_1 and 0_1 tide have the same phase they will practically give HW at the same time and LW at the same time.
The tidal motion will be strongest. HW rises most above mean level and LW falls most below mean level.

Neap-tide.

On the day that the M_2 and S_2 or the K_1 and 0_1 tidal constituents differ 180^o in phase, the HW of the M_2 tide and the K_1 tide coincide with LW of respectively the S_2 and 0_1 tide, or the other way around.
Tidal motion is then weakest. The rise at HW and the fall at LW with reference to mean level are the smallest.

3.3.2.7 Mean Sea Level (M.S.L.)

Mean Sea Level is given in the following symbols.

- Z_{oo} = the best practical figure which can be obtained for the height of mean sea level, referred to the datum in general use
- Z_o = the height of mean sea level, as obtained from any individual analysis, above chart datum
- S_o = the height of mean sea level, as obtained from any individual analysis, above the zero of observations
- A_o = the height of mean sea level, as obtained from any individual analysis, above an arbitrary datum, different from chart datum or the zero of observation.

The hydrographic surveyer will, in general, only be concerned with Z_o and S_o as Z_{oo} and A_o are symbols which are derived from extensive tidal computations.

M.S.L. may be calculated for a number of different periods as follows:
a) For short periods of a day or so to establish sounding datum for a hydro-

graphic survey.

b) For a single period of a month, usually calculated automatically in the course of the harmonic analysis.
c) For a succession of monthly periods, to calculate fluctuations.
d) For a period of a year, found automatically during the course of a harmonic analysis. This period gives a better value of Z_o than obtained from data only for one single month.
e) For successive yearly periods to determine the changes from year to year and the long period changes or trends.

The Admiralty method e.g. can be used to calculate the M.S.L. of short periods. This method is described as follows.
Take any 39 hourly observations and apply a filter, that is to say by using some observations once, some twice and some not at all. In this method the observations are written with the first observation numbered 0 and then up to 38. Multipliers are then applied as follows.

Hour	Multiplier	Hour	Multiplier	Hour	Multiplier	Hour	Multiplier
0	1	10	2	20	2	30	1
1	0	11	0	21	1	31	1
2	1	12	1	22	1	32	0
3	0	13	1	23	2	33	1
4	0	14	0	24	0	34	0
5	1	15	2	25	1	35	0
6	0	16	1	26	1	36	1
7	1	17	1	27	0	37	0
8	1	18	2	28	2	38	1
9	0	19	0	29	0		

The total sum is divided by the total of the multipliers, i.e. 30

The 39 hourly observations may start at any hour as the observations are successive.

3.3.2.8 Reference plane

The varying water-depth of rivers and estuaries is related to a datum which is known with respect to an accepted reference plane in the area under review or a national reference plane.

Sounding datum is the plane to which soundings are reduced in the course of a hydrographic survey.

Chart datum is the datum plane finally adopted for the published chart and is the level above which tidal predictions and tidal level are given in Tide tables and on published charts. It may or it may not be the same as the original sounding datum.

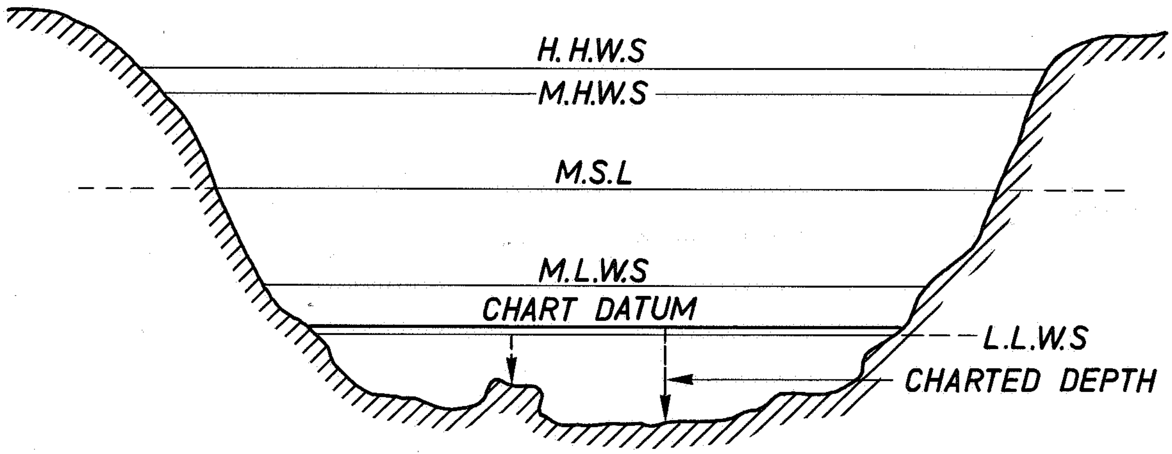
In selecting a datum for sounding the following considerations should be born in mind:

a the datum should be low enough for the navigator to be confident that, under normal weather conditions, there is always as least as much water as is shown in the chart.

b The datum should not be so low that it gives an unduly pessimistic idea of the least depth of the water likely to be found.

c The datum should be in harmony with datums of neighbouring surveys.

"A resolution adopted years ago is recommended that "Chart Datum" should be a plane so low that the tide will not frequently fall below it".



H.H.W.S. = high high water spring
M.H.W.S. = mean high water spring
M.S.L. = mean sea level
M.L.W.S. = mean low water spring
L.L.W.S. = low low water spring

Figure 3.3.3 Sea Levels

3.3.3 Transfer of datum (Admiralty method, at open coastline or in estuaries) (when tide is semi-diurnal)

When transferring datum by the methods described hereunder, it is assumed that "mean level" is the same at both gauges and that the ratio of the range at the two gauges is the same for a tide which falls to datum and the tide which is used during the transfer.
As the ratio of ranges may not be the same at neaptides and springtides, and as only at or near springtides the tide falls to datum, it is important that transfers should be carried out around springtides.
An additional reason for making transfers around springtide is that the relative effect of abnormal weather conditions is least when the range is greatest.
Two cases exist:

- Transfer from standard or secondary port, where the true value of mean level at springtide is known, see Figure 3.3.4.
- Transfer when true value of mean level at springtide is not known, "true spring ML" as used for transfer is: $\frac{1}{2}$(M.H.W.S. + M.L.W.S.) see Figure 3.3.5.

If the true value is known the differences between the observed and the true values at the established gauge can easily be calculated and it can be assumed that the same difference exists at the new gauge.
If the true value is not known at the established gauge, it can only be assumed that the difference at the new gauge is in proportion to the two ranges.
The observations taken are the same in both cases, but the formula differ, as will be shown. (see Figure 3.3.4).

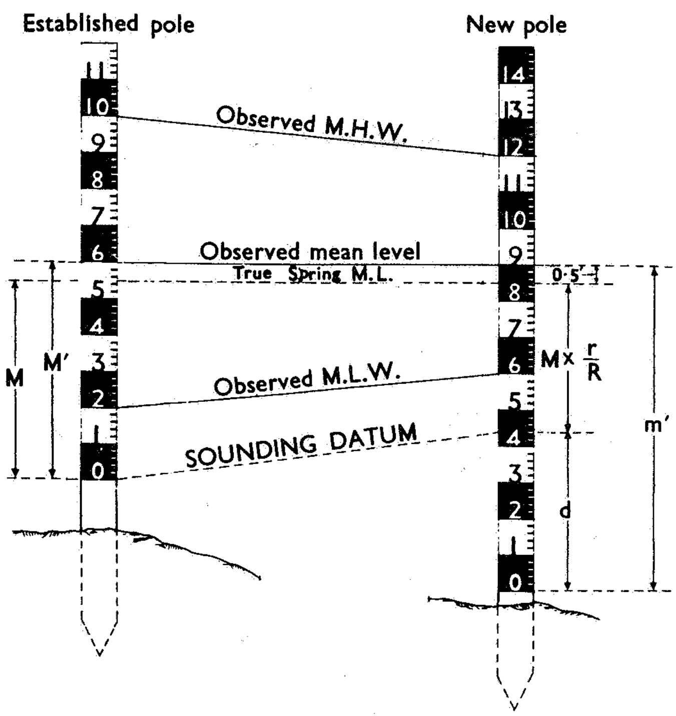


Figure 3.3.4 Transfer of datum (when tide is "semi-diurnal")

R : the observed mean range at the established gauge

r : the observed mean range at the new gauge

M': the height of observed mean level at the established gauge above chart datum

m': the height of observed mean level above zero of the new gauge

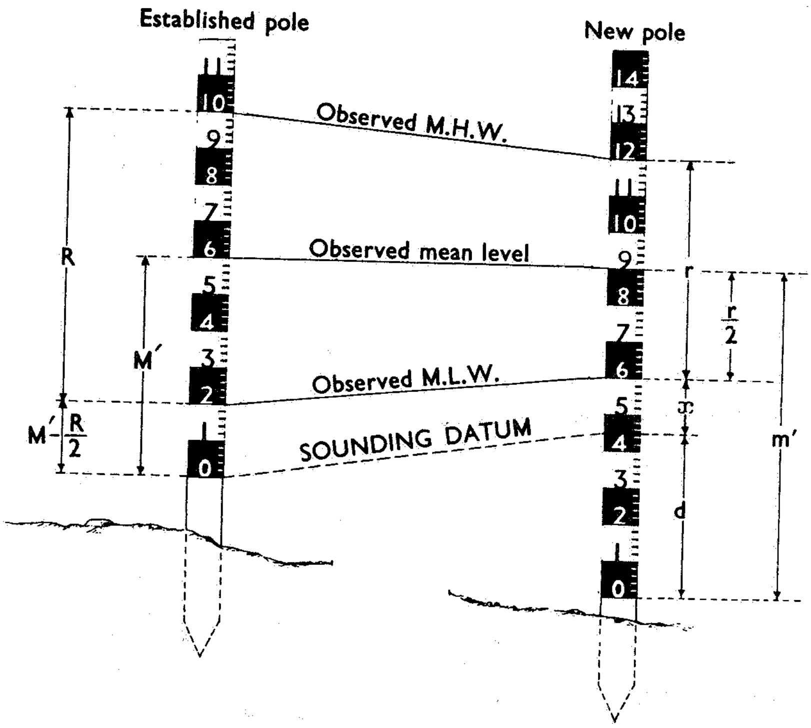
M : the height of true mean level at springtides above chart datum at the established gauge

d : the height of sounding datum above zero of the new gauge

It follows that: $d = m' - (M' - M) - (M \times \frac{r}{R})$

In the other case the problem is to find the amount by which the tide will fall on the new gauge, when the tide falls from the observed MLW to chart datum at the established gauge.

As previously indicated, it is assumed that the ratio of the two ranges at the time of observation is the same as when the tide falls to datum (see Figure 3.3.5).



It follows that: $x = \frac{r}{R}\left(M' - \frac{R}{2}\right)$

$$d = m' - \frac{r}{2} - x$$

$$= m' - \frac{r}{2} - \frac{r}{R}\left(M' - \frac{R}{2}\right)$$

$$= m' - \frac{M'r}{R}$$

Figure 3.3.5 Transfer of datum (semi-diurnal tide)

The necessary observations for the above method of datum transfer are four consecutive Low Waters together with three intermediate High Waters - though a similar result might be obtained with three Low Waters and four High Waters. The observations are entered on a form,

the Low Waters opposite the letters a, c and e, g

the High Waters opposite b, d and f.

The observations are taken in consecutive groups of four, giving a total of eight mean levels. The number is selected thus eliminating the effect of diurnal inequality and random meteorological effects.

The contributions are therefore as follows:

$$(a + b + c + d) + (b + c + d + e) + (c + d + e + f) + (d + e + f + g)$$

$$= a + 2b + 3c + 4d + 3e + 2f + g$$

The contributions taken separately with their divisors are therefore as follows:

For the Low Waters: $\dfrac{(a + 3c + 3e + g)}{8}$ = observed MLW

For the High Waters: $\dfrac{(2b + 4d + 2f)}{8}$ = observed MHW

	At established gauge					At new gauge				
	Heights above Chart Datum			Contributions for		Heights above zero of gauge			Contributions for	
	HW	LW	Factor	HWS	LWS	HW	LW	Factor	HWS	LWS
a										
b										
c										
d										
e										
f										
g										
Sums of contributions										
Observed MHW and MLW										

(Observed MHW = sum of HW contributions ÷ 4)

(Observed MLW = sum of LW contributions ÷ 8)

Obs. Mean Range (R) =		(r) =	
Obs. Mean Level (M') =		(m') =	

(Obs. Mean Range = Obs. MHW − Obs. MLW)

(Obs. Mean Level = $\frac{1}{2}$(Obs. MHW + Obs. MLW))

Date and time of 1st LW observation:

Date: Calculated by:

Established gauge: New gauge:

TRANSFER OF SOUNDING DATUM

WHERE TIDE IS MAINLY SEMI – DIURNAL

LN

A4

CALCULATION OF SOUNDING DATUM (d) AT NEW GAUGE.

Where "True Spring M.L." at established gauge is known.

From A.T.T. (Table V or part II)

MHWS mtrs
MLWS mtrs
½ sum mtrs = M (True Spring M.L.)

$$d = m' - (M' - M) - M\frac{r}{R}$$

=

= = mtrs above zero of gauge.

Connection between fixed marks and sounding datum on new gauge :

Remarks :

GENERAL LOCALITY :

Position established gauge	Position new gauge
Place :	Place :
Lat. :	Lat. :
Long. :	Long. :

TRANSFER OF SOUNDING DATUM

LN
A4

3.3.4 Transferring Datum when tide is "Diurnal"

When the tide is "diurnal" datum should not be transferred by the foregoing method. In such cases, it is assumed that the ranges of the tide at both places are in the same proportion as the sum of their main harmonic constituents. In other words, it is assumed that:

$$\frac{R}{(M_2 + S_2 + K_1 + O_1)} \text{ is the same at both gauges}$$

Two simple formulae are established, of which the first is used when the meteorological effects can be assessed, while the second assumes that the meteorological effects on mean level are proportional to the respective ranges. In a normal survey, datum is usually transferred from a position where the main harmonic constants of the tide are already known from at least one month observations. In such a case, harmonic constants at the new position are calculated from a long series of observations - up to one month - as the circumstances permit.

Normally the surveyer wishes to start sounding within a few days of the start of the survey, and he cannot afford to delay the soundings while a protracted series of tidal observations are being made. If possible therefore, a tidal party should be sent in advance of the main survey to obtain one month's observations, if this is impossible two series - at least of 25 hourly observations should then be obtained and analyzed.

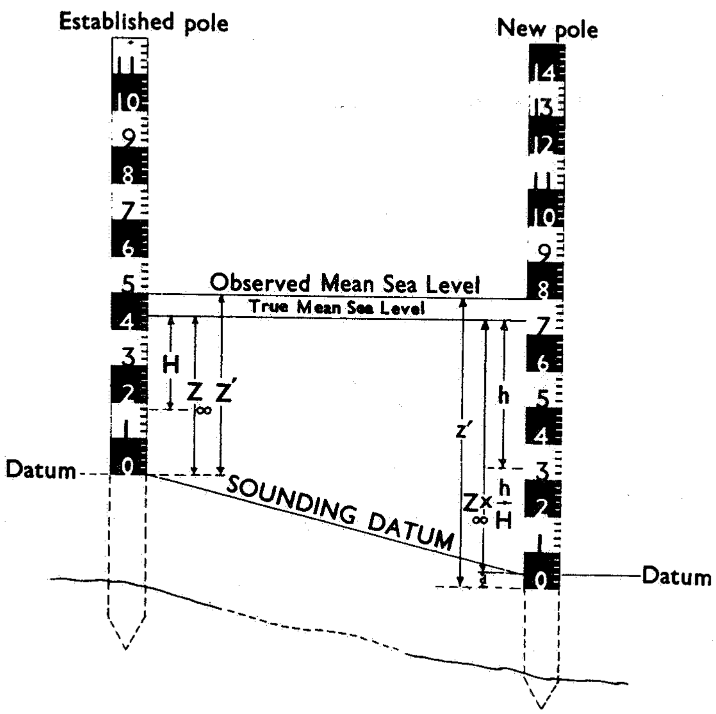


Figure 3.3.6 Transfer of datum (diurnal tide)

H is the sum of the four principal constituents at the established gauge ($M_2 + S_2 + K_1 + O_1$)

h is the sum of the same constituents at the new gauge

Z'is the height, above chart datum, of M.S.L. at the established gauge (Z_o from the analysis)

z'is the height of M.S.L. above zero of the new gauge (S_o from the analysis)

d is the height of chart datum above zero of the new gauge

Z_o is the true (average) height of M.S.L. above chart datum at the established gauge (from Admiralty Tide Tables).

From Figure 3.3.6 it will be seen that:

$$d = z' - (Z' - Z_{oo}) - Z_{oo} * \frac{h}{H}$$

As read from Figure 3.3.6

$$d = 0.8 \text{ m} - 0.06 \text{ m} - 0.44 * \frac{0.41}{0.26}$$

$$= 0.24 - 0.69 = 0.05$$

So chart datum is established at

0.05 m above the zero of the new gauge

The second case - where the true value of M.S.L. is not known - is illustrated in Figure 3.3.7.

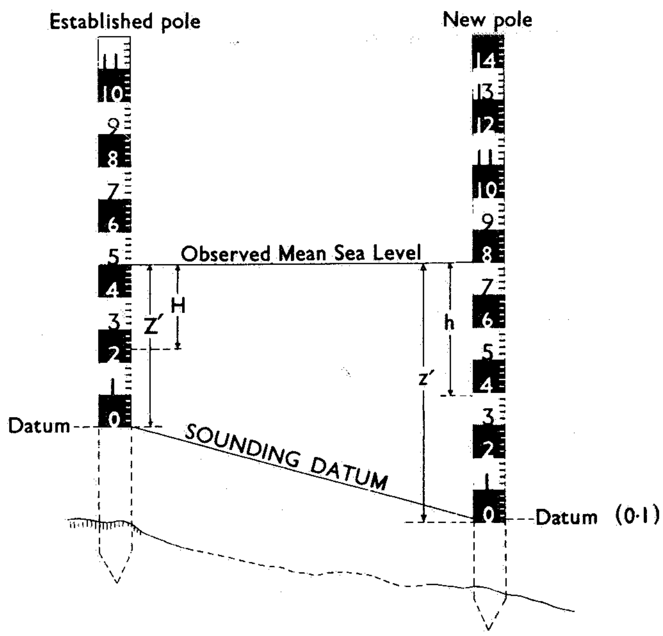


Figure 3.3.7 Transfer of datum (diurnal-tide)

Inspection of the figure will show that the formula is now as follows:

$$d = z' - Z' * \frac{h}{H}$$

In the example given in the figure: Z' = 0.5 m, H = 0.26 m, h = 0.41 m and z' = 0.8 m.

Then by the formula,

$$d = 0.8 \text{ m} - 0.5 \text{ m} * \frac{0.41 \text{ m}}{0.26 \text{ m}}$$

$$= 0.8 \text{ m} - 0.79 \text{ m} = 0.01 \text{ m}$$

In this case chart datum is established at 0.01 m above the zero of the new gauge.

3.3.5 Datums in river and estuaries

In order to understand the principle of establishing datums in estuaries and up rivers it is first necessary to appreciate the manner in which the tide behaves as it proceeds from open sea, first into the estuary and then up the river.

Most estuaries are funnel-shaped and this factor causes the tidal wave entering the estuary to be constricted; this, in its turn, causes a gradual increase in the range so that, at first, high waters begin to rise higher and low waters to fall lower as the wave proceeds up the estuary.

Further upstream the amplitude of the tide gradually decreases.

The propagation of the tidal wave is governed by inertia and friction. Upstream the river the tide is greatly influenced by topographical conditions, storage capacity inside and outside the river-bed, the bottom configuration of the river-bed, the slope of the river, the restricted cross-sections of the river etc.

It is evident, therefore, that in the upper reaches of a river, still under tidal influence the normal method of transferring datum cannot be used. The ratio of the ranges at two stations can be different for different tides. Another complicating factor is the effect of time- or season dependent river - discharges. This effect increases up the river as the tidal influence decreases.

It is therefore necessary to establish the tidal characteristics which are previously unknown by taking tidal observations at a series of selected positions spaced fairly evenly along a river.

The datum for each tidal station should be taken a fixed amount below the observed Mean Level of that station. The datum level thus determined should correspond to the lowest level to which the river can reasonably be expected to fall during the "dry" season or the season when the river is normally at its lowest.
It is very essential to connect the datum properly to benchmarks at each stage, it can always be altered at a later date when more observations are available.

No exact instructions can be given to cover each case, each river poses its own problem and the number of gauges necessary, as well as the distance between the gauges, depend on the length and width of the river as well as the slope of the riverbed.

3.3.6 Analysis of the vertical tidal motion

If the tide were a harmonic function answering the equation

$$y = \tfrac{1}{2}\, fH \cos(V_o + u + nt - g)$$

in which y = water-level with respect to mean water-level
f = factor (changing slowly with time)
H = tidal amplitude height of tidal wave
$V_o + u$ = initial argument (same as E + u in the A.T.T.)
n = angular velocity
t = standard time in hours
g = phase lag depending on the local meridian,

the water-level at any location at any time could be calculated, if the mean water-level, H and g at a particular location are known.
The vertical tide motion, however, consists of a number of harmonic tides, so called constituents.
Each of these constituents has its own angular velocity and at each location on earth its own specific H and g.

The latter two parameters have to be calculated. H and g are referred to as the tidal constants of a given location.

In order to be able to calculate the tidal constants, tide recordings of a sufficiently long time period are required at the location. The recordings have to be analysed and processed in order to determine the different harmonics. This process is called the harmonic analysis.
The harmonic analysis can be accomplished by different methods. One of the methods often employed, requiring only a slide rule, some tables and perseverance, is the "Admiralty Method", developed by A.T. Doodson, Director of the Tidal Institute in Liverpool U.K.

The Admiralty Method, requires 15 or 29 days of continuous hourly tide registrations. The latter, of course, produces the most accurate results.
For the sake of simplification and minimizing the chances of mistakes, special calculation sheets should be used.

Admiralty method

This section is intended to explain to the reader the philosophy behind this method. For a more theoretical explanation the reader is referred to the literature. As an example of this method a series of tidal observations of station Kampong Upang is evaluated (see Figure 3.3.9).
The tidal motion consists mainly of three types of tides:

Diurnal tides (once daily high water, once daily LW)
Semi-diurnal tide (twice daily HW, twice daily LW)
Secondary tides four times daily HW and LW).

The first step is to seperate the tide recordings into three groups corresponding to these three types of tides.
The respective tide periods are 24, 12 and 6 hours so the angular velocities are 15^o, 30^o and 60^o per hour respectively. The tidal motion (water-level as a function of time at a particular location) can now be expressed as:

$$y = S_o + R_1 \cos (15^o * t + \alpha_1) + R_2 \cos (30^o * t + \alpha_2) + R_4 \cos (60^o * t + \alpha_4) \quad (I)$$

in which S_o = mean water-level (with respect to zero of the staff gauge, invariably next to the recorder)
$R_{1,2,4}$ = amplitude of the partial tide
$\alpha_{1,2,4}$ = phase of the partial tide at time t = 0.

The water-levels for each hour are numbered y_o, y_1, y_2, y_3, etc. up till y_{23} (y_{24} is then y_o of the following day).
As origin of the time axis, $11^h\ 30^m$ is taken, so that for y_o:t = -11.5, for y_1:t = -10.5, for y_{11}:t = -0.5, for y_{12}:t = 0.5 and for y_{23}:t = +11.5.

For the sake of simplicity the following substitutions are made:

$R_1 \cos \alpha_1 = A_1$ $\qquad$ $R_1 \sin \alpha_1 = -B_1$
$R_2 \cos \alpha_2 = A_2$ $\qquad$ $R_2 \sin \alpha_2 = -B_2$
$R_4 \cos \alpha_4 = A_4$ $\qquad$ $R_4 \sin \alpha_4 = -B_4$.

$A_{1,2,4}$ and $B_{1,2,4}$ are constants for <u>one</u> particular location.
Substitution will give the following result:

$$y = S_o + A_1 \cos (15^o . t) + B_1 \sin (15^o . t) + A_2 \cos (30^o . t) +$$

$$B_2 \sin (30^o . t) + A_4 \cos (60^o . t) + B_4 \sin (60^o . t) \; ..... \; (II)$$

To determine S_o, A_1, B_1 etc., several summations have to be carried out which are indicated as X_o, X_1, Y_1, X_2, Y_2, X_4, Y_4.
The summation of S_o is adding up 24 hourly water-levels.
If $15^o . t = x$, then for the various hourly water-levels $x_o = -172^o.5$, $x_1 = -157^o.5$, $x_2 = -142^o.5$, ... $x_{11} = -7^o.5$, $x_{12} = +7^o.5$ ..... $x_{21} = 142^o.5$, ... $x_{23} = 172^o.5$.
The summation therefore consists of the following terms:

$24\ S_o$

$A_1\ (\cos x_o + \cos x_1 + \cos x_2 + ... + \cos x_{22} + \cos x_{23})$

$B_1\ (\sin x_o + \sin x_1 + \sin x_2 + ... + \sin x_{22} + \sin x_{23})$

$A_2\ (\cos 2x_o + \cos 2x_1 + \cos 2x_2 + ... + \cos 2x_{22} + \cos 2x_{23})$

$B_2\ (\sin 2x_o + \sin 2x_1 + \sin 2x_2 + ... + \sin 2x_{22} + \sin 2x_{23})$

$A_4\ (\cos 4x_o + \cos 4x_1 + \cos 4x_2 + ... + \cos 4x_{22} + \cos 4x_{23})$

$B_4\ (\sin 4x_o + \sin 4x_1 + \sin 4x_2 + ... + \sin 4x_{22} + \sin 4x_{23})$.

If we consider Figure 3.3.6 in which the ordinates represent the values of respectively cos x, sin x, cos 2x, sin 2x, cos 4x, sin 4x, it appears that the term of the factors of A_1, B_1 are equal but have opposite signs, as are A_2 and B_2, A_4 and B_4.

In the summation the factors A_1, B_1, A_2, B_2, A_4, B_4, become zero which give $X_o = 24 \times S_o$, so that S_o is determined.

The summation of X_1 consists of multiplying Y_o, Y_1, etc. with + 1 if cos x is positive and with -1 if cos x is negative.
The same applies for Y_1, X_2, Y_2, X_4, Y_4 which gives as result:

$X_1 = 15.322\ A_1$ $\qquad$ $Y_1 = 15.322\ B_1$
$X_2 = 15.455\ A_2$ $\qquad$ $Y_2 = 15.455\ B_2$
$X_4 = 13.856\ A_4$ $\qquad$ $Y_4 = 16\ B_4$.

A tidal motion, which consists of three partial tides with periods respectively 24 ^{hours} (S_1), 12 ^{hours} (S_2), 6 ^{hours} (S_4) is analyzed in three components.

For S_1 $\quad A_1 = X_1 : 15.322$, $B_1 = Y_1 : 15.322$

For S_2 $\quad A_2 = X_2 : 15.455$, $B_2 = Y_2 : 15.455$

For S_4 $\quad A_4 = X_4 : 13.856$, $B_4 = Y_4 : 16$.

But the partial tides do not have the exact periods of 24 hours, 12 hours and 6 hours and therefore corrections have to be made.

The Admiralty Tide Tables write each partial tide as the function:

$$y = R \cos (nt - r)$$

in which $\quad y$ = water-level
R = amplitude
t = time
$- r$ = phase at $t = 0 = 11^h\ 30^m$
n = angular velocity.

So for a particular location we have to determine for each partial tide the values of R and r. Again the now well known procedure is followed: with $R \cos (nt - r) = R \cos r \cos nt + R \sin r \sin nt$ the partial tides that are out of phase are transformed in components that are in phase. The values of R sin r and R cos r can now be determined. The values of cos r and sin r change daily, with the exception of the S_2- tide, but the values are known. These changes are taken care of by introducing $\sin (r_o - dm)$ and $\cos (r_o - dm)$ whereas r_o is the phase on the day in the middle of 15 of 29 days under observation. d is the number of the days minus the number of the day in the middle of the observed period and m is the increase in phase for each consecutive day. The right choice of the multiplication factors +1 and -1 brought out the partial tides of 24, 12, and 6 hours period. Likewise one can determine from the daily values of X_1, Y_1, X_2, Y_2, X_4, Y_4 additional tides that occur twice, three times or four times per 29 days. This is done in section III and IV of the calculation sheet.

The result of section IV give a series of equations with R and r as unknown parameters which can be determined by section V and VI and the tables.

The following relationship does exist:

$$360 - r_o = V_o + u + 11.5\ n - g$$

$360 - r_o$ = phase of the tide at $11^h\ 30^m$ of the centre day
$V_o + u$ = phase of the tide at the meridian of Greenwhich at 0 hours GMT according to the theory of equilibrium

$$g = V_o + u + 11.5\ n + r_o.$$

The values of V_o, u, f can be found in the Admiralty Tide Tables Part III. In this way $\frac{1}{2}$ fH (called A in section VIII of the calculation sheet) and g (= $V_o + u + 11.5\ n + r_o$) can be calculated for the most important partial tides, for a given location.

Besides the above mentioned method of analysis of the tide, the Admiralty has developed the semi-graphic method, which is described in the "Admiralty Tidal Handbook nr. 1".

Tidal prediction

The Admiralty has issued a manner to predict the tide, which is described in their publication NP 159 "Admiralty Tidal Prediction Form".

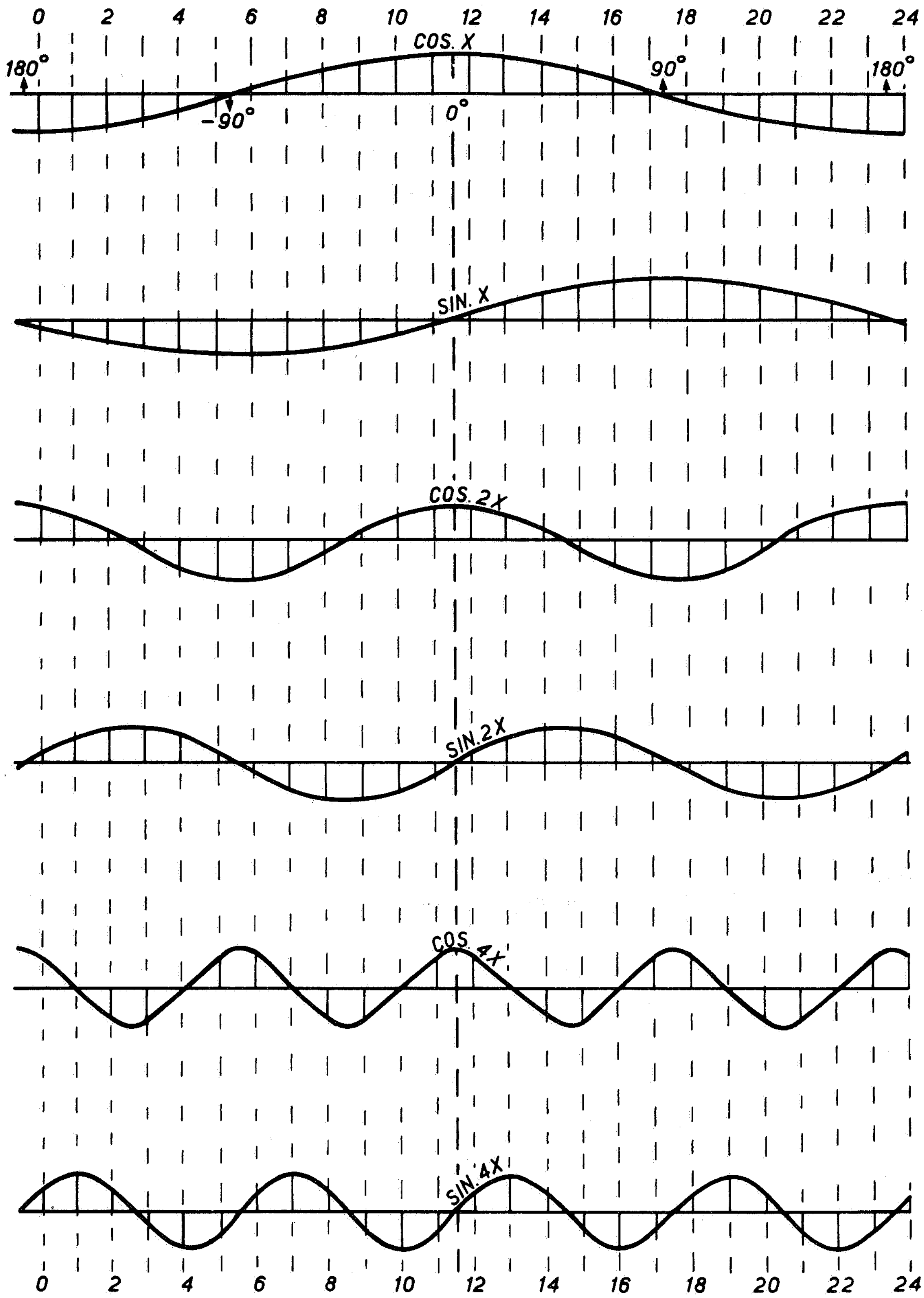


Figure 3.3.8 Harmonic curves

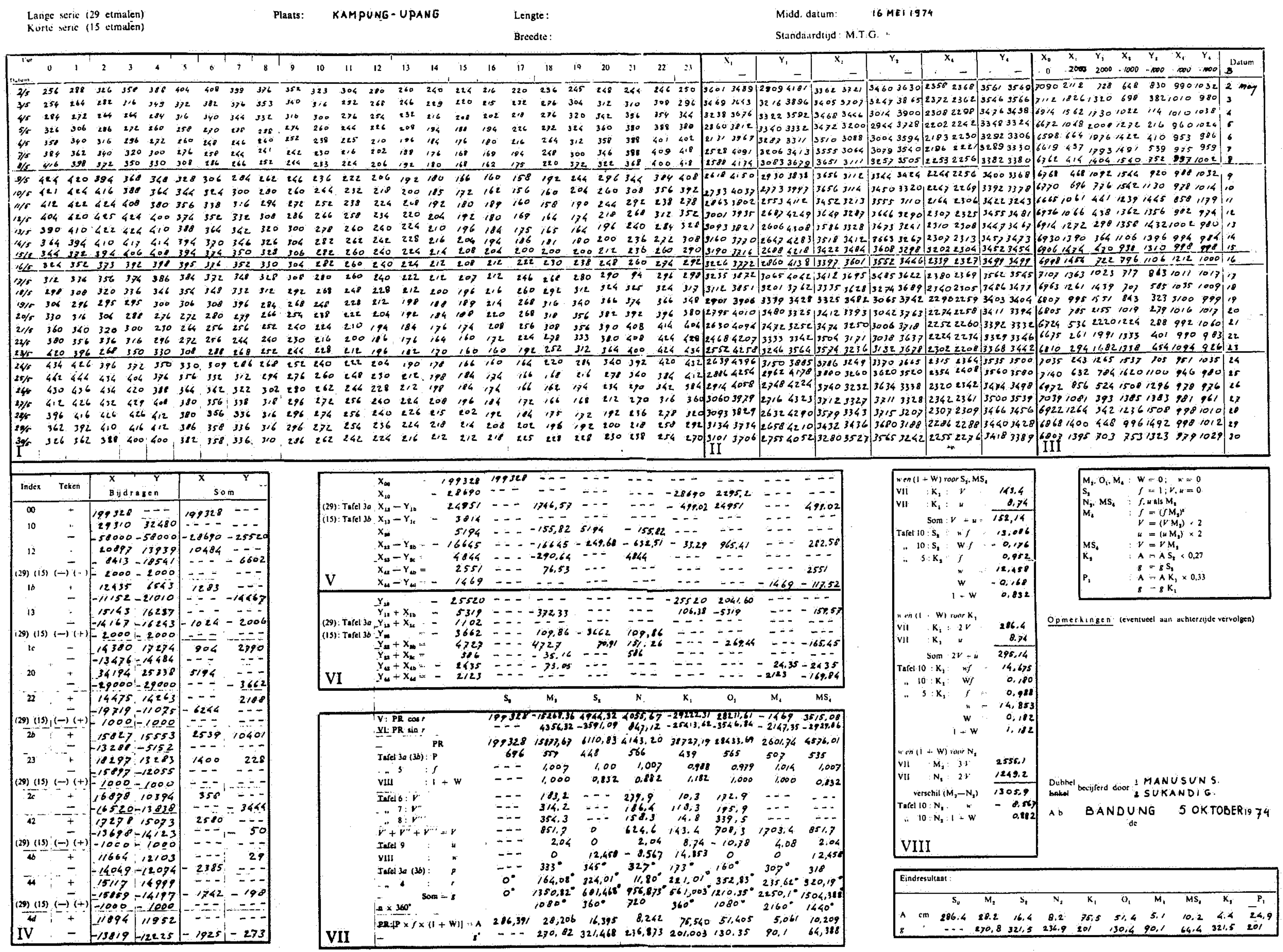


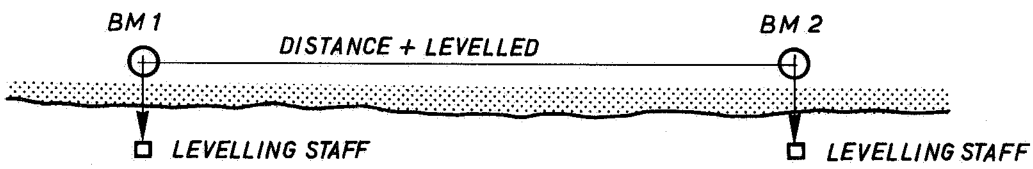
Figure 3.3.9 Tidal analysis Kampong Upang

3.3.7 Measuring the slope of the river

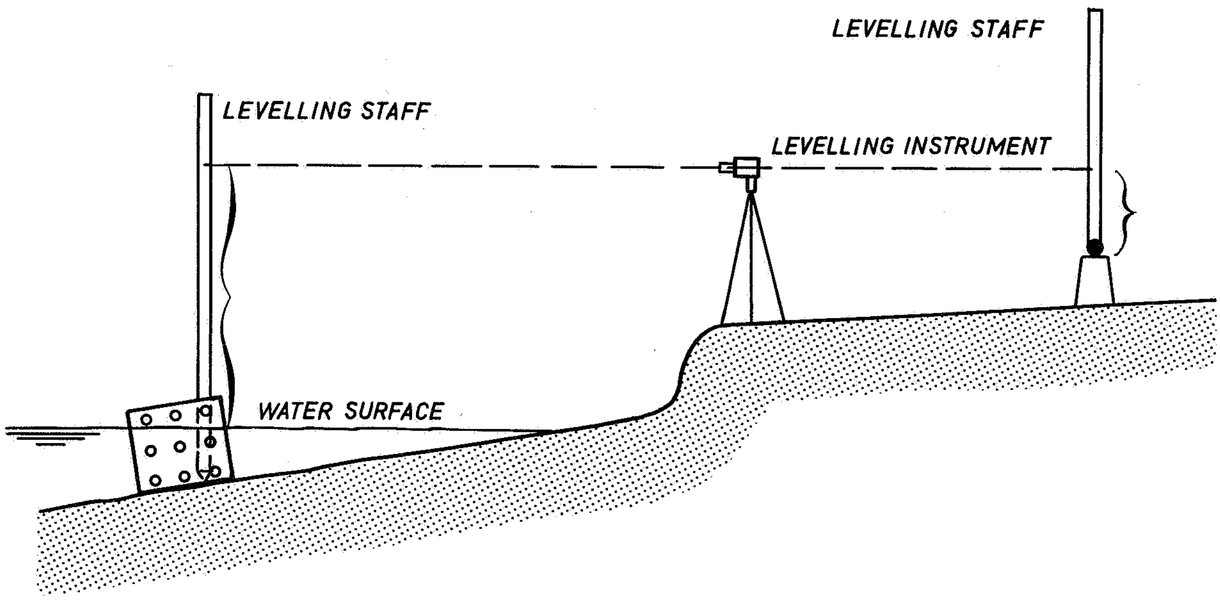
The roughness coefficient is used in many formulae of sediment transport and it has to be stressed that a good estimate of this roughness coefficient is very essential in the determination of a good mathematical description of the sediment transport phenomena. The roughness coefficient is determined using data of depth, water velocity and slope of the water surface of which the slope-data normally form the weakest link, that determines the overall accuracy to be obtained. As general the following rules have to be taken into account:

1. The selected river stretch and bank should be representative for the river and river banks should be as straight and parallel to the flow direction as possible, while no obstruction may be present (no eddies, no bridges or large trees in the water etc.).
2. Determining the length of the river stretch the absolute value of the river-slope has to be taken into account as well as the accuracy with which the water-levels can be determined. Moreover local influences can be reduced taking a longer stretch. As a general rule it can be said that distances of 500 m should be considered as a minimum for rivers with a slope of about 10-20 cm per km.

An acceptable accurate way to determine slopes is to determine the water-level by levelling starting from two benchmarks related to each other by levelling and with a known distance.



In order to reduce the influence of small waves a perforated drum can be used. The levelling staff is placed somewhere in this drum and simultaneous readings are made of the water-level height at the levelling staff and with the water-



level instrument this procedure can be repeated 3 or 5 times for several locations of the levelling staff within the drum.

If the distance is sufficiently great two permanently installed staff-gauges can be used. It should, however, be considered that the staff-gauge readings are less accurate than the water-level determination by levelling as described above.

In tidal areas care has to be taken that readings at both gauges or near both BM's have to be taken simultaneously. The differences in water-levels are then derived from observed water-level readings taken at the same time.

The slope of the river is calculated by dividing the difference in water-level by the distance. The slope of the river is expressed as a number.

3.4 Bathymetry

3.4.1 General

The bathymetric survey is an essential part of every field study as it gives information about the bottom configuration, the cross-sectional profiles in rivers, an insight into sedimentation or degradation and gives therefore the basic information to the engineer of the area under study.

It is therefore important to carry out this survey as accurate as possible and special attention should be paid to the calibration of the echosounder, the position fixing of the vessel, the water-level measurements to establish an accurate reference level to which soundings are reduced, and to select the right kind of echosounder attuned to the purpose of the study.
If soft mud layers are expected or the survey is intended as a pre-dredging survey or a post-dredging survey a low frequency echosounder should be used in order to detect layers and sedimentation or degradation, as the sound pulse of a low frequency echosounder penetrates more deeper in the bottom. A high frequency echosounder, however, gives only a recording of the top of the bottom regardless the bottom composition.

In rivers the cross-sections are sounded in transit lines at fixed intervals spaced along the river.
Distance to the transit beacons are taken by linear or angular measurements and at the same time the position fix is indicated on the echosounder recording.

In estuaries or along open coasts, soundings are taken on lines perpendicular to the bottom-contour lines in order to have the most accurate way of locating these lines. The intervals between the tracks are more or less dependent on the purpose of the survey and the scale on which data should be charted.

Position fixing can either be done by sextant angles on shore beacons or electronic positioning, while during the sounding-runs the position fixes are plotted on board of the survey vessel to obtain a check on the course of the vessel.
Fix numbers and times are noted down in a sounding book, in which each page is divided in columns for: date, time, fix number, first angle (always the left angle) with corresponding beacons, second angle with corresponding

RIVER :
CROSS-SECTION:
DATE :
TIME :

FIX NO. (LOCATION)	RANGING TO BEACON NO.	SAILING = FROM	TO	SOUNDED DEPTH [m]	CORRECTION [m]	REDUCTION [m]	DEPTH RELATED TO DATUM [m]	REMARKS
ALTERNATIVE FOR TRACK SOUNDING								
FIX NO.	FIRST ANGLE TO BEACON	SECOND ANGLE TO BEACON						

3.4.1 Example of sounding book

beacons, sounded depth, correction, reduction (to datum) and overall corrected depth. By using such a book all data are collected together and afterwards checks on doubtful figures easily can be made (see Figure 3.4.1).

3.4.2 Soundings

If position fixing is done by sextant angles, beside the crew, 3 observers should be on board. Two observers to take the sextant angles simultaneously and one observer to plot the position of the vessel and to instruct the coxswain on change in course to cope with drift etc..
At each angle measurement a fix is made on the echosounder and the time and fix number is noted down in the sounding book by one of the crew-members. The measured angles and the fix on the echosounding record are given the same running numbers.

If position fixing is done by electronic means, only 2 observers are required besides the crew of the vessel.
One observer to plot the fixes and to instruct the coxswain, one observer to handle the receiving units and echosounder.

Time intervals of fixes and distances between sounding tracks are dependent on required accuracies and chart-scale.
Beginning and end of each track (run) is marked with a double marked line (double fix) on the echosounder recording and in the sounding book a note is made.

3.4.3 Calibration of echosounder

At the beginning and end of each sounding day, the echosounder should be calibrated by means of a barcheck.
In case of an outboard transducer, a steel plate suspended to a marked line is lowered into the water and held at a certain depth beneath the transducer. The depth recording of the testbar should correspond with the actual depth of the testbar under the water-surface. This is checked at various depths.
If this is not the case, an adjustment has to be made in the speed of sound to cope with the density and temperature of the water, until the echosounder reading corresponds with the actual depth of the testplate.
Attention should be paid to the fact, that the zero-line of the recording,

being the bottom of the transducer is set on the scale reading corresponding with the depth under water of the transducer. Otherwise there is already a discrepancy between the echosounding recorded depth of the testbar and the actual depth of the testbar, equal to the depth of the transducer under water.

If the water is homogeneous in density and temperature, the adjustment on one depth will suffice to give correct readings on the other depths of the testbar.

If the water is not homogeneous in density and temperature the adjustment on one depth will not suffice as the reading on other depths may differ more and more from the actual depths of the testplate with increasing depth.
Therefore in such a case the echosounder should then be adjusted for speed of sound on the deepest testplate position.
From the other depths the differences should be noted down in order to make a correction graph, on which the x-axis represents the actual depths of the testbar and on the y-axis the corresponding readings on the recording.
The readings are plotted and a line of best fit is drawn which results in either a straight line, a broken line or a curved line (see Fig. 3.4.2).
During elaboration of the sounded data, for every recording reading then the actual depth can be derived from this graph.

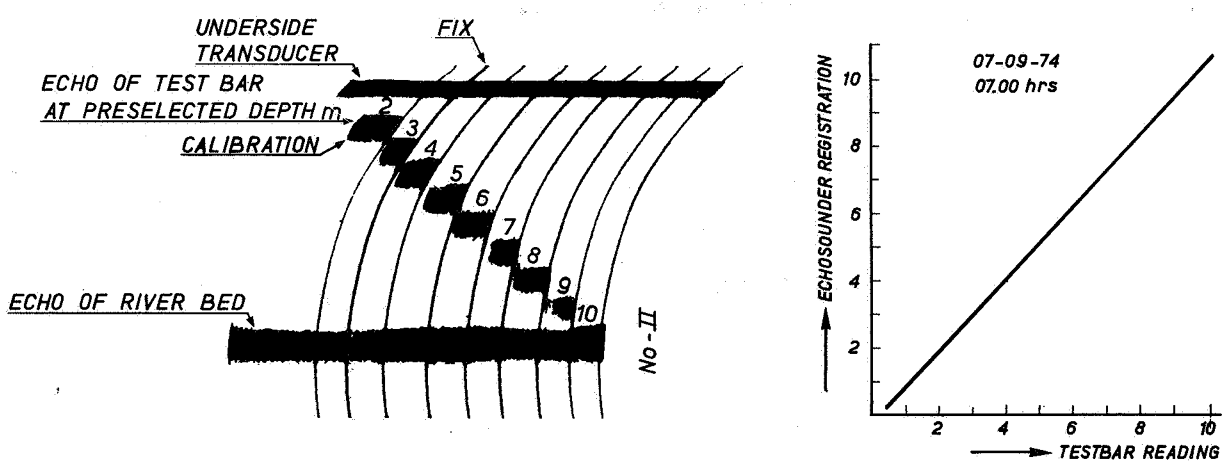


Figure 3.4.2 Calibration of echosounder

3.4.4 Reading scales

The sounded data are taken from the echosounder recordings by using a scale. Most manufacturers supply reading scales in addition to the scale attached to the echosounder in order to facilitate elaboration of sounding data in the office.

In cases that no extra reading scale is supplied, the scale on the echosounder has to be copied either on perspex or on a tracing film like Kodaktrace or True Scale.

If the styluspen of the echosounder is fixed to a rotating belt there will be no problem to copy the scale as all subdivisions have equal intervals. If the styluspen is fixed to a rotating disc, however, care must be taken that the subdivisions on the scale have intervals with increasing spacing from zero to the centre of the scale and with decreasing spacing from the centre downwards (see Figure 3.4.3).

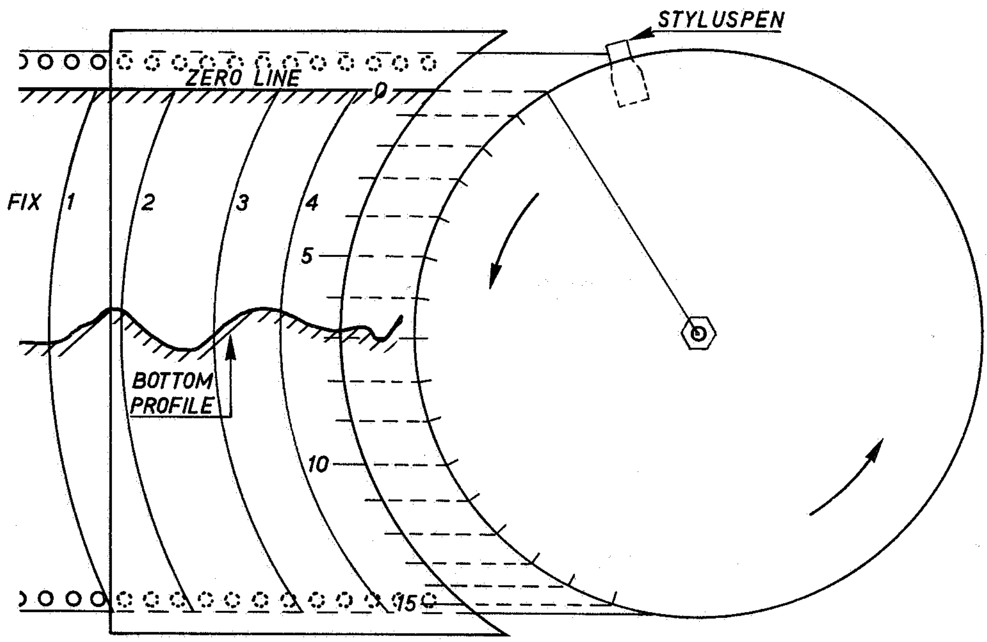


Figure 3.4.3 Example of scale for rotating disc

In order to take readings of the recording, the zero of the scale is set on the top of the transmitting signal-line and the depth of the river- or sea-bottom is read on the scale.

3.4.5 Correction and reduction

3.4.5.1 Correction

The total correction, in case that the zero-line on recording is put on scale-zero, equals to the algebraic sum of the distance between water-surface and transducer depth and the corrections found during calibration of the echosounder. If the zero-line of the recording is put on the corresponding value on the scale equal to the distance between water-surface and transducer depth, only the correction according to the calibration has to be applied.

3.4.5.2 Reduction in tidal areas

In order to obtain depths which can be compared with each other as the soundings are taken at different times and days and thus during various phases of the tide, the sounded water-depth has to be reduced to a certain reference plane, either MSL, or a plane below which the tide or water-level seldom falls (Datum).
The difference between the water-level at the moment of the fix at the tidal curve and a reference plane (Datum) to which de sounded depth are reduced is called the reduction.
The algebraic sum of the sounded depth, the calibration correction and the reduction will give the depth to datum, the so-called chart-depth.

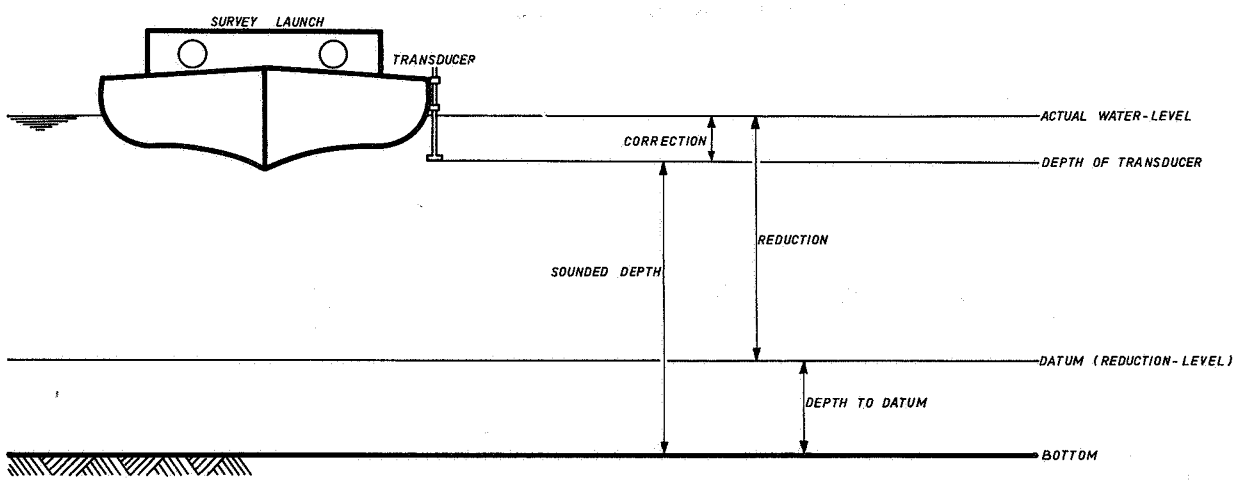


Figure 3.4.4 Correction and reduction of sounding

At an open coast it suffices to have two gauges at the border of the sounding area and to transfer the Datum from one gauge to the other. Reductions, which should be applied in locations between the gauges, can be found by interpolating liniary.

In tidal rivers, however, the reduction is more complicated and more cumbersome. The Datum is not necessarily a horizontal plane.
As mentioned before several gauges are required along the river and M.L. of each gauge should be established.(M.L. = mean level).
The M.S.L. of the gauge at the entrance (sea) should be transferred by levelling to the other gauges and the differences between the M.L. of each gauge in respect to the M.S.L. of the entrance gauge is then obtained.

Reduction can be carried out in relation to the M.S.L. of the entrance gauge or to a reference plane being L.L.W.S. or the equivalent.
In the last case, the difference between this reference plane and the M.S.L. of the entrance gauge should be applied to the M.L.'s of each individual river gauge so that the reference plane is known on the river gauge.

Regardless of the reference plane in use, the real problem lies in reduction of soundings for cross-sections in between two river gauges, as the tidal range and the phase of the tide can differ considerably between both gauges.
A method which can be applied is to plot the differences of the tidal curves in relation to the reference plane (M.S.L. or datum) for both gauges, for a period that soundings are taken on a certain date (see Figure 3.4.5 and 3.4.6).

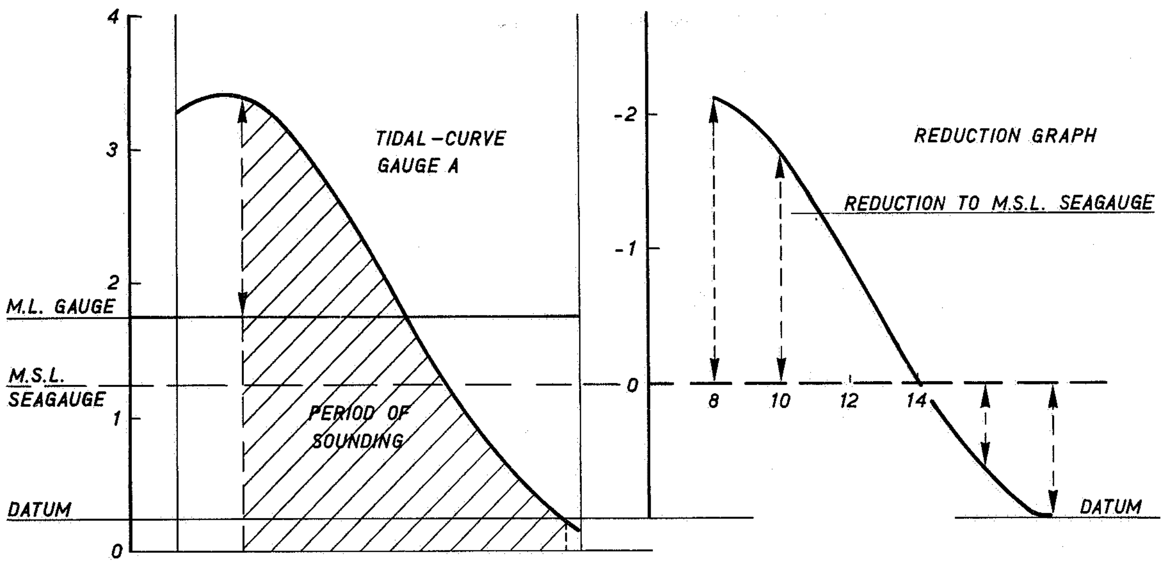


Figure 3.4.5 Tidal curve and reduction graph station A

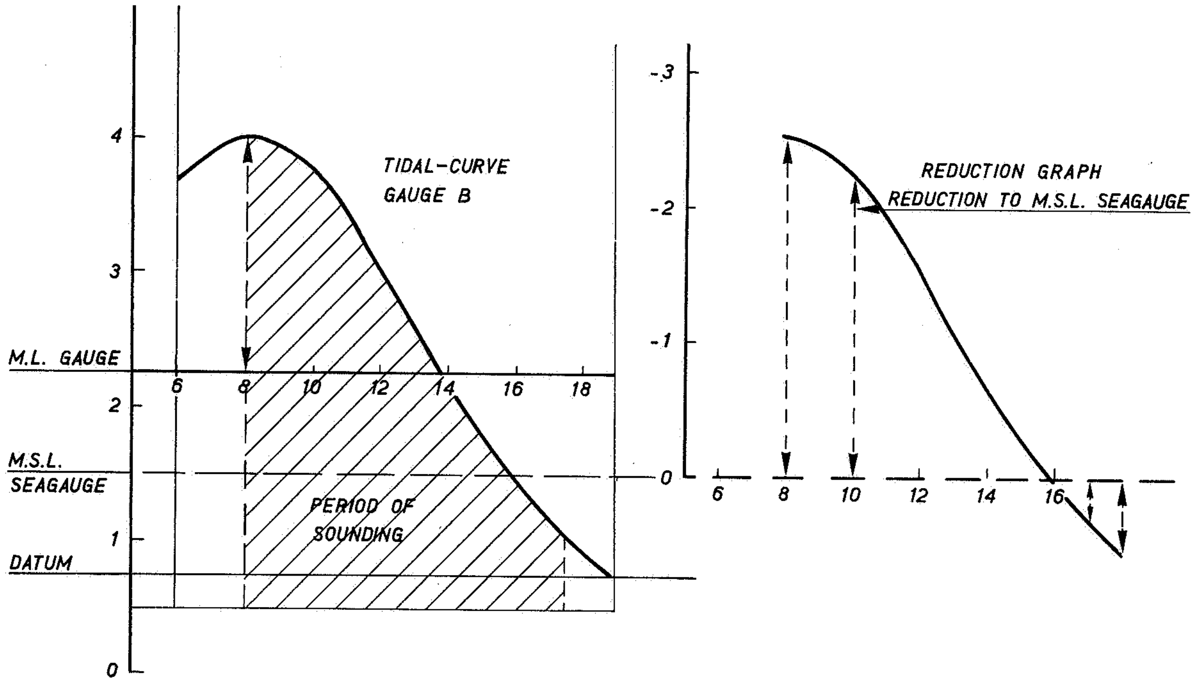


Figure 3.4.6 Tidal curve and reduction graph station B

A graph is made, with on the X-axis the distance between both gauges with in between the locations of the sounded cross-sections. At the position of the two gauges a Y-axis is erected, on which a scale with height differences between the tidal curve and M.S.L..

At a certain time that soundings are taken in a specific cross-section, the height differences are taken from the curves and plotted on the Y-axis for each gauge.

These points are connected with each other with a straight line; where this line intersects the Y-axis of the sounded cross-section, the height difference in relation to the plane is found and applied to the soundings (see Figure 3.4.7).

This reduction can either be + or - if M.S.L. is used as a reference plane.

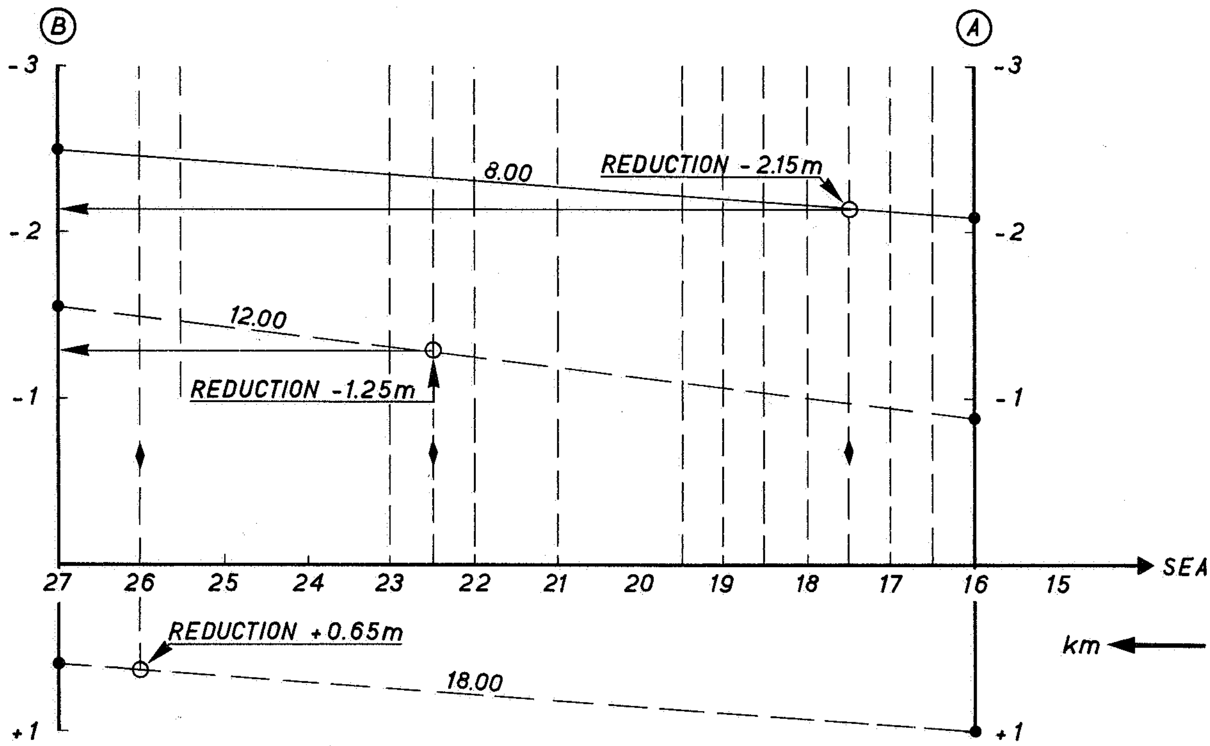


Figure 3.4.7 Example of reduction on a river

3.4.6 Charts involved for sounding purposes

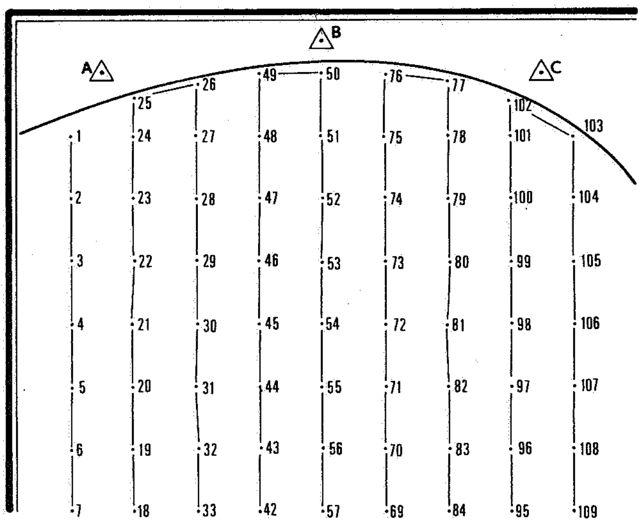
To present the data obtained by the soundings, several charts are used during the evaluation of the data.

Basic chart

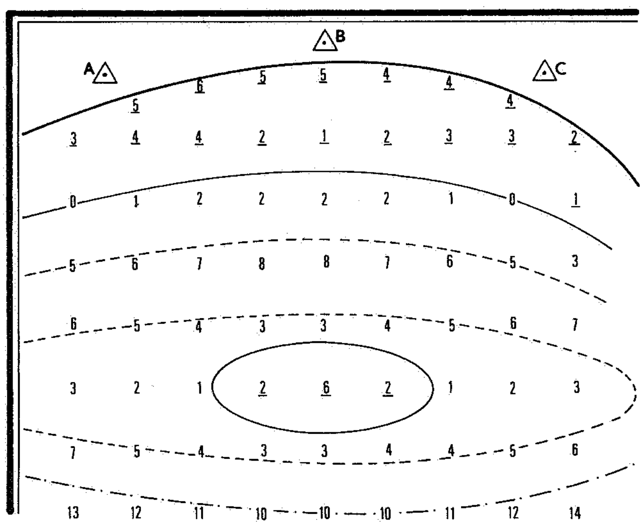
This chart is the mother-sheet for all charts mentioned below. It is preferably made on a thermal and hygroscopic stable (true scale) drawing film on which the grid is drawn and the triangulation beacons are plotted and if so required a complete grid of arcs of angles is drawn.

First chart

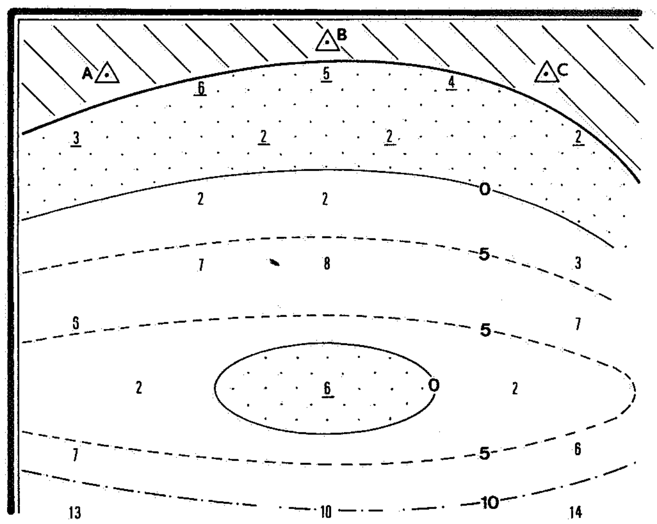
Mostly a photocopy of the original sheet, on which the actual sounding tracks in the field are plotted with their respective plot (fix) numbers, with a station-pointer or with compasses and a protractor.



*PART OF A FIRST CHART, EXAMPLE.
FIX NUMBERS OF THE SAME TRACK ARE CONNECTED BY A LINE TO MAKE INTERPOLATION FOR ADDITIONAL POINTS EASIER.*



*PART OF A SECOND CHART, EXAMPLE.
NEGATIVE DEPTHS (PARTS OF THE BOTTOM EXCEEDING THE REDUCTION-LEVEL) ARE INDICATED BY UNDERLINED FIGURES. THESE PARTS ARE DRY WHEN THE WATER-LEVEL IS ON OR BELOW THE REDUCTION-LEVEL.*



*PART OF A FINAL CHART, EXAMPLE.
UNDERLINED FIGURES INDICATE HEIGHTS ABOVE DATUM. DEPTHS AS WELL AS HEIGHTS IN DECIMETRES.*

Figure 3.4.8 Examples of charts involved for reproducing sounding data

Second chart (fair chart)

This chart is a transparent sheet of the same material as the mother-sheet, which is laid over the first chart and on which the corrected soundings are written according to their respective fix numbers.
On this chart the depth-contour lines and coast-lines are drawn. From this chart, copies can be made.

Final chart

This chart is a transparent sheet of the same material as the mother-sheet, and is laid over the second chart in order to copy the depth-contour lines and coast-lines and to draw the soundings only at regular intervals and at important positions.
In this way a more overall picture of the sounded area is obtained.

Chart of arcs

In order not to affect the mother-sheet by arcs of angles a separate transparent sheet of arcs can be made. A copy of this chart serves a dual purpose
a. to be used as a plotting chart aboard the sounding vessel (first chart)
b. to be used in conjunction with the second chart.

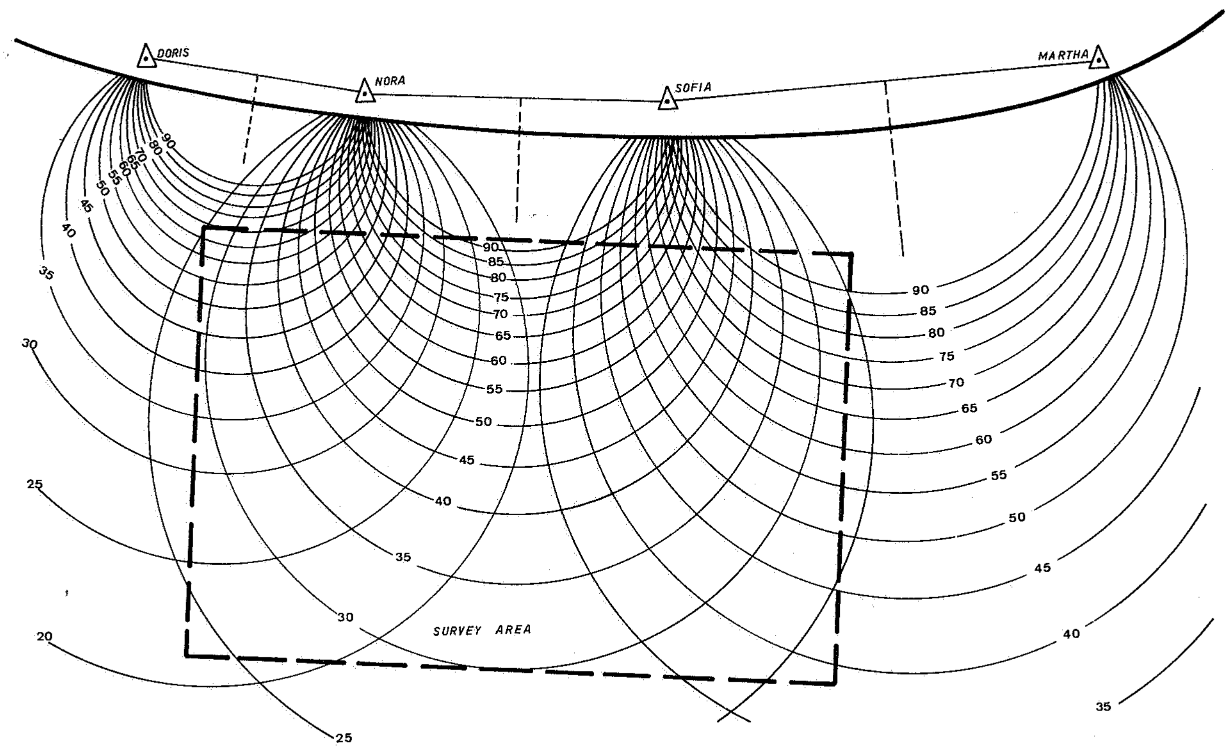


Figure 3.4.9 Chart of arcs

To produce a chart of arcs the following should be done:

The base (line connecting two beacons) is divided in two equal parts. In the centre of the base a perpendicular is drawn on that side of the base where the soundings will take place.
The centres of the circles corresponding with the angles which are required are then plotted on this perpendicular according to the formula $\frac{1}{2}a.\text{cotg}\alpha$ (AM in Figure 3.4.10) in which $\frac{1}{2}a$ is half the length of the base either in cm or mm. The formula gives the distance in cm or mm from the base on the perpendicular. Each circle is the locus of the observer from which he observes the two beacons at the corresponding angle (see Figure 3.4.10).
Constructing the most important circles for each part of the sounding area will give a useful chart, either for plotting quickly its position during the sounding tracks or for elaborating its position afterwards in the office.

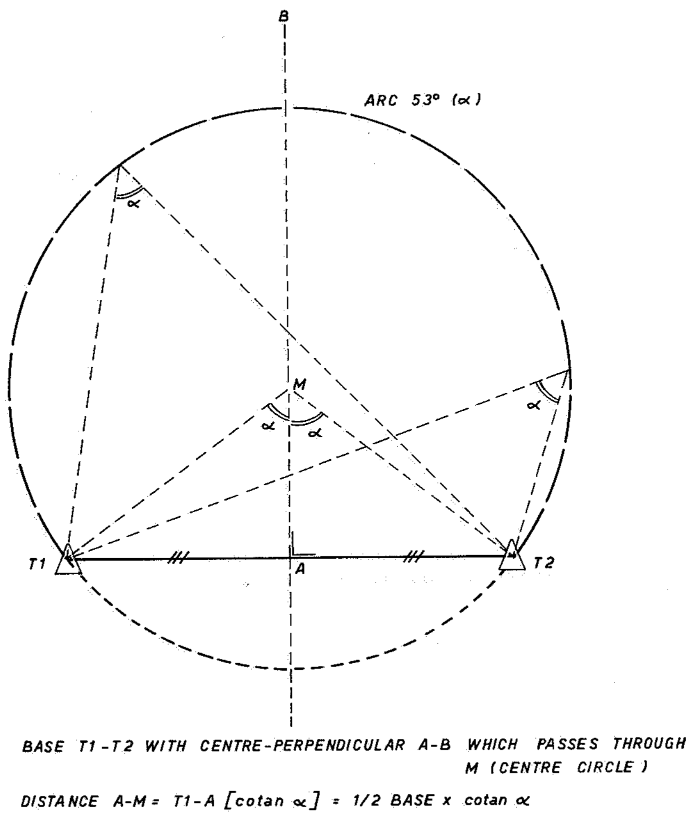


Figure 3.4.10 Goniometric principle used for plotting chart of arcs

Another method to produce a chart of arc quickly, less accurate but very useful for sounding purposes, is the method in which the station-pointer is used to find the centres of the circles on the perpendicular.
The fixed arm of the station-pointer is laid along the base (line connecting

2 beacons). One movable arm is set on angle value equivalent to the component of the angle ($90^o - \alpha$).
The intersection of this arm with the perpendicular gives the centre of the circle which is the locus of the surveyer who observes the two beacons at the angle α.
In this way the centres of the circles can be plotted.

3.4.7 Reproducing sounding data

For soundings taken at sea or in estuaries, the reduced soundings are charted at their respective fixes or if the interval between the fixes is too large, readings in between may be charted. A depth-chart is produced and contour-lines can be drawn (see Figure 3.4.8).

For soundings taken in a river according to cross-sections, the water-depths related to the reference plane for each fix in the cross-section can be plotted on mm-paper and a cross-sectional profile can be drawn (see Figure 3.4.11).
If so required a complete contour-chart of the river can be made derived from the cross-sectional profiles.

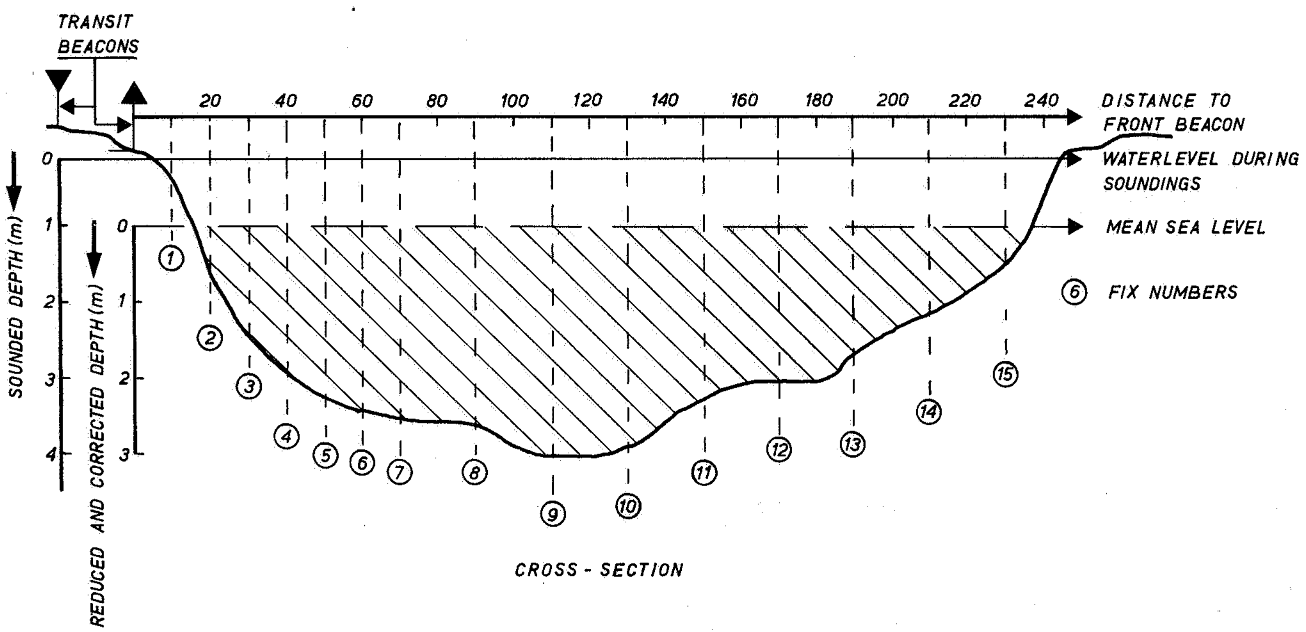


Figure 3.4.11 Example of a cross-sectional profile

3.5 Velocity

3.5.1 General

From a navigational point of view, surface velocities are important as the manoeuvring of a vessel is influenced by the velocity and direction of the current. Therefore it is essential when entering a harbour or moving to a jetty to know the velocity and direction of the current in respect to the vertical tide.
From stream rosettes the pilot can be informed about the direction and the velocity of the current at certain stages of the tide.

In the field of hydrology or coastal engineering it is often necessary to measure flow velocities in order to determine discharges.
In rivers outside the tidal zone or without any tidal influence, once a water-level discharge curve (Q/h) has been established, the discharge of the river is simply determined by measuring the water-level, unless the river-bed has changed considerably.
In tidal waters, however, this is not possible due to the mutual influence of the river run-offs and tidal phenomena which determine the strongly time dependent flow patterns and discharges for instance:

- a continuous change of water-level due to tidal influence
- greater velocity gradients in longitudinal direction than in rivers with flow in one direction
- change in distribution of velocities in time
- during the transition period from ebb to flood current, or vice versa there is a change in direction of the current with an intermediate zero velocity
- the occurrence of high and low water may not take place at the same time as the change in the direction of the current
- the change in direction of the current does not take place at the same time throughout the whole cross-section
- there may be stratification of flow by density current (a salt water wedge) with the upper strata flowing in one direction and the lower strata flowing in the reverse direction, while the change of direction of current and the maximum velocity occurring at different times and different depths
- some times large and rapid variation of width and capacity of section due to successive covering and uncovering of banks by the tide.

In case of morphological studies in rivers, estuaries or along open coasts often detailed information about current velocities, their direction and horizontal and vertical distribution, is required.

The current velocities in a river, estuary etc. can be determined by several methods.
The purpose of the survey, the site conditions, the type and number of vessels determine the method of measurements. Propeller instruments, pendulum meters, floats or a combination of these instruments can be used. It is also possible to determine velocities and/or discharges by using the dilution method or moving boat method.

3.5.2 Sampling the velocity profile

3.5.2.1 General

In practice only a limited number of observations can be carried out to determine a velocity profile.
From a statistical point of view the question will arise how large the sample should be to be representative with a certain confidence limit for the population (= velocity profile).
Much research was carried out to answer this question. From the results of discharge measurements in a large number of rivers without tidal influence of different sizes all over the world, it appears that to determine the river discharge it is more advisable to sample the velocity profile with many verticals in the cross-section than to observe only a few verticals with many velocity measurements in each vertical.
This is obvious, considering a sketch of a cross-section of a river on the same horizontal and vertical scale and not on a distorted scale as is presented in many cases: the depth of a river is in fact only a fraction of the total width.

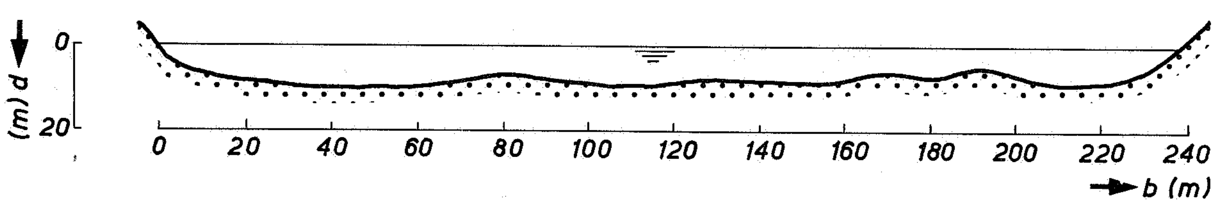
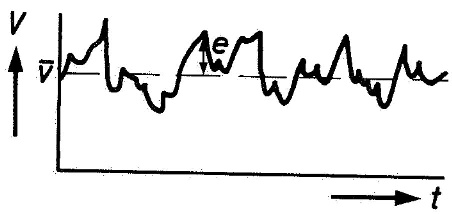


Figure 3.5.1 Cross-section of river (horizontal and vertical scale the same)

Because of the density currents which may occur in tidal regions it is, however, advisable to take velocity measurements at more or less close intervals in the verticals.
The normal measuring procedure is to measure the velocity profile of a cross-section in a river, but the observation of velocities is only taking a sample from the population of the whole velocity profile.
This velocity profile itself is not at any moment the same profile due to the fluctuations of the water velocity caused by turbulence.
The actual water velocity can be decomposed in a average component (deterministic) and in a stochastic component.



V = ACTUAL VELOCITY
$\bar{V}$ = MEAN VELOCITY
e = STOCHASTIC COMPONENT

Figure 3.5.2 Stream velocity

Using for instance a propeller current meter, the average velocity for a small period (seconds or minutes) is obtained by counting the number of revolutions of the propeller in a certain time lapse and is used as the velocity for one moment.
The problem to the hydrometrist is now how long should the time lapse be to reduce the effect of the stochastic component on "the" velocity.
It seems according to research and tests that an observation time of 30-60 seconds is advisable to reduce the stochastic component.

For measuring period of 30-60 sec

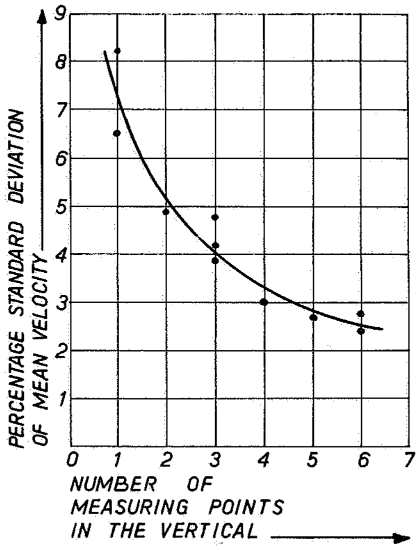


Figure 3.5.3 Influence of the number of points on the accuracy of the mean velocity

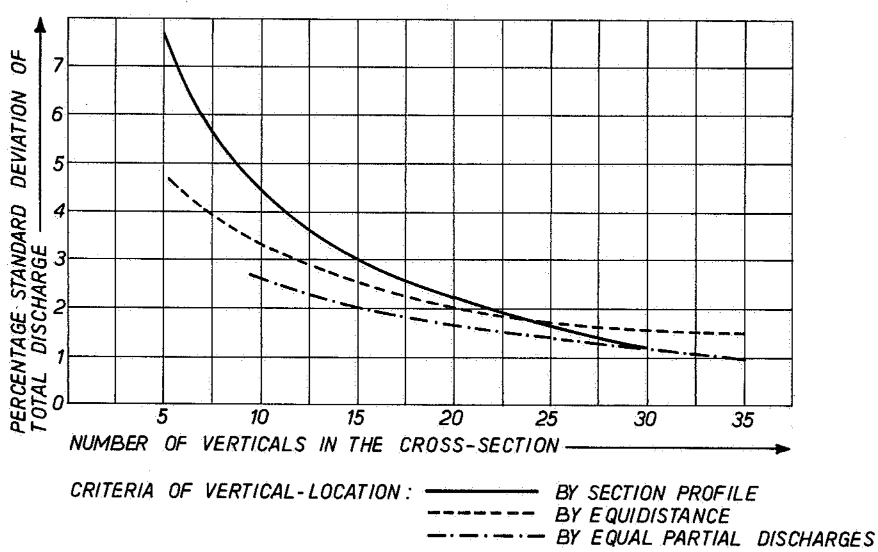


Figure 3.5.4 Influence of the number of verticals on the accuracy of the total discharge

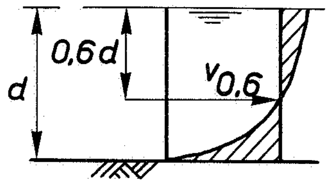
3.5.2.2 Methods to determine mean velocity ($\bar{V}$) in the vertical; non tidal rivers

Because of the relatively slowly changing conditions of the riverflow in non-tidal rivers, the velocity measurements can be carried out by one vessel only, which measures the velocities in one vertical after the other along the cross-section.

In order to measure the whole velocity profile in a reasonable short time, the velocity profile is sampled by a finite number of observations.

The mean current velocity in each vertical can be determined by any of the following methods depending on the available time and having regard to the accuracy, the width and depth of the water and the bed conditions and the changing stage;

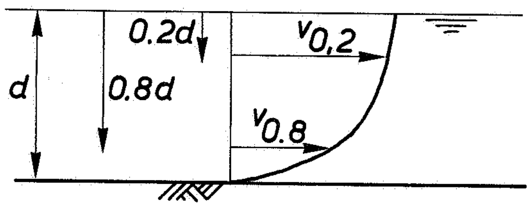
1. One point method



$\bar{V} = V_{0.6}$ (this method only to be applied in shallow water where the two point method or other methods cannot be used)

Figure 3.5.5 One point method

2. Two point method



$$\bar{V} = \frac{V_{0.2} + V_{0.8}}{2}$$

Figure 3.5.6 Two point method

3. Three point method

$$\bar{V} = \frac{V_{0.2} + V_{0.6} + V_{0.8}}{3} \quad \text{or} \quad \bar{V} = \frac{\frac{V_{0.2} + V_{0.8}}{2} + V_{0.6}}{2}$$

4. Five point method

$$\bar{V} = \frac{V_s + 3V_{0.2} + 2V_{0.6} + 3V_{0.8} + V_b}{10}$$

V_s = velocity at surface

V_b = velocity at bottom

3.5.2.3 Method to determine mean velocity ($\bar{V}$) in the vertical; tidal rivers

Because of the relatively rapidly changing condition of the riverflow, a result of the reasons mentioned in Section 3.5.1, such as change in distribution of velocities in time and the change in direction of the current does not take place at the same time throughout the whole cross-section, velocities in a cross-section should be measured simultaneously in as many verticals as possible.

Owing to the rapid change of the velocities, the individual measurement in the vertical has to be reduced to the same instant in order to get the current velocity distribution over the vertical.
The following procedures may be used:
- the velocities are measured at a suitable number of points in the vertical going from the surface to the bottom and back to the surface.
- at every point the velocity is measured twice except at the lowest point, when taking the average of both measurements at each point, the current velocity distribution in the vertical is found for the moment of the measurement at the lowest point.
- furthermore it is necessary to measure the direction of the current at each point of the vertical.

To obtain the mean velocity in the vertical the sum of the measured velocities can be divided by the number of points.
It is however advisable to plot the velocities in a velocity graph to obtain a check whether the measurement has been carried out correctly.
If the velocities show too much inconsistencies then the measurements have to be repeated immediately.
The mean velocity can also be derived by planimetring the velocity graph and dividing the result by the waterdepth.

3.5.2.4 Duration of measurements

In non-tidal rivers the velocity measurements can be carried out in a rather short time, measuring one vertical after the other across the cross-section.

In tidal rivers, however, the duration of the measurements depends on the type of tide in the area but should cover a full tidal cycle.
If the tide is diurnal or semi-diurnal with a large inequality in amplitude the observation should cover a period of at least 25 hrs, if the tide is pure semi-diurnal an observation period of 13 hrs will suffice.

If the object of the tidal hydrometry is to find the magnitude of the upland discharge only, it is immaterial during which tide the measurements are carried out.
If however, the object is to obtain an insight into the horizontal tide, the most complete information is obtained if observations are made during both extremes, that is during spring and neap tide.

Unlike the method of measurements in non-tidal rivers, in tidal rivers the measurements should start just before HW or LW slack and should be continued to another HW or LW slack and dependent on the type of tide this can be after 25 hrs (diurnal tide) or 13 hrs (semi-diurnal tide).
If the upland discharge has to be derived from the observations the measurements should end at the same waterlevel as at the start of the observations, regardless whether it is slack water or not at that specific moment.
This is done to make sure that no bankstorage by bankoverflow of previous flood tides is included in the measurements.
It often occurs that especially the low water slack will be delayed beyond the normal tidal cycle, due to outflowing bankstorage upstream and the outcome of the upland discharge will be influenced by this.

In Figure 3.5.7 is illustrated the vertical and horizontal tide while in Figure 3.5.8 the shape of a Q-graph composed of low and high riverdischarge and the tidal prism is illustrated.

3.5.3 Velocity observations

The velocity of water can be observed directly or indirectly.
Directly means that the moving water "particle" is followed and the time lapse in which the particle travels a certain distance is measured. To distinguish the particles under review from the surrounding water a cloud of dye may be used. The cloud moves with regard to the bottom of the river at the same rate as the surrounding water as far as the mean (surface) velocity of the river is concerned.

Current velocities and current directions can also be determined directly by float observations. Sometimes it is even the only method to be applied because of the circumstances under which the current measurements have to be carried out.
In estuaries and along the coast wave action might be such that measurements with other methods (propellers, pendulum) are not feasible: the vessel from which the measurements are to be done will probably pitch and roll in such

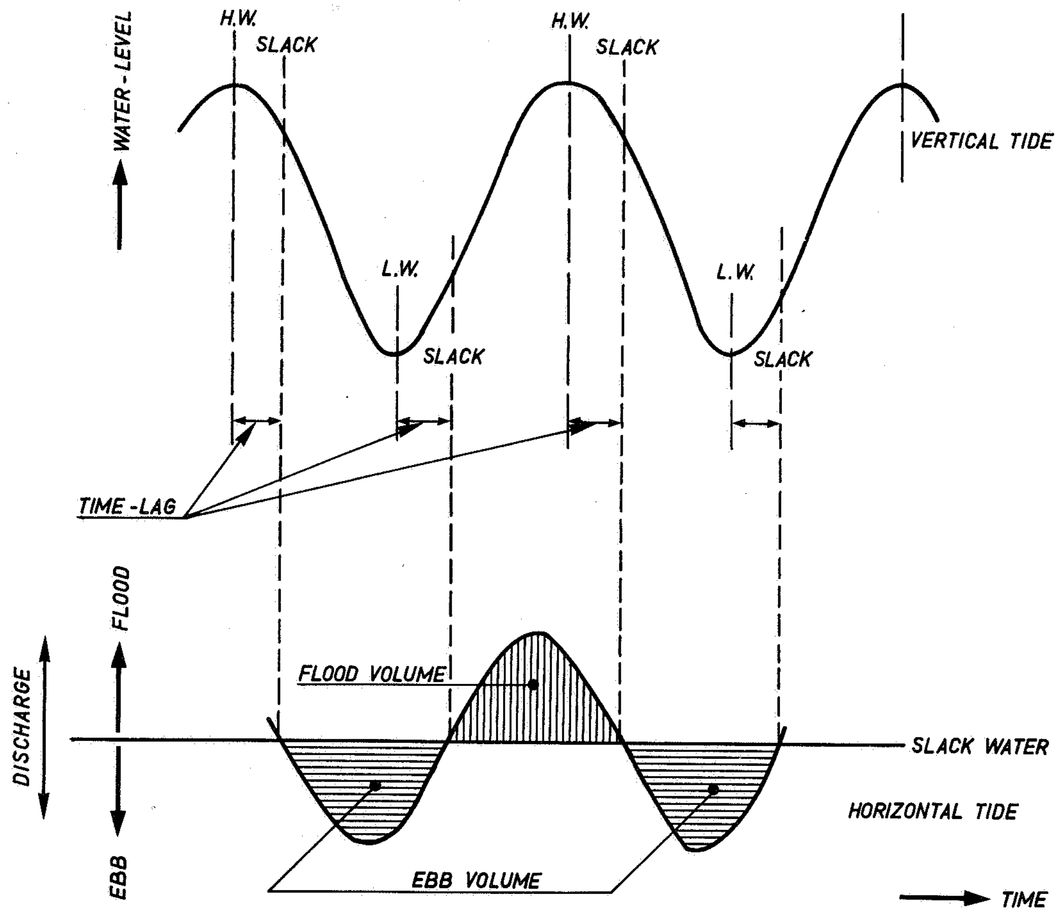


Figure 3.5.7 Vertical and horizontal tide

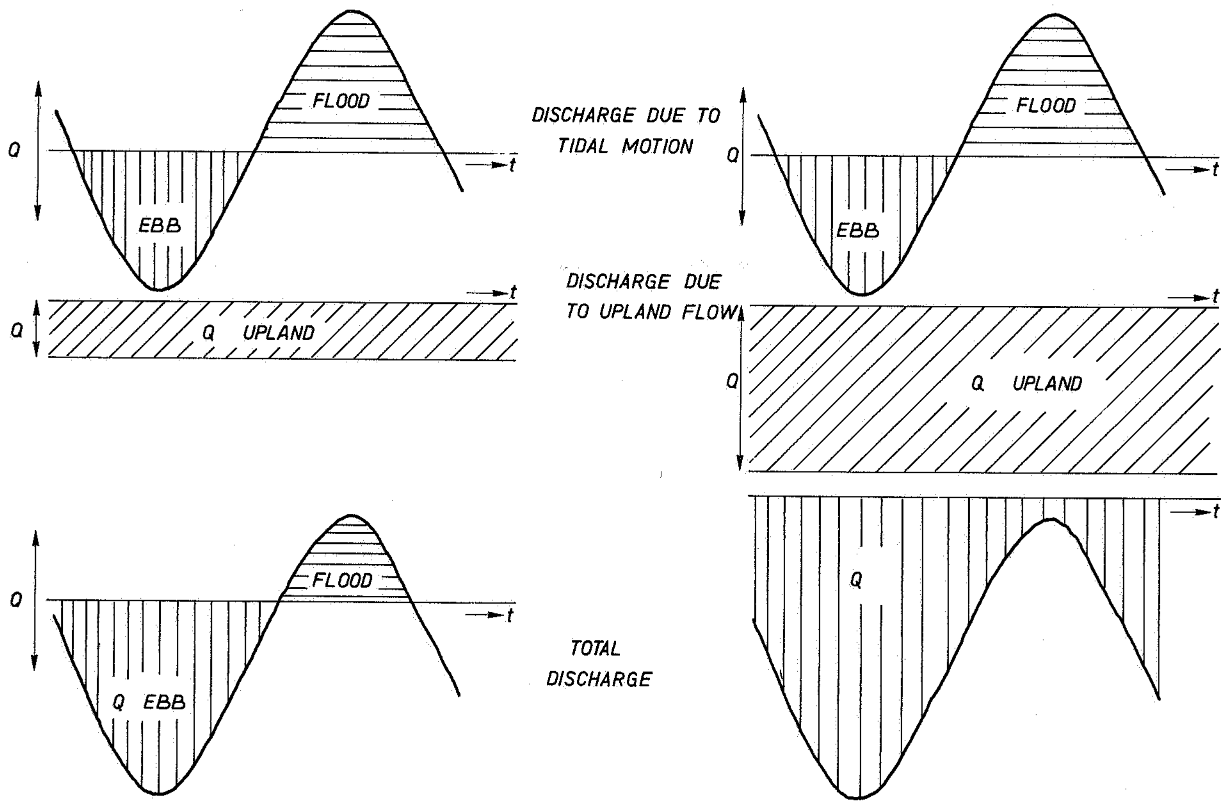


Figure 3.5.8 Shape of Q-graph composed of upland flow and tidal prism

way that the current-meter is moved up and down or even sideways - causing other components than the current velocity to act on the propeller.
Moreover the orbital velocities, due to waves, may severely obscure the true current velocity.

Measuring the current velocity indirectly is carried out by propeller or cup current-meters, savonius rotor current-meters and pendulum meters.
For the propeller type instruments the passing water particles turns the propeller.
For the pendulum type instruments, a resistance body suspended on a thin wire is deflected from the vertical and the angle of deflection is a measure of the current velocity, dependent on the mass and area of the resistance body.

3.5.3.1 Propeller current-meters

Propeller current-meters can be used either suspended from a cable when measurements are carried out from a vessel in waterdepths exceeding 1½ mtr or connected to a wading-rod in waterdepths up to 1½ mtr.

The number of revolutions in a certain time lapse is a measure of the current velocity dependent on the pitch of the propeller as a waterparticle travels a distance equal to the pitch of the propeller in one revolution.
With a propeller of 0.50 m pitch, the waterparticle travels 0.50 m in one revolution and when the time is measured as well, the velocity in m/sec can be obtained.
The pitch determines also to what maximum velocity the propeller can be safely used:

Pitch	Maximum velocity
0.125 m	1.25 m/sec
0.250 m	2.50 m/sec
0.500 m	5.00 m/sec
1.00 m	10.00 m/sec

Before the current-meter is used, the propeller must be calibrated, for the OTT-current-meter this is done in a calibration towing tank in the factory and the calibration formula is given in the cover of the case.
Normally the calibration results are expressed in a set of formulae, such as given below.

Propeller 10821/2	125 mm dia. pitch 0.250 m
if $n < 2.22$	$v = 0.2543\ n + 0.018$ m/sec
if $n > 2.22$	$v = 0.2615\ n + 0.02$ m/sec

These formulae are derived from the general formula $v = k.n + \Delta$, in which:

v = current velocity (m/sec)
k = the pitch of the propeller (m)
n = number of revolution (1/sec)
Δ = constant in m/sec

Gauging with the ideal current-meter, having a propeller of purely helical contours, and being absolutely free of all outside influences such as friction and backwater caused by the support of the meter, would render between v and n the simple relation

$$v = k \times n$$

where k is identical with the geometric pitch of the propeller, if the n-values are plotted in a graph along the abscissa and the v-values along the ordinate, the above equation becomes a straight line with a slope equal to

$$k = \frac{v}{n}$$

But in fact only an approximation to these ideal conditions is really possible. It is for this reason that the v-n curve of a current-meter deviates from a straight line and present itself as a more or less flat hyperbola which starts a small distance up on the ordinate. This point corresponds to the responsive starting speed V_o of the current-meter.
In higher ranges of revolution (n) the curve will run straight.

Be aware that the right calibration formula is used for the propeller because the constant Δ depends also on the backwater effect due to the support of the current-meter.
For a current-meter calibrated as a suspended meter the calibration will be different if the instrument is used on a wading-rod.

3.5.3.2 Floats

The measuring procedure may be different according to the purpose of the measurement.
In general it can be said that the procedure is to measure the time lapse in which the floats travels a certain distance.
Several measuring procedures are mentioned below.

Float attached to a string. From an anchored vessel the float on a thin string is released (see Figure 3.5.9)

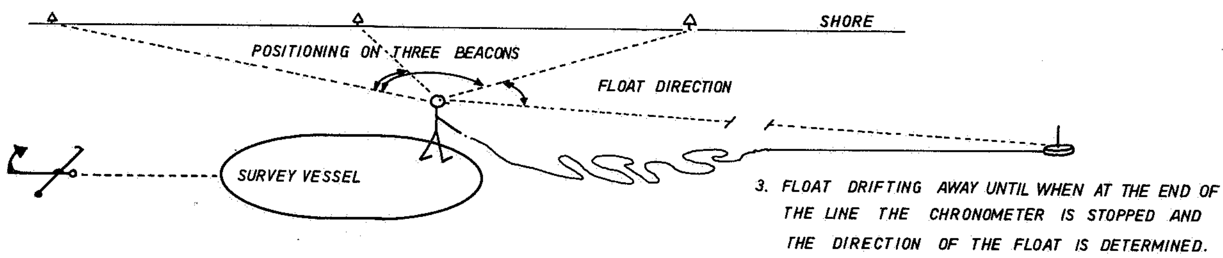


Figure 3.5.9 Float measurement from anchored vessel

The string is divided in three parts with a total length of 30 meters, the first part of 5 meters being the precursor and the end of it marked by a knot. This knot indicated the start of the measuring string of 20 meters - which again ends in a knot. The remaining part of 5 meters is tied to the vessel.

The precursor is tied to the floating part of the float and serves the purpose to give the float an initial velocity, equal to the stream velocity and to get away from any disturbance due to the boathull. As soon as the first knot passes the hand of the observer, a chronometer is started and the second part of the string up to the second knot is eased off into the water in order to prevent restrictions in the movement of the float. The second knot is kept by the observer and as soon as the drifting stretches the string and pulls the knot, the chronometer is stopped.
The velocity of the current is obtained by dividing 20 meters by the time lapse. The direction of the current may be obtained or by compass bearing or by sextant angles between a known direction to a beacon on shore and the float.
Another procedure is the free-floating way. The floats are now followed by ship

and at specific intervals the location of the floats in their track are determined by sextant angles (see Figure 3.5.10) or by means of theodolite bearings from ashore (see Figure 3.5.11).

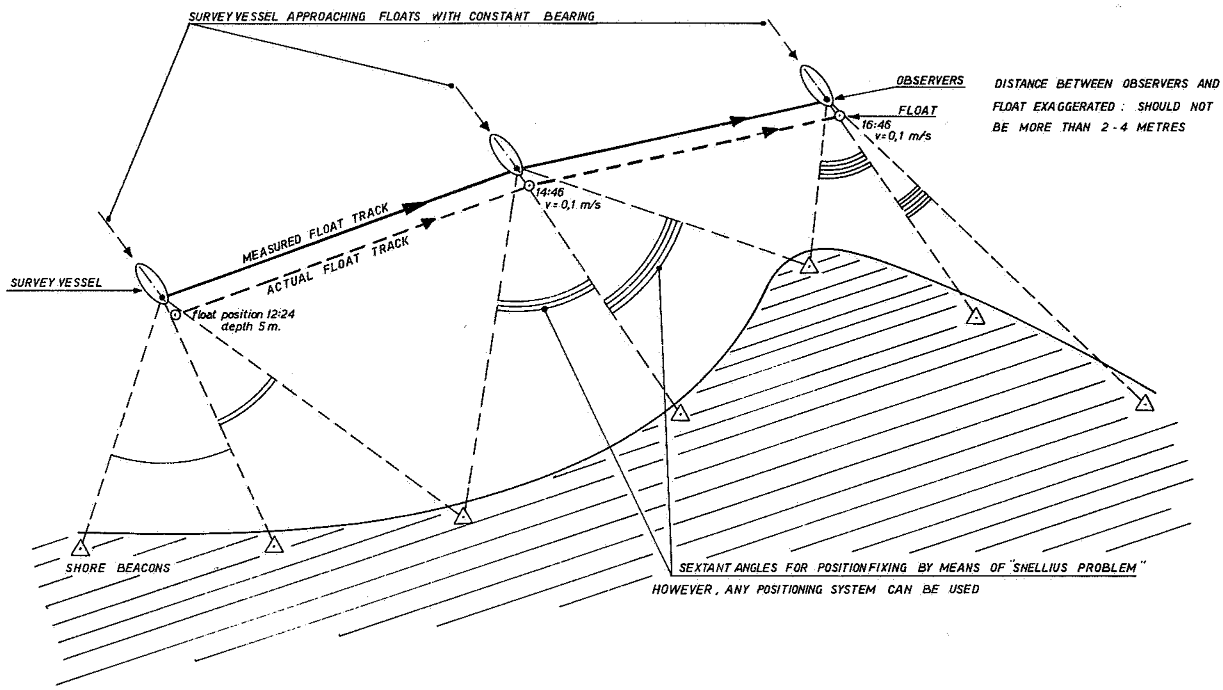


Figure 3.5.10 Positioning of the float by sextant angles

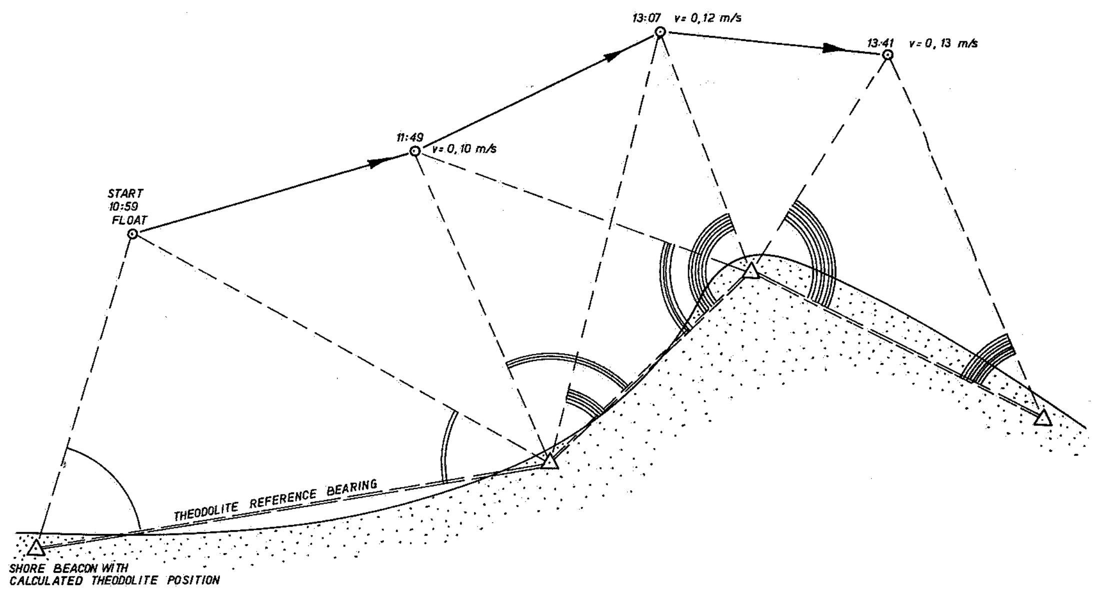
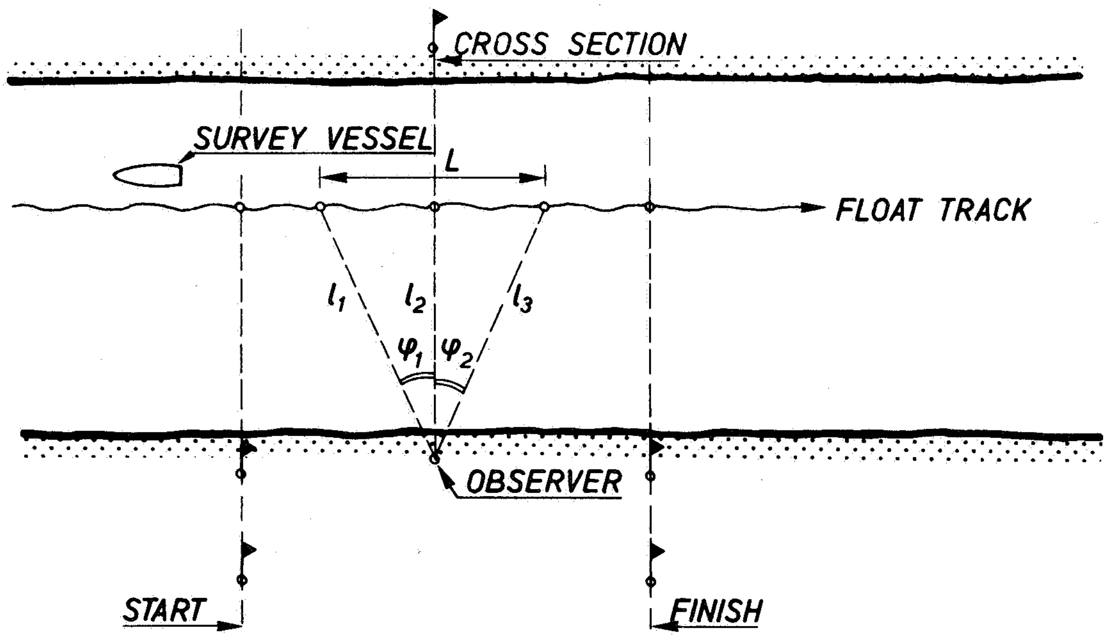


Figure 3.5.11 Positioning of the floats by theodolite bearings

The average velocity in each track is obtained by dividing the travelled distance measured on the plotting chart by the time interval between two plotted position-fixes, while the direction of the current is derived from the bearing between the two position-fixes in respect to the North of the plotting chart. Measuring in this way gives also an indication of the general direction of the current flow.
Nowadays using scientific calculators no plotting is required, but the average velocity and the direction of the floats can be calculated directly if the co-ordinates of the shore beacons are known and the angles are put in as variables.

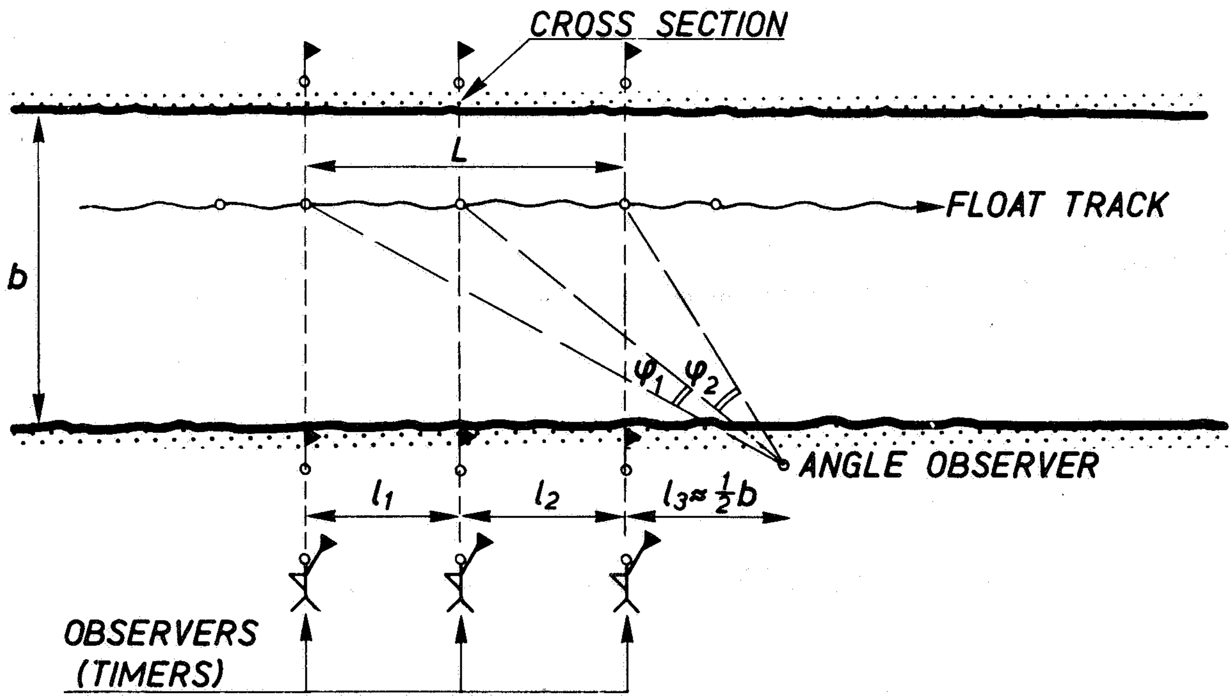
If the floats with their resistance crosses set at various depths, are dropped at the same position every half hour during a tidal cycle and recovered after say 10 minutes (depending on how strong or weak the current is) - the floats should have travelled a long enough track to determine on the plotting chart the velocity distribution and the current pattern at this position can be established.
By using floats it should be kept in mind that wind velocity and direction should be measured as well in order to determine the wind influence on velocity and direction of the float.
Therefore it is essential to reduce the influence of the wind, by making the floating body as small as possible and to keep the top of the floating body flush with the watersurface by attaching some scrap iron as weight under the float.
In any case floats with resistance crosses up to $1\frac{1}{2}$ mtr under the water surface are most effected by wind influence as the toplayer of the water is influenced by it.
By using floats also the flow pattern in a certain area can be measured by letting the float drift during a full tidal cycle, taking position-fixes of the floats at regular intervals.

The above mentioned procedure is used in estuaries and along coasts, but also in rivers. Because of the fact that in rivers the float is in general closer to the observer(s), its position on its track can be obtained by the following method: a combined distance and angle measurement or angle measurement and a range line. Some possible arrangements are presented in the Figures 3.5.12 to 3.5.14.
The float track is plotted and the velocity is again calculated from the travelled distance divided by the time lapse.



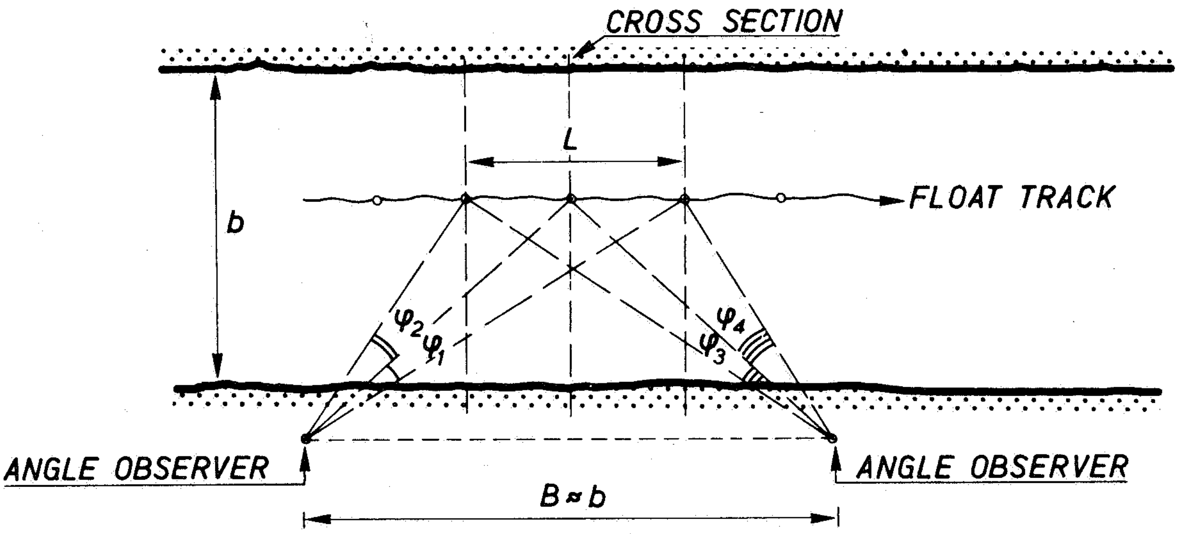
Measurements: l_1, l_2, l_3 = distance
ϕ_1, ϕ_2 = angle
time for traveling L

Figure 3.5.12 Positioning by angle and distance measurement



Measurements: l_1, l_2, l_3 = distance
ϕ_1, ϕ_2 = angle
time for traveling L

Figure 3.5.13 Positioning by angle measurement and fixed distance (l_1, l_2, l_3)



Measurements: $\phi_1, \phi_2, \phi_3, \phi_4$ = angle (with respect to reference bearing)
B = distance

Figure 3.5.14 Positioning by angle measurement and fixed distance (B)

As mentioned above a resistance cross is fixed at a certain depth below the float. The velocity of the float equals the current velocity at the depth of the resistance cross.
As the velocity distribution in the vertical can be assumed to be parabolic the mean current velocity is obtained if the resistance cross is fixed on 0,6 times the waterdepth (see Figure 3.5.15).

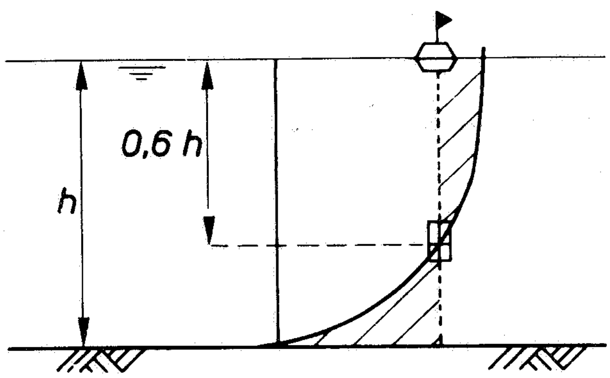
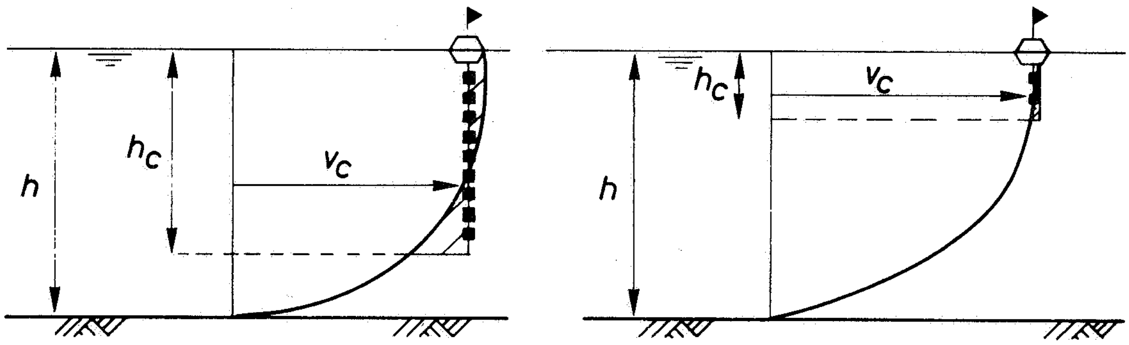


Figure 3.5.15 Cross at 0,6.h

Sometimes chain depth floats are used to measure the mean water velocity over a certain depth. At this type of float a number of small resistance crosses are attached to the float. The upper crosses will be slowed down by the crosses closer to the bottom and the lower crosses will be speeded up by the upper crosses, were the current velocity is higher than closer to the bottom. Because of the irregular bottomprofile and to prevent grounding of the float the chain cannot reach exactly to the bottom so that a part of the vertical is not sampled for velocity but can be estimated under the assumption that the velocity distribution function is a parabola (see Figure 3.5.16).



$$\bar{v} = \frac{(h_c \times v_c) + \frac{1}{2} v_c \cdot (h - h_c)}{h}$$

$$\bar{v} = \frac{(h_c \times v_c) + \frac{2}{3} v_c \cdot (h - h_c)}{h}$$

rest plane $\frac{1}{2}\ v_c \cdot (h - h_c)$

rest plane $2/3\ v_c \cdot (h - h_c)$

Figure 3.5.16 Rest plane

With emphasis it is stated that because of the non-parabolic shape of the velocity distribution in tidal areas it is not recommended to use the chain-depth float and interpret the results as mentioned above.
However, float observations can give good results, they are sometimes even the only method for measuring current velocities and direction at specific depths, but the results should be checked with other methods when converted to average velocities and for determining total river discharge.

3.5.3.3 Pendulum-meter

The measuring procedure is that the resistance body is lowered with a cable from a winch with depth indicator from the anchored survey vessel.
The cable is led over a support which carries also the dial construction for reading the deflection of the wire from the vertical axis.
This deflection can be read on the cupola by means of concentric rings while the deflection in respect to the longitudinal axis of the ship can be read by means of the radials, this last deflection gives the azimuth of the current if a compass heading of the vessel is observed at every measurement.
Because of the fact that the wire-bending has to be taken into account, the

velocity distribution along that part of the cable that has already been lowered should be known.
This means that at rather many points in the vertical the velocities have to be measured.
The deflection of the cable from the vertical, read on the so-called cupola, is a combination of the force of the velocity current againt the resistance body (ϕ body) and the deflection of the wire due to resistance against the current.

$$\phi_{read} = \phi_{body} + \phi_{cable}$$

These deflections related to the velocity have been calibrated and are given in calibration graphs (see Figure 3.5.17 and 3.5.18). Graph 1 gives the relation v - ϕ_{body} and α - v, in which α is a factor to determine the cable bending. α depends on the drag coefficient of the cable and resistance body, the density and velocity of water.
Graph 2 gives the relation between ϕ_{read}, $\Sigma\alpha\Delta L$ and reduction of $\Sigma\alpha\Delta L$ and $\Sigma\Delta L$, in which ΔL is the increase in cable-length with respect to the previous observation.
When the velocity has such a value that the ϕ_{read} is smaller than 5^o or larger than 30^o then another resistance body has to be used. (For different ranges in velocity different types of bodies are available.) As a rule of thumb the following formula may be used to estimate the required weight of the resistance body:

$$h_{max} * v_{max} * 5 = \text{required weight } [kgf]$$

in which: h_{max} = maximum depth in cross-section [m]
v_{max} = maximum velocity in cross-section [m/s]

Due to the fact that for a good determination of the cable-bending at a certain depth, velocities have to be measured at preceding depths, this instrument is not suitable for one point, two point etc. sampling in the vertical.
Sampling intervals of 1 meter are necessary to calculate the wire-bending in good order. So in fact a more or less continuous velocity profile is obtained.
The integration method as used by propeller instruments cannot be applied.
The discharge per unit width is calculated in the same way as for the propeller instrument.

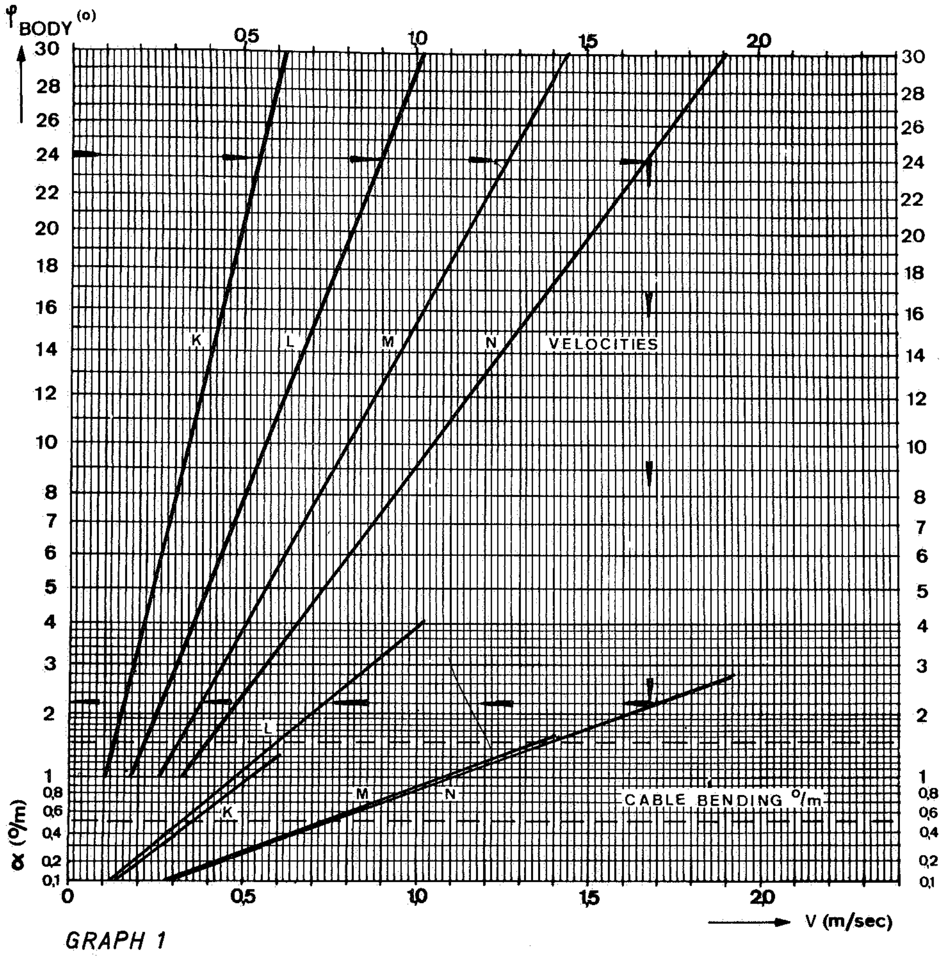


Figure 3.5.17 Calibration graph Handpendulum-meter

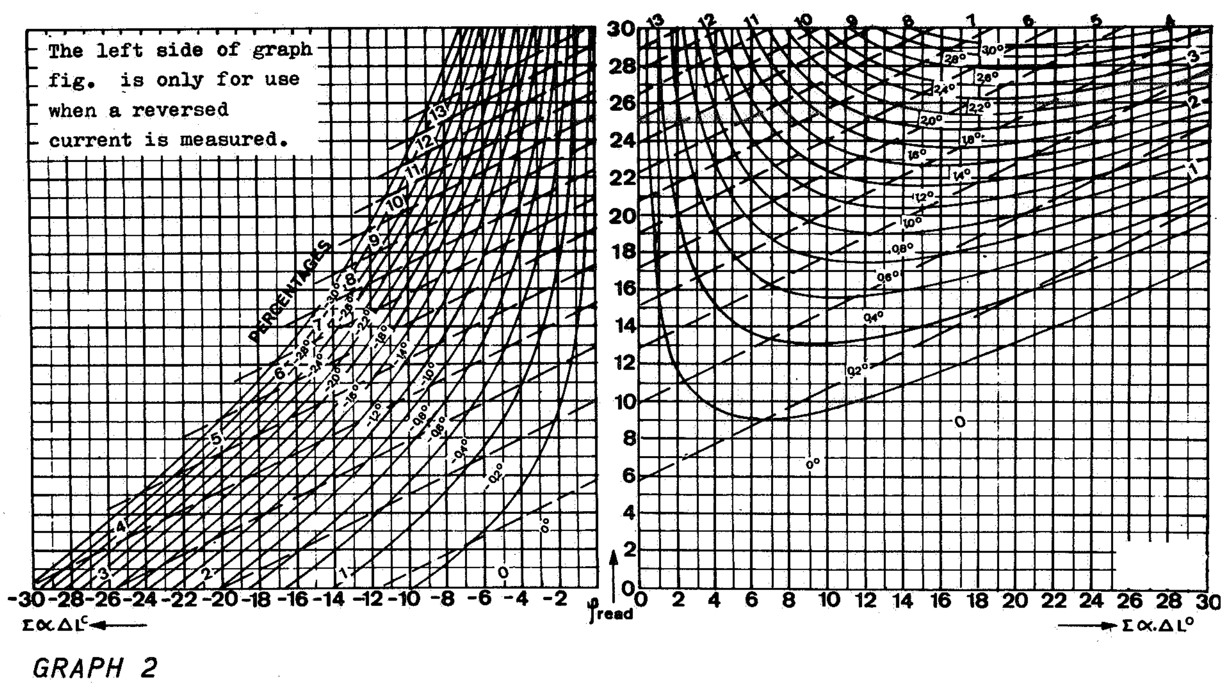


Figure 3.5.18 Correction graph for Σ α AL and depth of body

3.5.3.4 Practical informations for measurements in tidal rivers

The following should be included in the measurements:

a) Gauge readings from a recording or staff gauge, which should be read every 15 minutes or half hour. Care should be taken that the clock of the gauge reader or from the tidal recorder is checked with the clocks on board of the survey launches.

b) Every hour the velocities are measured at intervals of 1 or 2 meters along the vertical, starting at 0.5 m below the water surface and finishing at 0.5 m to 0.2 m above the river bed.
The discharge per unit width (q) must be calculated immediately after the observation in order to check whether wrong observations are made.

c) The depth below the survey vessel must be recorded each time after finishing the velocity observation in a vertical. The depth, recorded by the echo-sounder or by a well-calibrated lead-line, may deviate from the depth found from the current-meter due to the fact that the latter in a strong current does not go down vertically and is therefore often recording too great a depth. The calculated q may be corrected rectilinearly, thus:

$$q_{corrected} = \frac{h_{recorded}}{h_{current\text{-}meter}} * q_{calculated}$$

in which h = water depth.

d) On each form on which the velocities of a vertical are recorded the current direction during observation should be noted. To eliminate possible confusion it is customary also to mention to which nearby well-known location or water course the water is flowing.

e) The time of each slack water, when the flood changes in ebb flow or the reverse, has to be noted.

f) At the start of the observations, vessels are anchored in the chosen locations. During the course of the observation period it is essential that the observer checks the position of the vessel repeatedly, at least before each hourly observation begins.

3.6 Discharge measurements

3.6.1 Introduction

3.6.1.1 General

Although the principle to obtain the discharge per unit width (m^2/sec) (e.g. the product of the mean velocity in the vertical and the area per unit width) remains the same whether the measurements are carried out under permanent - or non-permanent river flow conditions the method of calculating the total discharge Q(m^3sec) is very much dependent on the above mentioned conditions.

The availability of observers, vessels and instruments may also be the reason that in river cross-sections, velocities are only measured in a few verticals, so that the total discharge Q (m^3sec) has to be calculated from the measurements in a limited number of verticals. See par. 3.5.2. about the influence of the number of velocity observations in the vertical and the measurement number of verticals on the overall accuracy of the discharge.

Although it should not be applied indiscriminately, good use can be made of the formula of Chézy for a number of cases that only too few verticals have been measured to obtain a good estimation of the q-h curve for the other verticals.

The formula of Chézy is:

$\bar{v} = C \sqrt{dI}$ (The average velocity can be obtained as described in paragraph 3.5.2.2.)

in which $\bar{v}$ = average velocity in a vertical m/sec
C = roughness coefficient of Chézy $m^{\frac{1}{2}}$/sec
d = water depth
I = longitudinal slope of the water surface.

3.6.1.2 Site selection

The site of the cross-section whether it is located in tidal or non-tidal rivers should comply with the following conditions.

- the cross-section must be in a straight reach and of uniform profile.
- it must be avoided to have the site close to confluences or bifurcations to prevent backwater influences.
- the waterdepth in the selective cross-section should be sufficient to provide effective immersion of current-meters or floats whichever are to be used.
- the bed of the reach should not be subject to change during the period of measurement.
- the orientation of the reach should be such that the direction of flow is as closely as possible normal to that of the prevailing wind.
- sites at which vortex or backwater flow or deadlines tend to develop should be avoided.
- all discharges should be contained within a defined channel with stable boundaries (banks) and well defined geometrical dimensions.
- the selective cross-section should be marked by means of transit beacons at both banks.
- a staff gauge or waterlevel recorder should be established at the cross-section.

3.6.1.3 Determination of cross-section

To determine the profile of the cross-section the measurement of depth should be made at such spacing as to define the cross-sectional profile accurately, therefore an echosounder should be used.
During the depth measurement the waterlevel should be noted down.

The cross-sectional profile is plotted from the echosounder registration and position-fixes.
In non-tidal rivers the cross-section is divided in as many verticals as are required to obtain an accurate as possible discharge calculation.
In tidal rivers the location for the measurement's vertical, attuned to the available vessels, are chosen in such a way that the observed velocity profile at a specific location may be considered representative for that section of the cross-section where the measuring vessel is anchored.
Care should be taken not to locate a vessel too close to a riverbank.

3.6.2 Calculation of discharge per unit width

In non-tidal rivers the discharge per unit width q (m^2/sec) is the product of $\bar{v}$ (average velocity in the vertical (m/sec)) and the water depth (d) at the vertical at the moment of measurement).

In tidal rivers, where more points in the vertical are measured the normal procedure is to make a velocity graph to check firstly the velocity measurement and to determine the area of this graph by planimetring.
The mathematical expression is

$$q = \int_{o}^{h} v_y . d_y$$

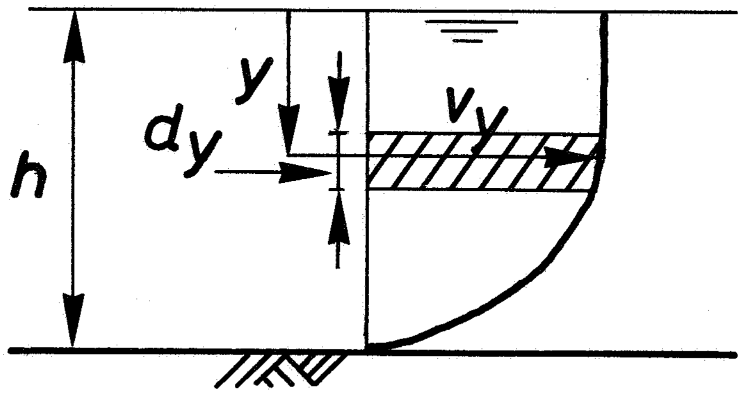
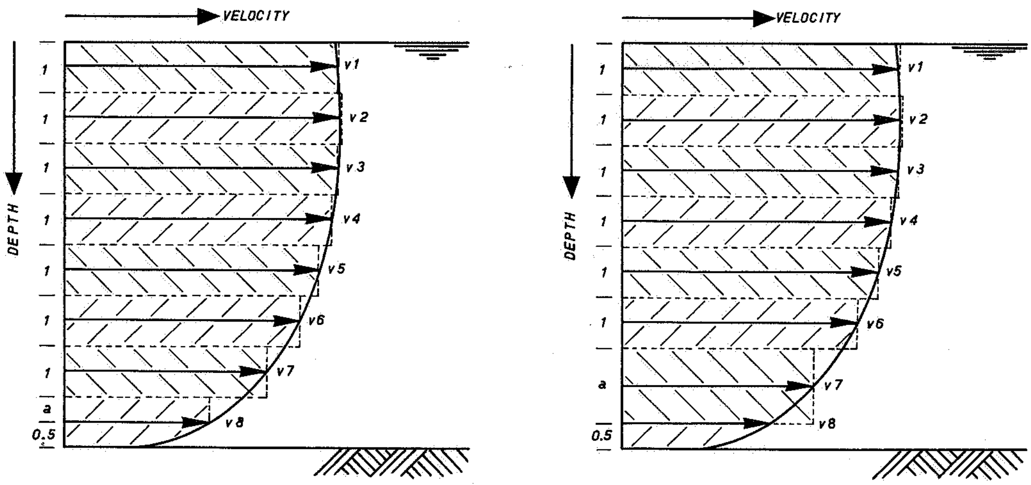


Figure 3.6.1 Velocity profile to determine q

If no planimeter is available, the discharge per unit width (q) can be found as illustrated in Figure 3.6.2. Calculation of q graphically.



$q = v1x1 + v2x1 + v3x1 + v4x1 + v5x1 + v6x1 + v7x1 + v8x(a + 0.25)$ $\qquad$ $q = v1x1 + v2x1 + v3x1 + v4x1 + v5x1 + v6x1 + v7xa + v8x0.25$

Figure 3.6.2 Calculation of q graphically

When using the planimeter to determine the discharge per unit width care must be taken not to make mistakes in the scale factor.

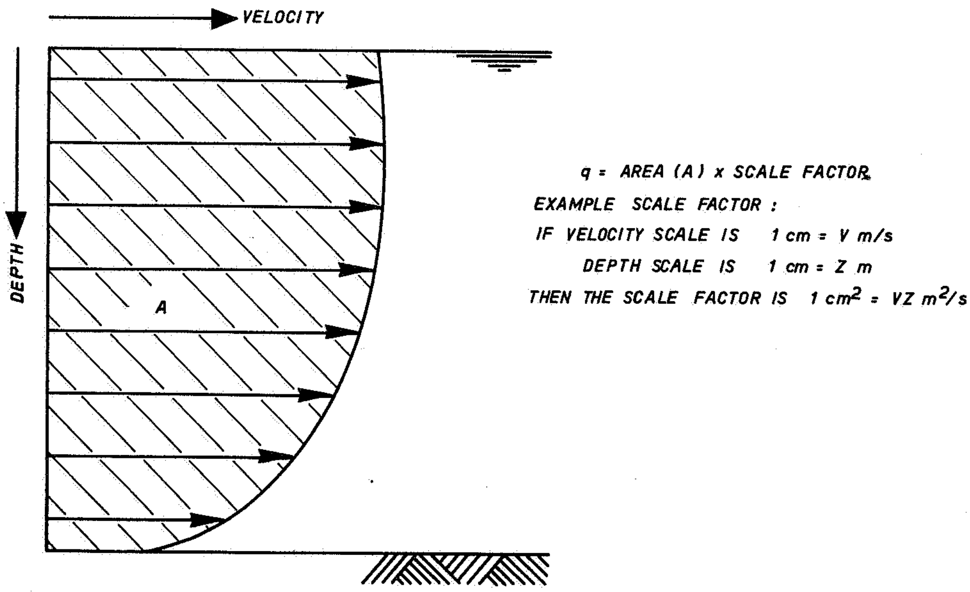


Figure 3.6.3 Example of scale factor

As mentioned in Paragraph 3.6.1 the discharge per unit width for other verticals than the ones which has been measured can be obtained by the formula of Chézy.

If we consider that the discharge per unit width = $q = \bar{v}.d$ and we substitute $\bar{v} = C\sqrt{dI}$ then $q = d * C\sqrt{dI} = d^{3/2} * C\sqrt{I}$.

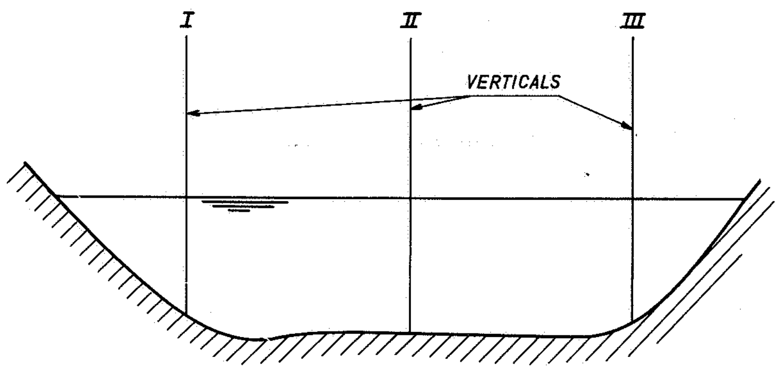


Figure 3.6.4 Cross-section

So in vertical I the discharge per unit width = $q_1 = d_1^{3/2} * C\sqrt{I}$ in vertical II $q_2 = d_2^{3/2} * C\sqrt{I}$, in vertical III $q_3 = d_3^{3/2} * C\sqrt{I}$.
If we assume that the roughness coefficient and the slope (I) have the same value for the whole cross-section then $C\sqrt{I}$ can be eliminated and the discharge per unit width are in relation to each other as:

$$q_1 : q_2 : q_3 = d_1^{3/2} : d_2^{3/2} : d_3^{3/2} .$$

So the dischargesper unit width are in proportion to $d^{3/2}$, this gives us the opportunity to obtain a weighed estimate of discharge per unit width for other not measured verticals by the $d^{3/2}$-curve.

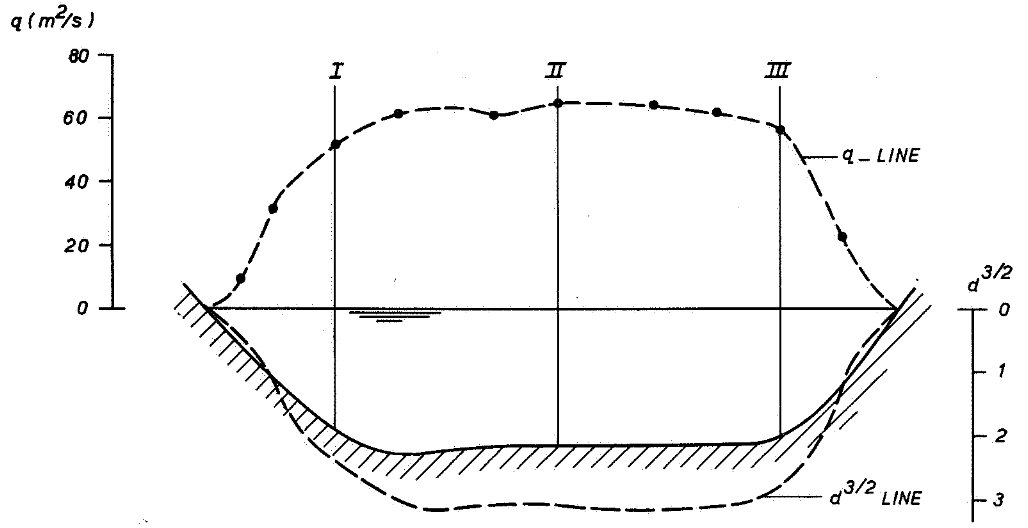


Figure 3.6.5 $d^{3/2}$-curve and discharge per unit width curve

3.6.3 Methods to calculate the total discharge

3.6.3.1 Graphical method

Depth velocity-integration method

The discharge per unit width q or the product of the value of the mean velocity $\bar{v}$ at each vertical and the corresponding depth ($\bar{v}$.d), should be plotted over the water surface line and a smooth curve drawn up connecting the $\bar{v}$.d points as shown in Figure 3.6.6.

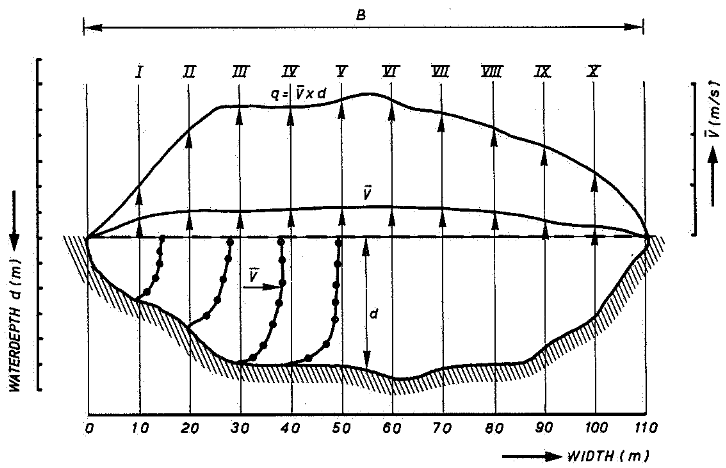


Figure 3.6.6 Depth velocity graph

To obtain a good $\bar{v}.d$ curve the rules described in Paragraph 3.6.2 can be used indicatively and total Q is obtained by planimetring the area enclosed by the q-curve and the water surface.
In tidal rivers this same method can be applied, but will take considerably more time to compute the discharge for a full tidal cycle as for every hourly observation in this cross-section $\bar{v}.d$ curves have to be drawn and the area to be determine.

3.6.3.2 Arithmetrical methods

Mean Section method

The cross-section is regarded as being made up of a number of panels each bordered by two adjacent verticals. If $\bar{v}_1$ and $\bar{v}_2$ are the mean velocities at the first and second vertical respectively, and if d_1 and d_2 are the depths measured at the verticals I or II respectively and "b" is the horizontal interval between the said verticals than the discharge of the panels is to be calculated as

$$Q_p = \frac{\bar{v}_1 + \bar{v}_2}{2} \times \frac{d_1 + d_2}{2} \times b$$

where Q_p is the partial discharge through the considered panel.
This is to be repeated for each panel and the total discharge is the summation of the discharges per panel.

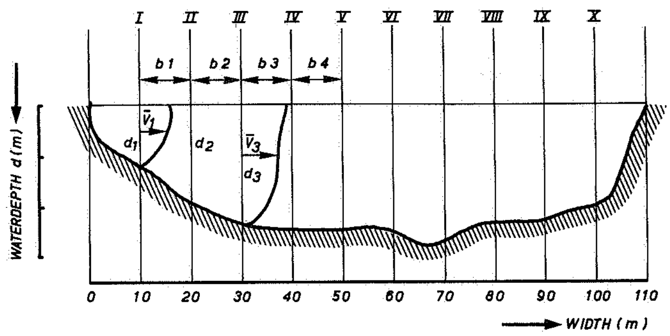


Figure 3.6.7 Mean Section method

For the panels at the side (close to the bank) the same equation can be used as above, whereas the velocity at the bank is taken as zero and the depth at that point is also taken as zero.
It must be realised, however, that the mean velocity in vertical direction towards the banks in many cases has a parabolic form and therefore it may give a better estimate to calculate Q_p for the panels near the banks as

$$Q_p = 2/3\ \bar{v}_1 * \tfrac{1}{2}\ d_1 * b$$

b = the width from the vertical I to the bank.
The total discharge is the sum of all the calculated Q_p's.

Mid Section method

Assuming a straight line variation of $\bar{v}.d$, the discharge in each section should be computed by multiplying $\bar{v}.d$ by the corresponding width measured along the water surface line. This width should be taken to be the sum of half the width from the adjacent vertical to the vertical for which $\bar{v}.d$ has been calculated, plus half the width from this vertical to the corresponding adjacent vertical on the other side.

$$Q_p = \bar{v}.d * \tfrac{1}{2}(b_1 + b_2)$$

in which b_1 = the horizontal distance between vertical I and II
b_2 = the horizontal distance between vertical II and III.

The value of $\bar{v}.d$ in the two half-widths next to the bank should be taken as zero.
The total discharge is a summation of all the calculated Q_p's.

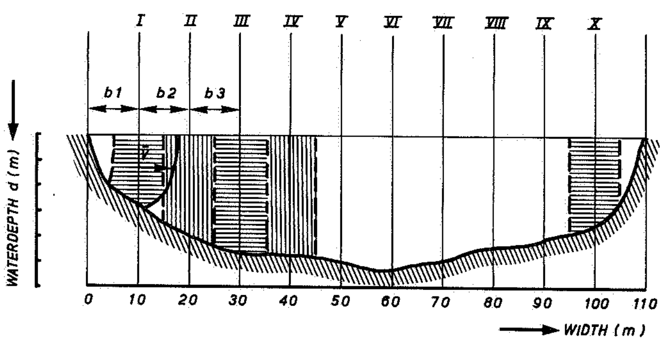


Figure 3.6.8 Mid section method

3.6.3.3 Velocity-contour method

Based on the velocity distribution curves of each vertical, a velocity distribution diagram for the cross-section can be prepared showing curves of equal velocity, see Figure 3.6.9.
Starting from the maximum velocity contour-line, the areas enclosed by the equal velocity curves and the water surface should be measured by a planimeter and should be plotted in another diagram with on the ordinate (y-axis) the velocity and on the abscissa (x-axis) the corresponding area enclosed by the respective velocity curve and the water surface.
The total area enclosed by the velocity area curves and the x and y axis (obtained by a planimeter) represents the discharge of the cross-section.

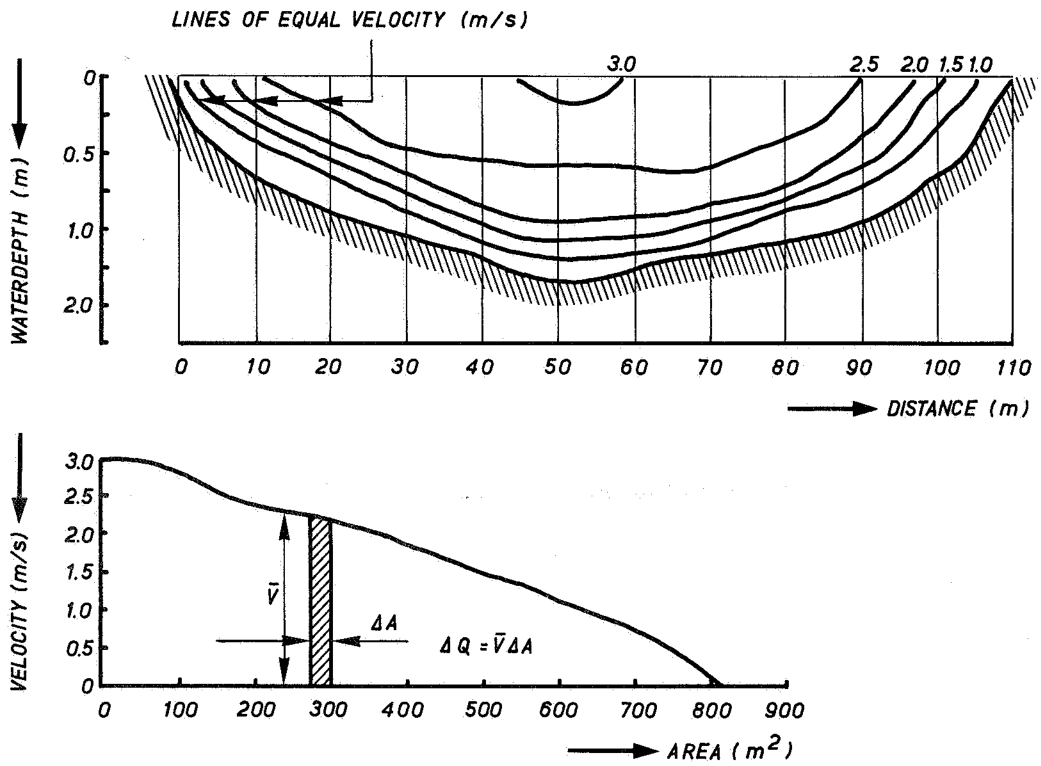


Figure 3.6.9 Velocity-contour method

3.6.3.4 Cruette's method

Another method to obtain the total discharge in a cross-section is the Cruette method.
This method can be applied for instance, if a Q-h relation curve for a certain station is difficult to be obtained, due to the long distance to travel to the station so that not enough discharge measurements can be taken at various water-levels.

By this method measurements can be taken during raising and falling stages in a short periode of time, in one visit to the station.
Measurement should start just before an upcoming flood wave and should be continued till the end of the falling stage.

The current velocity measurements are carried out from bank to bank at pre-selected verticals and repeated so that many verticals are measured with various water-levels.
During each measurement at a vertical the water-level is observed at the start and end of the measurement and noted down. To expedite the measurements each vertical to be measured at 0.2 and 0.8 of the water-depth, in case that the water-depth is too small only the 0.6 method is used.

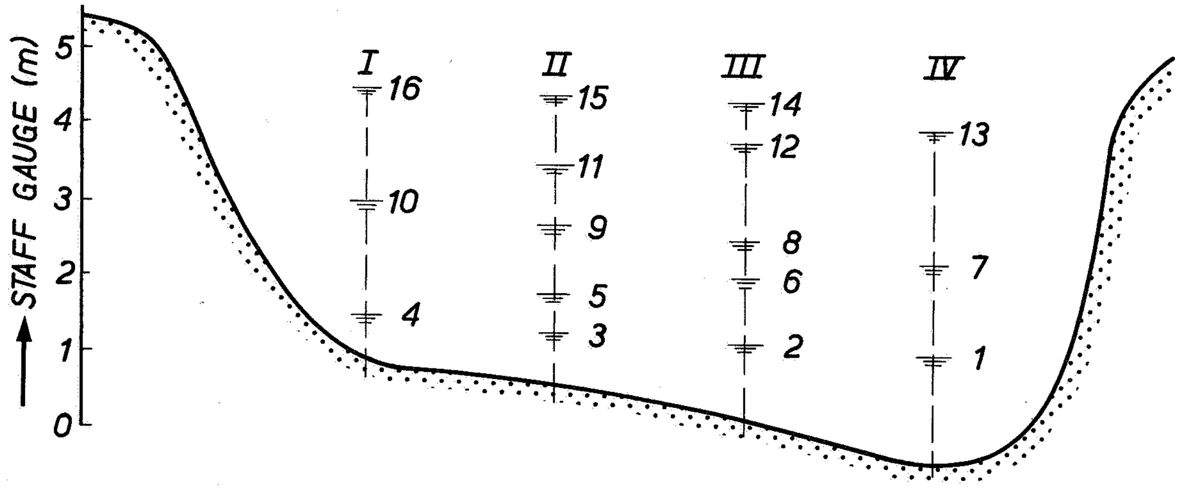


Figure 3.6.10 Cross-section and verticals with water-level sequence of measuring velocity profiles 1, 2, 3 etc.

For each vertical the q-h curve is determined

q = discharge per unit width

h = gauge reading at each measurement.

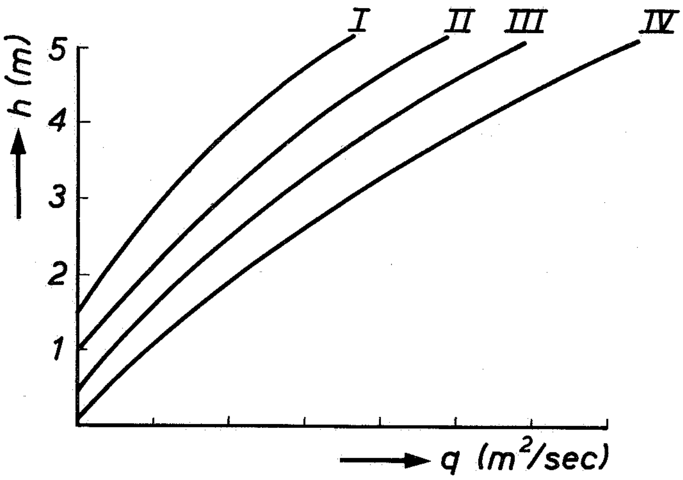


Figure 3.6.11 q-h relation for each vertical

From the obtained q-h curve a family of curves would be found by plotting at each vertical the q's corresponding to a same water-level. In this mass-curve diagram the q is plotted on the vertical axis and b (width of the river) on the horizontal axis (see Figure 3.6.12).

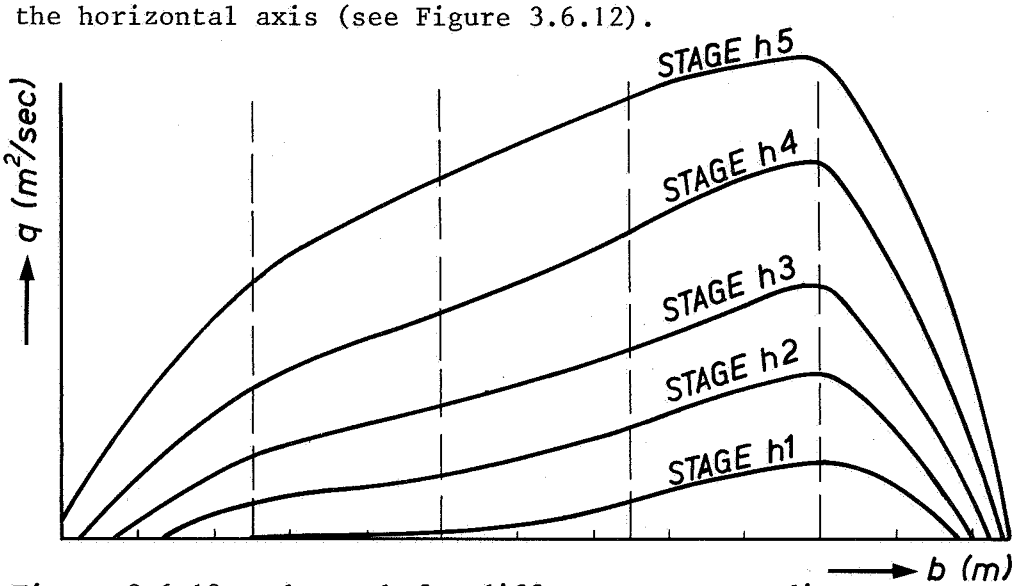


Figure 3.6.12 q-b graph for different gauge readings

The total discharge for a particular gauge reading (water-level) can then be obtained by planimetring the area contained by the curve and the horizontal axis and a Q-h curve can be constructed (see Figure 3.6.13).
The above applies for the falling stage as well and it might be discovered that the Q-h graph shows a hysteresis (see Figure 3.6.14)

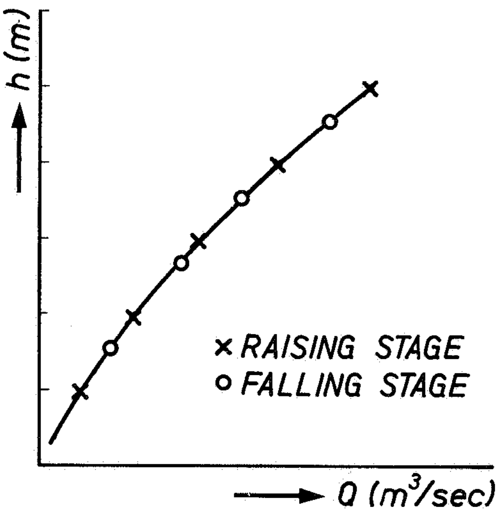


Figure 3.6.13 Q-h curve

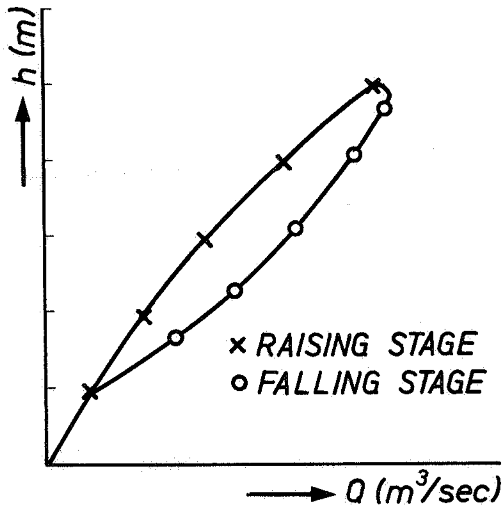


Figure 3.6.14 Q-h curve

3.6.4 Calculation of discharge in tidal rivers

For reasons mentioned in Paragraph 3.5.1 measurements in tidal rivers have to be carried out simultaneously in as many verticals as is reasonably

possible with the number of vessels available.
For each observation of velocities in the verticals, the discharge for unit width can be calculated by one of the manners as described in Paragraph 3.6.2.

With the graphical method as described in Paragraph 3.6.3 the q for time of observation can be plotted and the total Q passing the cross-section at that specific moment can be derived by planimetring the area between the $\bar{v}.d$ line and the water surface.
This is, however, a rather cumbersome procedure as for every hourly observation during a full tidal cycle the method should be repeated due to the varying water-level and therefore another method can be followed where the formula of Chézy is used.

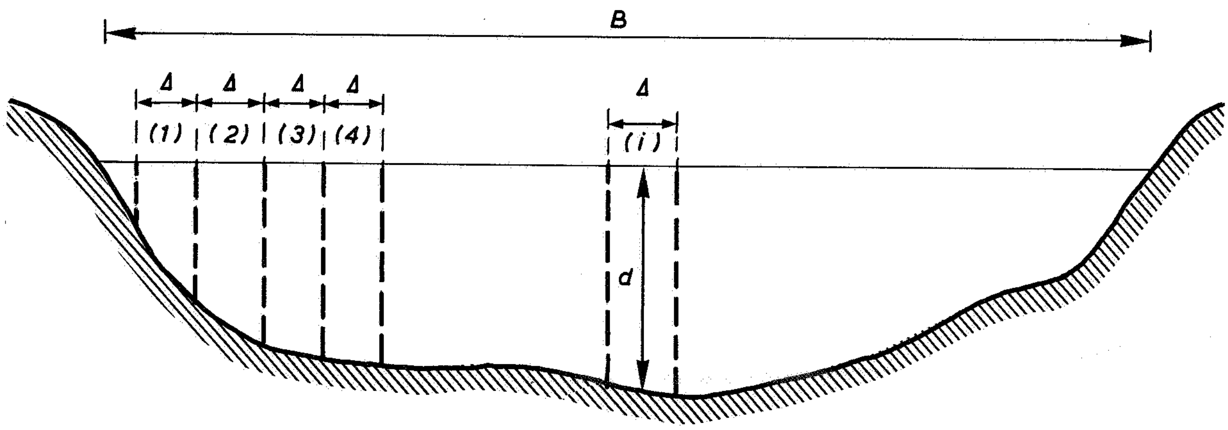


Figure 3.6.15 Cross-section

For a section with a width of Δ (m), perpendicular to the river cross-section

$$Q_p = \bar{v} * d * \Delta$$

in general

$$Q_{p_i} = \bar{v}_i * d_i * \Delta$$

Applying Chézy ($\bar{v}_i = C\sqrt{d_i I}$) $\rightarrow Q_{p_i} = C\sqrt{I} * d_i^{3/2} * \Delta$

The tidal discharge Q through the cross-section:

$$Q = Q_{p_1} + Q_{p_2} + \dots\dots\dots\dots + Q_{p_n}$$

$$= \sum_{i=1}^{n} Q_{p_i} = \sum_{i=1}^{n} C\sqrt{I} * d_i^{3/2} * \Delta = C\sqrt{I} * \Delta * \sum_{i=1}^{n} d_i^{3/2}$$

The term $\Delta * \sum_{i=1}^{n} d^{3/2}$ denotes the area between the water-level and the $d^{3/2}$-line

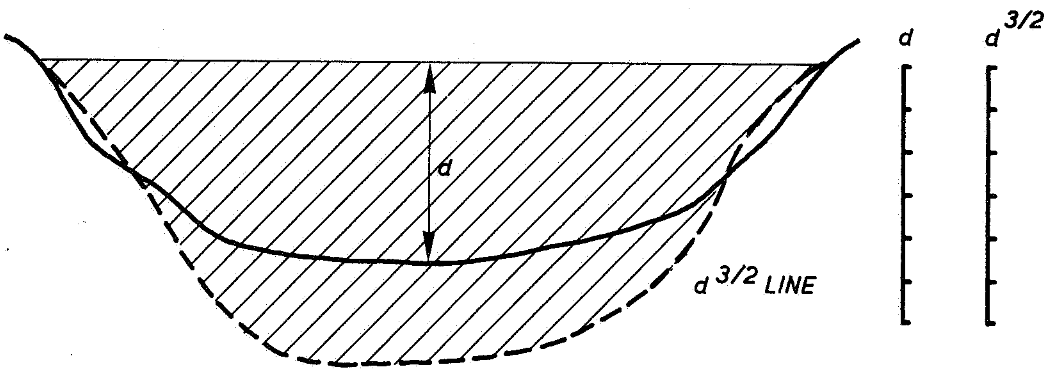


Figure 3.6.16 $d^{3/2}$-area

The cross-section is divided in as many zones as there are measured verticals, the $d^{3/2}$-curve is drawn and the area of each zone between the $d^{3/2}$-curve and the water-line is determined by planimetring. See Figure 3.6.17

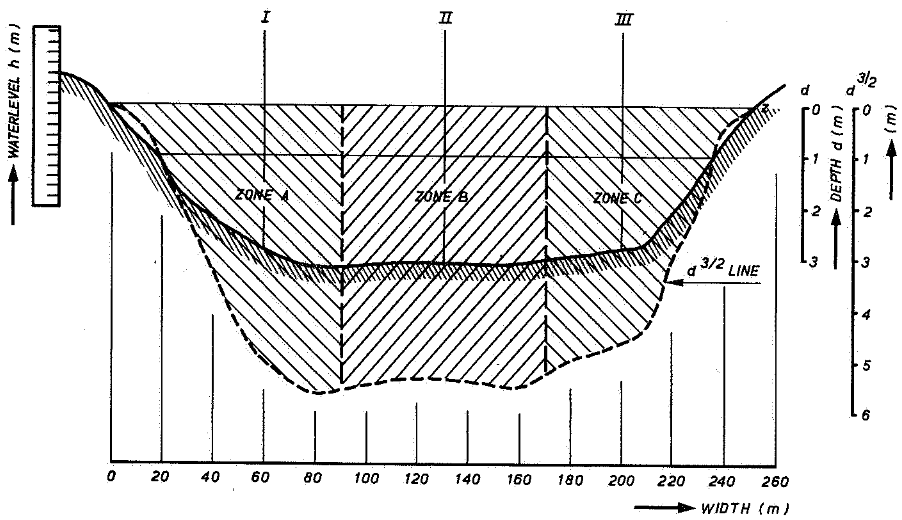


Figure 3.6.17 Cross-section divided in zones

As the water-level varies with the tidal motion the value of $d^{3/2}$ varies also (see Fig. 3.6. 18).

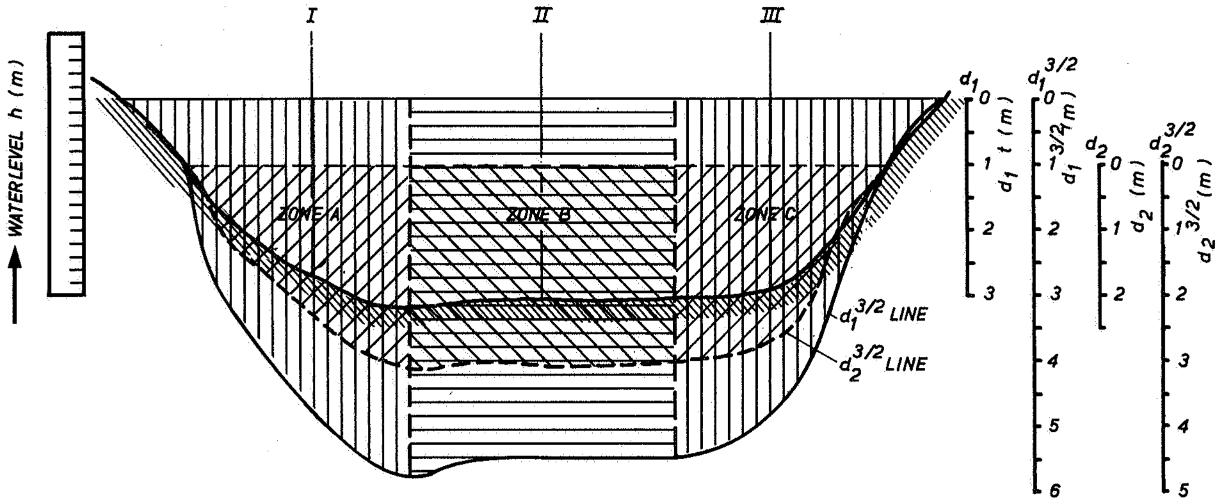


Figure 3.6.18 $d^{3/2}$-line varying water-level

Therefore to reduce the amount of work involved in determining the area between the water-line and the $d^{3/2}$-line, a sub-graph is made as illustrated in Figure 3.6.19 in which the value of $\Sigma\ d^{3/2} * \Delta$ is given as a function of the water-level (h). For (h) is taken the lowest, highest and mean waterlevel occurred during the discharge measurement.

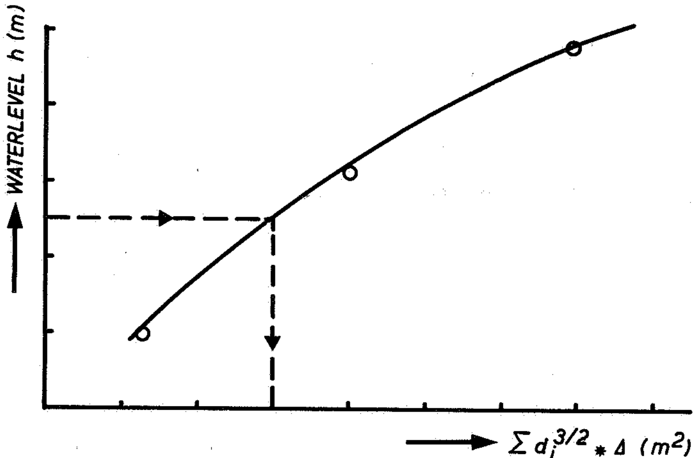


Figure 3.6.19 $\Sigma\ d^{3/2} * \Delta$ versus water-level h

In several verticals the velocity profiles are measured simultaneously. Each vertical represents a zone, for each zone a curve h versus $\Sigma\ h^{3/2} * \Delta$ is established and compiled in one graph, see Figure 3.6.20

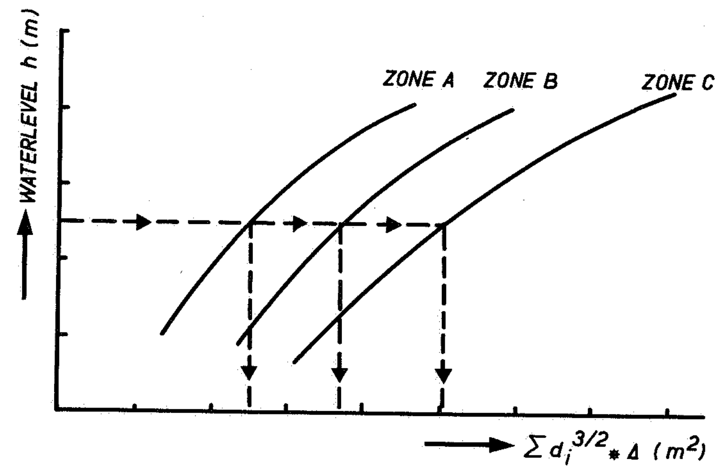


Figure 3.6.20 Compiled curves $\Sigma\ d^{3/2} * \Delta$ versus water-level

To calculate the discharge per zone at one particular time is the same as described earlier.

Per zone $Q_z = C\sqrt{I} * A$ in which

A = taken from the graph Figure 3.6.15 = $\Sigma\ d^{3/2} * \Delta$

$C\sqrt{I}$ = mean velocity in vertical divided by $\sqrt{d} = \frac{\bar{v}}{d^{\frac{1}{2}}}$.

To facilitate calculation the tabel on the next page can serve as example.

Another method of measuring the discharge with only a few vessels in a cross-section is to position one vessel at a reference vertical for the whole period of observations and let the other vessel take velocity measurements at the other verticals moving from one vertical to the other.

This method, however feasible, will put a heavy burden on the crew of the vessel as they are constantly at work during a full tidal cycle, either anchoring the vessel or weighing anchor and in the mean time carrying out the velocity observation.

In the night this operation is hampered by darkness, and although the various verticals can be marked by buoys, still findingthe proper location will take time and therefore also put an extra strain on the crew.

<u>Numerical example of calculating river discharge</u>

The following example is part of the calculation done for the S. Upang from field data acquired on 7408 27/28.

Time	Staff gauge (m)	Depth (h) (m)	Section (Ship)	Average velocity $\bar{V}$(m/sec)	$C\sqrt{I}$ ($m^{\frac{1}{2}} * sec^{-1}$)	A (m^2) ($\Sigma\ d^{3/2} * \Delta$)	Q ($m^3 * sec^{-1}$)
09.00	0,17	5,64	Kenten	0,12	0,051	1300	66
		4,64	Tawar	0,01	0,004	1070	4
		5,44	KP 1	0,02	0,008	1390	11
			Total				81
11.00	0,88	6,35	Kenten	0,45	0,179	1550	277
		5,35	Tawar	0,39	0,169	1320	223
		6,15	KP 1	0,39	0,157	1680	264
			Total				764
18.00	2,53	8,00	Kenten	1,08	0,382	2260	863
		7,00	Tawar	0,69	0,261	1950	509
		7,80	KP 1	0,71	0,254	2390	607
			Total				1979

$Q = A * C * \sqrt{h * I}$

$\bar{V} = h^{\frac{1}{2}} * C * \sqrt{I}$; $C\sqrt{I} = \frac{\bar{V}}{h^{\frac{1}{2}}}$

But in case that this method will be applied, the discharge per unit width q or the $\bar{v}$ of each vertical measured by the moving vessel have to be plotted versus the time and a smooth curve is drawn through the plots.
The discharge per unit width or the $\bar{v}$ at the time that measurements has been carried out at the reference vertical can then be derived from the various $\bar{v}$ or q curves.
The elaboration of the total discharge can then be carried out as described earlier.

3.6.5 Discharge measurements in tributaries of a main river

Non-tidal river

In order to find the distribution of the discharge in tributaries of a main river, measurements in the main river and branches has to be taken simultaneously.
This can easily be achieved by using in each cross-section one vessel which measures the velocities in the required verticals in a rather short period of time.

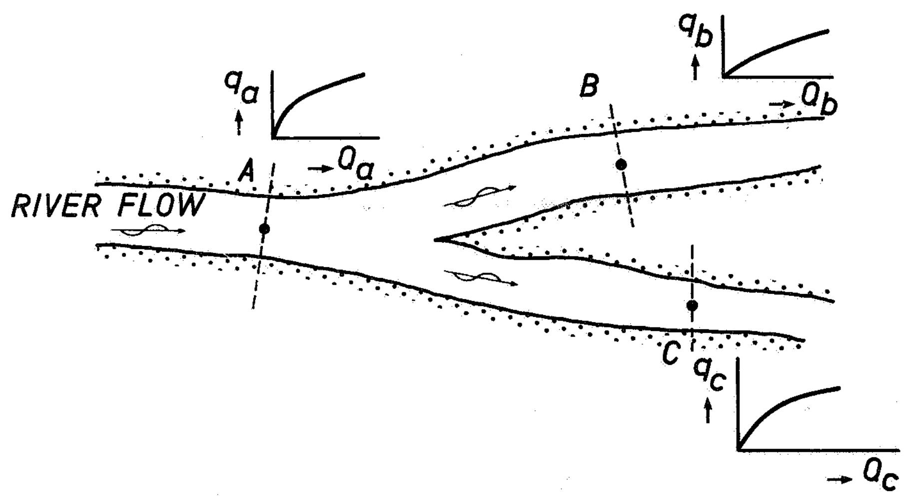


Figure 3.6.21 Reference vertical method

Tidal-river

In tidal-river, however, measurements have to be carried out during a tidal cycle, and as described in the previous chapter rather impossible to measure all verticals with one vessel.

Therefore in tidal-rivers a method can be applied of the reference vertical. Each cross-section is measured first by all available vessels during a full tidal cycle in order to calibrate the cross-section that is, to obtain the velocity distribution in the cross-section for every phase of the tide.
If this has been accomplished, the cross-sections can be measured simultaneously, with in each cross-section one vessel in the respective reference vertical.

The reference vertical is often the deepest part of the cross-section but heavy traffic of ocean-going vessels may make it impossible for the sake of safety and undisturbed measurements to comply with this and another reference vertical has to be chosen.
With the aid of the established relations, the total discharge of the main river and the distribution in the tributaries can then be calculated.

3.6.6 Calculation of upland-flow in tidal rivers

The upland-flow is the resulting discharge of the river in the directions of the sea. So it is the run-off of the basin of the river.
It is essential, however, to start and finish the measurements at the same water-level, thus not only during slack water, in order to prevent wrong figures by storage which can occur during previous high tides and which is discharged in the period of observations. Another disturbing factor could be wind set-up at sea which can give a higher ebb volume with an equal Q-upland. Thus provided that the above mentioned discrepancies are reckoned with, the upland flow may be computed in the following way:

Semi-diurnal tide

For a semi-diurnal tide with a period of approximately 13 hours the principle of computation is given in Figure 3.6.22.

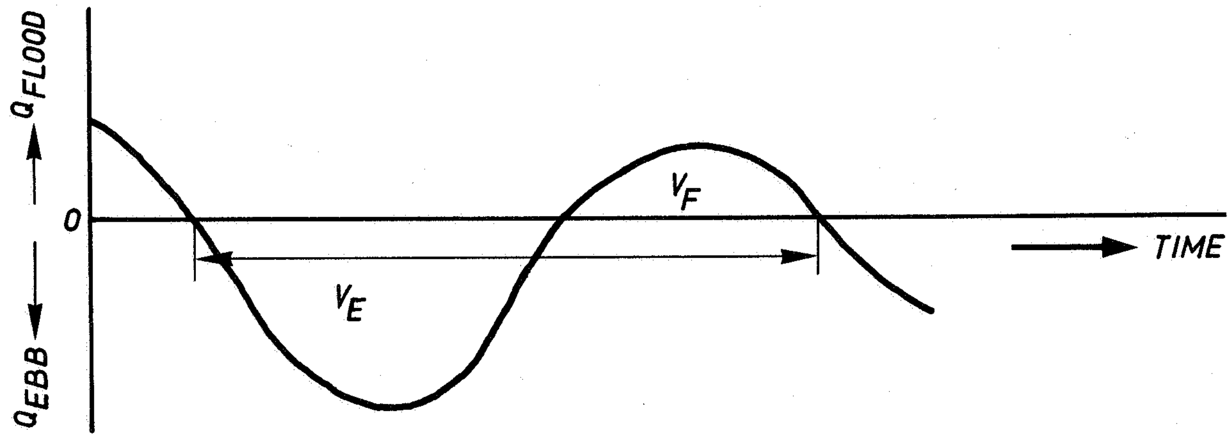


Figure 3.6.22 Semi-diurnal tide and upland discharge

Diurnal tide

For a diurnal tide with a period of approximately 24 hours the principle of computation is given in Figure 3.6.23

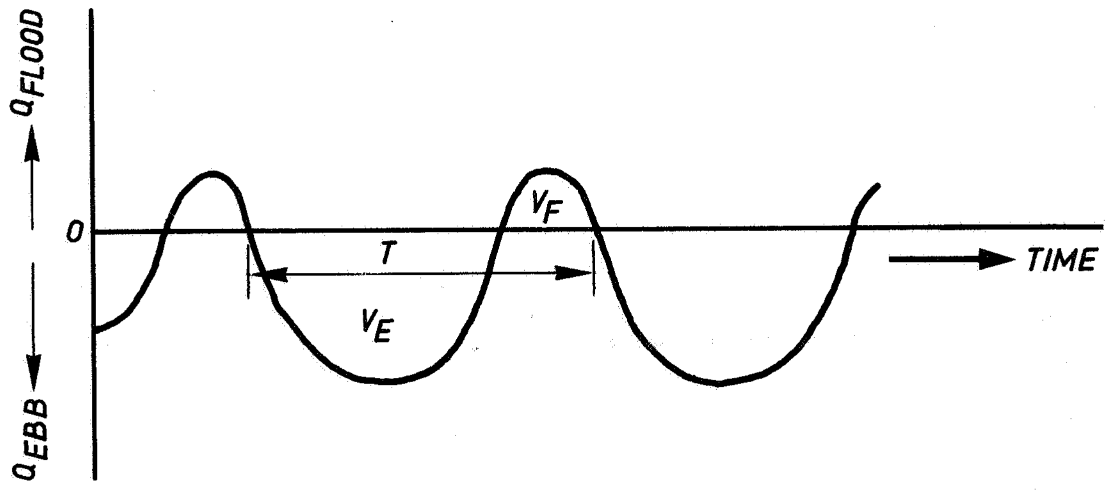


Figure 3.6.23 Diurnal tide and upland discharge

The upland discharge can now be computed as the residual volume in the cycle of approximately 25 hours.

$$Q_{upland} = \frac{V_E - V_F}{T} \quad \frac{\text{in } m^3}{\text{in sec}} \qquad T = \text{approximately 25 hours (in seconds)}$$

Mixed tide

For a diurnal tide it may occur during neap, that the semi-diurnal constituents will dominate, resulting in a semi-diurnal character with large daily inequality in water-level.

In such a case, discharge measurements should at least cover 25 hours.

The computation of the upland discharge can now be carried out as given in Figure 3.6.24

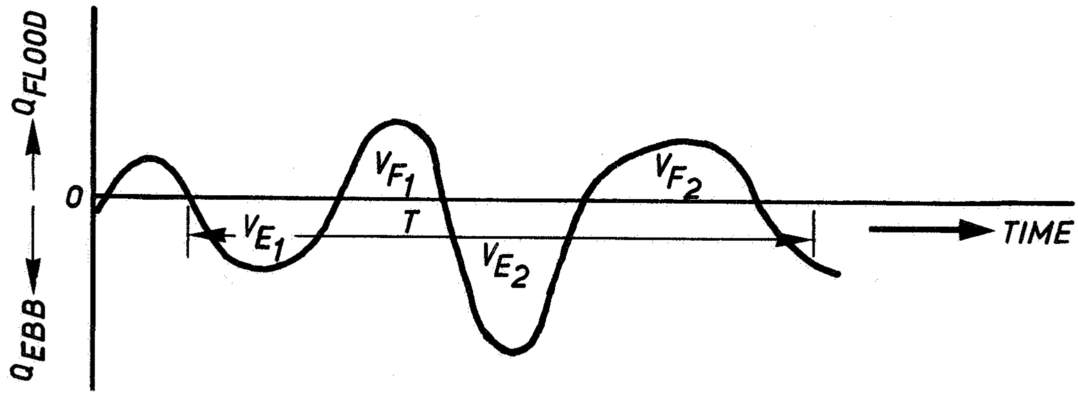


Figure 3.6.24 Diurnal tide with semi-diurnal character

$$Q_{upland} = \frac{(V_{E_1} + V_{E_2}) - (V_{F_1} + V_{F_2}) \text{ in } m^3}{T \text{ in sec}}$$

T = approximately 24 hours (in seconds)

3.6.7 Special measuring procedures

3.6.7.1 Float measurement

As an illustration of the theory given in Paragraph 3.5 the following numerical example is given:

For a river Aqua with a width of 102 m at the location of the cross-section that will be sampled for determination of the river discharge, the observations and calculations are given in Figure 3.6. 25.

Comments to this Figure

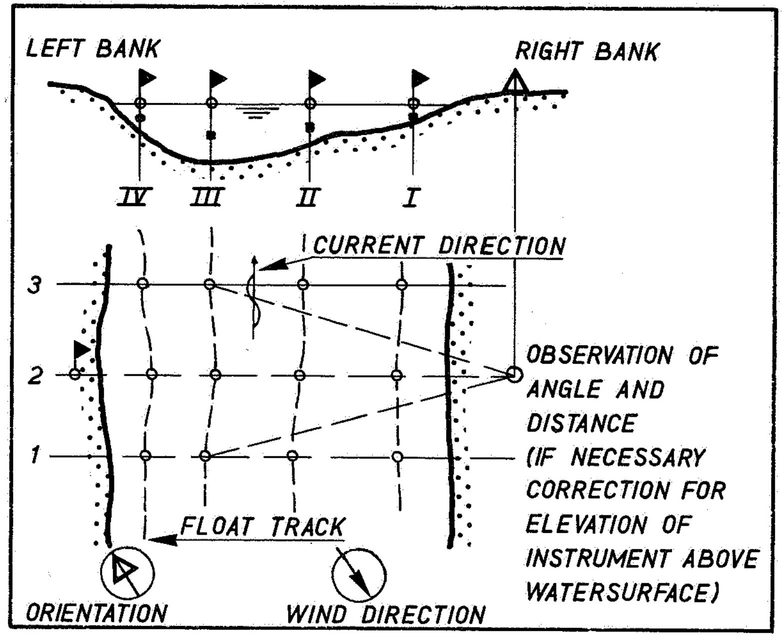
1 4 float tracks are observed
2 observed
3 observed (dimension: degrees and minutes)
4 distance, observed
5 time = 0, start of measurement, using stop-watch
6 observed
7 observed
8 observed
9 (6) - (3)
10 float track in time interval (11) = (4) sin (9)
11 (8) - (5)
12 (10) : (11)
13 observed
14 observed
15 observed
16 (13) - (6)
17 float track in time interval (18) = (14) sin (16)
18 (15) - (8)
19 (17) : (18)
20 observed
21 ((12) + (19)) : 2
22 (20) * (21)
23 remarks (ship traffic, repeated etc.).

It is advisable to plot the data for a check. If the stream vectors are not perpendicular to the cross-section only the component perpendicular to the cross-section should be taken into account for determining the river discharge. For this reason it is more advisable to calculate (10) and (17) as (4) sin (9) and (14) sin (16) than as (7) tg (9) and (7) tg (16).
Figure 3.6.26 gives the plots, the observations and then evaluation of the discharge according to the graphical method.

FLOAT MEASUREMENT

DATE : 730914
RIVER/LOCATION : AQUA/NEAR BRIDGE TOWN
WIND VELOCITY : 0.4(m/s)
WIND DIRECTION : 0°
GAUGE READING : 3.01(m)+ DATUM
TYPE OF FLOAT : FLOAT + RES.CROSS AT 0.6d.
CROSS-SECTION : DISTANCE FROM RIGHT BANK(m) / DEPTH(m)

DISTANCE FROM RIGHT BANK (m)	0	10	20	30	40	50	60	70
DEPTH (m)	0	5.0	6.5	8.0	9.1	11.9	13.9	14.2
DISTANCE FROM RIGHT BANK (m)	80	90	100	102				
DEPTH (m)	12.3	9.0	1.2	0				



1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23
FLOAT TRACK	OBS. TIME	CROSS-SECTION 1			CROSS-SECTION 2							CROSS-SECTION 3							d	$\bar{v}$	q	Remarks
		BEARING	DIST	TIME	BEARING	DIST	TIME	ϕ	s	Δt	v	BEARING	DIST	TIME	ϕ	s	Δt	v				
NUMBER	hr	°/,	m	sec	°/,	m	sec	°/,	m	sec	m sec⁻¹	°/,	m	sec	°/,	m	sec	m.sec⁻¹	m	m.sec⁻¹	m² sec⁻¹	
I	15.30	324/50	34.1	0	358/20	32.6	363	33/30	18.7	363	0.05	34/60	32.9	817	35/58	21.4	454	0.05	6.0	0.05	0.30	
II	15.50	338/10	57.7	0	0/40	52.1	228	22/30	21.6	228	0.09	20/100	59.0	428	19/20	19.4	200	0.10	8.8	0.10	0.88	
III	16.10	344/00	81.0	0	359/00	78.0	158	14/59	21.0	158	0.13	14/56	79.0	338	15/36	21.4	180	0.12	14.2	0.12	1.70	
IV	16.30	349/08	106.1	0	0/50	103.2	362	11/42	21.2	362	0.06	10/54	106	672	10/04	19.0	310	0.06	9.2	0.06	0.55	
																			$Q_{tot.}$		75.9	m³/sec.

Figure 3.6.25 Evaluation of float measurement and Table

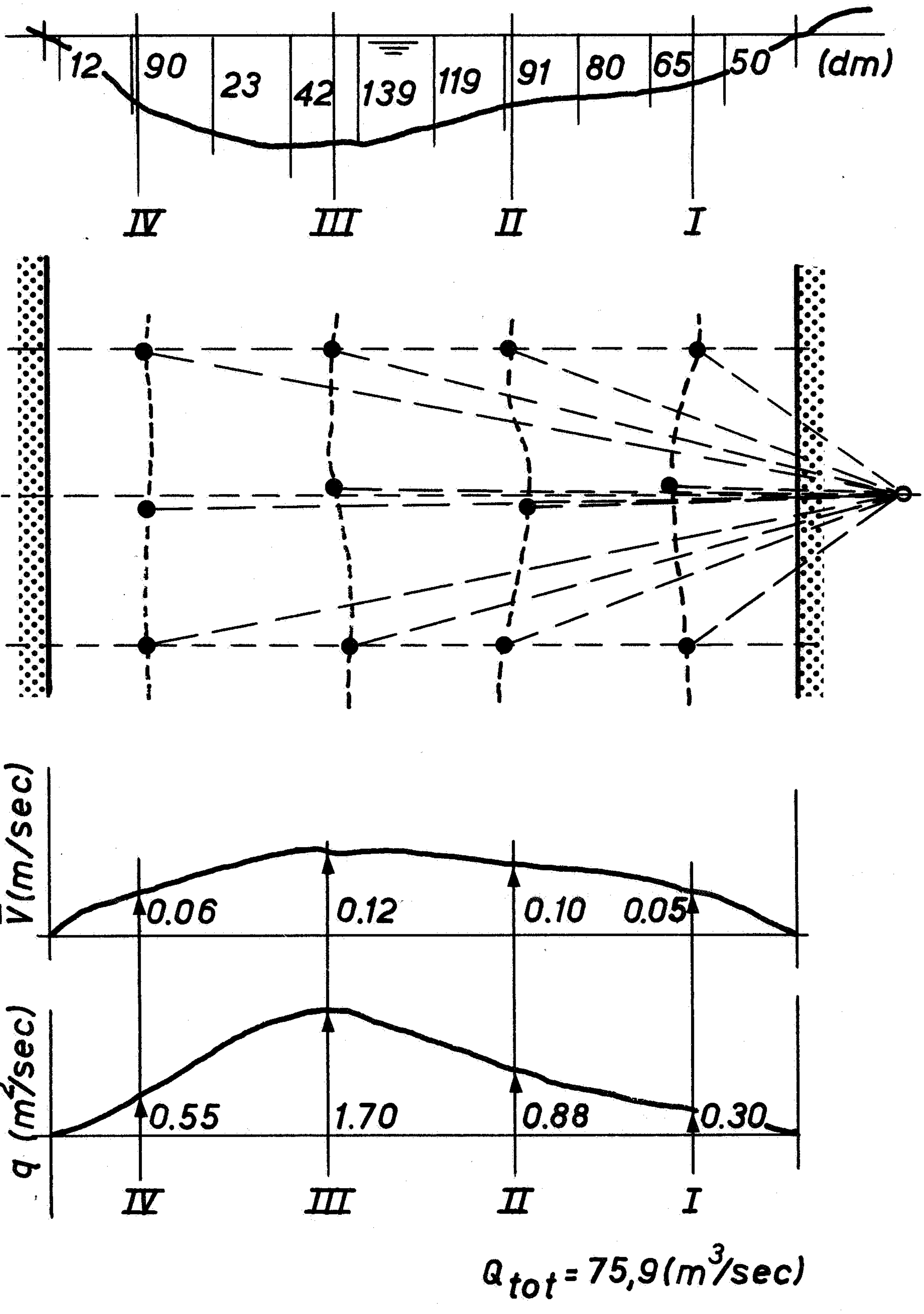


Figure 3.6.26 Evaluation graphical method

3.6.7.2 Pendulum instrument

In this paragraph a part of the standard form for a planeta measurement is given and elaborated.

1	DATE :					
	RIVER :					
	LOCATION CROSS-SECTION:					
	LOCATION VERTICAL :					
	OBSERVER :					
	SURVEY VESSEL :					
		dimension				
2	time	(hr)	9.00			
3	type of resistance body		B			
4	cable length (Σ Δ1)	(m)	0.5	1.0	2.0	3.0
5	compass bearing	(^{o})	300	300	290	290
6	horizontal angle	(^{o})	20	20	20	25
7	error (hor. + dev.)	(^{o})				
8	azimuth current	(^{o})	140	140	130	135
9	vertical angle (ϕ_{read})	(^{o})	19	19	17	19
10	α Δ1	(^{o})	-	0.2	0.4	0.3
11	Σ α Δ1	(^{o})	-	0.2	0.6	0.9
12	reduction on Σ α Δ1	(^{o})	-	0	0	0.2
13	$\phi_{bending}$	(^{o})	-	0.2	0.6	0.7
14	ϕ_{body}	(^{o})	19	18.8	16.4	18.3
15	velocity (V)	(m/s)	0.68	0.67	0.63	0.66
16	reduction on Σ Δ1	(%)	5	5	4	5
17	do	(m)	0.03	0.05	0.08	0.15
18	depth of body	(m)	0.47	0.95	1.92	2.85
19	water-depth	(m)				

In the following table comments are given for the elaboration.

1	data for storing the results (date, place, etc.)
2	observed
3	observed
4	observed
5	observed
6	observed
7	compass deviation table
8	observed
9	observed
10	from graph 1; preceding velocity V is used to obtain α from the graph (Figure 3.5.13)
11	previous $\Sigma\ \alpha\ \Delta 1$ + (10)
12	graph 2; figure between the curved lines (Figure 3.5.14)
13	(11) - (12)
14	(9) - (13)
15	graph 1 (Figure 3.5.13)
16	graph 2; figure between straight lines (Figure 3.5.14)
17	(4) $\times$ (16)
18	(4) - (17)
19	echosounder or hand-lead

It is possible that at a certain depth it will be required to change the resistance body because of the decreasing current velocity with the increasing depth ($\phi_{read} < 5^o$). If this is the case, care should be taken that for the rest of the profile $\Sigma\ \alpha\ \Delta 1$ should be converted for the new type of resistance body (conversion tables are delivered with the bodies).

3.6.7.3 Tracer methods

The principle of the dilution or tracer method is that the velocity and so the discharge, can be determined from the concentration of a tracer solution added to the flowing water. The tracer solution, existing of colouring matter, salt or radio-active material, is injected into the current either at a constant rate or with a sudden injection Figure 3.6.27.
This method is particularly suitable in rivers where no cross-sectional pro-

file can be determined.

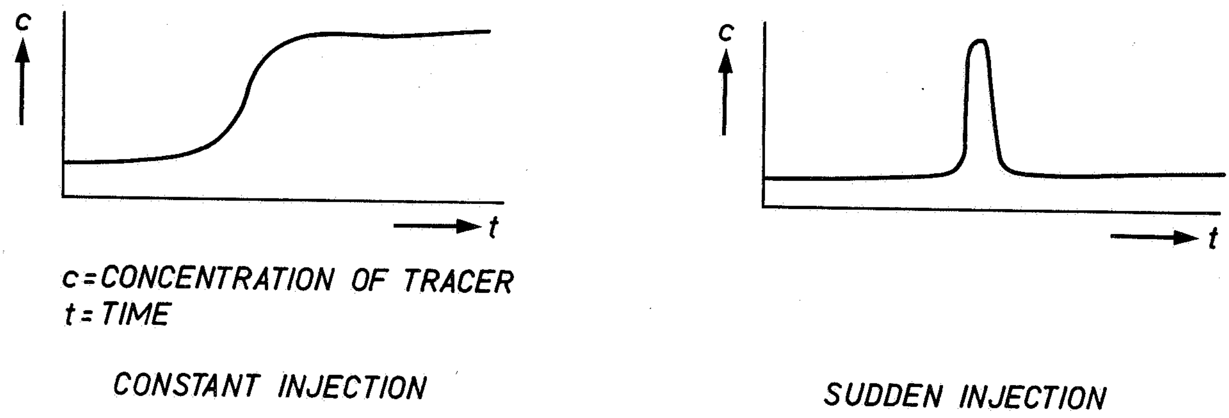


Figure 3.6.27 Injection methods

At a location downstream from the injection point, where a complete mixing of the tracer solution throughout the whole cross-section has occurred the concentration of the tracer is measured.

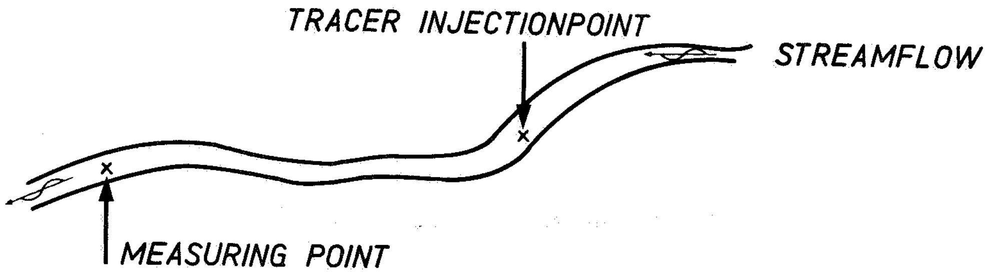


Figure 3.6.28 Tracer method

The accuracy of this method depends upon the complete mixing of the tracer solution in the water and upon whether or not tracers become tied to suspended or bottom material of the river.
The discharge should not change in the measuring reach. The reach has to be free of tributaries or bank overflow.
When dye is used as tracer material the dilution can be measured with a fluori-meter. When salt is used the dilution can be detected with an electrical conductivity-meter.
In case of radio-active tracer material scintillator counters are used.

Radio-active tracers turn-out to be quite troublesome because it may be dangerous to public health. Also, there is a change that the river water contains some radio-activity due to natural causes. Therefore the radio-activity should also be measured before the test is carried out.
These so-called background measurements have to be carried out too in case of using dye or salt as tracers.

To gather quantative data from the tracer method, the tracer has to fulfill a number of special requirements.

a. Easy to dissolve.
b. No or very small amounts of material with the same characteristics upstream from the injection point.
c. No absorption of the tracer by suspended - or bottom sediment.
d. Easy to measure dilution.
e. No environmental hazards.
f. Cheap because of large quantities involved.

In case of the constant rate method a constant amount - but relatively small compared with the river discharge - of tracer is added continuously to the river. To maintain a constant rate either pressurized tanks or a relatively simple Mariotte-bottle are used. The principle of the latter is explained in Figure 3.6.29.

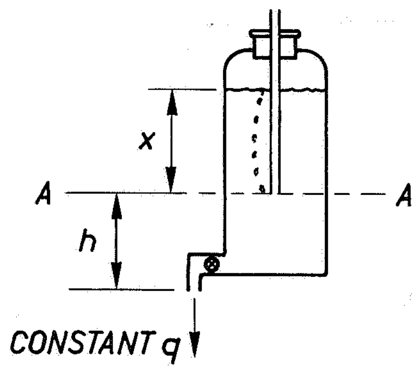


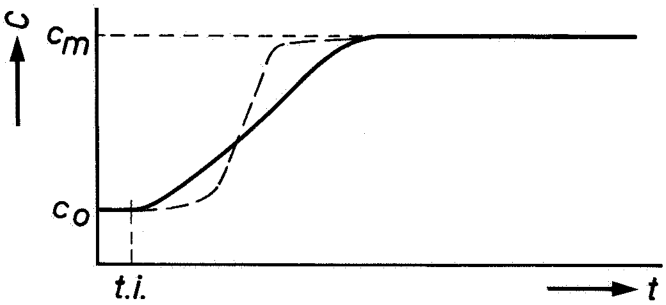
Figure 3.6.29 Mariotte-bottle

The bottle is closed but air can enter through a vent pipe. The pressure at level A is constant and equal to the atmospheric pressure, no matter what distance X may be. The distance h determines the (constant) outflow.

The dilution of the tracer is determined with the help of titration technique or an electric conductivity-meter, fluori-meter, scintillator with counter, depending on the kind of tracer applied.

To reduce the mixing length in the river, a pipe with a large number of holes can be used to speed-up the diffusion of the tracer solution in the cross-section of the river.

As is mentioned before there are two techniques to determine the discharge of a river with a tracer method namely the constant rate and the sudden injection method.
Figure 3.6.30 shows a graph in which the concentration of tracer material is plotted against the time in case of constant rate injection.



c_o = natural concentration in river water $[g/m^3]$
c_m = concentration after injection $[g/m^3]$

Figure 3.6.30 Constant rate injection

The calculation of the river discharge (Q) between the injection and measuring station is done with a so-called tracer balance:
tracer in = tracer out + increments in tracer storage
tracer in = $Q * c_o + q * c$
tracer out = $(Q + q)\ c_m$
in which Q = river discharge $[m^3/sec]$
q = discharge tracer solution $[m^3/sec]$
c = concentration tracer sol. $[g/m^3]$

after c_m has reached a constant value, the increment in tracer storage is equal to zero.
So:

$$Q * c_o + q * c = (Q + q)\, c_m$$

$$Q = \frac{c - c_m}{c_m - c_o} * q$$

$\frac{c - c_m}{c_m - c_o}$ is called the dilution factor

The Figure 3.6.31 shows the c versus t graph in case of the sudden injection.

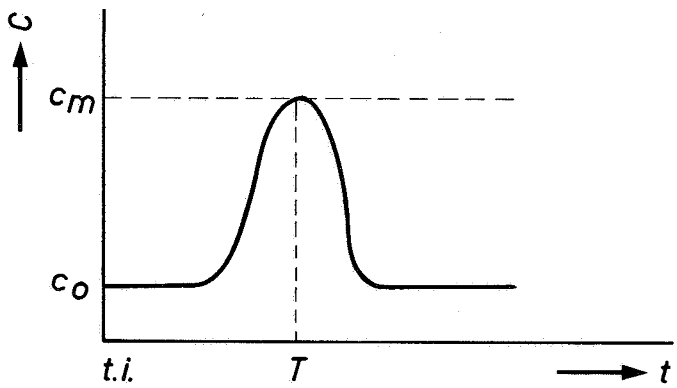


Figure 3.6.31 Sudden injection

When the total amount of injected tracer [M] has passed the measuring station then:

$$M = \int_o^\infty Q(c_m - c_o)\, dt = Q \int_o^\infty (c_m - c_o)\, dt$$

$$Q = \frac{M}{\int_o^\infty (c_m - c_o)\, dt}$$

This method involves a lot of work because the integration might have to cover an extensive time period.

A third method to compute the discharge of a river with tracer material is the multiplication of the mean water velocity with the area of the cross-section (A). The mean water velocity is determined by the time lapse between the moment of injection and the "centre of gravity" (T) of the c-t graph (Figure 3.6.31).

When the distance between injection station and measuring station is S the mean velocity is $\frac{S}{T}$ [m/sec] and the total discharge $Q = A * \frac{S}{T}$.

As was mentioned earlier the accuracy of the tracer method depends on whether or not complete mixing occurs such that the concentration of injected tracer over the whole cross-section at the measuring station is constant.

There are two methods to determine the minimum mixing length:
1. calculation using channel and flow parameters
2. dispersion observations of preliminary injections.

The former method is most suitable for small streams, the latter for larger rivers. One formula for small streams according to Rimmar is:

$$L = 0.13\, b^2\, C(0.7\, C + 2\sqrt{g}) * (gd)^{-1}$$

L = mixing length	m	
b = mean stream width	m	$5 < b < 50$ m
C = Chézy coefficient	$m^{\frac{1}{2}}\ t^{-1}$	$15 < C < 50$
d = mean stream depth	m	
g = acceleration due to gravity.		

It is emphasized that the length obtained from this formula may only be used as a preliminary guide and the actual length required should be established by practical tests.
The formula is invalid for stream widths being less than 5 m and exceeding 50 m. When the preliminary injection method is desirable a strong colouring agent is injected during a short period and at the measuring station the whole cross-section is checked to see whether or not the concentration is constant.

3.6.7.4 The moving boat method[1)]

Frequently on larger streams and in estuaries convential methods are impractical and involve costly and tedious procedures, this is particularly true during floods when facilities may be inundated or inaccessible, at remote sites where no facility exist, or at locations where unsteady flow conditions

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1) partly coppied of Draft International Standards ISO/DIS 4369: Measurements of liquid flow - open channels - Moving boat method. UDC 532.573.

require that measurements be made as rapidly as possible.
In other cases floating obstackels or rivertraffic require a flexible kind of discharge measurement that not has to be interrupted due to this kind of unforseen happening.
The moving boat technique is applicable to rapid measurements of rivers. In tidal rivers or estuaries where density currents occur this technique with which only limited information in vertical direction is obtained, can not be applied, as the river water in the upper layer can flow seawards while in the underlayer sea water will flow river inwards.
Furthermore this phenomenon may only be existing in part of the cross-section.
The moving boat technique is based on the velocity area method of determining discharges. According to this method the total area of the cross-section is divided in sub-areas for which a representative velocity is determined.
The principal difference between a conventional measurement and the moving boat measurement is in the method of data collection. The mean velocity in the segments of a cross-section of the stream in the case of a conventional technique is determined by point velocities or an integrated mean velocity in the vertical. The moving boat technique measures the velocity over the width of a segment by suspending the current meter at a constant depth during the traverse of the boat across the stream. The measured velocity and the additional information of the depth sounding gives the required data for determining the discharge.

Three types of data are required:
a) location of observation points across the stream
b) stream depth
c) stream velocity.

During the traverse an echosounder normally records the geometry of the cross-section and a continuously operating current meter senses the combined stream and boat velocities.
A third set of data needed is obtained by measuring at intervals the angle between the current meter, which aligns itself in a direction parallel to the movement of the water past it, and the true course of the boat or by measuring the distance to a fixed point on the bank.

The velocity measurement taken at each of the observation points in the cross-

section is a vector quantity which represents the relative velocity of water past the current meter. This velocity v_r is the vector sum of v_w, the component of the time integrated stream velocity perpendicular to the boat path in the interval, and v_b, the velocity of the boat with respect to the stream bed along the selected path, as this vector sum is the relative velocity of the water past the current meter assembly.
The following vector diagram demonstrates this relationship.

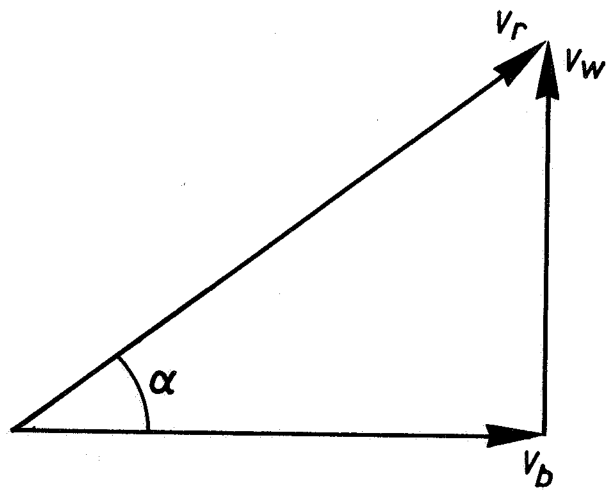


Figure 3.6.32 Diagram of velocity vectors

In the velocity-area method for determining discharges the number of measuring points in each vertical are weighed against the number of verticals. From Figures 3.5.3 and 3.5.4 can be concluded that with 3 points in each vertical and 15 verticals (sampling time 30-50 sec) a good practical optimum seems to be obtained in most situations. If more accuracy is required it is more efficient to measure a greater number of verticals and not to increase the number of sampling points in each vertical. The moving boat method can be considered as a method where many verticals are measured in one point of the depth.

Distinction can be made between various methods. In the following two methods are described as being representative for the different approaches. Combinations of the described techniques of course are possible.
In method 1 (as it is called in the following) the current meter is located at a fixed depth below the water surface and the determination of the location of the observation points in the cross-section is based on observations of the angle between the section and the orientation of the current meter.

In (the afterwards called) method 2 the current meter is freely suspended while the location of the observation points is directly measured with for example sextant readings.
In the following a more detailed description is given of the two methods, the way of measurement and the principles of the calculation procedure.

Method 1

Equipment and measurements

A specially equipped boat is used as indicated in Figure 3.6.33. A vane with an indicating mechanism is mounted on the bow. The angles (α) between the direction of the vane and the cross-section is read via a pointer mounted in line with the vane. A sighting device attached to the free swivelling dial provides a means of aligning the index point on the dial with the cross-section.

The current-meter (Ott) is mounted on the leading edge of the vane, whereas an electronic counter is used as pulse indicator. A stopwatch is used for time interval measurements. Finally an echosounder is installed for depth registration.

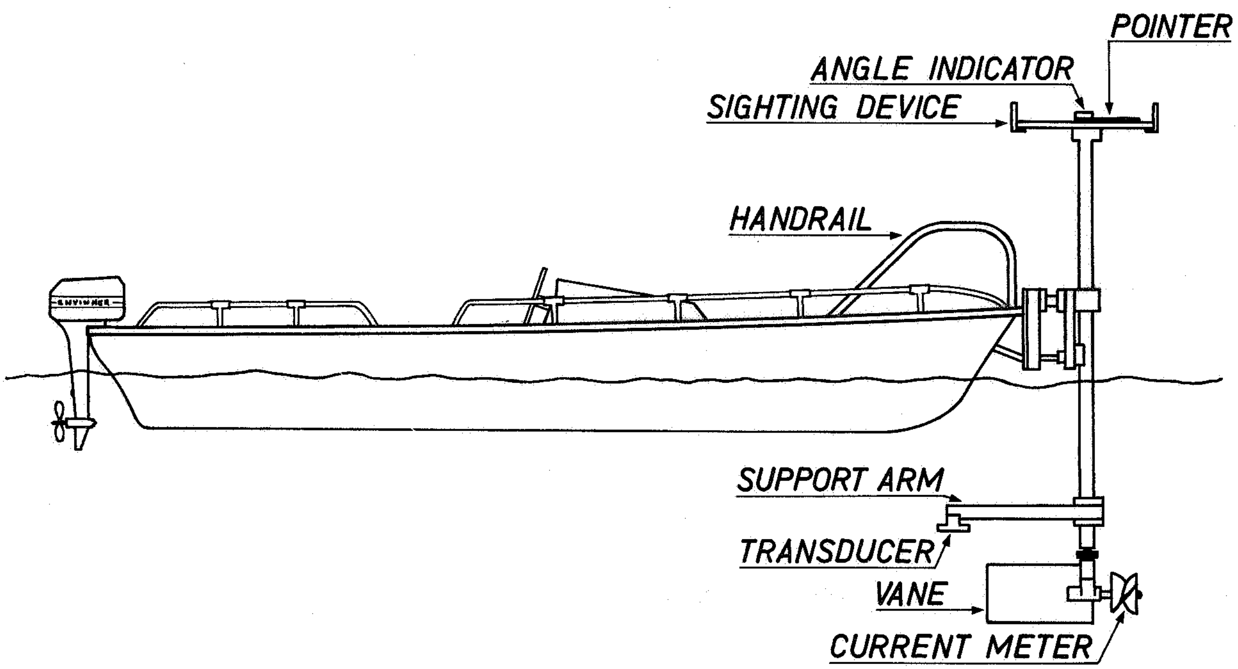


Figure 3.6.33 Possible lay-out of equipment on board of a ratamaran boat

A crew of four persons is required:

1. A boat operator.
2. An angle observer.
3. A revolution counter operator.
4. A note keeper, echosounder operator, stopwatch operator.

After selection of the site and installation of the reference marks or beacons that define the orientation of the cross-section, the width of the stream and the location of the beacons(s) near the river are determined for example by triangulation. For rivers with a width of more than about 600 m alignment references are used on both banks.

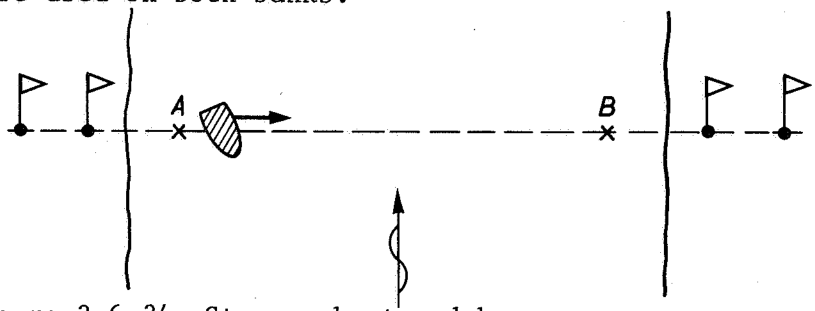


Figure 3.6.34 Stream, boat and beacon

The location of the water-line is fixed in relation to a nearby beacon. Using method 1 the exact locations of the starting and finishing position of the boat have to be determined (A and B). This can be done by rangefinder observations or sextant readings in the case a third reference point on the bank(s) is available Buoys can be located in A and B and moving towards B continuously. Starting in A the revolution counter and vane direction indicator are read and the echosounder registration is marked simultaneously. The observations can be done at regular time intervals for calculation purposes (20 or 30 sec); it is not essential for the method itself. The boat velocity is taken more or less equal to the water velocity. However, a minimum of 25 à 30 observations is required.

Computation

The boat velocity is calculated by

$$v_b = v_r * \cos\alpha$$

and the distance ΔL covered in time interval Δt with:

$$\Delta L = v_b * \Delta t$$

The total distance L between A and B (Figure 3.6.35) is calculated as the sum of the distances ΔL

$$L = \sum^{n} v_b \; \Delta t$$

with n = number of intervals.
Knowing the real distance between A and B (L_r) a correction factor k_1 can be determined:

$$k_1 = \frac{L_r}{L}$$

The water velocity is calculated according to

$$v_w = v_r * \sin\alpha$$

And the partial areas and discharges as:

$$\Delta A = \Delta L * d$$

$$\Delta Q = \Delta A * v_w$$

where d = average depth between two observation points.

The total area and discharge between A and B (n intervals) is calculated according to

$$A_{AB} = \sum_{n} \Delta A$$

$$Q_{AB} = \sum_{n} \Delta Q$$

The corrections for the areas and partial discharges between the points A and B and the banks are made assuming a straight bottom slope between the water line and the depth in A or B and using a parabolic velocity distribution. Taking into account these corrections the total area and discharge A_{TOT} and Q_{TOT} are calculated.

Two more adjustments are made:

a) Adjustment for the mean velocity in the vertical (k_v).
It is assumed that in larger streams the relation between the velocity on the depth of the current-meter and the mean velocity in the vertical can be considered as fairly constant. In order to arrive at a representative average value of k_v several vertical velocity distributions have to be determined in strategically placed verticals. k_v is calculated as a weighed average with weights in proportion to the discharge in the segments. As such a coefficient depends on the average water depth these kind of measurements have to be repeated at several stages in a non-tidal river and at various moments in a tidal river or estuary. It is, however, not necessary that k_v is determined for each different discharge measurement.
k_v normally ranges between 85 and 92, and depending on the required accuracy of the measurement it seems that a good estimation is obtained using k_v = 90.

b) Width adjustment.
The correction factor k_1 is used to correct both the calculated area (A_{TOT}) and discharge (Q_{TOT}).

Summarizing the real area (A_r) and discharge Q_r are calculated with the following formulae:

$$A_r = k_1 \times A_{TOT}$$

$$Q_r = k_v \times k_1 \times Q_{TOT}$$

Method 2

Equipment and measurements

No special equipment with reference to the conventional techniques are required in the most basic execution form of method 2. The current-meter is freely suspended in the same way as for a conventional discharge measurement and distances are determined by rangefinder or sextant. However, it is recommended

to use an echosounder for simultaneous depth registration and special counting device for revolutions and time interval observations; the objective of the special counter is to "freeze" for a moment the observations that have to be copied. This can be done by two counters that operate alternatively or one counter with a special memory system.
Due to computational difficulties in cases that observation errors are made a one counter system with a memory is highly recommendable.

A crew of four persons is required:
1. A boat operator.
2. A sextant or rangefinder observer.
3. A revolution, time interval observer.
4. A note keeper, echosounder operator.

Field practice showed that for experienced groups, dependent, however, on the facilities offered by the counting device, a crew of three persons is sufficient.

As far as the site selection is concerned reference is made to the description given above for method 1. The reference points on one bank consist of 3 beacons, two for alignment of the launch and two as a base for the sextant readings. For rivers over about 600 m wide these reference marks are installed on both banks.

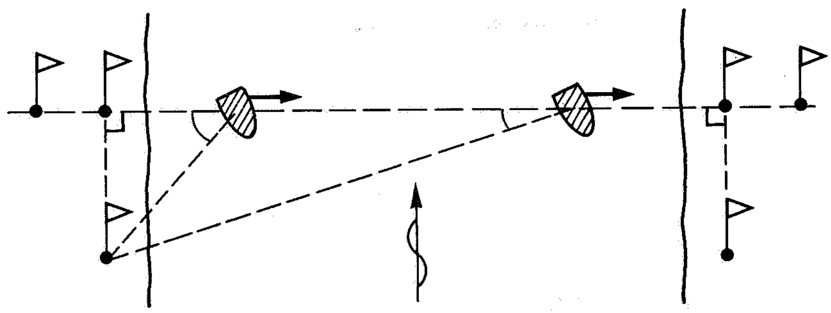


Figure 3.6.35 Stream, boat and beacons for sextant readings

No special measurements as described in method 1 are required to locate points A and B.

Computation

The computational procedure is slightly different from method 1.
ΔL results directly from the location determined by the sextant readings and v_b is calculated with

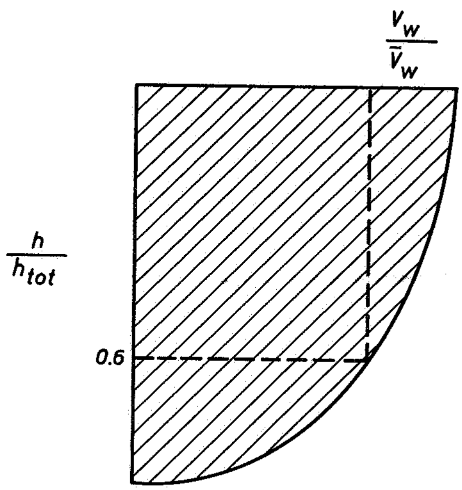
$$v_b = \frac{\Delta L}{\Delta t}$$

where ΔL is the distance between the observation points and t the time interval between two readings.
The water velocity is determined by

$$v_w = \sqrt{v_r^2 - v_b^2}$$

where v_r is the measured relative water velocity and v_w the water velocity.

The mean velocity adjustment is made directly afterwards the computation of v_w (and before calculation of the partial discharges) by way of a unitary velocity distribution in the vertical, relating the relative depth with the relative velocity.



where $\bar{v}_w$ = mean water velocity in the vertical
h = depth of the current-meter
$\bar{h}_{tot}$ = total depth.

Figure 3.6.36 Vertical velocity distribution

The curve is presented in discrete values every 10% of the water depth. Interpolations between two points on the curve are made linearly. It has to be mentioned that before using the curve it is checked if the total area 0 equals 1, if not the values $v_w/\bar{v}$ are corrected proportionally.

As far as determination of a representative vertical velocity distribution is concerned, reference is made to the observations with respect to the determination of k_v in method 1. Depending on the required accuracy of the discharge measurements the velocity distribution can be determined for the measurement itself, for the cross-section in general, for the river or a certain river stretch etc..
As an example some velocity distributions ($v/\bar{v}_s = f(h/h_{tot})$) are given with their respective standard deviations (S). These values are measured in the lower Magdalena River in Colombia (non-tidal). The presented average value for the river are composed with 120 observations in several cross-sections and during all stages of the year. In order to show the possible variation two extreme distributions for separate cross-sections are given. These examples clearly show that a variation in $v/\bar{v}$ of 10% is no exeption.

h/h_{tot}	.1	.2	.3	.4	.5	.6	.7	.8	.9
	Average values for the Lower Magdalena River in Colombia (120 observations)								
$v/\bar{v}$	1.20	1.18	1.15	1.11	1.07	1.03	.96	.89	.78
S	.09	.11	.07	.07	.07	.08	.07	.09	.10
	Average value for Tenerife cross-section in Lower Magdalena River in Colombia (20 observations)								
$v/\bar{v}$	1.11	1.10	1.09	1.06	1.06	1.05	1.01	.96	.86
S	.08	.08	.05	.11	.06	.05	.06	.07	.16
	Average value for Arneenia and Barbosa cross-sections in lower Magdalena River in Colombia (28 observations)								
$v/\bar{v}$	1.21	1.20	1.17	1.13	1.08	1.03	.94	.87	.79
S	.09	.07	.05	.06	.08	.06	.06	.08	.09

After calculation of the mean velocity in the vertical $\bar{v}_w$ the partial areas and discharges are calculated as follows

$$\Delta A = \Delta L * d$$

$$\Delta Q = \Delta A * \bar{v}_w$$

and the total area and discharge between A and B:

$$A_{A-B} = \sum_n \Delta A$$

$$Q_{A-B} = \sum_n \Delta Q$$

The corrections with respect to the unsampled areas between the points A and B and the riverbanks is done in the same way as described under method 1: a straight bottom slope is assumed between the water-line and the depth at in A or B and a parabolic velocity distribution is used. Taking into account these corrections the total area and discharge A_{TOT} and Q_{TOT} are calculated and as no width adjustment is required and the velocity adjustment is made already if the following relations hold:

$$A_r = A_{TOT}$$

$$Q_r = Q_{TOT}$$

Special attention has to be paid to the velocity adjustment in case method 2 is used to measure the velocity at various depths. In this case each crossing can be considered as representative for the whole cross-section or can be related to a representative layer. In the first case the computational procedure is as it is described above. In the second case the velocity correction is made as follows:

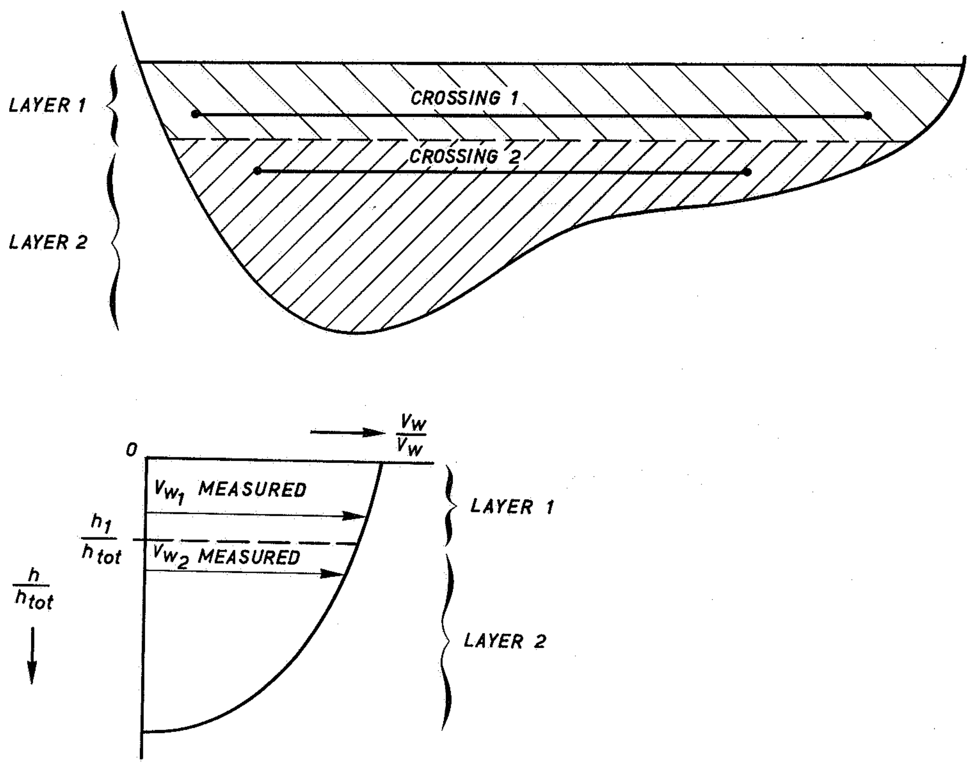


Figure 3.6.37 Moving boat measurements at two different depths

The mean velocity in layer 1 ($\bar{v}_{w1}$) is defined according to

$$\bar{v}_{w1} = \frac{1}{h_1} \int_o^{h_1} v_w \, dh$$

$$= \frac{\bar{v}_{w2}}{h_1} \int_o^{h_1} \frac{v_w}{\bar{v}_w} \, dh$$

In this relation $\bar{v}_w$ can be estimated with the unitary velocity distribution as was explained before. The estimation of $\bar{v}_w'$ in the computation of layer 1 is based on v_{w1} and for layer 2 on v_{w2}.
The values of

$$F = \int_o^{h_1} \frac{v_w}{\bar{v}_w} \, dh$$

are a function of the depth and the unitary velocity distribution. These values can be determined once the velocity distribution is known. The values of F can be calculated for discrete values of h_1 (or h_1/h_{tot}). Intermediate values are obtained via linear interpolation:

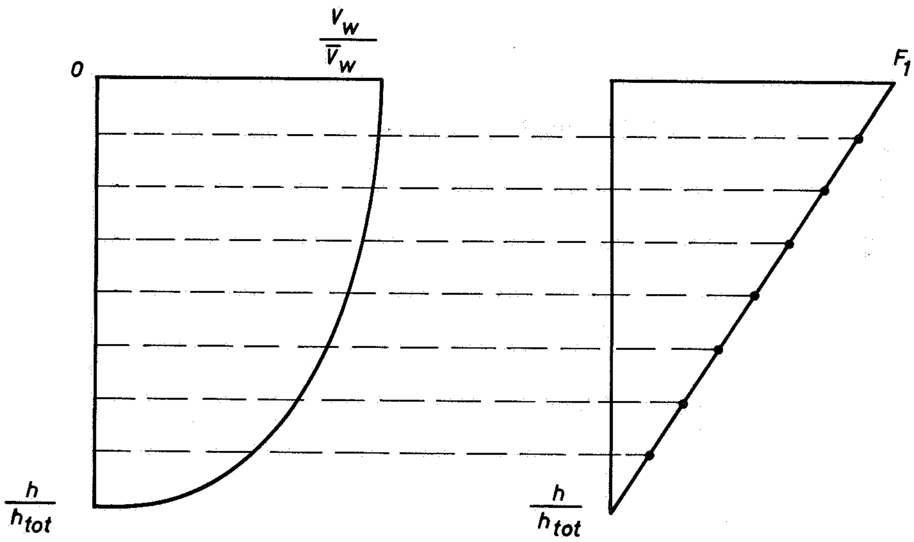


Figure 3.6.38 Determination of the partial areas of the unitary velocity distribution as a function of the depth

Summarizing the main advantages and disadvantages of the moving boat method can be listed as follows:

Advantages

- Flexible with respect to floating obstacles etc.
- Effective as far as the sampling procedure concerns. During a measurement of say 15 minutes, almost all this time is used for measuring; during a conventional discharge measurement 15 min sampling time of water velocities may require much more overall time, due to hoisting and lowering of the current meter and position-changing of the measuring vessel.

Disadvantages

- A more expert and experienced crew is required than for a conventional measurement.
- A special equipment is necessary, especially for method 1.
- Computational procedure is too difficult and too long to carry out during the survey.
- Only applicable in situation where the vertical velocity distribution is more or less normal.

3.7 Sediment transport

3.7.1 General

The bottom of alluvial water courses, like rivers and estuaries, is composed of granular material. These sediments can be transported by flowing water if the flow velocities are sufficiently high. Alterations in the flow velocities, either caused by nature (wet and dry seasons) or by human interference in the natural conditions (construction of structures in the river) will influence the magnitude of the sediment transport. The effect of this changeable sediment transport is erosion of the river-bed at some places and sedimentation at other locations.
In order to facilitate the design of civil engineering works in rivers and estuaries and to predict the morphological changes resulting from these works, many formulae have been developed, based on the results of experiments in nature and in hydraulic laboratories.
However, computed sediment transports are rather inaccurate. The reason for this is:

1. The interaction between the water movement and the sediment transport is very complex and therefore difficult to describe mathematically
2. Sediment transport measurements are inaccurate, hence sediment transport formulae can not be properly checked.

Measurements of sediment transport are executed in order to select a suitable formula and to adjust this formula in such a way that the sediment transports in the specific river or estuary can be estimated as accurately as possible.

The measurement of the sediment transport is, like the water velocity measurement, taking a sample out of the population of the whole transport mechanism. Because of the much more complex behaviour of the sediment, the sampling of the phenomenon is more difficult.
Sediment transport has been a subject for investigation for many centuries, but only recently modern hydraulics have given a proper scientific back-ground to the experiments.
Sediment transport can be classified as follows:

classification based on source	transport of bed material	bed load	classification based on mechanism
	wash load	suspended load	

In alluvial rivers bed material is transported, depending on hydraulic conditions. This bed material can be transported as bed load or as suspended load. Besides this bed material, very fine sediments are transported (wash load). The transported amount does not depend on the hydraulic conditions but on the conditions of the catchment area.

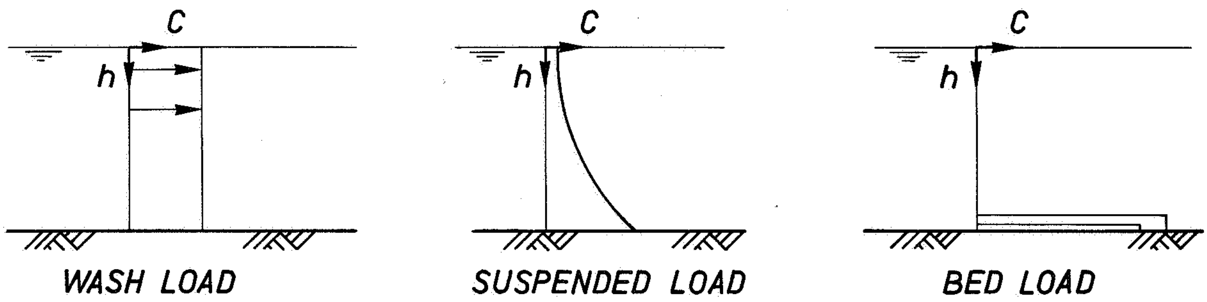


Figure 3.7.1 Different kinds of sediment transport

3.7.2 Wash-load

Wash-load is the transport of the finest particles of silt and dust that are brought into a river. They remain in suspension until the sea is reached, or get stuck in some ox-bow lake with slow flowing or stagnant water.
Wash-load has little or no influence on the river behaviour, although it may quantitatively be the largest amount of sediment transported by the river.
Its main source is the weathered top layer of the drainage basin; dried and loosened during the dry season, it is easily washed (by rain) or blown (by wind) into streams and creeks.
For that reason, the largest quantities of wash-load are found during the early part of the rainy season, during the rise of the river. It seems that almost an unlimited amount of wash-load can be carried by the river. In very large quantities the wash-load can cause a change in viscosity of the water in the river, which in that case carried almost mud only.
There is no relationship between discharge and wash-load.
The distribution of the concentration of wash-load in the vertical is uniform (see Figure 3.7.1), except variations due to turbulences.

To measure the wash-load a simple but laborious method is to take water samplers and to determine the concentration of the wash-load. The water sampler is lowered into the water to a certain depth, the cork is pulled out of the bottle and the water can pour into the bottle. The bottle is taken up and taken out of the sampler, provided with a cork and labelled and numbered. In this manner the vertical can be sampled. All the numbers and the depths of the samples are noted on a water-sampler form.

Care should be taken not to pick up the sampler too soon, but to allow for sufficient time for the bottle to be completely filled. After the measurement the bottles are taken to the laboratory to be analysed on silt content and if necessary on salinity.

The bottles have to be kept in the shadow to prevent evaporation. Labels should be fixed properly on the bottle and the numbers and other notes should be written with waterproof ink.
The identifying of each sample is very important and should at least contain the following essential information: site, date, time, section no., vertical no., bottle no., temperature, initials observer.

The data about the sediment content are expressed in units of concentration

so in mg/l or ppm $\left(= \frac{\text{weight dry sediment in kg} \times 10^6}{\text{weight water sediment mixture in kg}}\right)$.

The wash-load contains the smallest grains of the grain-size distribution of the total sediment transport. This portion is flowing through the Delft Bottle, because this instrument catches only particles > 50 (μm).
For this reason the concentration of the wash-load should be sampled with a water sampler (which may lead to errors, see Paragraph 3.7.3.5) or other types of instruments that will catch material < 50 (μm) (e.g. the US Depth-integrating sediment sampler or the US Point-integrating sampler).
In general the wash-load can be assumed to be evenly distributed over the vertical. Wash-load concentration may vary considerably over the width of the river.

Furthermore in general the grain-size distribution of the particles has to be determined.
This can be done by applying the pipette method (see Paragraph 4.2.4).

A pipette method, using a special Andreasen-Esenwein pipette, is decribed in that chapter. Also micro-sieves can be used to determine the grain-size distribution (see Paragraph 4.2.3).

3.7.3 Suspended load

Suspended load can be considered as bed material in suspension and consists mainly of fine sand grains that are nearly continuously supported by the water and only interact with the bottom material with a very low frequency, because their weight is mainly compensated by the upward directed force of the turbulence of the flow.
Over a short stretch of river, the suspended load can be assumed to remain in suspension at a constant rate and the river bed does not change due to this type of sediment transport.
Regarded over the full length of river, however, the concentration of the suspended load may vary that means that the particles can settle and other particles can be picked up from the bed, in different quantities. Some relationship exists between bed- and suspended load.
Pick up velocity is higher than settling velocity so that a weak correlation between discharge and suspended load exists. The peak in suspended load will not coincide with the peak of the hydrograph.
The distribution of the concentration in the vertical is not exponential. If a distribution curve has been adopted then it is sufficient to determine the concentration at only two depths, which reduces the number of measurements considerably. This is only theoretical and in practice especially in tidal regions, more samples should be taken in the vertical to determine the total suspended load.

To measure suspended load the following instruments can be used:
1. Delft bottle (sediment transport meter)
2. US Depth-integrating Sampler US D-49 (sediment concentration meter)
3. US Point-integrating Sampler US P-61 (sediment concentration meter).

The Delft Bottle is a transport meter as its principle is a flow-through system, the sediment containing water flows through a bottle shaped sampler and therefore a large volume of water is sampled and the sediment is a direct transport measurement.

The US-integrating samplers,however, are concentration meters as there is no flow through system but a bottle is filled with a relative small sample of the sediment containing water. The volume of water sediment mixture is fixed to the size of the bottle.

3.7.3.1 Delft Bottle (DF_{12})

The instrument can be used in two ways:

A. Suspended on a wire for all depths from surface till 0.5 m above the bottom. A tail-fin keeps the nozzle in up-stream direction.

B. Standing in a frame on the bottom, for distances of 10 - 20 - 30 - 40 - 50 cm from the bottom.
The sampling body is attached in an inclined position in order to prevent disturbance of the flow close to the bottom. This implies the use of bent nozzles in order to situate the orifice of the nozzle perpendicular to the flow direction.

For use with the sampling body there are four nozzles, two straight and two bent ones. For measuring in moderate and high velocities 1 m/sec < v < 2.6 m/sec the small one is to be used, the internal diameter is 1.55 cm, the area 1.9 cm^2. For measuring in small velocities v < 1 m/sec the larger nozzles with an internal diameter of 2.2 cm, the area 3.8 cm^2, may be used.
Measuring range in velocities up to 2.5 m/sec, grain-size exceeding 50 µ (0.05 mm) (1 µ = 1 micron = $\frac{1}{1000}$ mm).

The operational instructions for the Delft bottle are as follows:

Start measuring with the small nozzle. Although the sand catch is smaller than with the big nozzle, the accuracy is better due to the smaller correction and due to the fact that losses during hoisting up may be neglected. In most cases a measuring time of 10-15 minutes gives a sufficient catch.
If the sand catch proves to be too small with the small nozzle and doubling of the measuring time is not possible (tidal waters) the big nozzle may be applied in case of velocities smaller than 1 m/sec.
After expiration of the measuring time the DF must be hoisted up. The best hoisting up velocity is about 10-20 cm/sec. Faster hoisting up causes loss of sediment.

Launching for a measurement

- Lower the DF into the water surface.
- Stop as soon as the DF has been fully submerged; DF will incline backwards. due to air content; air will escape from nozzle and small opening on top of the rear side.
- Air venting can be accelerated by lifting the tail-piece by means of a boat hook.
- Wait for the DF to reach a horizontal position.
- Adjust wire counter to zero.
- Lower DF quickly till the depth to be measured or bottom.
- Start stopwatch.

After expiration of measuring time

- Hoist the instrument up; not faster than 20 cm/sec. As soon as the DF emerges stop the stopwatch. The sampling body will slightly incline forward, thus preventing loss of sand through openings in the lid.
- Manoeuvre rear side of DF above the funnel.
- Open the lid with tail-piece as lever.
- Empty the contents of the chamber in the funnel.
- Rinse the chambers by means of a deck wash hose.
- Remove DF and make ready for another measurement; take care that the lid is well closed.
- Let the sand catch settle in the measuring glass at the bottom of the funnel.
- If leaves or small twigs are in the sample, remove them before the caught volume is determined.
- Detach glass from funnel and read the caught volume.

The caught quantity of sediments can be measured volumetrically on the site or poured into a jerrycan, properly labeled and administrated, and sent to the laboratory for further analysis. In the laboratory the fractions smaller than 50 μ normally will be separated from the fractions larger than 50 μ by sieving through a sieve with openings of 50 μ or by settling procedure. This separation of fractions is necessary because of the fact that for smaller than 50 μ fractions the loss coefficient of the Delft Bottle has increased so much that a determination with any accuracy of the transported quantity of sediments for these small fractions is not possible. If the caught quantity of sediments is measured volumetrically on the site this separation of fractions will be achieved more or less automatically. The volume of the settled sand in the

special measuring glass at the bottom of the funnel, in which the Delft Bottle sample has been poured, is read at the moment that the division between settled sand and water is clearly visible and more or less stable.
At that moment the greatest amount of sediments with diameters smaller than 50 μ is still in suspension and hence is not included in the measured volume. Sediment transport of particles smaller than 50 μ will have to be determined by means of taking water samples. The limit of 50 μ (greater than 50 μ suspended sediment transport to be determined by Delft Bottle; smaller than 50 μ suspended transport to be determined by water samples) is rather arbitrarily and will have to be checked occasionally.

<u>3.7.3.2 Computation of sediment transport</u> (transport-meter)

For computation of the sediment transport concerning suspended load > 50 μ measured with the Delft Bottle, the following data are required:
- cross-sectional profile
- velocity distribution in the vertical (current-meter)
- sampling depth (by wire counter block)
- measuring time (stopwatch)
- average grain-size
- sediment catch (DF measuring glass in cc, sometimes this quantity caught is dried and weighed later on in a laboratory. In that case the catch is expressed in kg (or grams)).

To obtain the total depth in the vertical firstly a cross-section profile has to be sounded. In this cross-section the verticals to be measured should be chosen in such a way as to give the best possible measuring result.
The velocity distribution is obtained by a current-meter which is lowered into the water and the velocity is measured at all depths at which the sediment transport is going to be measured.
The average grain-size is obtained in the laboratory in the following manner:
- by comparing with a standard-ruler (instantaneous but rough)
- by means of a settling tube (V.A.T. Visual Accumulation Tube) (accurate and rather quick, see Paragraph 4.2.3).
- by means of drying and sieving (accurate but slow, see Paragraph 4.2.2).

<u>If all data are collected the computation can start</u>

- Knowing both the average grain-size and the flow velocity the correction factor by which the sand catch must be multiplied can be determined; this correction factor is the ratio of the loss coefficient and the hydraulic coefficient. The factor is given in the manual of the instrument.

- The loss coefficient is the ratio of the total sand volume that enters the nozzle and the part that settles down in the DF.
  The loss coefficient increases with increasing flow velocity and with decreasing grain-size.
- The hydraulic coefficient is the ratio of the discharge through the nozzle and the discharge through the same imaginary orifice after removal of the instrument (obtained by model tests).

The suspended sediment transport of particles with diameters greater than 50 μ, measured with the Delft Bottle, is computed in the following way.
For each measured point in the vertical the transport is calculated.

$$S = \frac{G.\alpha}{T'.F} \qquad \text{or} \qquad T = \frac{V.\alpha}{T'.F}$$

in which S = transport in kg/s (dry material) per m^2

T = transport in m^3/s (pores included) per m^2

α = correction factor depending on nozzle used, average grain-size and flow velocity (see manual)

G = catch in kg (dry material)

V = catch in m^3

F = area of nozzle used in m^2

T' = sampling time in seconds.

By multiplying S (or T) by the vertical distance for which the measuring point in the vertical is representative and by summation of these values over the total vertical, the transport through the vertical is obtained, expressed in kg/s per m (width). Taking into account the total width, represented by the vertical, and adding the transports of all verticals, the total transport through the cross-section is obtained.
For registration and elaboration of the Delft Bottle - data the standard form could be used (see Figure 3.7.2).

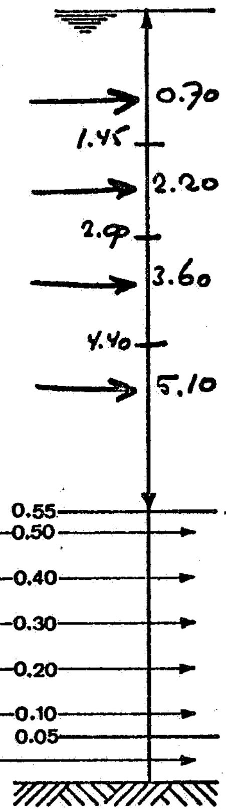
RIVER: Magdalena
STATION: Ballena
DATE: 25-10-71 VESSEL: Explorador

MEAN WATERLEVEL: –
TIME: 9:50 TILL 10:40
OBSERVERS: HJO, HG

DELFT BOTTLE										TRANSPORT COMPUTATION	
time	height above bottom	depth below surface	current velocity	sample time	caught	correction factor	caught	nozzle area	volum. transport	repres. height	transport
	m	m	m/s	min	cm3		cm3	cm2	m3	cm	m3
1	2	3	4	5	6	7	8	9	10	16	17
9:50		0.70		3	23	1	23	1.9	0.58	145	84.1
		2.20		3	12	1	12	1.9	0.30	145	43.5
		3.60		3	20	1	20	1.9	0.51	150	76.5
		5.10		3	30	1	30	1.9	0.76	135	102.6
	0.50			3	36	1	36	1.9	0.91	10	9.1
	0.40			3	76	1	76	1.9	1.92	10	19.2
	0.30			3	14	1	14	1.9	0.35	10	3.5
	0.20			3	53	1	53	1.9	1.34	10	13.4
	0.10			3	128	1	128	1.9	3.23	10	32.3
											0.4

VERTICAL Nº I

total depth: 6,30 m



B.T.M.A.

no.	sample time	caught	caught in 2 min.	transport
	min.	cm3	cm3	m3
11	12	13	14	15
1			28	0.48
2			22	0.37
3			25	0.42
4			25	0.43
5			20	0.34
6			30	0.51
7			20	0.34
8			25	0.42
9			15	0.25
10			18	0.31
			mean:	0.4

TRANSPORT IN VERTICAL $m^3/m^1/24hrs$:	385

$$(10) = \frac{(8)}{(9) \times (5)} \times 0.144 \ m^3/24 \text{ hours}/m^1$$

$$(14) = \frac{(13) \times 2}{(12)}$$

$$(15) = 0.017 \times (14)$$

$$(17) = (16) \times (10)$$

Figure 3.7.2 Calculation sheet

3.7.3.3 Depth-integrated sampling with US-D 49

To obtain a sample, a bottle is inserted in the sampler and the instrument is lowered at a uniform rate from the water surface to the bottom of the river, it is instantly reversed on touching the bottom, and then raised again to the surface at an uniform rate.
The sampler continues to take its sample throughout the time of submergence.
At least one sample should be taken at each selected vertical in the river cross-section.
A clean bottle is used for each sample.

The transit rate depends on the mean velocity in the vertical, the water depth and the inside diameter of the nozzle.
This can be derived from the graph, see Figure 3.7.3.
Stream less than 4,5 mtr are usually depth-integrated on a round-trip basis. The sampler is lowered to the bottom of the stream at a uniform rate and raised back to the surface at a uniform rate, but not necessarily the same rate.

Transit rates for depth integration

Accurate depth-integrated samples partly depend on a reasonably uniform transit rate of the sampler. An experienced operator who uses the depth-integration method frequently, will seldom have any trouble in maintaining an adequately uniform transit rate throughout a single sampling trip. A satisfactory volume of sample is more difficult to obtain than a reasonably constant transit rate.

The following techniques often aid in attaining satisfactory transit rates:

a. If the stream velocity is known from previous measurements or can be estimated from an observation of surface velocity, the sampling time can be obtained from a previously prepared velocity-sampling time chart. Also, if the sampling time is known for the sampler nozzle size and one velocity, it can be computed for other velocities because sampling time varies inversely proportional with the velocity.

b. Determine the depth to be sampled, and for round-trip integration divide twice the sampling depth by the sampling time to obtain the required rate for lowering or raising the sampler.

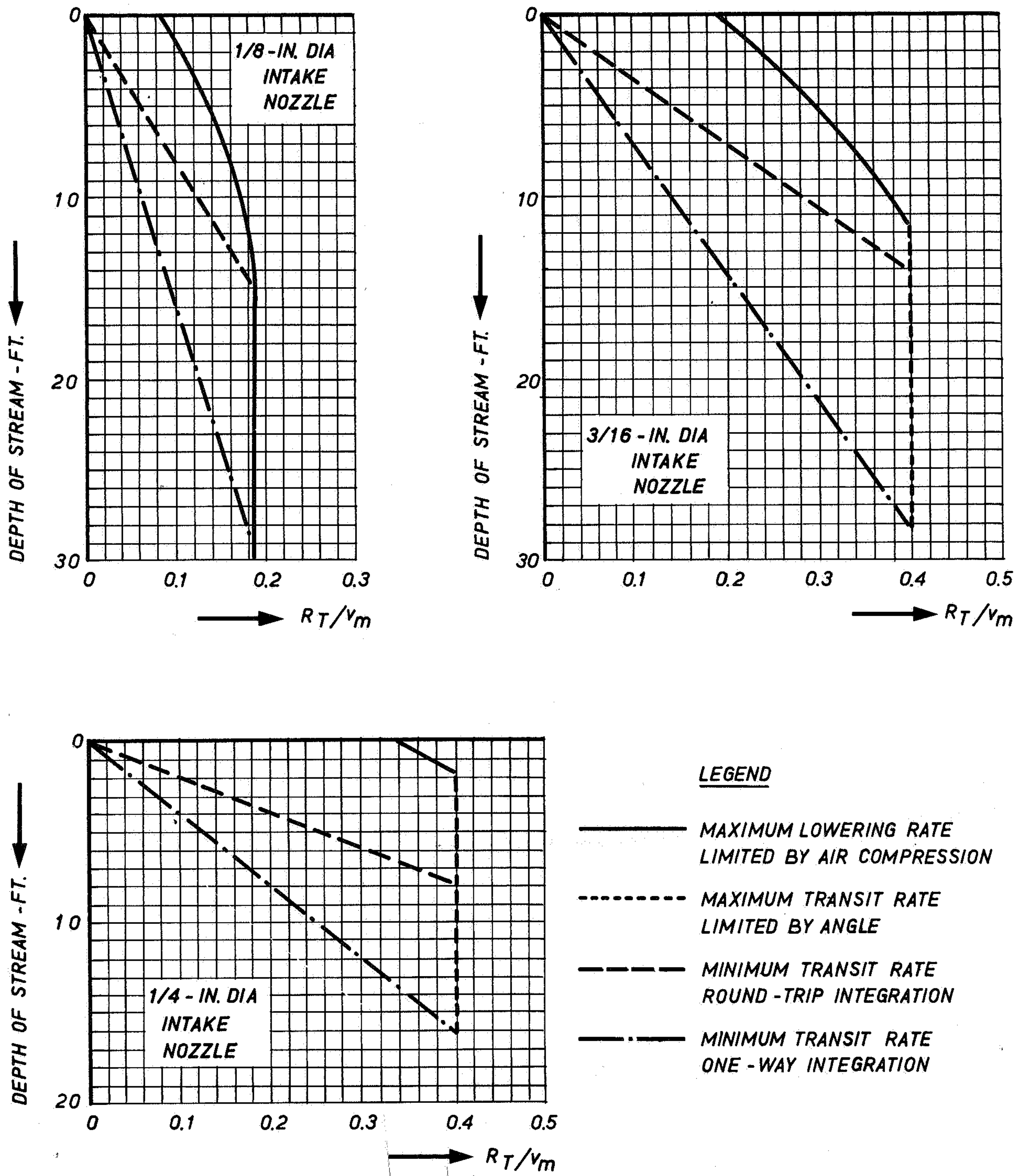


Figure 3.7.3 Allowable ratio of uniform transit rate RT to mean velocity V_m for depth-integration ½ ltr milk bottle

c. The sampler should then be lowered or raised the necessary number of feet and tenths of feet per second. An operator working alone can count slowly to approximate a one-second interval. If two operators are available one can use a stopwatch as a time control.

d. In round-trip integration the time required for lowering the sampler may be noted and if it is more or less than half the total time desired, the raising rate can be increased or decreased to improve the total time.

e. One should observe sample volumes and sampling rates carefully, so that the transit rate can be adjusted from one sample to the next.

f. In the ETR (Equal Transit Rate) method, the transit rate should be the same in all verticals. Establish a transit rate and then determine the time (which depends on the depth) for sampling in each vertical. By watching the sampling time it is possible to maintain a nearly constant transit rate in the vertical. Because the sediment concentration often varies more in the vertical than in the horizontal direction, a uniform transit rate in the vertical is more important than a constant rate from one vertical to another.

g. Sampler transit rates should be kept within certain specified limits.

h. Depth-integrating suspended sediment samplers do not sample down to the stream-bed. Suppose that the intake nozzle of a sampler is 10 cm above the bottom of the sampler. The sampler will not sample the lowest 10 cm of the vertical if the stream-bed is firm and level. If the bed is soft and dunes are present, the sampler may sink into the bed or may be lowered on top of, or downstream from, a dune.
   Presumably one should consider the sampling depth as 10 cm less than the total depth and reverse the sampler travel without hesitation when the sampler touches bottom. Because the reversal will almost never be instantaneous, too large a sample will be taken at the reversal point. Another method is to consider the whole depth as the sampling unit and allow the reversal of sampler travel to take up the same length of time that would normally be spent in integrating 15 cm of depth. Sampling procedure should be adapted to sediment discharge computation procedure at this point. Also a record should be kept of the sampling procedure that was actually used.

3.7.3.4 Point-integrated sampling with US-P 61

To obtain a suspended sediment sample at any point beneath the surface of a stream, called point-integrating method, the sampler is lowered to the sampling point with the valve in the equalizing and closed position. When the sampler reaches the sampling point, the operator manipulates and holds the electrical switch to set the valve in the sampling position. At the end of the sampling time, the operator permits the valve to close by releasing the switch, raises the sampler out of the stream and removes the sample container. A clean bottle is required for each sample. The capacity of the equalizing chamber will permit sampling to a maximum depth of about 55 metres.

The sampler may be used to obtain a sample continuously over a range in depth, called depth-integrating method. If the stream is not over $5\frac{1}{2}$ m in depth with moderate velocity, the sampler valve is electrically held in the open position and the sample is obtained by lowering the sampler at a uniform rate from the water surface to the bottom of the stream, instantly reversing it, and raising the sampler to the water surface again at a uniform rate.

If the stream is between $5\frac{1}{2}$ m and 9 m deep, or has a high velocity, the sampler may be used to depth-integrate in one direction, i.e., from the bottom of the stream to the surface. The sampler valve is set in the equalizing position and the sampler is lowered to the stream-bed. The valve is then opened to the sampling position by holding the electrical switch closed and the sampler is immediately raised to the surface at a uniform rate.
Streams which are too deep or flow too fast to be depth-integrated by either of the above methods may be sampled by dividing the vertical into fractions each of which is depth-integrated individually.
Tests have shown that fluctuation in sediment load is so great during a period of only 15 seconds, that any one sample may not be representative of the average sediment concentration. If an instrument will not sample continuously for as long as 1 minute, the average of several consecutive shorter sampling periods may be used. Not less than six 15 seconds samples and not less than three 30 seconds samples will probably be required to determine the average concentration at one point.

3.7.3.5 Computation of sediment transport (concentration-meter)

For computation of sediment transport concerning suspended load measured with an US depth-integrating sampler or water sampler the following data are required:

- cross-sectional profile
- velocity distribution in the vertical
- sampling depth
- sediment concentration of each sample (milligram/lts or ppm)
- measuring time.

Velocities can be expressed quantitatively as the transport of a volume per unit of time and per cross-sectional area ($m^3/sec/m^2 = m/sec$).
Concentration can be expressed as the weight of dry sediment per unit volume (kg/m^3), therefore, the product of the two equals the weight of dry sediment transported per unit of time per unit cross-sectional area ($m^3/sec/m^2 * kg/m^3 = kg/sec/m^2$) see Figure 3.7.4.

The suspended sediment concentration in a stream fluctuates with time and varies from the surface to the bottom, and also from side to side. The sediment transport in a vertical is obtained by determining the sediment transport in each measurement point by multiplying the velocity and the corresponding concentration and by integrating these point-values over the vertical.
By summation of the transports through the various verticals, taking into account the width that each vertical represents, the total suspended sediment transport in the cross-section can be established. Care has to be taken which dimensions are used (kg or m^3, m^2 or cm^2, m or cm, hours or seconds etc.).

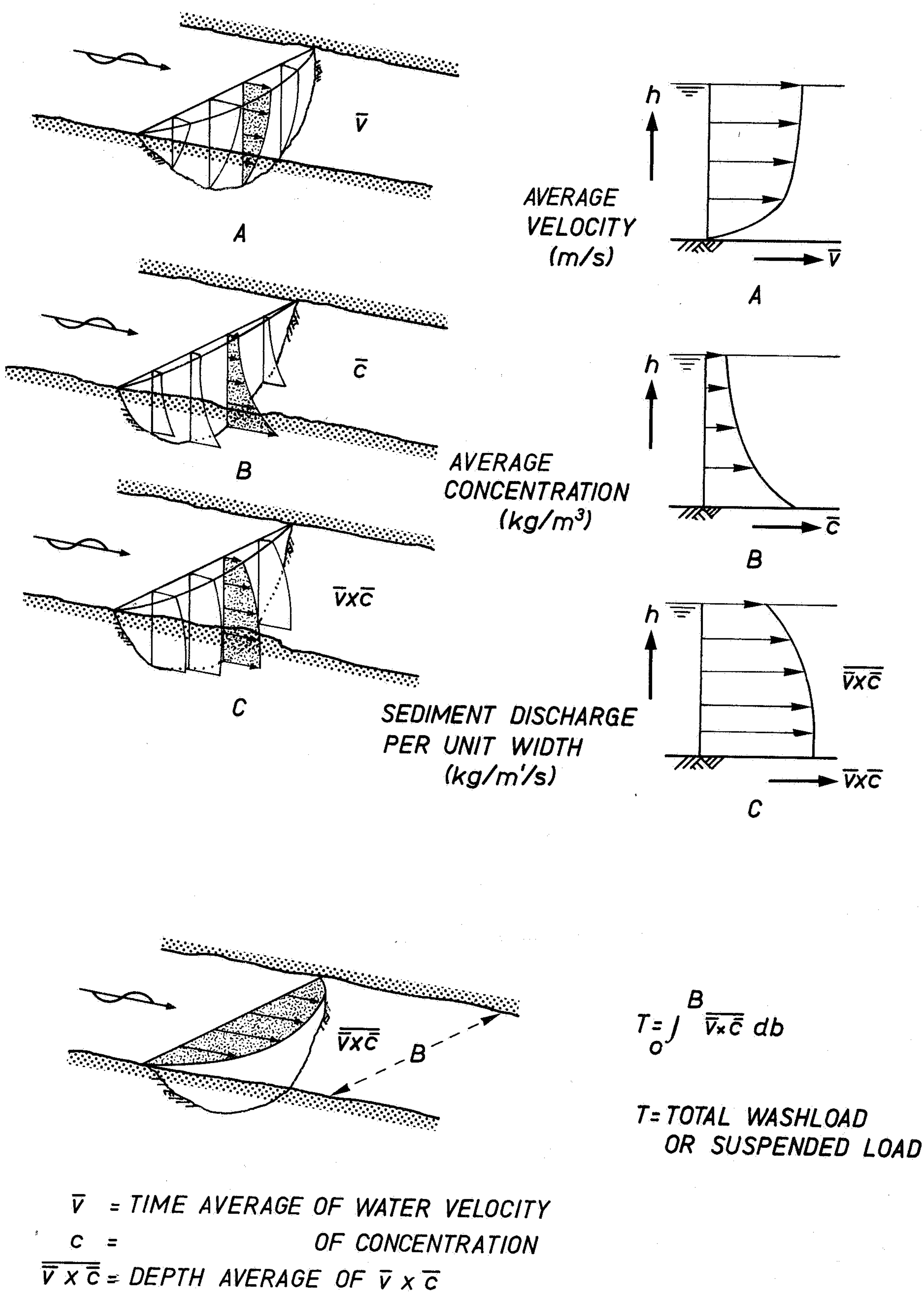


Figure 3.7.4 Illustration of calculation of sediment transport

In case that the sediment transport is calculated indirectly, thus by multiplying average flow velocities by average concentration determined from water samples an error is introduced as is indicated in the following.
The transport in a point in a period F has to be considered as the average of the product of instantaneous flow velocities (v) and concentration (c) measured during a period T (seconds). This average transport equals

$$\overline{v.c} = \frac{1}{T} \sum_{t=1}^{T} v.c$$

$$v = \bar{v} + v'$$
$$c = \bar{c} + c'$$

$\bar{v}$ = average velocity over period T (deterministic component)
v' = fluctuating velocity around $\bar{v}$ (stochastic component)
$\bar{c}$ = average concentration over period T (deterministic component)
c' = fluctuating concentration around $\bar{c}$ (stochastic component)

$$\overline{v.c} = \frac{1}{T} \sum_{t=1}^{T} (\bar{v} + v') * (\bar{c} * c')$$

$$= \frac{1}{T} \sum_{t=1}^{T} (\bar{v}.\bar{c} + \bar{v}.c' + v'.\bar{c} + v'.c')$$

this can also be written as

$$\frac{1}{T} \sum_{t=1}^{T} (\bar{v}.\bar{c}) + \frac{1}{T} \sum_{t=1}^{T} (\bar{v}.c') + \frac{1}{T} \sum_{t=1}^{T} (v'.\bar{c}) + \frac{1}{T} \sum_{t=1}^{T} (v'.c')$$

$$\frac{1}{T} \sum_{t=1}^{T} (\bar{v}.c') = 0 \quad \text{and} \quad \frac{1}{T} \sum_{t=1}^{T} (v'.\bar{c}) = 0$$

thus

$$\overline{v.c} = \frac{1}{T} \sum_{t=1}^{T} (\bar{v}.\bar{c}) + \frac{1}{T} \sum_{t=1}^{T} (v'.c') \quad \text{or}$$

$$\overline{v.c} = \bar{v}.\bar{c} + v'.c'$$

if, however, v' and c' are correlated to each other then v'.c' has a certain value otherwise $v'.c' = 0$.

Thus $\overline{v.c} \neq \bar{v}.\bar{c}$. The transport is computed as $\bar{v}.\bar{c}$ which differs from the value $\overline{v.c}$ (Delft Bottle).

3.7.4 Bed load

This transport refers to the larger grains of material carried by the river rolling and sliding over each other but hardly rising from the bottom. Their movement is extended some distance into the bottom with an exchange of grains from different layers.

In the upper-river the bed load transport may be the largest portion in the sediment transport.
The initial power for the motion is the drag force of the flow upon the particles, which have a certain capacity to move. Generally from the bed an unlimited source of supply of materials is available to meet this capacity. In that case it can be said that the transporting capacity is always satisfied to the full extent. This brings about that there is a correlation between the flow velocity and the sediment transport, while another consequence is that, as soon as the drag force somewhat decreases for one reason or the other, also the transport decreases, which immediately affects the bed.

Bed load is a phenomenon that takes place in pulses, never as a smooth and continuous flow of sand; this is more or less contrary to suspended load and wash-load. Suspended particles move with the same mean velocity as the surrounding water, bed load particles move considerably slower. However, in the upper-river the particles in the bed load transport may reach a considerable velocity.
The distribution of the concentration of bed load in the thin layer close to the bottom is very high in comparison with the other kinds of sediment transport.

The accuracy of the <u>bed load</u> sampling is not only affected by the complexity of the sediment transport phenomenon but also by the accuracy of the sampler, the measuring procedure and the calibration of the sampler.

The measurement is highly influenced by the roughness of the sand bed of the river (ripples, dunes, plane bed, bed material).

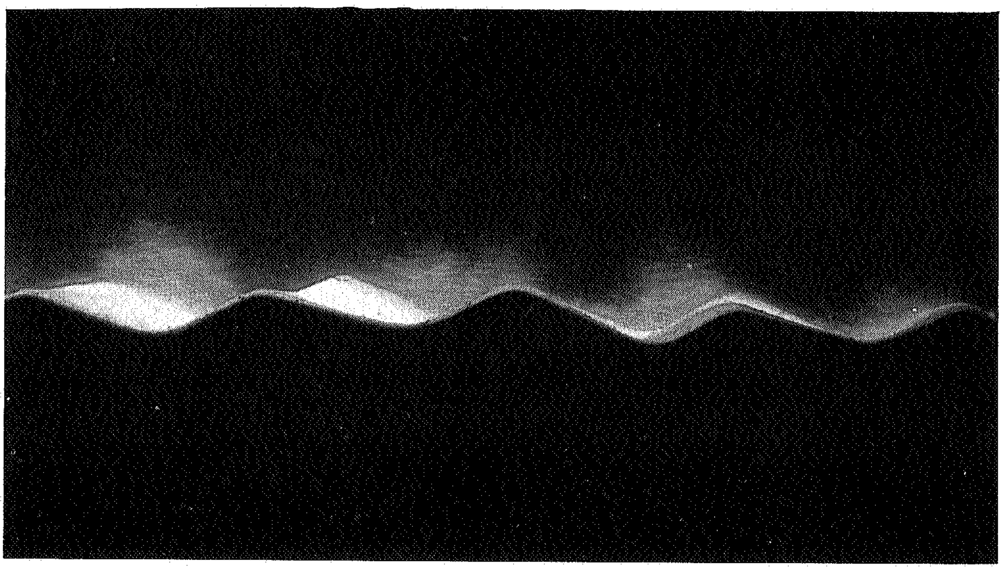


Figure 3.7.5 Sediment transport over a ripple
(wave and current in shallow water)

Figure 3.7.5 gives a picture of bed material that is transported over a ripple. It is obvious from this picture that the position of the sampler in the bed-form affects greatly the catch of material. The maximum catch is present at the crest of the ripple or dune and the minimum in the trough.
Moreover in almost every case the bed-form is not stable but changes. So the bed load transport is fluctuating in time and in space, as well as in longitudinal- and transverse directions.

This results in a large variation of the catch in the sediment sampler. So a single observation can only have a poor accuracy. Therefore a number of observations have to be taken in order to arrive at a better estimate of the average transport. In Figure 3.7.6 a graph is given for the relationship of the variance of the mean bed load transport as a function of the number of observation (De Vries, 1973).
This is only a theoretical curve (originating from the phenomenon) for a theoretical probability distribution of the samples. Additional errors as for instance instrument errors are present.

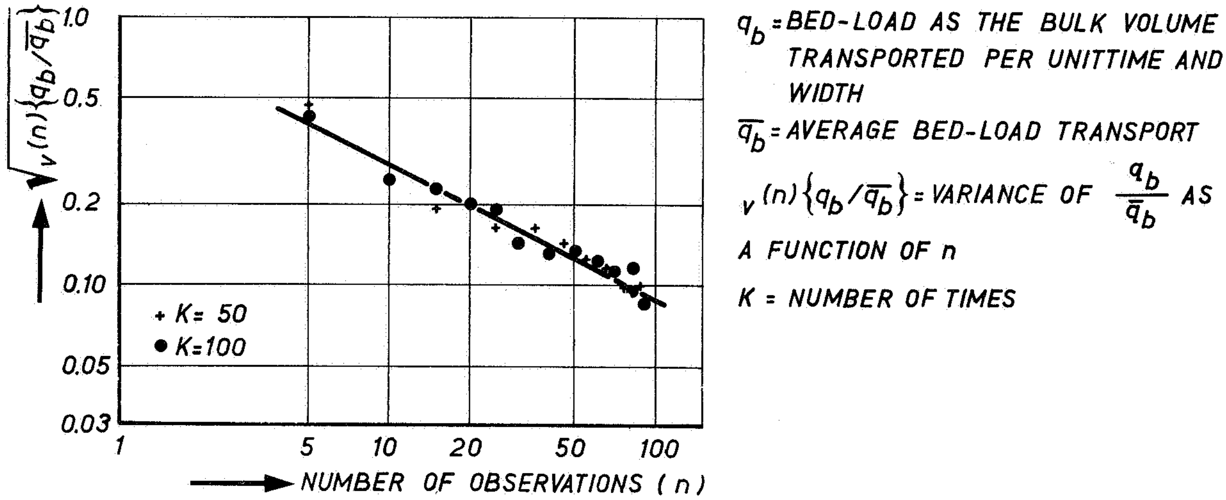


Figure 3.7.6 Relationship of number of individual bed load samples and standard deviation

For measuring the bed load transport the Bed Load Transport Meter Arnhem (BTMA) is widely used. For the principles and description of the instrument see Section 2. The measuring procedure is as follows.

- Make a cross-sectional profile of the river.
- Make a longitudinal sounding of the river (to establish the sort of bottom form, large or small sand ripples etc.).
- According to the bed-form decide on the measuring procedure.
- Lower the instrument in the water till the bottom is reached; make sure that this is not done too fast, to give the instrument time to settle itself in the current direction.
- Leave the instrument for two full minutes on the bottom.
- Hoist up the instrument.
- Empty the basket into the trough, while rinsing the basket with deck wash water.
- Measure the volume of the catch.
- Repeat this measurement e.g. 10 times depending on the required accuracy (see Figure 3.7.6) and depending on the bed-form either at random from one position of the launch or as many times shifting the launch in longitudinal direction (see Figure 3.7.7).

If the sampler is lowered from an anchored survey vessel then it is placed at random in a reach with length L. If the predominant wave length (λ) of the ripple pattern of the river bed is small compared to L, then random sampling is assumed by operating from the vessel, anchored at one location. However, for $L < \lambda$ this procedure would lead to systematic errors. Now the vessel has to leave its fixed position in the cross-section, shift in longitudinal direction and anchor again in order to obtain random sampling. This may be repeated a few times (see Figure 3.7.7).

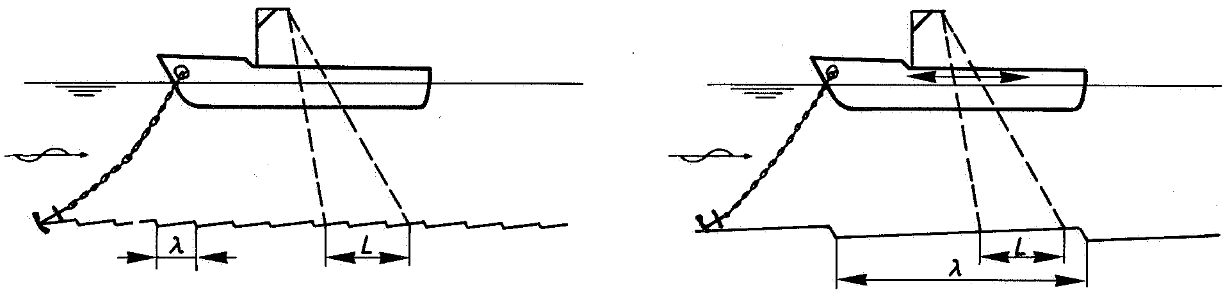


Figure 3.7.7 Location of the survey vessel for bed load sampling

3.7.4.1 Computation of bed load

The catches of the sampler are averaged and the volume of the average catch or the complete catch is converted into daily transport (in m^3/24 hours/m') by means of the BTMA calibration curve (see Paragraph 3.7.4).
This calibration curve is based on tests carried out by the Delft Hydraulics Laboratory

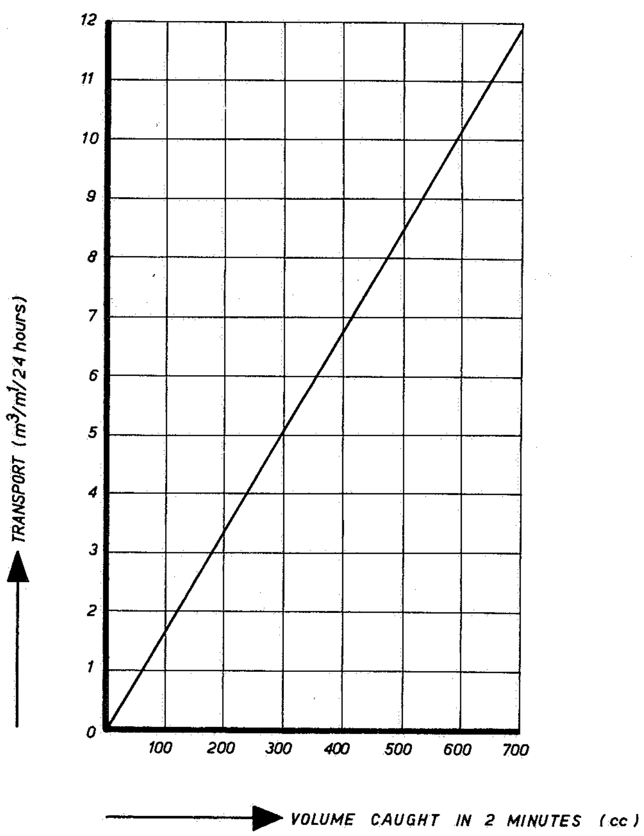


Figure 3.7.8 Calibration curve BTMA

If the sediment transport has to be expressed in kg/s the sample is taken to the laboratory and the dry weight of sediments in the sample is determined.

$$S = \frac{2.G.B.}{0.085.T}$$

in which S = transport in kg/s through cross-section

G = average dry weight of all samples taken in the cross-section in kg

B = width of river or estuary in m

T = sampling time (of one measurement).

The factor 2 is the efficiency factor (see manual), the factor 0.085 represents the width of the basket of the instrument in m.

Normally in the laboratory also the grain-size distribution of the BTMA samples will be determined.

On the standard form for sediment transport (see Figure 3.7.2) the calculation for the bed load transport (and also suspended load, measured with the Delft Bottle) can be evaluated in an easy way.

3.7.5 Bottom sampling

In order to gain some insight in the morphology of the river, the erodibility of the river-bed and to determine the characteristics and origin of the bed material, bottom samples are taken at several locations in the river.
These locations should be chosen in such a way that a representative sampling is made across the cross-sectional and along the longitudinal profile of the river (see also Section 2.6).
To describe the samples a standardised nomenclature should be used, e.g. the M.I.T. standardisation. A sand rule can be a good help. In Figure 3.7.10 M.I.T. standardisation is given on a standard form for presenting the grain-size distribution data.
The colour of the samples should be noted on the measuring form also according to standards (a standard may be the US Soil Conservation Service Soil Colour Charts). Caution: the colour of wet material could differ from the same dry material. If possible a description should be made of the different minerals that occur in the sample and their relative occurrence.
Also the shape of the grains should be noted.

A table in which strength and structual characteristics are given can be a good guide for field indentification of bottom samples (Figure 3.7.11). As much as possible descriptive remarks about the sample and the location where the sample is taken should be noted down.
Even a description of the river-banks may be useful for interpretation of the results or to use the bottom sample data for studying the river-banks or the morphology in general.

Bottom samples can be taken with a Van Veen bottom-grab.
Two types of this grab are available: small size (weight = 2.4 kgf), content $\frac{1}{2}$ dm^3, medium size (weight = 5.25 kgf) and medium size + extra lead blocks (weight = 11 kgf), content 2 dm^3.

The size of a drawn sample mainly depends on the compactness of the bottom. With the same type of grab the caught sample of fine compact sand is smaller than in coarse sand or fine gravel. A heavy grab catches more than a lighter one. Therefore the small size is always fitted with lead blocks. For the medium size extra lead blocks can be supplied. These blocks can be attached with a few screws. If used, the caught sample is often twice the catch without these blocks. Moreover when a strong current prevails, the cable of the heavier grab deviates less from the vertical than the lighter one. The buckets are provided with air escape holes; this makes that the grab meets the bottom with its full weight and that when the buckets close there is no disturbance by the escaping air. However, these holes cause also that the water can percolate easier through the fissure between the buckets when the grab is hoisted above the water surface, and with it the captured fine particles. Therefore screws are supplied with the grab, to close the holes if fine parts are important.

In spite of the heavy closing force it can happen, when the grabs are sampling gravel or a mixture of sand and gravel, that a pebble sticks between the buckets. Be aware of it that in such a case the sample is not representative; the smaller parts have been lost during hoisting. It is always a good rule to take at least 6 samples on every location and base the conclusions on the total of samples. This is especially important when the bottom is less regularly shaped and the bottom material consists of a mixture of materials.

For a heavy gravel bottom the "van Veen" grab is less useful. For this purpose a drag grab is better. This grab is a heavy bucket with larger sharp circumference. It should be towed over the bottom over a distance of a few metres.

The investigation on the composition of the material of the river bed is carried out, like determining the river discharge and the total suspended load, by sampling the river bed. Again the question will arise "what are the requirements for the sampling which will result in a grain-size distribution for the bed material within a beforehand stated accuracy"?
From investigations by Dr. de Vries (DHL publication no. 90 of January 1971) it seems that to determine a sample size to ensure a proper sampling, coupled to an error limit, the relationship as given in Figure 3.7.9 may be applied.

It stands to reason to use D_{84} to characterize the coarse end of the grain-size distribution as the largest inaccuracies can be present in that end due to the fact that the mass of a grain is proportional to D_i^3.
As in the field D_{50} is more easily to estimate , D_{84} can be found by taken $2 \times D_{50}$ (G = 2) and $M = D_{84}^3 = 20\ (2D_{50})^3$.

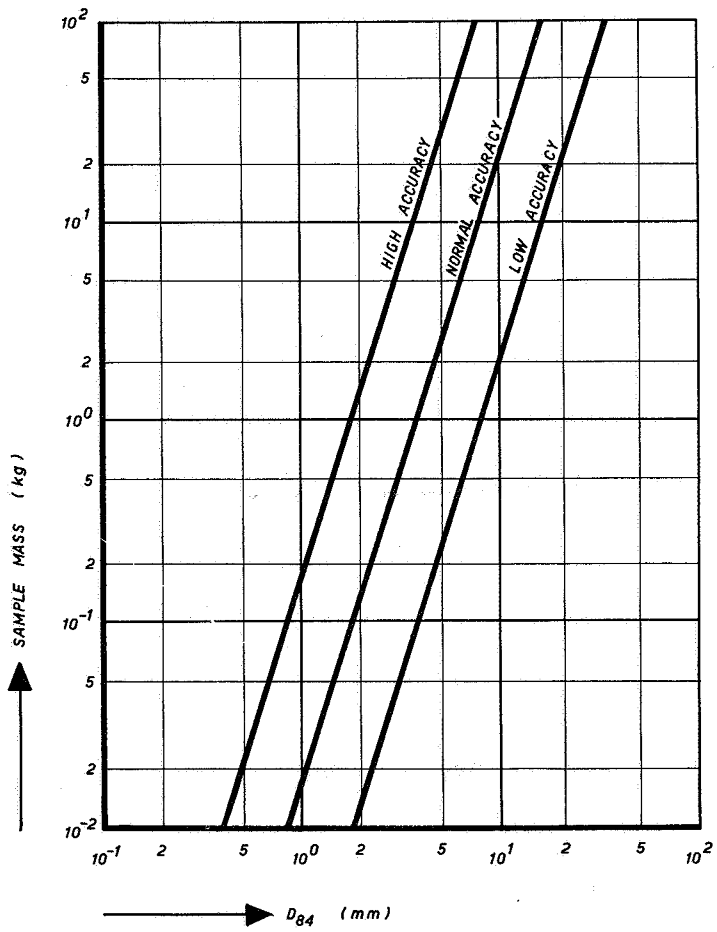
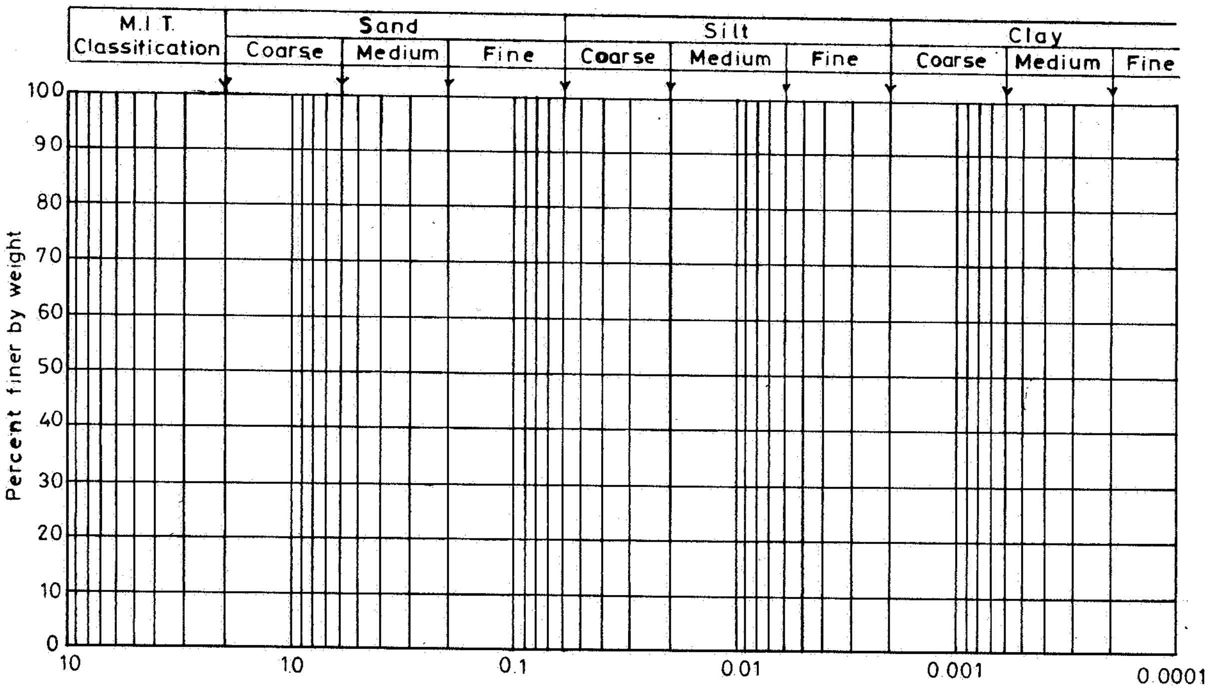
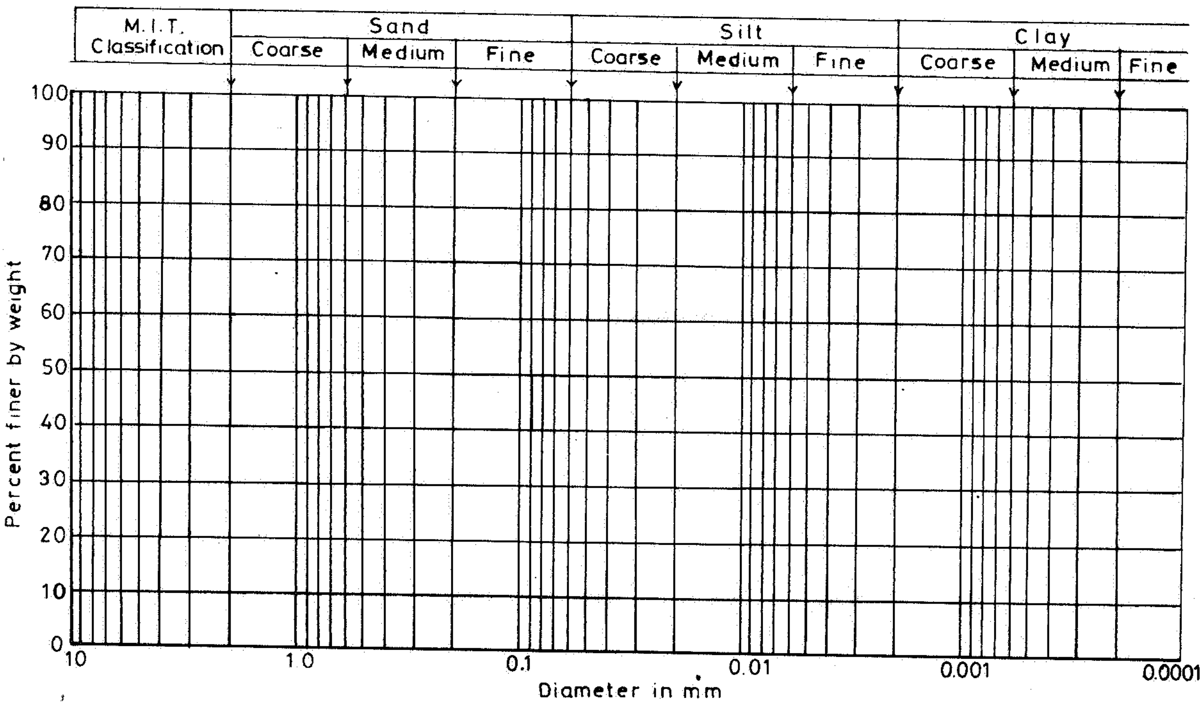


Figure 3.7.9 Design curve for sample mass M (D_{84} means that 84% of the sample passes through the sieve with openings D)





SAMPLE :
LOCATION :
DATE

Figure 3.7.10 M.I.T. Standard for classification bottom samples

				Strength	Structure	
	Types		Term	Field Test	Term	Field Identification
Coarse grained, non-cohesive	Boulders Cobbles Gravel			Can be excavated with spade, 2" wooded peg can easily be driven in	Homo-geneous	Deposit consisting essentially to one type
	Uni-form	Sands	Compact	Require pick for excavation, 2" wooded peg hard to drive more than a few inches	Strati-fied	Alternatively layers of varying types
	Graded		Slightly cemented	Visual examination. Pick remove soil in lumps which can be abraded with thumb		
Fine grained, cohesive	Low plasti-city	Silts	Soft	Easily moulded in fingers Particles mostly barely or not visible: dries moderately and can be dusted from the fingers	Homo-geneant	Deposit consisting essentially of one type
			Firm	Can be moulded by strong pressure in fingers		Alternating layers of varying types
	Medium plasti-city	Clays	Very soft	Exudes between fingers when squeezed in fist	Fissured	Breaks into polyhedral fragments along fissure planes
			Soft	Easily moulded in fingers	Intact	No fissures
	High plasti-city		Firm	Can be moulded by strong pressure in the fingers general: dry lumps can be broken, but not powdered; disintergrates under water; sticks to the fingers; dries slowly with cracks	Homo-geneous strati fied	Deposits consisting of essentially one type Alternating layers of varying types if layers are thin, the soil may be described as laminated
			Stiff	Cannot be moulded in fingers		
			Hard	Brittle or very tough	Weathered	Usually exhibitscrumbs or columner structure
Organic	Peats		Firm	Fibre compressed together, colour brown to black		
			Spongy	Very compressible and open structure, colour brown to black		

Figure 3.7.11 Table of strength and structural characteristics

3.7.6 The river bed in tidal regions

In general the river bed is not a stable channel and the course of the river is not always straight but shows many bends and sometimes a braided pattern. The cross-section will seldom have a smooth rectangular shape. Especially in river bends the cross-section will have some deeper- or shallow parts and in general the deepest part in the cross-section will be shifted somewhat downstream compared to the location with the maximum geometric curvature of the river bend (see Figure 3.7.12).

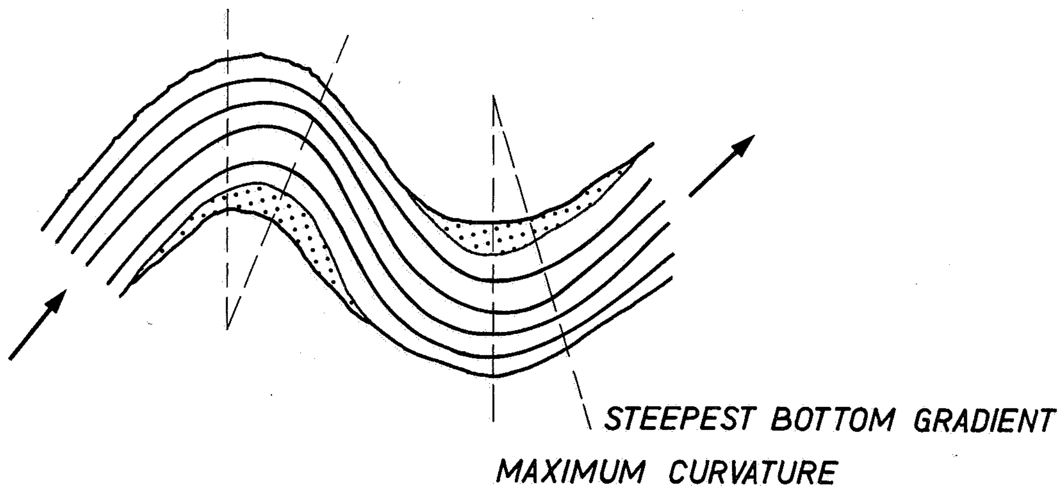


Figure 3.7.12 River bend, current in one direction

In coastal regions where not only an upland river discharge but also tidal effects cause water movements, the patterns of the deep and shallows parts of the river will be even more complex because of the changing of the magnitude and the direction of the current velocity. Sediment transport is highly influenced by the tide; usually different natural channels develop for the flow during flood- and ebb tide.
Hence in tidal regions, where the current has alternating directions, a double channel can develop as shown in Figure 3.7.13.

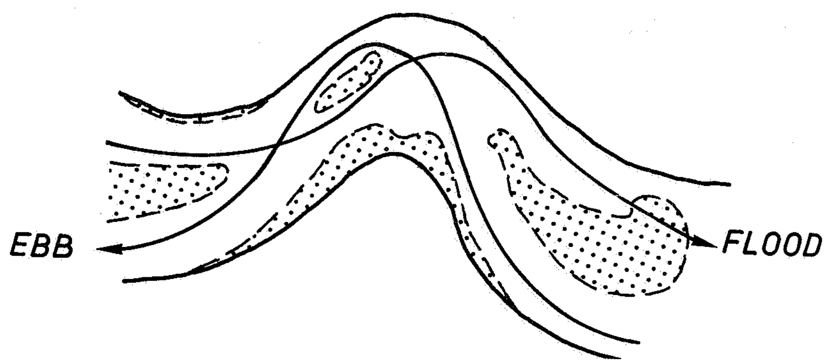


Figure 3.7.13 River bend in a tidal region

Due to the fact that the water-level during flood tide is normally rising, the flood channels have the tendency to "die out" into flats or shoals. At a certain moment in the tidal cycle the water-level is so high that at the end of the flood channel the water will not be confined anymore in one specific channel but will flow over the flat to the next bend.

The contrary will occur during ebb tide. The water-level goes down and the ebb current will be even more restricted to a narrow channel, causing the effect that this channel is normally deeper.
Also the fact that the upland river discharge is only in seaward direction, results in deeper ebb channels than flood channels.

3.8 Salinity

3.8.1 General

Knowledge about the behaviour of the salt water that penetrates from the sea into estuaries is undispensable for designing and situating inlets for irrigation systems, drinking water supply systems etc.
Due to the difference in specific density of salt and fresh water, density currents occur. These currents can cause difficulties in manoeuvring ships into harbours etc..
Due to chemical reactions between salt water and clay particles in the suspended-and wash-load transported seaward by the river, coagulation occurs. This results in sedimentation and decreases the depth of the river, which may cause navigational problems.
For these regions an investigation of the salt water movement in time and in space is necessary.

3.8.2 Density currents

Density currents arise from differences in salinity (or density) between two interconnected or periodically separated bodies of water.
Suppose there are two bodies of water; one with fresh water and the other with salt water as shown in Figure 3.8.1.

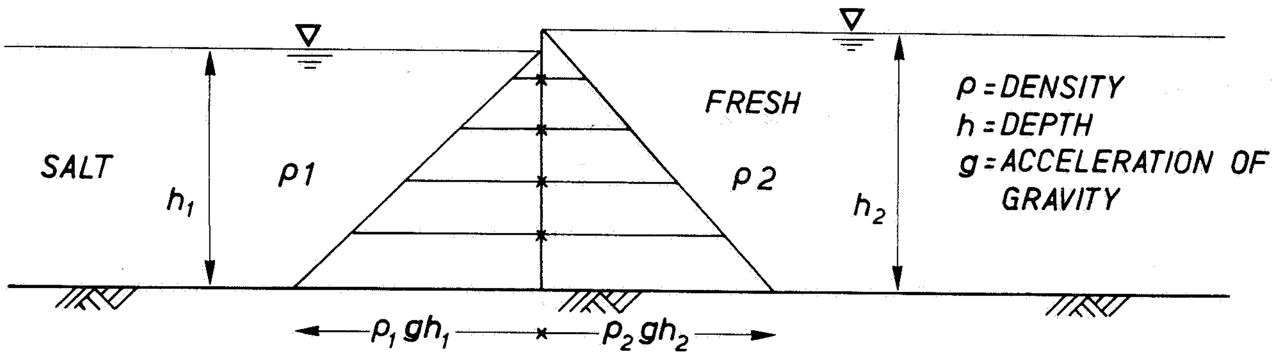


Figure 3.8.1 Pressure distribution

The pressure forces acting on the separation are in equilibrium when:

$$\tfrac{1}{2}\,\rho_1\, g\, h_1^{\,2} = \tfrac{1}{2}\,\rho_2\, g\, h_2^{\,2}$$

$$\rho_1 > \rho_2 \qquad h_1 < h_2$$

The net pressure distribution at the separation is given in Figure 3.8.2.

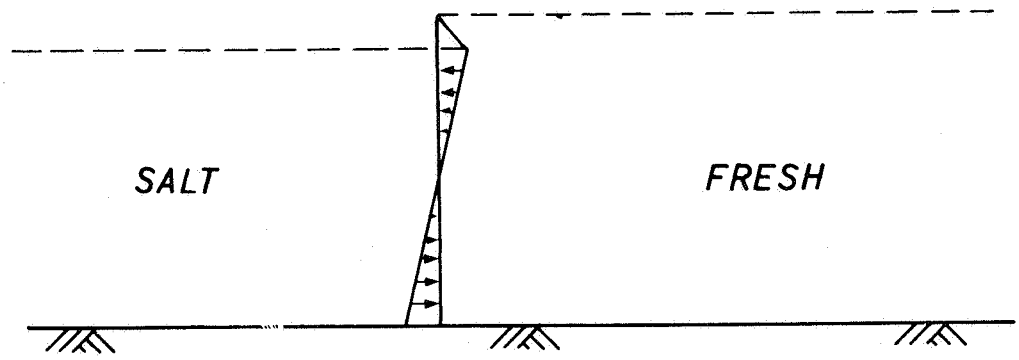


Figure 3.8.2 Nettpressure distribution

When the separation is removed these pressure differences will result in a flow of salt water near the bed into the section with fresh water and a flow of fresh water near the surface into the section with salt water. After some time the interface between the two fluids of different density will have the following appearance (often called the dry bed curve):

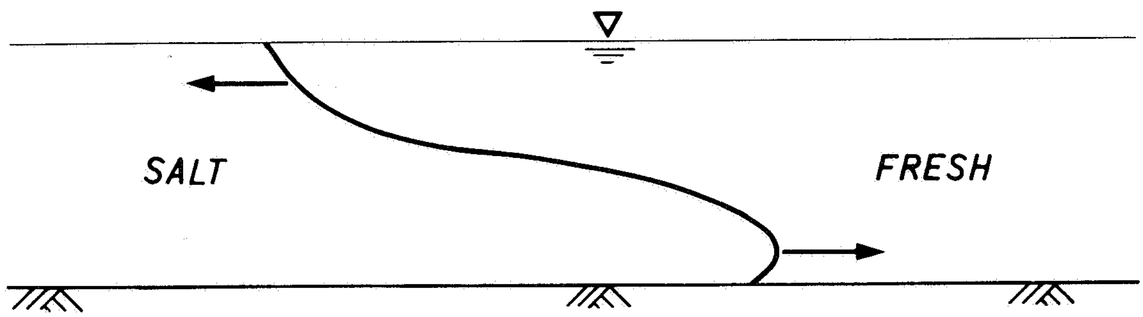


Figure 3.8.3 Dry bed curve

The distortion near the bed is caused by the bed resistance.
The shape of this curve is roughly the same as the shape of the water-surface in case of collapse of a reservoir dam.

With density currents in tidal rivers one is also confronted with the mixing of the fresh and salt water. The rate of mixing depends on the ratio river run-off to tidal prism.
Complete mixing occurs when:

$$e = \frac{Q \times T}{V} < \frac{1}{10}$$

$Q \times T$ = run-off during one tidal period (m^3)
V = tidal prism (m^3).

Partial mixing when $0,1 < e < 1$

Little mixing when $e > 1$

In case of complete mixing the salinity distribution is nearly constant in the vertical. The salinity of the river increases going downstream while in case of little mixing one finds almost horizontal layers of different salinity.
These phenomena are shown graphically in Figure 3.8.4 up to 3.8.7.

3.8.3 Salinity measurements

To obtain an idea regarding the variation of the salinity in the vertical, the variation along the river and the variation during the tidal period, a large number of measurements is necessary. Therefore it is very important to make sure that all data are written down in good bookkeeping fashion.
A data form may be as follows:

date	time	water depth	depth of probe	e.c.	sal.	temp.	remarks

e.c. = electrical conductivity (for Beckman RS 5 - 3 [mmho/cm] .
sal. = salinity (for Beckman RS 5 - 3 [^{o}/oo] .
temp. = temperature (for Beckman RS 5 - 3 [^{o}C] .

The Beckman type RS 5 - 3 is capable to measure the electrical conductivity and temperature. This is highly valuable when it comes to interpolation between field data.
When the water velocities are high it is not possible any more to lower the measuring probe vertically from an anchored vessel. In a case like that the measurements are taken from a drifting vessel. Some times it is necessary to add some weight to the probe assembly. The extra weight has to be attached far enough away from the probe itself such that the inductive measurement is not disturbed.

The measurements in one vertical are done every meter. In the column "remarks" conditions like rain, wind, passing ships etc. are written down.

From the data the salt distribution in one vertical at one particular time (for example High Water) can be determined.

This can also be done during a whole tidal cycle or during short periods.

The vertical as well as the horizontal distribution along a river may be obtained by a vessel moving upriver on the high water top and sampling verticals in the deepest part of the cross-sections at intervals of 1,2 or more kilometers.

The data from one vertical can be written down in tables or can be presented in a graph with on the horizontal axis the actual salinity and on the vertical axis the depth of the probe .

When comparing the results of the different verticals it is recommended to make the horizontal and vertical values dimensionless, so on the vertical axis divide the depth of the probe by the water depth and on the horizontal axis divide the actual salinity by the average salinity in the vertical (see Figure 3.8.4 up to 3.8.6).

For a condition at one moment a graph can be made depicting lines of equal salinity with on the horizontal axis the distance and on the vertical axis the water depth. In the vertical,the measured values of the salinity are written. After this a map with lines of equal salinity can be drawn (see Figure 3.8.8.

3.8.4 Units for salinity and chlorinity

The salinity or chlorinity can be expressed as follows:

S (salinity) : amount of all in water dissolved solids in grams per kilogram, in $^{o}/oo$

Cl^- (chlorinity): amount of dissolved Cl^- - ion in grams per kilogram of water, in $^{o}/oo$.

Because of the fact that the density of salt or brackish water differs only a little from 1000 kg/m^3 the above mentioned definitions are also valid for a liter of water instead of a kilogram of water.

3.8.5 Relations between density, salinity and chlorinity

The temperature influences the density of water. The relation between temperature, salinity and density can as a first approximation be expressed as follows:

$$\rho = 1000 + 805\ S - 0.0065\ (t - 4 + 220\ S)^2$$

in which:

ρ = density [kg/m^3]
S = salinity [$^o/_{oo}$]
t = temperature [^oC]

The relation between salinity and chlorinity may be expressed:

$$S = 0.03 + 1.805\ Cl^-$$

The Beckman RS 5 - 3 salinometer gives the values for Cl^- and S reduced for the temperature at the moment of measurement.

Elaboration

The elaboration of salinity measurements mainly exists in presentation of the gathered date in a conveniently arranged way. The data are written in tables or presented in a graph with depth on the vertical axis versus salinity on horizontal axis often the results are expressed in dimensionless units (actual depth over mean depth, actual salinity over mean salinity) see Figures 3.8.4 up to 3.8.6.
To get an idea of the extent of the salt-water wedge often a graph is given of lines of equal salinity (along a longitudinal profile of the river) and for one moment (e.g. high water) in the tidal cycle (see Fig. 3.8.8).

Salinity may be an important factor in selecting a site for a water-intake structure for irrigation purposes.
Density currents may influence the behaviour of transported sediments (flocculation).

At first instance by calculation the estuary-number ($e = \frac{Q \times t}{V}$, see paragraph 3.8) an impression can be obtained about composition of the water and whether a salt-wedge exists.

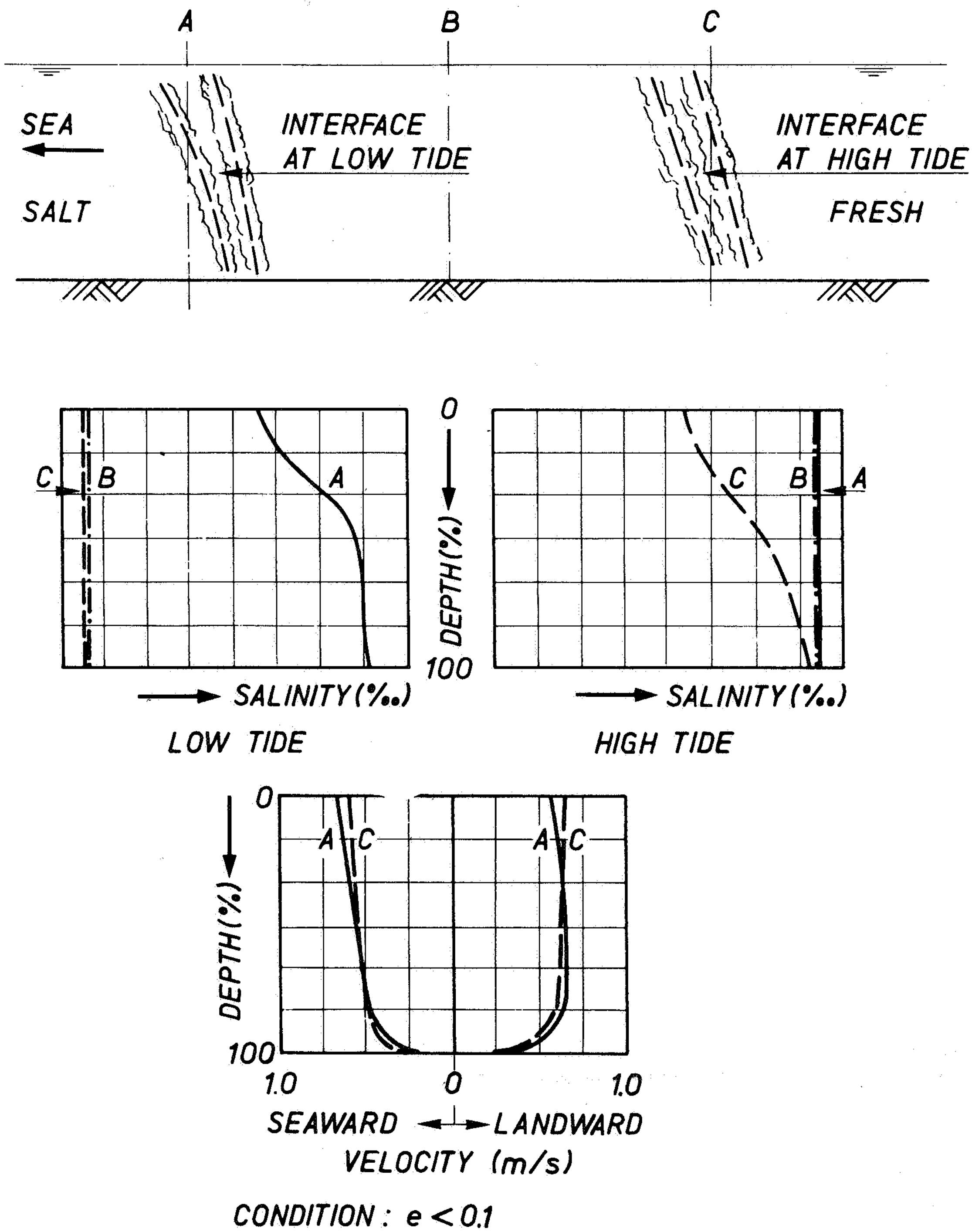


Figure 3.8.4 Example of the vertical salinity - and velocity distribution - complete mixed estuary

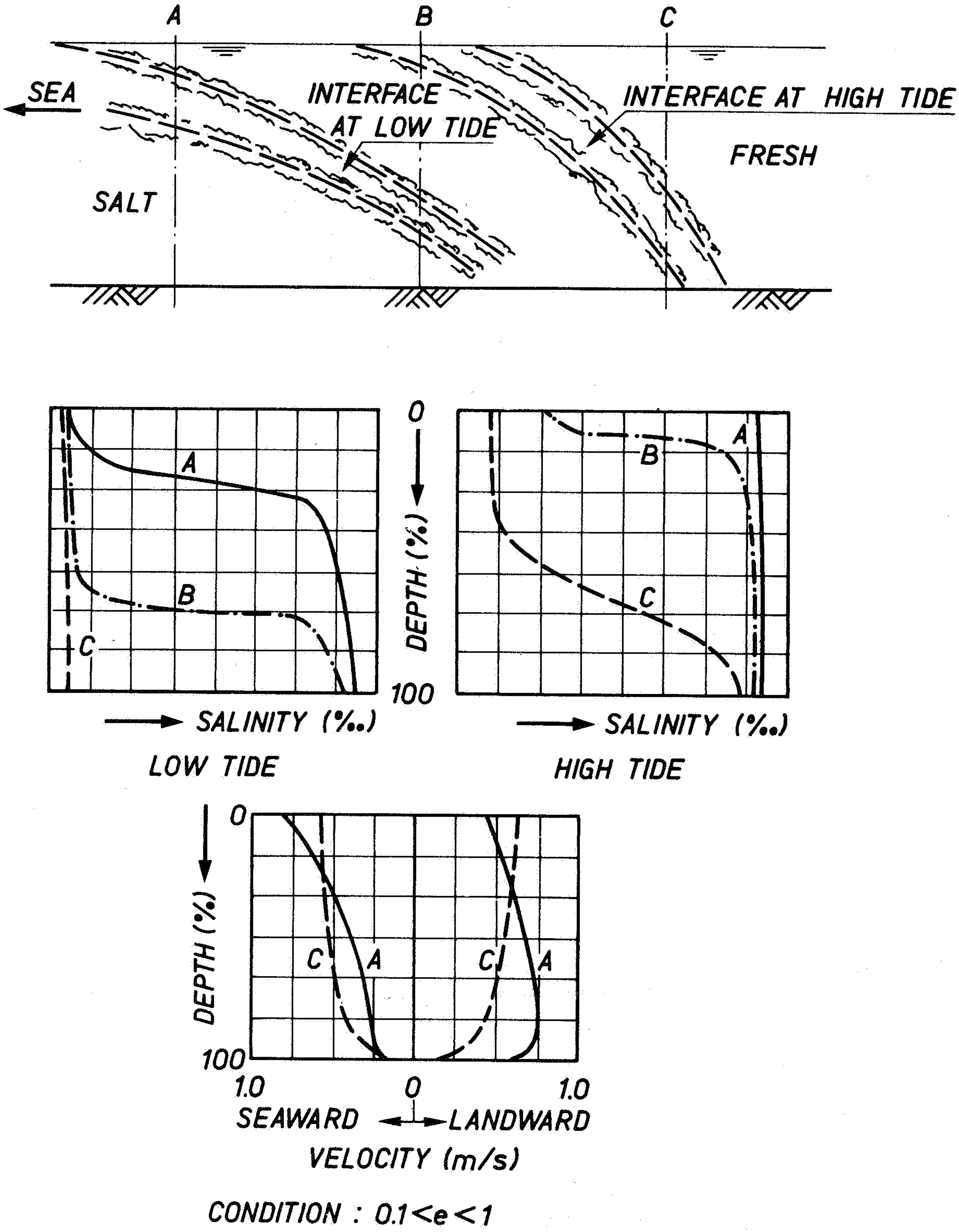


Figure 3.8.5 Example of the vertical salinity and velocity distribution - partial mixed estuary

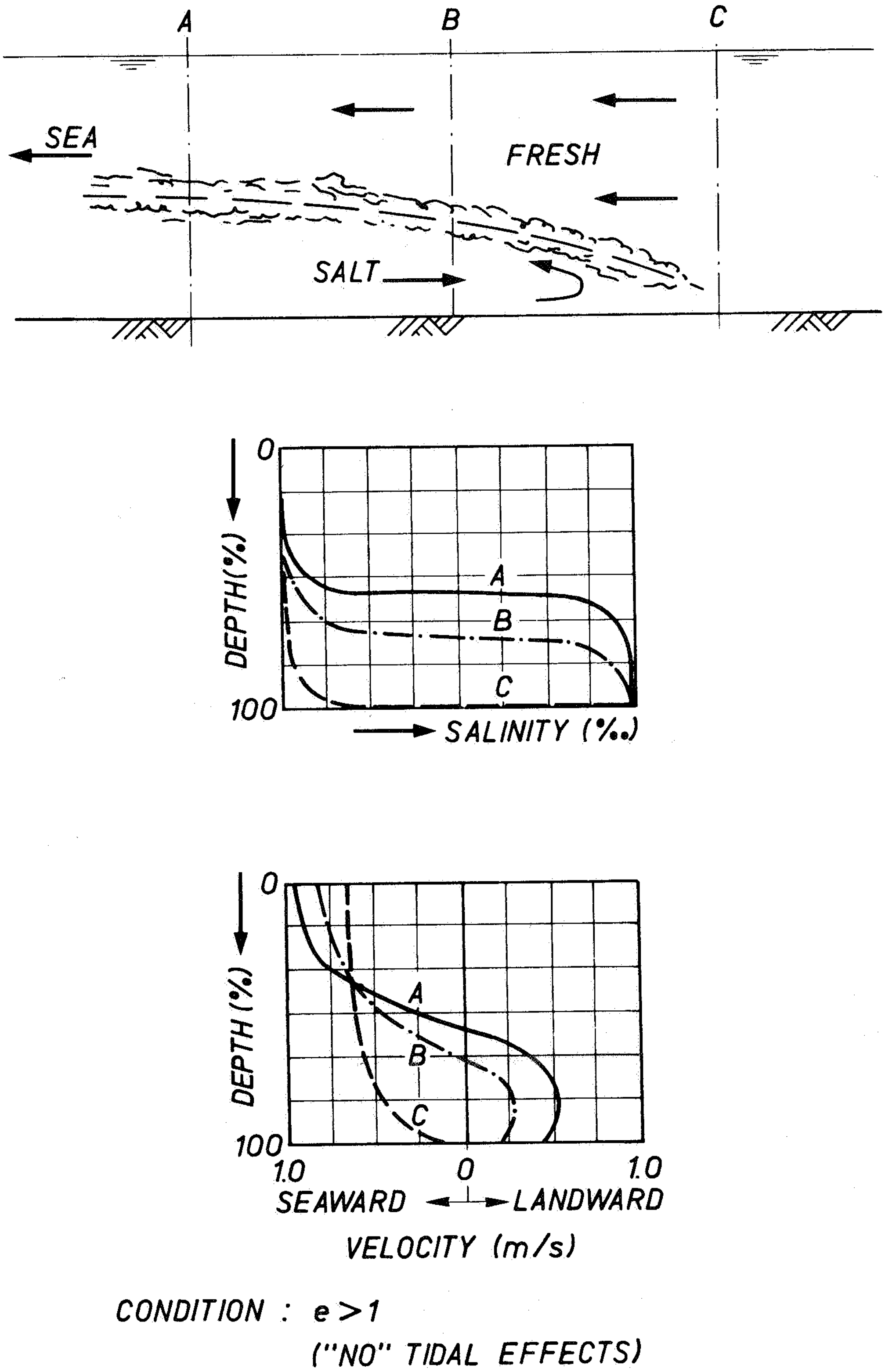


Figure 3.8.6 Example of the vertical salinity and velocity distribution - not (little) mixed estuary

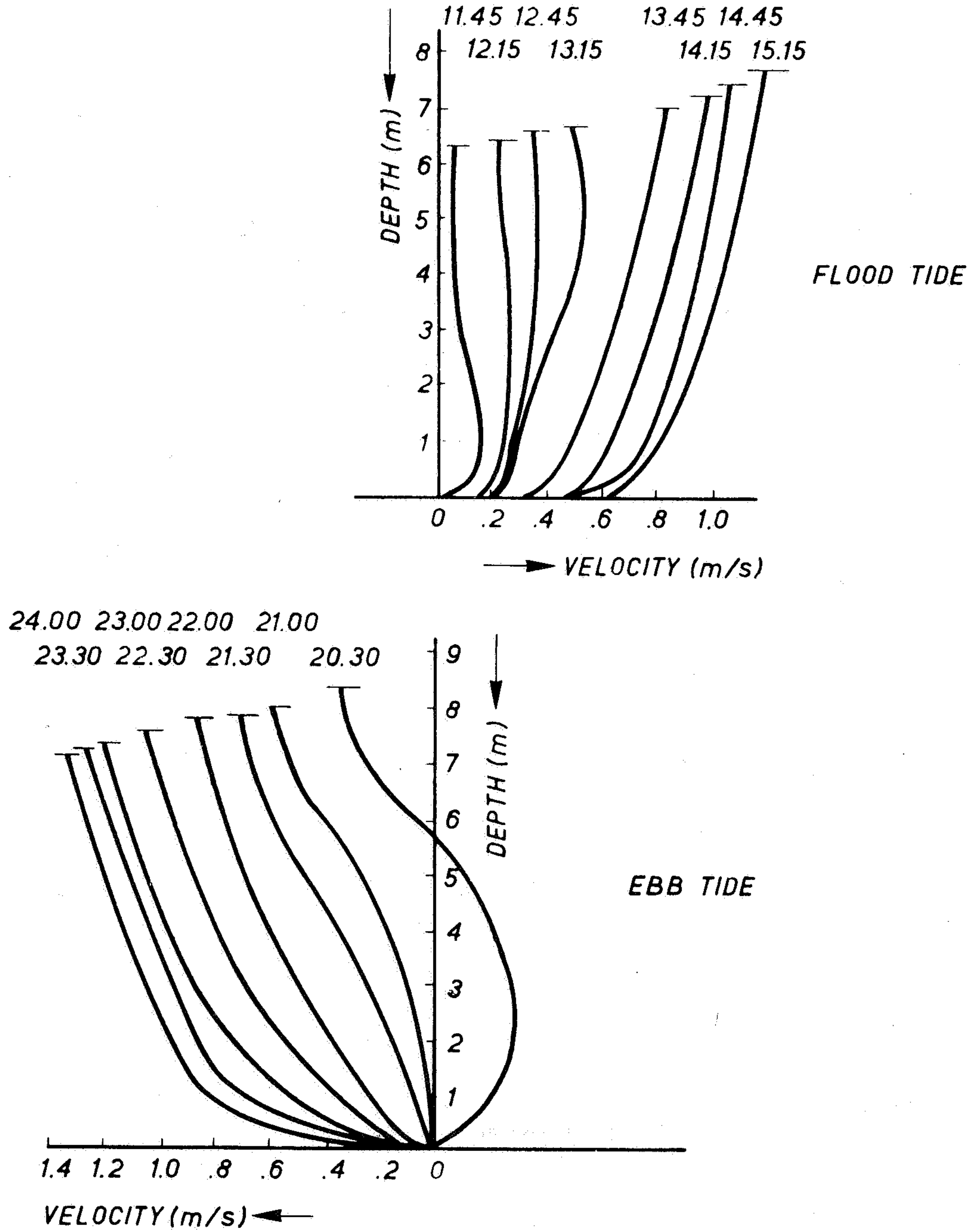


Figure 3.8.7 Development of velocity profile after slack water until maximum velocities
Date: 740829
Location: river Musi (vessel "Tawar")

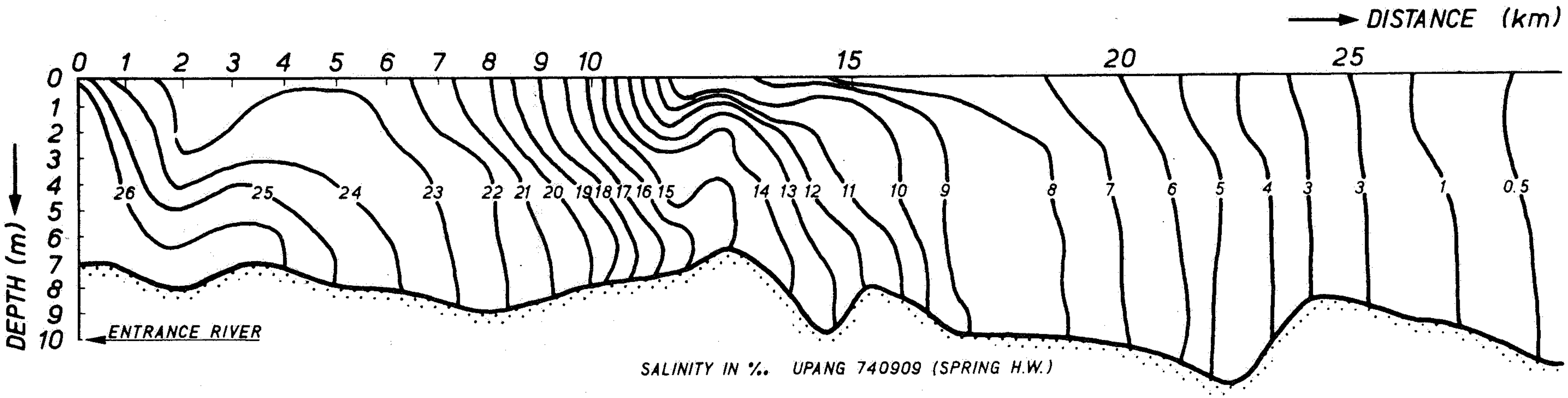


Figure 3.8.8 Longitudinal profile

4.0 Analysis of samples

4.1 General

During a hydrographic survey usually 2 kinds of samples are taken:

1. Bottom and bed-load samples.
2. Suspended sediment samples, whether it belongs to wash load or suspended load.

For each group of samples different kinds of analysis are required to obtain certain parameters viz.

Bottom and Bed-load samples

Bottom	and	Bed-load samples
- specific gravity		- sand-content
- water-content		- grain-size distribution
- sand-content		
- grain-size distribution		

Suspended sediment samples
- sediment-content
- grain-size distribution.

4.1.1 Definition

Specific gravity

The specific gravity of soils is the ratio of the weight in air of a given volume of soil particles at a stated temperature to the weight in air of an equal volume of distilled water at a stated temperature.

Water-content

The water-content is defined as:

$$\text{water-content in weight percent} = \frac{\text{wet weight - dry weight}}{\text{dry weight}} \times 100$$

The procedure to determine the water-content is:
- to weigh a part of the sample (wet weight)
- dry it for 24 hours at 110° C
- to weigh it again (dry weight).

The water-content is also often expressed in volume percent. Instead of weighing now the volume has to be determined.

$$\text{water-content in volume percent} = \frac{\text{total volume} - \text{volume dry material}}{\text{total volume}}$$

Sand-content

The sand-content is defined as:

$$\text{sand-content in weight percent} = \frac{\text{dry weight sand}}{\text{total weight of sample}}$$

in which sand is defined according to grain-size diameter between 0.050 mm and 2 mm, according to United States Geological survey between 0.062 mm and 2 mm.

Silt

All particles with a grain-size between 0.004 till 0.062 mm.

Clay

All particles with a grain-size smaller then 0.004 mm.

4.2 Separation of sand and silt

Two procedures can be used to separate the sand and silt fraction of a sample. After weighing the dried sample, the sample is sieved and washed through a sieve with openings of 50-60 μ.

The sand, remaining on the sieve is dried and weighed and percentage is determined.

(Method less accurate as silt particles can stick to the sand particles.)

Another method to separate the sand and silt fractions of a sample is based on the difference in settling velocities of particles with various diameters. According to Stokes Law the fall velocity of a grain can be expressed as:

$$w = \frac{1}{18\,\nu}\,\Delta\,.\,g\,.\,d^2$$

in which w = fall velocity in m/s

ν = kinematic viscosity of the liquid in which the settling occurs. For water of 20° C the kinematic viscosity equals about 1.10^{-6} m^2/s (see Figure 4.1.3)

Δ = relative density of sediments $\left(= \frac{\rho\ \text{grain} - \rho\ \text{water}}{\rho\ \text{water}} \approx 1.65\right)$

ρ = density in kg/m^3

ρ water = 1000 kg/m^3

ρ grain = normally 2650 kg/m^3

g = acceleration of gravity (= 9.81 m/sec^2 dependent on latitude)
d = diameter of grain in m.

If sand is defined as particles having a diameter greater than 60 μ, from Stokes Law can be calculated that the fall velocity of these particles in water of for instance 20° C is larger than about 0.3 m/s.
The sample is brought into suspension by shaking in a cylinder filled with water to a height of h cm (e.g. 40 cm). Now the particles are permitted to settle down during $\frac{h}{0.3}$ seconds (e.g. $\frac{40}{0.3} \approx 135$ seconds).
After this period all particles greater than 60 μ and a part of the finer fractions is lying on the bottom of the cylinder. Then the remaining suspension is carefully poured into another container and kept (in case some other analysis will have to be performed with these fine fractions). The cylinder is filled up again with water till h cm. The procedure is repeated several times. Finally only grains of a diameter greater than 60 μ remain in the cylinder.

To obtain the grain-size distribution of a sample, the following could be done:
Divide the sand part of the sample in two groups by sieving (dry or wet)
- coarse fraction
- finer fraction.

- For the coarser fraction of the sample the method of sieving is a common procedure
  - weigh the dried sample
  - sieve it, or in reverse order if impossible because of the sticking together of the particles after have been dried
  - make a grain-size distribution and plot this in a graph with a linear and a logarithmic distribution on the axis.
- For the finer fraction of the sample two methods can be used:
  - sieving if the sample is of sufficient weight (± 100 gram)
  - visual accumulation tube, if sample is small (see Section 4.2.4).

To obtain the grain-size distribution of the silt fraction of the sample a pipette method or micro sieves can be used (see Section 4.2.2).

4.2.1 Sieving

If sufficient sample is available a choice can be made out of the following possibilities:

a) dry or wet sieving
b) manual or mechanical sieving
c) several sieving machine principles
d) sieving time - and intensity.

For several countries standard sieves are in use according to the national norms:

Netherlands	N 480
Germany	DIN 4188
England	B.S. 410 1796
France	AFNOR NF XII - 501 - 1938
United States	ASTM E 11 - 39.

In the Netherlands the sieve analysis is divided in two parts, one part is above a certain particle size and the other below this particle size. The border lies at 40 - 100 (μm). Above this border the analysis is executed with sieves according to standard N 480, below this border with micro-precision sieves.

The normal sieving procedure goes as follows:

1. Split the sample into a suitable quantity for sieving (with sample splitter, or quarter method).
2. Put the sieves of the required sizes on top of each other coarse on top to fine at the bottom.
3. Under the bottom-sieve a catcher is placed.
4. A dried sample of at least 100 grams depending on the grain-size (measured on an analytic balance to an accuracy of 2 decimals), grinded so that no lumps remain, is put on the top-sieve and closed by a cover.
5. Put the whole set in a sieving machine.
6. Sieve for ten minutes; after 10 minutes the meshing and the sides of the sieve pan should be cleaned with a brush (check the meshing for clogged up holes).
7. Sieve again for 3 minutes.
8. From each sieve the amount of material is weighed on an analytic balance and the percentage calculated. (The percentage is obtained from the weights of each fraction.)

9. The amount of material which has fallen through the finest mesh and is on the catcher, should be weighed as well.
10. Make a grain-size distribution graph.

If the sieve rest in the bottom dish is less than 5% of the sample weight, no further analysis is necessary.
If the sieve rest in the bottom dish is more than 5% of the sample weight, its grain-size distribution can be determined by using the pipette method.

4.2.2 Micro-precision sieves

An accurate analysis of particle sizes < 100 µm is not possible with the normal sieves, and therefore such analysis has to be carried out by micro-precision sieves.

The sieve plate of a micro-precision sieve is made from nickel-plate in which electrolytic holes are made with an accuracy of ± 2 µm.
The sieve plate is mounted to a cylindrical shaped brash edge.
Sieves are available with a diameter of 75 mm, with openings of 3, 5, 10, 20, 30, 45, 60, 75 and 90 µm.

Micro-precision sieves are always used in a liquid, either water + teepol or aceton or white spirit.
The micro-sieves are used in a set, one on top of the other, the sieve with the largest opening on top.
Before sieving is commenced all sieves are dried and weighed.
After a sample has been brought in the top sieve, this sample is washed with water on top of the other sieves, so that the water drains through all the sieves.
The subfraction of the top sieve is transferred to a weighed dish.
Then the next sieve is "washed" and so on. Everytime the subfraction is transferred, a detergent (e.g. Teepol) may be added to the sample, but not more than 0.1% Teepol solution ought to be used.

If a sieve clogges up, further sieving of this particular sieve can be carried out in an ultra-sonic bath of 40 kc, but the danger exists that the particles will be broken.

4.2.3 Visual Accumulation Tube (V.A.T.)

The V.A.T. can be used to determine the particle size distribution of a sample quantity of sand (0.2 - 1.0 gram) with a median particle size of d = 0.05 - 0.4 mm (50 - 400 μm).
The principle is based on the stratified system, in which a mixture of particles of different size starts falling from a common source and becomes stratified according to their settling velocity.
For this purpose the increase of the deposited quantity of sand is measured as a function of the time. By means of a calibration the grain-size of a certain percentage can be determined by the settling time of that certain percentage.

The V.A.T. consists of a settling tube B (see Figure 4.2.1), inside diameter about 25 mm, with on top a cup A which is separated from the tube by a clamp-device on a rubber hose.

Under the settling tube is a capillary C in which the deposit can be measured with a measuring tape behind the capillary tube.

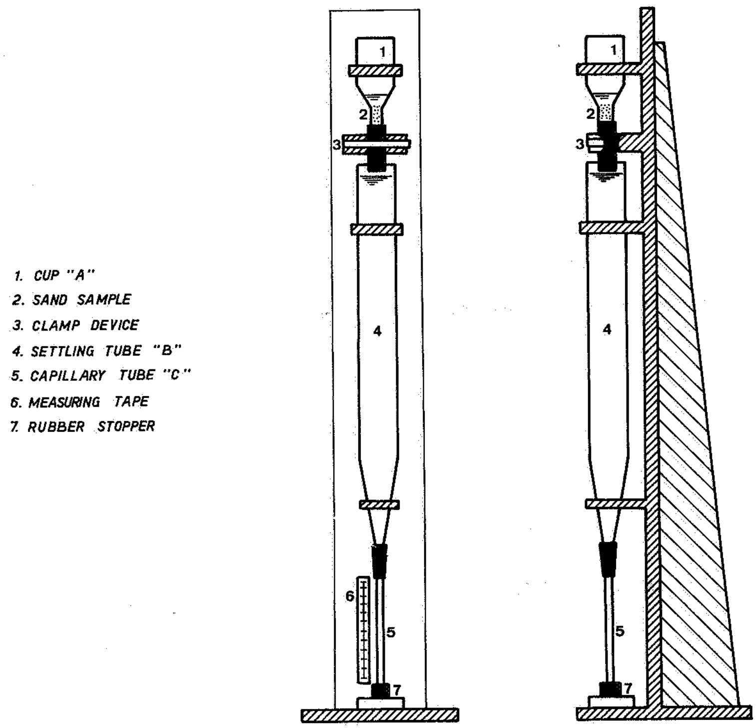


Figure 4.2.1 The V.A.T.

The procedure is as follows:

1. Clean the cup A and settling tube B with water while the clamp-device is open (after each measurement).
2. Close the capillary tube with a rubber cork in such a way that the top of the cork is level with a grid line on the measuring tape.
3. Fill the settling tube B via cup A with water up to 2-3 cm below the top egde of tube B and measure the water temperature.
4. Close the clamp-device.
5. Put the sample in cup A and pour some cm^3 water on this sample so that all the sand is under water.
6. Open the clamp-device and start the stopwatch at the moment the sand begins to fall in tube B.
7. Note the time when the first sand grains reach the bottom (top of rubber cork) and subsequently the times when the height of the deposit reaches, for instance, 0.5, 1.0, 1.5 cm, etc..
8. Determine after 10 minutes the total height of the deposit and calculate the percentages corresponding with the height of e.g. 0.5, 1.0, 1.5 cm, etc..
9. Determine the grain-size from the settling time corresponding with the percentages.

The V.A.T. should be calibrated with a number of samples with a know grain-size distribution.
Any change in the instrument or conditions (e.g. temperature) needs a recalibration. For deviations in temperature, correction factors for the grain-size can be determined.
Usually the accuracy of the size distribution is $\pm$ 5 à 10%.

4.2.4 Pipette method

The determination of the grain-size distribution, using the pipette method is based on the application of Stokes Law. In a settling column of suspended sediments, differences in concentration will occur as a consequence of the settling with different fall velocities of particles of various sizes on a certain level in the suspension (e.g. h cm below the surface) the concentration will decrease in time compared to the initial concentration. At a certain moment (t) the particles at depth (h) have diameters smaller than the diameter (d) calculated with Stokes Law in which are substituted the values of h and t.

The percentage of particles with diameter smaller than d equals:

$\frac{\text{concentration at time t}}{\text{initial concentration}} \times 100\%$.

The silt-content at a certain height is determined by taking a sample at that height with a pipette, drying the particles in an oven and weighing the dry particles on a balance (accuracy 0.1 mg).

The pipette method is suitable to determine the grain-size distribution of particles smaller than 60 (μm). The required equipment is:

- a perspex cylindrical tube 1 meter high, diameter 15 cm
- an accurate stopwatch
- a pipet of 25 ml
- equipment to determine sediment concentrations (see Section 4.3).

The sample or a number of samples together is brought into the perspex tube (settling tube) which has been filled completely with fresh or saline water and is stirred thoroughly.

Immediately after stirring and re-establishment of still water, a sample is taken 25 cm under the water-surface with a pipet of 25 ml (sample S_o).

After 1, 2, 3, 4, 8, 16, 30, 60, 120, 240 and 480 minutes (samples S_1, S_2 etc.) are taken in the same way.

The following data should be determined:

a. density of the silt suspended in the water and density of the water without silt in suspension (see Section 4.4).

b. temperature of the solution

c. height of the water column above the place where the sample is taken (normally 25 cm).

Now the content of sediment in each sample (S_o, S_1, S_2 ...) is determined by a colorimeter or by filtration and weighing (see Section 4.6.6).

The initial sediment concentration (S_o) should not be less than 1000 ppm.

According to the Law of Stokes the grain-size distribution can be calculated from the time concentration relation

$$v = \frac{1}{18\,\nu}\,\Delta\, g\, d^2$$

v = fall velocity in m/sec ($= \frac{h}{t}$, h = depth of sampling below water surface in m
t = time in seconds after the taking of sample S_o)

Δ = relative density

$$\frac{\rho \text{ grain} - \rho \text{ water}}{\rho \text{ water}} \approx 1.65$$

ρ = density in kg/m^3

g = acceleration of gravity ($= 9.81$ m/sec^2)

d = diameter of grain m

ν = kinematic viscosity in m^2/sec, is a function of the temperature (see Figure 4.1.3).

A graph for particles sizes ($\rho = 2650$ kgm^3), dependent on sampling time and using a sampling depth of 0.25 m in pure fresh water ($\rho = 1000$ kg/m^3), is given according to Stokes Law in Figure 4.2.2 With this calibration graph it is possible to determine the grain-size distribution. (Caution: this graph has been calculated for a temperature of 20^o C.)

A disadvantage of above described method is the fact that each time when a sample is taken with the pipette the settling of the particles is disturbed. This can be prevented as far as possible using the Andreasen-Esenwein pipette (see Figure 4.2.4).
The required volume of the suspension amount to only 350 cc. The initial concentration has to be at least 1000 ppm. With the three-way tap the sampling procedure can be executed very easily.
First the sample is lead to the pipette of 8-9 ml and then by turning the three-way tap the sample can be transferred to a dish.

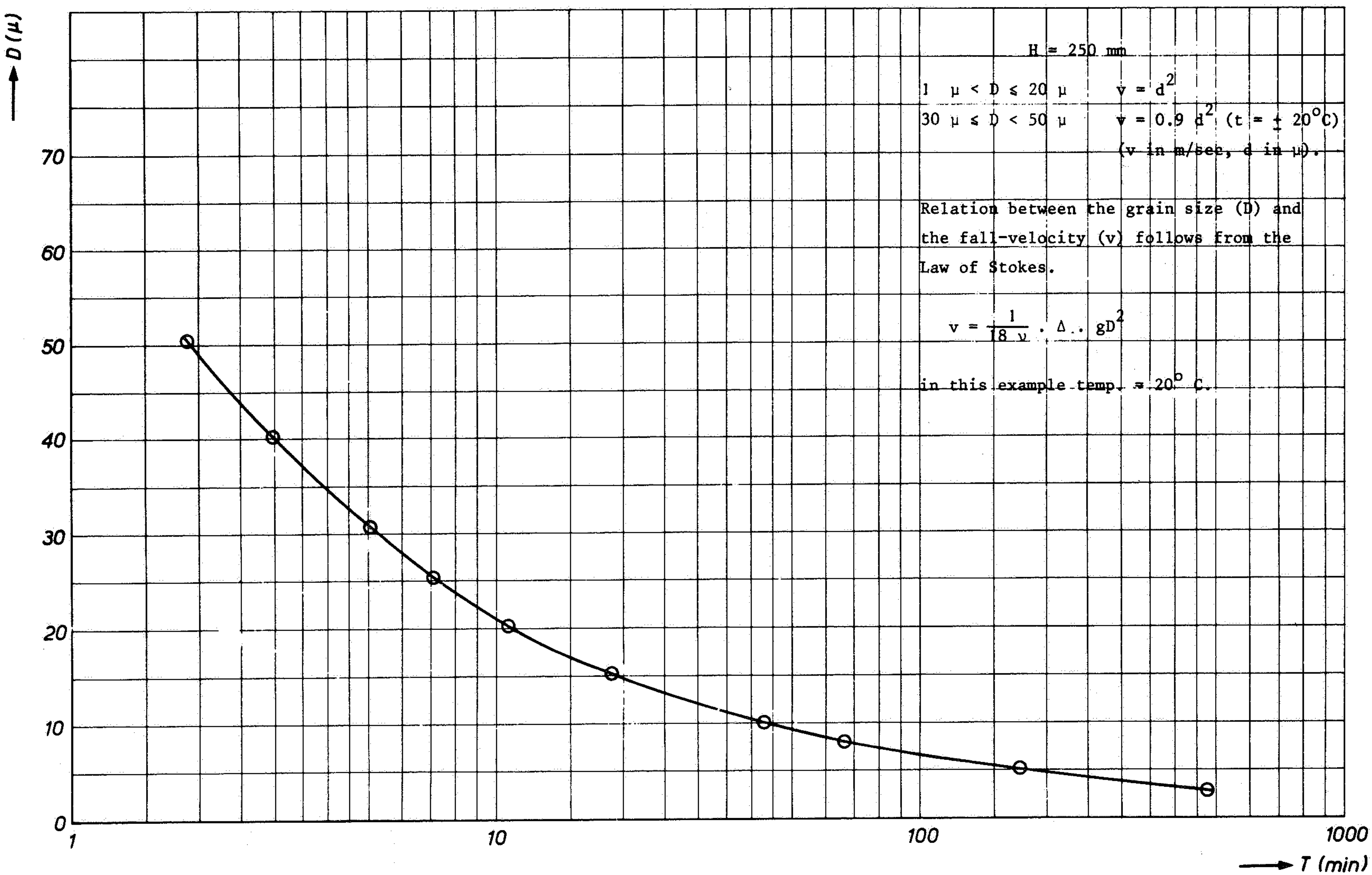


Figure 4.2.2 Calibration graph for grain-size determination according to Stokes Law

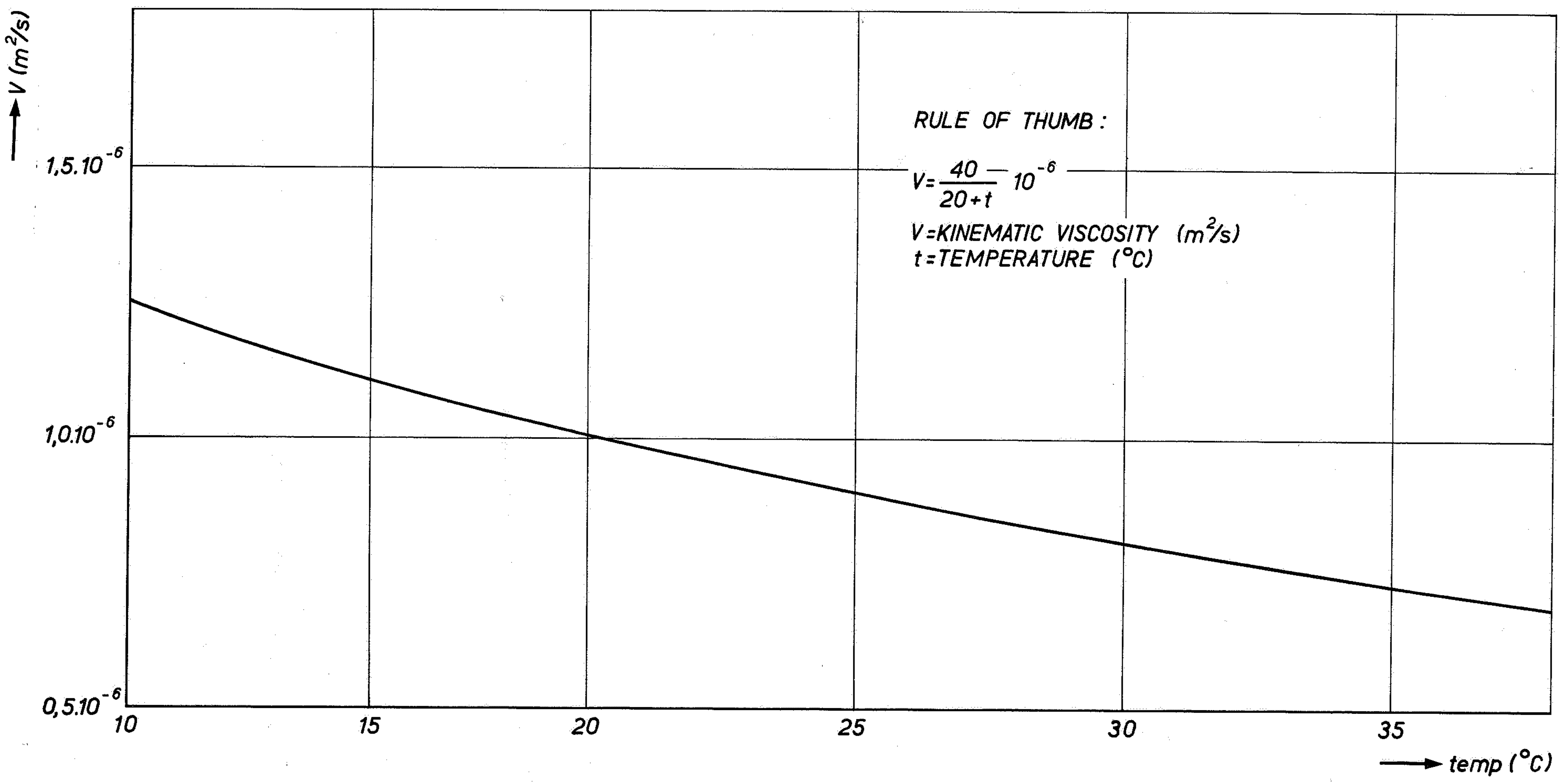


Figure 4.2.3 Graph of kinematic viscosity

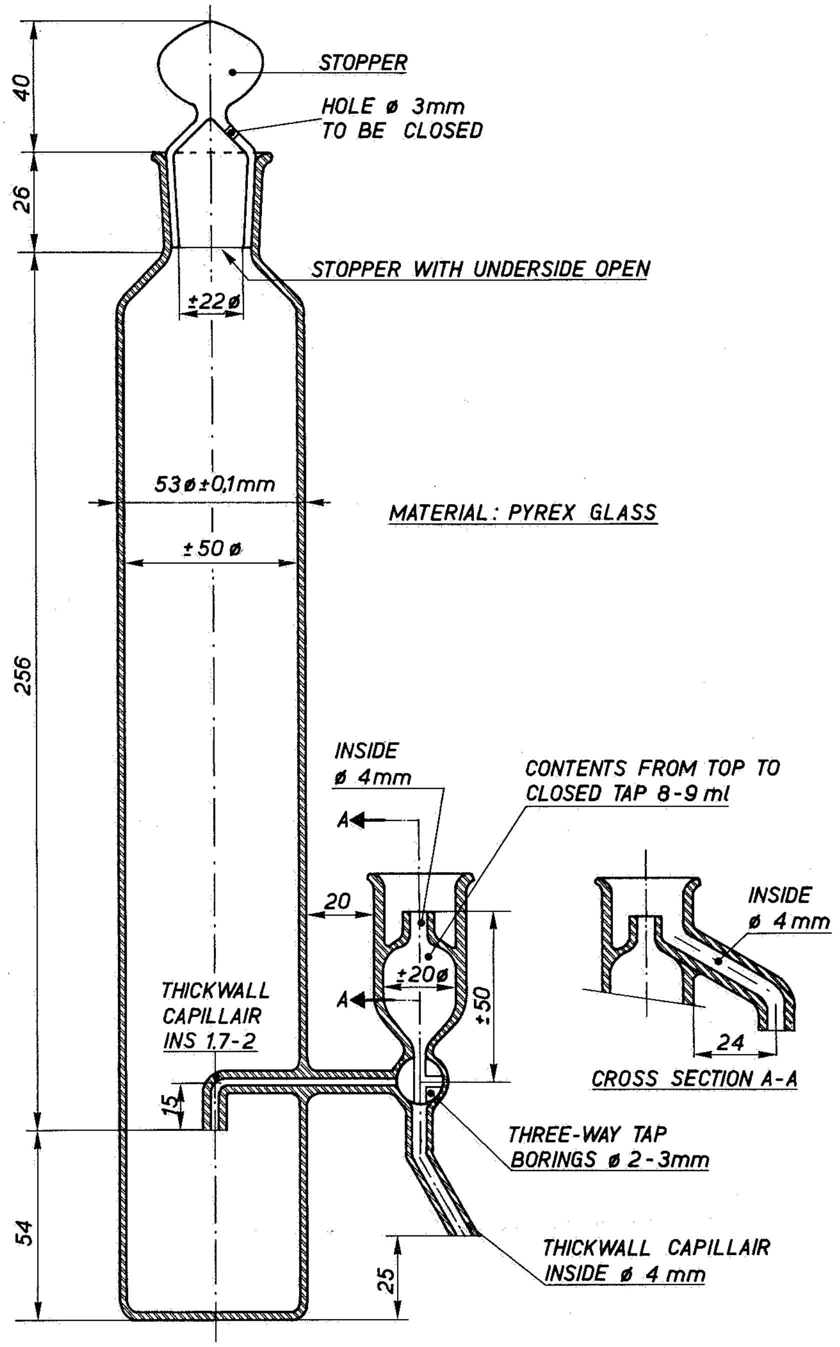


Figure 4.2.4 Andreasen-Esenwein Pipette

4.3 Analysis of suspended sediment

4.3.1 Determination of suspended sediment concentration

To determine the concentration of suspended- and or wash load, the sediment-content of a water sample or depth-integrated sample has to be analysed. There are two methods which can be used:

- evaporation method
- filtration method.

The filtration method is prone to a greater loss of material, while the evaporation method as a small ratio of sample weight to the tare-weight. Therefore, no hard and fast guide lines can be provided for their choice and for each case the usefulness of both methods should be judged on its results. If no vacuumpump and filtering glassware is available, the evaporation method is a good alternative.

4.3.2 Evaporation method

a. Determine the volume of the sample or the total weight of the sample (sediment + water) plus bottle to the nearest 0.5 gram and record this weight as gross weight.
b. Let the sample stand undisturbed so that the sediment will settle from suspension.
c. Decant the sediment - free liquid.
d. Wash the remaining sediment from the bottle into an evaporation dish, whose weight has been determined before up to an accuracy of 0.0001 g; be sure that no sediment is lost during the transfer.
e. Determine the weight of the empty bottle after drying, and record this weight as tare-weight.
f. Dry the sample in the evaporation dish until all visible water is lost, then heat the contents for one hour in an oven at 110^{o}.
g. Cool the evaporation dish in an desiccator.
h. Determine the weight of the dish + sample to the nearest 0.0001 g.
i. Substract the weight of the dish to get the sediment weight.
j. Substract the tare-weight from the gross weight to get the nett weight of the sample (sediment + water) and compute the concentration as equal to:

$$\frac{\text{nett weight sediment}}{\text{nett weight of water and sediment}} \text{ or } \frac{\text{nett weight sediment}}{\text{total volume of suspension}}$$

In dealing with coarse sediment particles this method is satisfactorily, with finer grained sediment, however, the settling time increases until a point is reached where the method becomes impractical.

4.3.3 Filtration method

The filtration method requires the following equipment:

- vacuumpump
- special glass-ware
- filterpapers
- analytical balance (accuracy 0.1 mg)
- oven.

The volume of a water sample is measured in a graded cylinder and thereafter filtered under suction through a filterpaper. Especially in humid climates it will be necessary to dry the filterpaper in the oven just before determining the dry weight of the filterpaper.
After the filtration the filterpaper with the sediments on top of it is dried again. The dry weight of the sediments divided by the volume of the sample gives the sediment-content.
The following procedure should be applied:

a. Filterpaper is dried in the oven.
b. The dry weight of the filterpaper is determined.
c. Numbered filterpaper is put in the funnels of the vacuum flasks.
d. The volume of the sample is determined.
e. The sample is poured into the funnel, the vacuum pump is started and the sample is filtered.
f. Distilled water is filtered in order to remove traces of salt (in case the samples were taken in a tidal region).
g. Filterpaper is removed and dried for three hours at 100° C.
h. Filterpaper with sediments are weighed immediately after having been taken from the oven.
i. Weight of dry sediments is determined.
j. Concentration in ppm is computed as $\frac{\text{net weight of sediment}}{\text{volume of sample}}$ $\begin{matrix}\text{(milligrams)}\\ \text{(liters)}\end{matrix}$

4.4 Determination of density of sediments

The density of sediments can be determined in a pycnometer or in a narrow measuring glass.

$$\rho_s = \frac{G\ \rho_w}{B+G-C}$$

in which: ρ_s = density of sediments in kg/m^3
G = dry weight of sediments in kg

ρ_w = density water in kg/m^3

B = weight pycnometer and water in kg

C = weight pycnometer and water and immersed sediments in kg.

In the case a narrow measured glass is used the water-level in the glass must be the same when determining the weights B and C. This could be effectuated by using a pincet and a filterpaper which sucks up the excess of water while determining weight C.

4.5 Presenting the results

The results of the analysis of the samples can be given in diagrams such as a cumulative grain-size distribution (see Figure 3.7.11) or a Piper diagram (see Figure 4.5.2 and 4.5.3).
Different parameters of the grain-size distribution can be calculated as they are required for different formulae concerning river morphology.
These parameters are e.g. median diameter, mean diameter, standard deviation percentages etc..

The results of the determination of sediment concentration can be illustrated in a drawing with on the x-axis the time scale and on the y-axis depth scale. For each time of observation the sediment concentration per sample is written at the specific depths at which each sample is taken.
Lines of equal concentration are then drawn and an overal picture of suspended load movement is the result (see Figure 4.5.1).

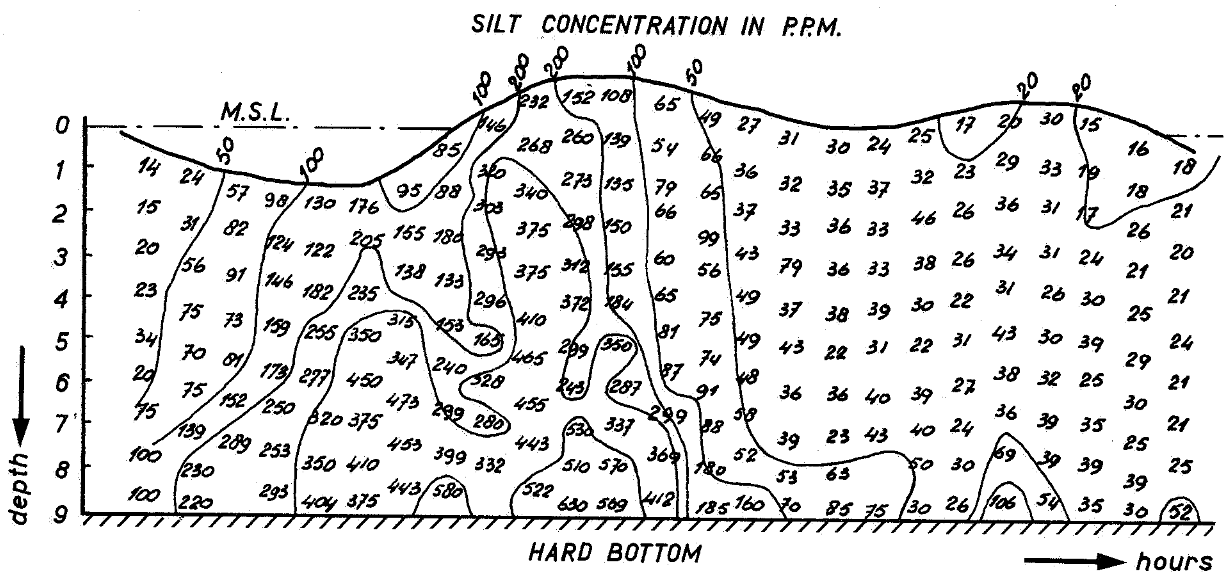


Figure 4.5.1 Suspended load concentration versus time

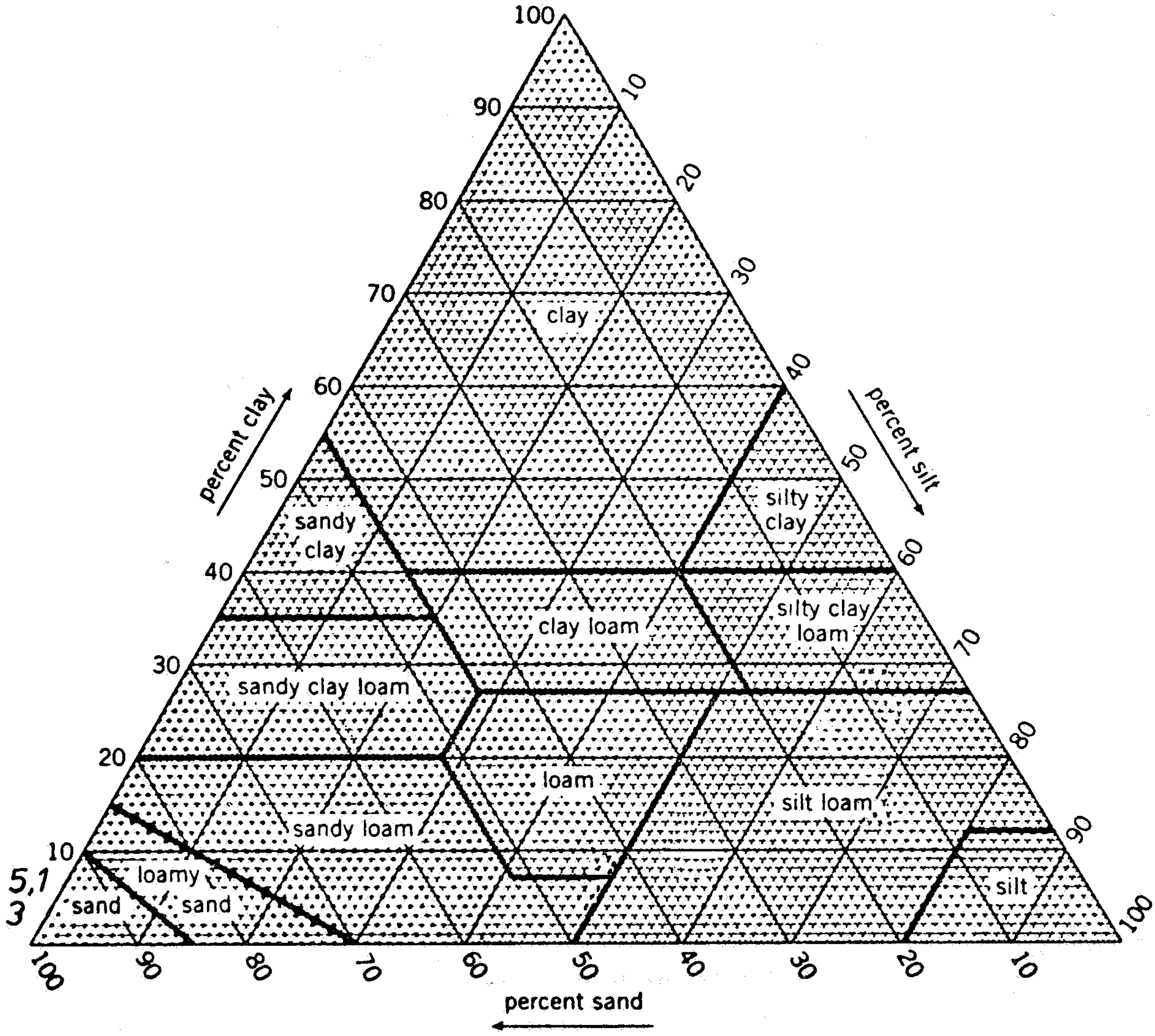


Figure 4.5.2 Piper diagram

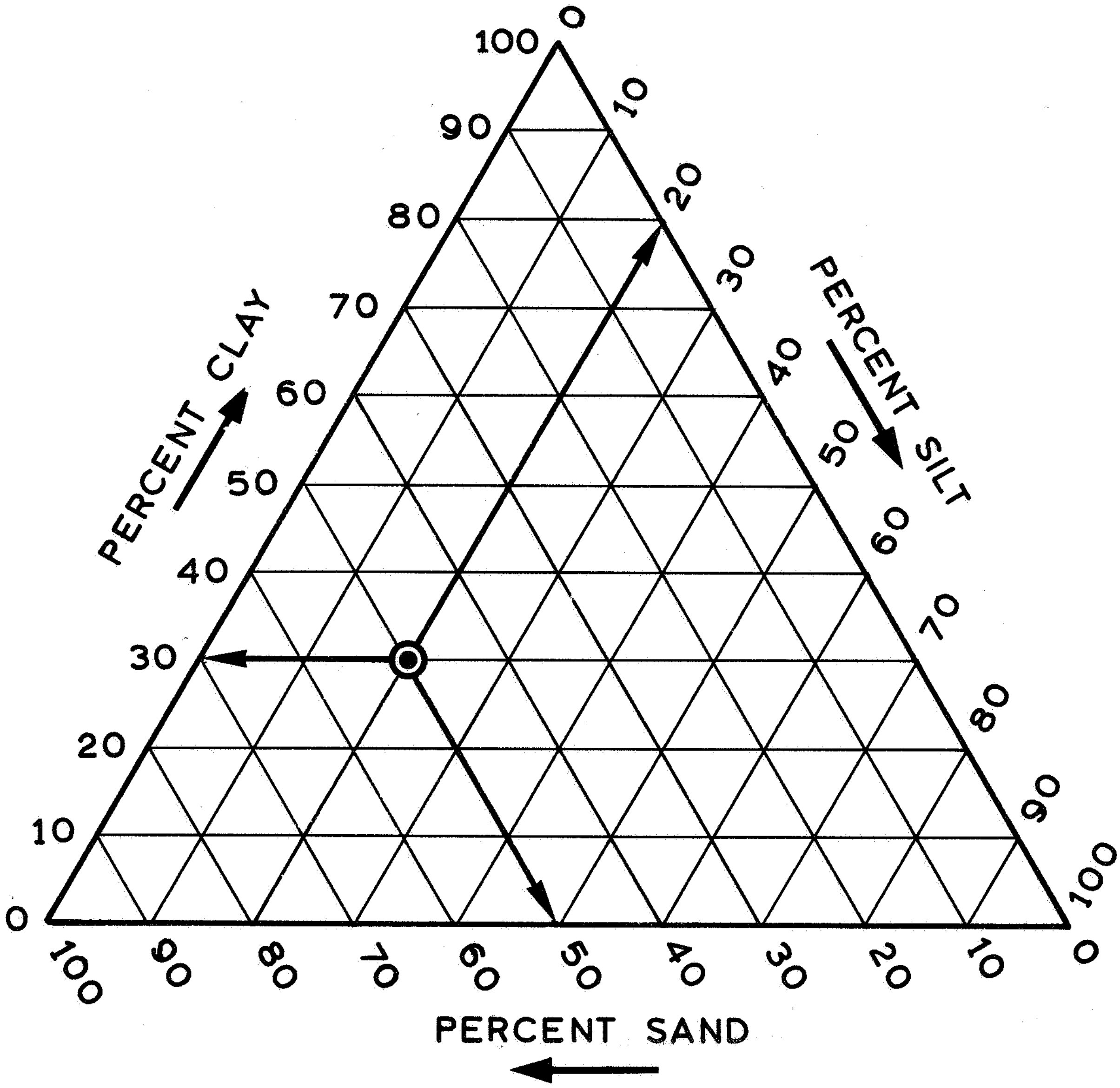


Figure 4.5.3 Example of reading a piper diagram