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DESIGN CONSIDERATIONS
OF
UNDERWATER PIPELINES

THESIS

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OF

UNDERWATER PIPELINES

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INTRODUCTION

A review of the developments in the trends in underwater pipeline construction over the years serves to illuminate an important point. That is, regardless of development, there is always room for improvement. As long as submarine pipelines continue to provide the most efficient means for moving oil in nearshore zones, and feasible applications in marine disposal of sewage, submarine aqueducts and power plant cooling water intakes; underwater pipelines will continue to expand at a proporationate rate. In the past, a principal deterrent to the use of submarine pipelines has been the cost of installation and a scarcity of available information on marine environmental design. However, with the development of new construction methods and equipment and rapid advances in marine research, submarine pipelines now can be more economically utilized for various purposes.

It is the intent of this thesis to study the diverse oceanographical and constructional factors that have a direct bearing on the planning of underwater pipelines. There is need for better definition between the nature and effect of the forces to which the pipeline is subjected, whether these forces are of the permenant type assaulting on a laid pipeline, or of a temporary character as those encountered during installation.

The thesis also presents the analysis leading to the final design concept, as well as presenting up-to-date information on construction technology and materials. Some emphasis is directed towards the analysis of waves and their action in relation to submarine pipelines.

Careful planning in advance may save a great deal of money, as well as help avoid severe headaches for engineers and owners. Installation of a submarine pipeline and keeping it in a safe condition against forces of nature, is a sort of job that requires much more in design and planning time than in construction. It might take years in preperation and be executed in few days. This planning depends mainly on the considerations outlined in the following study.

It is also the purpose of this study to point out, so far as possible, the source of the data which have been cited and the further sources of information which are available for a more complete study on the various phases of the subject. The broadest investigation and study are essential to a sound concept of the various problems which the engineer must meet.

CHAPTER ONE

GENERAL CONSIDERATIONS

A. Purposes of Laying Underwater Pipelines:

The term underwater or submarine pipeline is usually applied to pipes of any diameter and length laid on the bed of the sea with the majority of its length below low water.

Underwater pipelines have been used for many years, particularly in the oil industry where lines of thirty kilometers in length are not uncommon, They are used for the transportation of oil from tankers to shore or vice versa especially where there is shallow water nearest the shore.

With the advances made in construction methods and size of tankers, many ports discovered that they did not have the necessary installations to deal with the large tankers. One solution was the construction of artificial islands in the open sea to serve as discharging stations and a pipeline link between the island and the shore. Between the island and the tanker, the connecting pipes are usually of rubber hoses to provide the required flexibility and strength.

Cities located on islands or peninsulas may find out that their water supply should be delivered through submarine aqueducts or intakes. Large neuclear and steam power plants need huge volumes of water for cooling, and if situated on a coast, it will be found most economical to utilize submarine feed and return pipelines.

Nearly every coastal community in the world discharges storm water and sewage, raw or treated, into the sea. The effects of these discharges may be studied from the standpoint of both public health and public nuisance (grease, oil slicks, visible floating solids, odors, etc.). The principles of dilution, together with the aforementioned factors limit the minimum distance from shore. Sewage is carried to the diffusion points by an underwater pipeline system that should be designed and installed nearly the same way as oil pipelines.

In all these types of applications, as well as in other uses, the continuous operation of the pipeline is usually of great importance. Pipelines on the bottom of the sea or bay are difficult to inspect except during summer months, and are nearly always difficult and expensive to maintain and repair. In many locations they cannot be repaired during long intervals of time because of severe sea conditions.

B. Causes of Failures in Underwater Pipelines in the Past:

A knowledge of the types of failures that have occured in underwater pipelines and the causes for these failures is of value to this study.

The more important causes of failures are listed below (1):

1. Breaks caused to pipes struck by ships or ship anchors.
2. Damage by sagging when pipes crossed over boulders or ledges.
3. Sagging due to erosion of material from under the pipes.
4. Abrasion causing the loss of thickness of exposed pipes.
5. Corrosion of bolts and nuts (with the use of nickel steel bolts and nuts, this cause of failur has been eliminated.)

C. Shore Caracteristics:

By definition the shore is that strip of ground bordering any body of water which is alternately expossed, or covered by tides and waves. A shore of unconsolidated material is usually called a beach.

Shores are continually changing because of the movement of the unconsolidated material to and fro along the beach.

The various terms applied are defined in Fig. 1.

1. Descriptive classification of beaches:

A descriptive classification of readily accessible shores with respect to their constituent materials can be simply viewed as follows:

a. Sand, gravel, shingle and cobble beaches, or those consisting of coarse materials.

b. Muddy, silty, or clayey beaches, or those beaches consisting of fine-grained materials.

c. Bedrock and reefs, or those shores which consist either of rock which has been worn away by the waves to form a more or less gently sloping underwater offshore bench, or those which consist of coral.

The course - grained beaches have the common property of exposure to the open sea and the action of more or less large waves.

Beaches in area protected from large waves (such as in bays, lagoons, or marshes) are composed generally of fine particles, and constituents are not so well sorted. As a general rule, the finer the grain size, the greater is the water content, and the poorer the bearing capacity.

Many shores consist of rock which has been worn away by the waves to form a more or less gently sloping underwater offshore bench. Some beaches consist of firm rock and are smooth, but in others the bedrock consists of shale or clay which may be softened by the continual exposure to water, such soft bedrock is not uncommon.

Coral reefs commonly found in tropical waters are special types of rocky features. They are characterized by a broad shallow bench of coral rock, varying in width from a few meters to more than a kilometer or more. The surface of this rock is irregular and cut here and there by channels of deeper water.

Any appraisal of beach characteristics involves the three problems of statics, dynamics and stability. Statics refer to the physical state of the individual components of the beach, such as configuration, grain size, and strength.

Dynamics relate to the physical, chemical, and biological processes that control the beach and its constituent materials: i.e., the processes by which the components of the beach are formed or change once they have formed. Stability is related to the rate of change of the beach characteristics after formation. Stability is of particular concern in operating on shores because of the desirability of ascertaining whether the beach will or will not change significantly from one period of time to another, and if any change is likely to occur under what conditions, and to what extent it will occur.

2. Shore changes:

Shores change rapidly in character compared with many other geographical features. Nevertheless, these changes may be relatively slow, such as the erosion by waves of resistant headlands or coral reefs; they may be rapid as the erosion

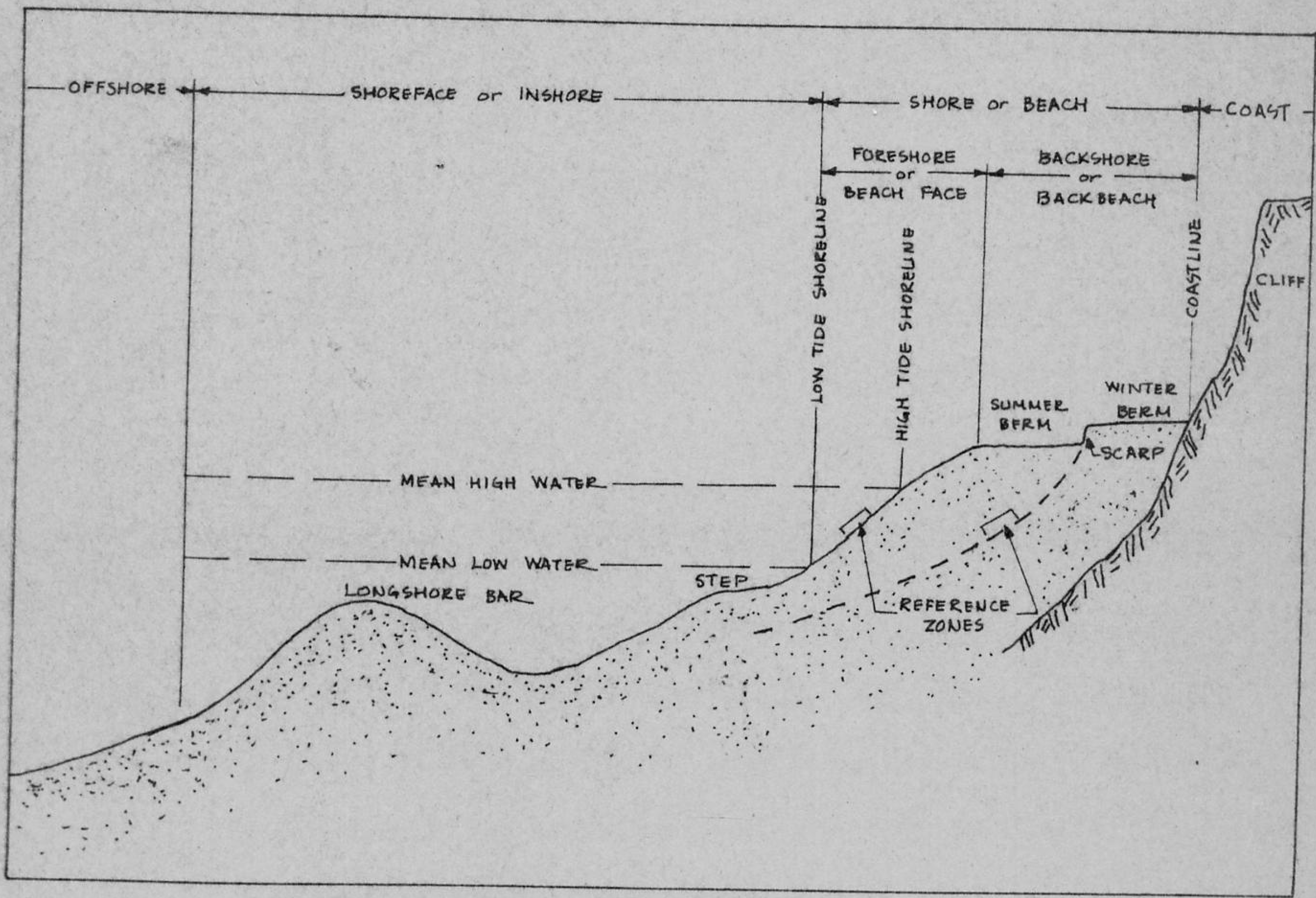

Fig. 1 Definition sketch of shore profile

of a beach foreshore during the course of storm which lasts perhaps but a few hours; they may occur at intermediate rates, like the building of dunes by winds along sections of beaches being furnished with a surplus of sidements by littoral drift; shore changes may also alternate, receding in winter and advancing in summer, or between spring and neap tides. Whatever the physical causes and whatever the rate, all shores are changing constantly.

The four primary factors that control the formation and configuration of a beach are the geomorphology of the land adjacent to the beach, the type and quantity of beach material available, and the waves impinining on the beach. Each factor gives the beach different characteristics, though they may be difficult to seperate. The first two factors tend to change only gradually, whereas the second two tend to change periodically over intervals of years, seasonally throughout the year, and even at shorter intervals, perhaps daily or hourly.

The first factor, geomorphology, depends upon long term geologic circumstances. The type of beach material is normally fairly constant for a given area. The type of material largely determines the slope of the beach face; the finer the material, the flatter the beach. The third factor, quantity of material available, is largely dependent upon rivers and streams carrying sediments to the ocean, and the erosion of land in contact with the ocean with subsequent transport of material. The fourth factor,the interaction of waves and beach, is responsible for the details of a beach. This phenomenon causes sand to move along the coast as well as onto or off a beach. It causes the rapid change in a beach and is the most difficult to predict or evaluate quantitatively.

D. Climatology:

Probably the most important factor to be considered in offshore construction is the weather. With present methods of construction it is the controlling factor most of the time, and as the line progresses seaward, it becomes even more problemitical. The occurence of an unforecasted storm can be disasterous to a contractor if he is caught a long way from shore.

An every day effect of weather is the waves resulting from strong winds. The heights of waves and the percentage of time that they occur vary according to the time of the year, depth of water and distance from shore.

Information on the direction, velocity, duration and frequency of the prevailing winds is of great value in the study of waves and currents since they are directly affected by wind. Wind waves are formed when wind velocity averages about 10 knots and over.

Littoral drift is frequently in the direction of the prevailing wind along the shore, and it may accumulate a large quantity of material about any obstruction similar to a pipeline.

All pertinent records of weather, temperature and precipitation records should be studied and thoroughly investigated.

E. Hydrographic Investigations:

The measure of a successful submarine pipeline study is the hydrographic survey and investigation.

Having determined maximum flows to be discharged, approximate pipe diameters can be established. The exact diameter cannot be established finally until the length and gradient of the pipeline is determined. However, having established approximate diameters, maximum and minimum radii of curvature (relative to maximum allowable stresses) can be determined for the pipe sections to be built on land. This assists in the selection of the construction areas.

The collection and examination of local information is most important. The type of information which can usually be obtained includes the position of shell fish beds; the set and direction of tides and currents; coastal erosion and whether beaches are making or losing; the study of old photographs and charts together with weather records, storm data, wave heights, wind data, etc.

The sequence of hydrographic investigations is as follows:

1. Echo sounding:

Having roughly established the extent of the pipeline the sea bed is surveyed by echo sounders.

Echo sounders with an accuracy of $\pm$ 30cm., are now available. These instruments are now calibrated to record the depth of water assuming a constant sound velocity of 800 fathoms (1460m) per sec. (2). However, since the sound velocity is a function of water temperature and salinity, corrections must be made.

The data collected are used to prepare a bathymetric map and a chart is prepared with contours at about 0.5 meter intervals, over an area extending a distance equal to the length of the whole line on each side of the pipeline. From this it is possible to assess the best alternative routes for the pipeline and the maximum depth of water available. Rock outcrops, if they exist, can be located on the echo sounding graph, nevertheless the alternative routes selected are investigated along their whole length by divers or frogmen.

2. Float tests:

Floats are released at various points along the pipeline route. They are released over full tide cycles on all ranges from neaps to springs and at varying wind directions. Their tracks are plotted on the contoured chart. The results obtained indicate shoreward drift, speed and direction of surface currents; also the most favourable positions at which to originate the remaining investigations as likely points of connection to stations or inlets. More information about float tests will be given in Chapter Three.

3. Underwater currents:

It has been found that surface currents or drifts can sometimes be quite shallow, and that over the same vertical plane, different directions and speeds of currents can exist simultaneously. This variation is usually measured by sensitive current meters.

4. Temperature readings:

Variations in sea temperatures are not merely confined to the seasons, but can vary considerably throughout the twenty four hours period. Therefore, information on temperature gradients from sea bed to surface within the surrounding area of the proposed location of pipeline must be obtained. It is valuable in the design of the pipe in relation to thermal stresses.

F. Marine Soil Explorations:

The purpose is to determine the type of material over, or through which the pipeline must be pulled or installed.

1. If the echo sounding survey has shown that there is likely to be rock outcroppings along the route, then it is necessary to determine exactly the location of such outcrops. This is done by sending down a diver or a forgman to walk the route of pipeline and take samples either by checking or boring.

2. Jet probing:

Jet probing permits the geologists to determine the thickness of bottom sediments, and the general type of sedimentary material found beneath the sea floor in the area of the probe. The probe itself consists of a four-meter long pipe of 1.90cm. diameter which is attached to a hose furnishing high-pressure water from a pump located on a tending boat (3). With experience, a geologist using the jet probe can tell whether the pipe is jetting through sand, gravel, shells or clay, or hitting bed rock by the sound and feel of the pipe.

3. Sparker seismic system:

This system is an innovation for studying the bottom and subbottom structure of the oceans. It is similar to an echo sounder in that it is used for determining water depth. The Sparker, however, introduces a more powerful sound pulse from an electric discharge in the water to produce a high energy, low frequency, relatively broad-band pulse. The acoustic energy reflected from the sea bottom and differing subbottom horizontal layers is detected by a towed hydrophone array, then amplified, filtered and recorded on a moving paper chart presenting a vertical cross section profile of the geologic structure beneath the track of the survey vessel.

The Sparker seismic reflection method utilizes the effect that a discontinuity between two strata has upon the transmission of sound. Thus the Sparker delineates the geologic structure underneath the sea bottom, giving information about the vertical thickness of sediments and their stratification.

4. Coring:

Samples are obtained by a piston coring device, or the more lately devised Vibracore which is a pneumatic-powered coring device designed to vibrate itself into the sediments up to 6m, depth and collect a cylindrical core of the bottom and sub-bottom materials. The Vibracore provides a sample of the quality of sediments at different depths.

Samples are evaluated by laboratory tests to identify and establish specific gravity, water content and other index properties of soil. Usually the unconfined compression test is performed to measure the shearing strength, and a consolidation test to measure the propable maximum settlement expected. By computational procedures the frictional coefficient of the sea bed medium is determined to assist in calculating the pulling force required.

If it has been found that the underwater currents acting transversly to the pipeline are so strong that trenching is needed, then borings must be taken at regular invervals along the route.

If the bottom should contain boulders and the depth of water is such that it is thought likely that distrubance due to wave action could extend to the sea bed, perhaps causing the boulders to move and in consequence damage an unburied pipe, the following action could be taken (4).

Samples of marine growth are taken from the base of the boulders and these are examined by marine biologists who are able to determine the age of the growth. To illustrate the technique let us assume that the growth was definitely not less than 3 years old, then by checking the weather records over the last 3 years it could be established whether or not a severe storm had occured. If a storm of any magnitude had occured within that period then it would be reasonably safe to assume that it had failed to move the boulder and in consequence it would be safe to lay an unburied pipe.

The presence of mud in the bottom of the sea is an indication of the absence of wave action, as mud is readily eroded and washed away by such action. The absence of mud is not an indication of wave action as conditions may not have been fovorable for its formation.

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CHAPTER TWO

CONSTRUCTION TECHNIQUES

The techniques of underwater pipelaying have in common the fact that the pipelines is constructed at some point remote from the position at which it is to be laid, and no jointing or construction is done on the sea bed. The construction site would be either on land or afloat on a barge. The handling and moving of the pipeline from its construction location to the sea bed clearly involves considerable stressing of the pipeline and it is primarily for this reason that the design of the pipeline has to be closely studied with, and related to, the method of installation.

A. Pipe materials, their uses and fabrication:

The principal pipe materials used for submarine construction are steel, cast iron, plastic, wrought iron and asbestos bonded corrugated metal pipes. Relative economy plays a big role in the selection of pipe material, with due consideration to the construction method adopted.

1. Cast iron pipe:

Cast iron pipe is used because of its high resistance to corrosion and consequent long life. This type of pipe is manufactured with either the conventional bell and spigot joint or a ball and socket joint. Because of the rigidity of the bell and spigot joint, this type of joint would require the driving of piling in water up to 100 meters depth.

The ball and socket, on the other hand, permits a relative deflection of 15° without leakage, resulting in a very flexible line. The joint further allows the pipe to be towed into place with pulls of up to 84 tons without over-stressing. This may be sufficient to permit the entire line to be pulled, if skids are placed at each joint to minimize drag.

It is apparent that the ball and socket cast iron pipe has several distinct advantages. The great degree of flexibility takes care of settlement, minor undercutting, and other irregularities better than any other type, as the pipe will tend to follow any minor variations in the ocean bottom. However, cast iron pipe is the most expensive of the various materials listed. This greater cost cannot be justified on the basis of good corrosion resistance alone. The thick cast iron pipe is undoubtedly more corrosion resistant than unprotected steel pipe, but it would not have a longevity equivalent to concrete or well protected steel pipe. To protect the cast iron pipe with a coating would be rather difficult and unreliable due to the large flanges and inherent flexibility of the joints.

2. Wrought iron pipe:

Wrought iron pipe has been used for several sewage outfalls. Joints may be welded, or dresser and victaulic couplings can be used.

Wrought iron is quite expensive but has been claimed to have corrosion properties even better than cast iron, particularly in salt water environmments, protective coatings when warranted can be applied to wrought iron welded pipe with relative simplicity.

3. Asbestos bonded corrugated metal pipe:

This pipe has been used for various sewage installations over the past 25 years and has apparently given excellent service. A recent development in the manufacturing process made possible a smooth interior thereby providing the same friction characteristics as other smooth pipes. It is not, however, suitable for high pressure service.

4. Plastic pipe:

During recent years, developments in the field of plastics has made possible the manufacture of plastic pipe in sizes up to 90 cms. diameter. Some manufacturers have developed a resin and fiber glass laminate resulting in a competitive pipe placed in corrosive environments. While the pipe is substantially more expensive than either protected steel or concrete, its lightness (25kg/m) will facilitate handling and may reduce installation costs. Several types of joints can be developed to permit the use of any method of laying. Its lightness, however, would be a detriment once installed, requiring the extensive use of anchors at frequent intervals to hold the pipe in place.

5. Concrete pipe:

Concrete pipe in small diameters has not been used extensively in underwater pipeline construction. The special care and additional effort required to make up underwater joints have generally led to the use of cast iron or steel pipe, even so concrete pipe is far less expensive initially.

For a design head of 75 meters, the type of concrete pressure pipe normally used is known as steel cylinder. This type is a steel cylinder designed to withstand the internal pressure with an 18mm mortar lining and a minimum of 18mm gunite coating. It is manufactured in lengths of 9.0 meters and utilizes the lock - joint steel ring joint. This pipe has an inherent disadvantage in that when laid directly on ocean bottom, portions of the steel joint rings would be exposed to the action of sea water.

Many attempts were made at manufacturing a reinforced or prestressed concrete pipe, with emphasis on the jointing systems. The objection to the conventional bell and spigot system is the protruding portion of the bell which creates a difficulty in construction. Some of the joints used in past projects are illustrated in Fig. 2 and 3.

6. Steel pipe:

Steel pipe has been used almost exclusively in the oil gas industry of installations comparable to sludge outfalls. The pipe is manufactured in lengths up to 15 meters and wall thickness up to 13mm. They are usually butt-welded together into lengths depending upon the space available on the construction site, generally not exceeding a kilometer, and not less than 250 meters. Yeild strengths up to 3600 kg/cm2 are readily attainable at a relatively smqll premium cost. This type of pipe allows a great degree of flexibility in construction techniques, particularly in designs utilizing the long section method of installation, It is estimated to be more expensive than concrete pressure pipe but should be capable of much more rapid fabrication and installation.

The obvious disadvantage to steel is its limited

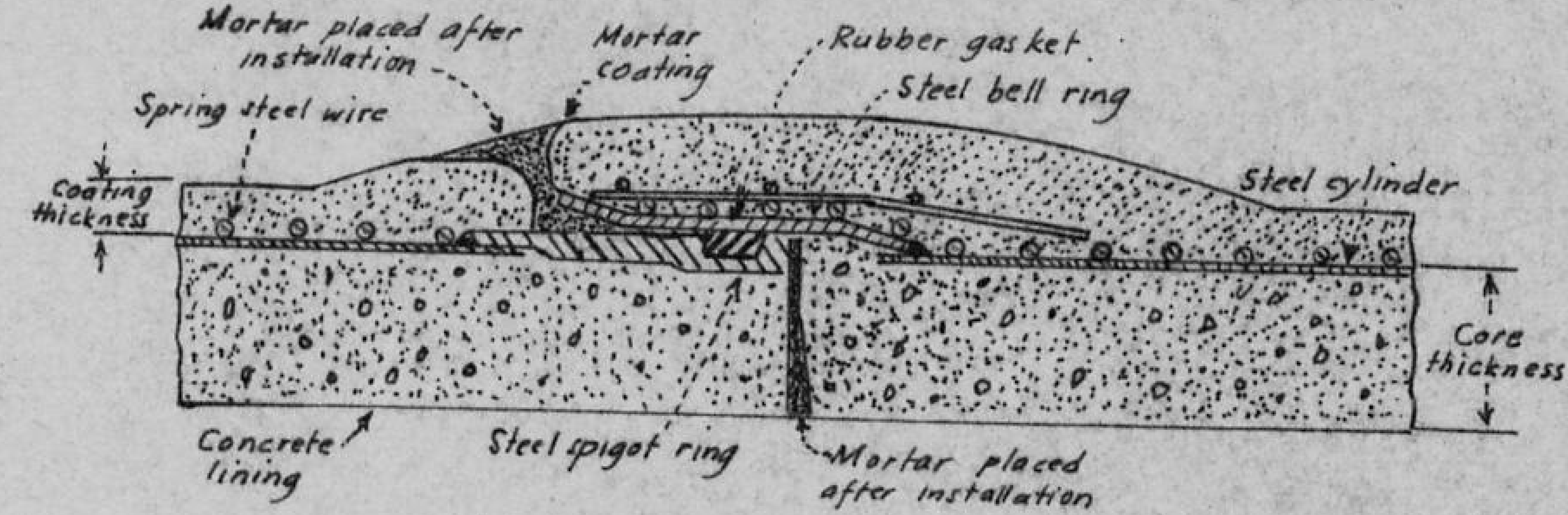
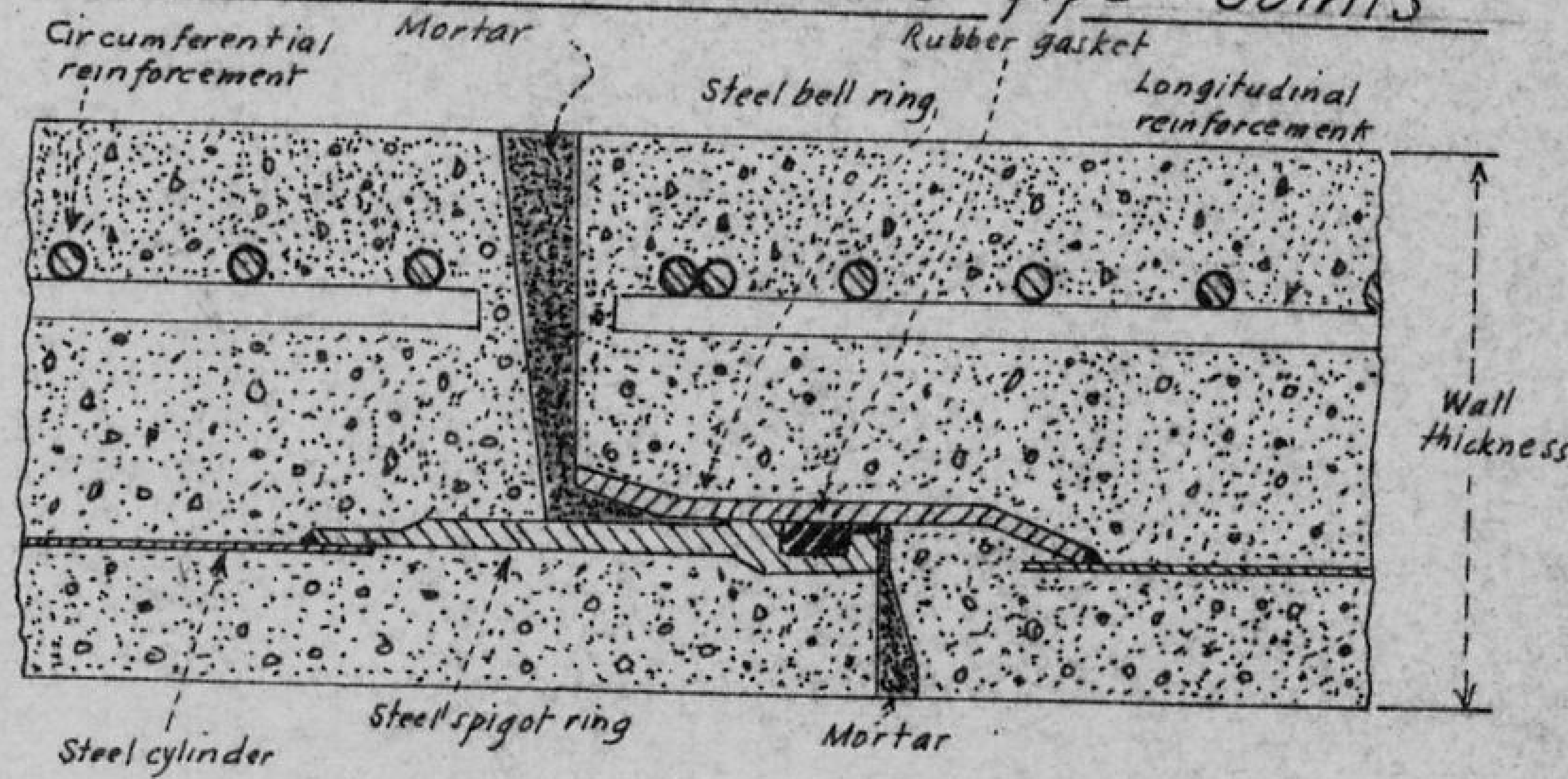
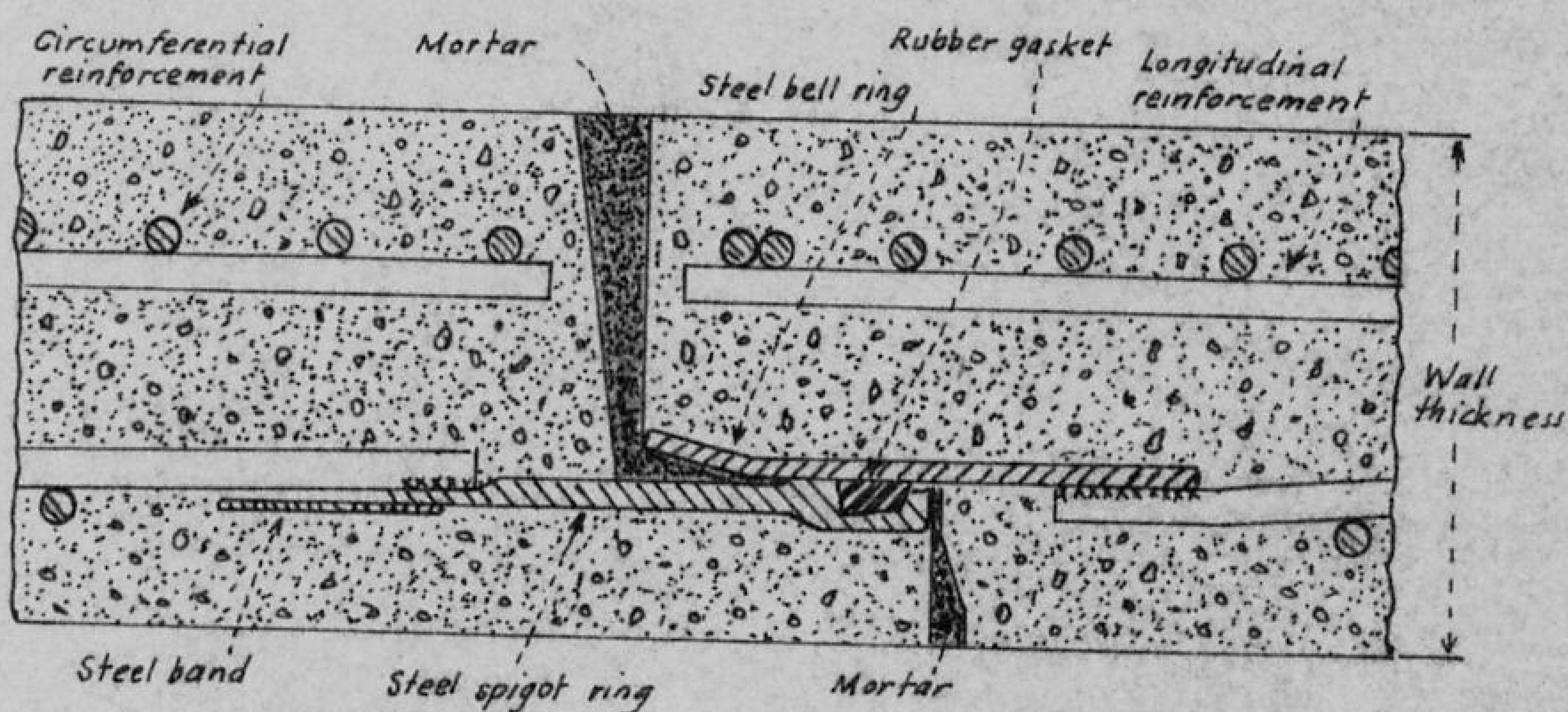


Fig. 2 - Prestressed concrete pipe - Joints



Concrete Cylinder Pipe (not prestressed): rubber and steel joint



Concrete Pressure Pipe: rubber and steel joint

Fig. 3 Concrete pipe:- joints

resistance to corrosion, particularly in a sea water environment. There are several well tried methods of protection, however, which include protective coatings and cathodic protection or both.

B. Pipeline Corrosion Protection:

Underwater Pipelines are usually designed for a long design life in the range of 60-100 years. Investigations into corrosion protection to insure the required 100-year life must be thoroughly run. The problem of corrosion protection resolves itself into two parts, namely interior and exterior corrosion .

1. Interior corrosion:

For sludge or sewage outfalls some type of protection is necessary because of the existence of oxygen in the effluent. During the construction period and until the line is actually placed in service, the interior may also be subject to the corrosive action of sea water and in a limited way to the action of marine life. The three types of lining used most commonly at present for interior protection are coal tar enamel, epoxy resin and cement mortar.

a. Coal tar enamel, applied hot to a thickness of 1/16 to 3/32 inch cover over a primer has established records of satisfactory water works service approaching 40 years. Coal tar enamel has a flow coefficient slightly superior to that of cement mortar. The main problem with this lining is the difficulty of applying the enamel in the field, if two long sections are to be joined, as required by the long sections method of construction.

b. Epoxy resin lining is applied under pressure after welding has been completed. The total thickness of lining varies between 15 and 25 mills (thousandth of an inch) applied in coats of 5 mils each. The advantages of epoxy resin are that it is flexible, easily applied, and has a very high antiabrasive characteristics.

c. Cement mortar lining has been used extensively for both water works and sewage mains with excellent results. A cement lining is not wholly imprevious to water, but water that penetrates forms an alkaline solution in contact with steel which tends to inhibit corrosion. In addition, if any cracks develop in the lining during spinning, the phenomenon of autogenous healing will probably heal most of the cracks, provided the lining is continuously wet. If welded joints are used, precautions must be taken during the welding of the field joints to prevent damage to the lining. Cement mortar is slightly cheaper than coal tar enamel lining.

2. Exterior corrosion:

It is well known that steel pipe left unprotected in sea water would have a very short life. The corrosive action in salt water may be accelerated by marine growths, such as barnacles and bacteria, or by stray currents making use of the excellent electrolytic properties of sält water. In addition, any abrasive action of the surrounding sand tends to accelerate the process of corrosion. It is obvious, therefore, that some type of coating or other type of protection is essential.

The investigation of coating used for similar steel pipelines indicate that there are four coating materials which are most common.

a. Coal tar coating: This type of coating is usually applied by layers of tar reinforced with fiberglass. This is usually applied to individual pipes either on the site or at works, and after welding the joints are coated with similar materials.

   The main disadvantage to this type of coating is due to its creep characteristics under load and need for very close inspection during application.

b. Somastic coating: Somastic is a dense mixture of asphalt mastic which is applied to the pipe by a continuous process as uniform seamless coating. It is applied very thick (12-18mm) in order to insure complete waterproofing. Field joints can be made easily resulting in continuous protective shield along the pipe. When used on a subaqueous pipeline subject to bending resulting from variations in the ocean bottom, the coating may crack and pop off the metal surface, leaving exposed areas subject to corrosion.

c. Plastic tape: A relatively new type of wrapping is the cold supplied plastic tape consisting of a laminate of plastic and butyl rubber. Tests indicate that these tapes are tough and pliable and have excellent moisture resistant properties. They are also high in dielectric strength, insulation resistance, and chemical resistance. This tape makes an excellent pipeline coating and is now priced to be competitive with applied coatings. The tape is manufactured in widths of 30cms. and is wrapped over a prime coat.

d- Gunite coating: Gunite coating is usually applied over the bitumen or tape either by spraying pneumatically or by being placed in shutters.

Gunite coatings have been used sloley as the means of protection on water lines, but these have always been buried lines on shore. However, while each of the three coatings previously mentioned are effective in protecting the steel pipe, a gunite coating is usually added for several reasons: First, concrete is the only known pipe coating material which resists penetration by barnacles and which is unaffected by all forms of marine life. Secondly, it is generally desirable to have as heavy a pipe as possible both from the standpoint of stability in place and in order to provide the proper negative buoyancy for laying operations. It is more economical to provide this extra weight by increasing the thickness of the gunite coating rather than thickening the steel wall. In order to keep the thickness of the gunite to a reasonable value, the use of heavy aggregates, such as barium sulfates, is sometimes justified to produce concrete weighing about 3 tons per cubic meter.

3. Cathodic protection:

Cathodic protection is an important part of the system to insure the achievement of the anticipated design life of a submarine pipeline. Fundamentally this process consists of artificially introducing an electropotential in reverse to that existing naturally between the pipe and the moist soil or water. This is accomplished by supplying electric current by the use of sacrificial anodes or rectifiers.

The sacrificial anodes method consists of connecting a metal anodic to the metal to be protected, depending on the natural potential of metals. All metals have a natural potential. A part of the galvanic series together with each metal's potential is as follows:

METALS	VOLTS
Magnesium	- 2.34
Aluminum	- 1.67
Zinc	- 0.76
Iron	- 0.44
Brass	- 0.28

METALS	VOLTS
Brass	- 0.28
Hydrogen	0.00 Reference
Copper	+ 0.34

Using hydrogen as a reference, the metals have been classified as to negative and positive voltage in proper sequence. Connecting any two of these metals together in an electrolyte will cause current to flow resulting in the corrosion of the anode metal. Magnesium is the most anodic of common metals, hence it is the most popular type of anode. The potential difference in voltage is the aljebraic difference between the anode and cathode potentials. Although the difference in potential between Magnesium and Iron is 1.90 volts, the actual measured difference in the field varies between 0.90 volts and 1.50 volts depending on the electrolyte, whether water or soil. The potentials may be measured by a voltmeter.

The Magnesium anodes, usually made of cylindrical castings weighing 15 kgs., are placed in specially prepared backfill enclosed in cloth bags and buried within 3 meters of the pipeline and at a depth such that the top of the anode is at the same level as the invert elevation of the pipe. One disadvantage of using Magnesium anodes is that anodes have to be replaced every 10 or 20 years depending on the environment in which they are installed.

The rectifier type of cathodic protection consists of applying direct current to the sea bed, which is in turn transmitted to the pipeline. This type is less expensive when large amounts of current are required. The break-even point between the use of sacrificial anodes and rectifiers is between 2 and 3 amperes. When more current that this is required to protect the pipeline, it is usually more economical to install a rectifier station.

The general criteria for determining the need of cathodic protection is that any soil with a resistivity of less than 10,000 ohm cm. can be considered as corrosive. Soil with a resistivity of less than 1,000 ohm cm. is extremely corrosive while soils with a resistivity of over 10,000 ohm cm. are usually non- corrosive. The ohm cm. which is the basic unit of resistivity is the resistance in ohms of one cubic centimeter of the material in question (1).

It is possible, in some cases, to protect the steel pipe in sea water without the use of coating, but it is generally more economical to use a combination of a coating and a cathodic protection system.

At times it may be desirable to consider applying cathodic protection to a previously coated pipeline if the coating appears to be deteriorating. The criteria for making this decision is the coating leakage resistance of the pipeline coating. This is determined by applying a temporary current to the coated pipeline using batteries and a ground bed. Test leads are installed approximately 60 m. apart along the pipeline. Voltage readings are taken and the current flowing in the pipeline is computed using Ohm's Law. At the same time, pipe to soil potentials are taken with a copper sulfate cell between the two ends of the section under consideration. The total resistance of the coating in ohms is calculated using the pipeline current and pipe to soil potentials, The resistance is multiplied by the total area of the section to be protected. A criterion of 30,000 ohm sq. m. is considered good coating leakage resistance. Any coating with a resistance of less than 10,000 ohm sq. m. should be considered for application of cathodic protection. If repairs are decided upon, they are usually made be experienced divers or frogmen.

C. Testing of Materials and Workmanship: (2)

1, Ultrasonic tests:

Before pipes are fabricated the plate from which they are made is batch-tested ultrasonically to detect laminations and piping.. It will be appreciated that due to the heavy stressing of the pipes during launching and the marginal tolerances allowed for in design, it is essential that the material of the pipe wall should be absolutely perfect. In addition to ultrasonically batch-testing of the steel plate prior to manufacture, after manufacture the pipes are similarly batch-tested.

2. Stress testing:

An additional works test is stress testing, where sections of pipe are cut at random and stress tested to destruction.

3. Radiography of welded joints:

It is imperative that every weld on the pipeline is checked over 100 per cent of its length, This is achieved by radiography using gamma rays from Iredium 192. Radiography of welded joints is carried out both at the works and on site. For the sake of brevity, the system can be described as taking x-ray photography of every weld and examining the negative for flaws. A welding supervisor is employed to examine the negatives and also to test and grade the welders themselves periodically.

4. "Holiday" detection:

This is an extremely important test and is carried out with great patience and skill. It is used to test the coating and lining materials for continuity and to make absolutely certain that there are no pin-holes, or, as they are called, "Holidays". A very high voltage is applied to the lining or coating through a metal brush . The slightest flaw or holiday, even though it be invisible to the naked eye, will cause a spark to jump from the brush to the pipe wall. This is accompanied by a loud crackling, and an audible warning signal, and at the same time a severe voltage drop is recorded on the apparatus. This apparatus is portable and can be carried over the shoulder in much the same way as modern transister sets. Each flaw or holiday is marked, repaired and checked again. This process is repeated on three separate occasions, the last one being immediately prior to the placing of reinforced concrete.

The lining is checked in much the same way but the holiday detector assembly is a self-contained unit which is placed inside the pipe, the metal brushes being cylindrical and baffle plates at each end enabling the assembly to be blown along the pipe by compressed air.

The holiday tests prove that the linings are intact and also that they will provide sufficient insulation to keep the current applied for the cathodic protection down to an absolute minimum.

5. Electrical tests:

Immediately prior to and after launching, electrical tests are applied to determine the amount of current which will be used for cathodic protection during the lifetime of the pipe. However, the amperage required is a fair indication as to the efficiency of the lining and coating and can therefore be considred as a further test after installation.

6. Pressure tests:

The pipes are inevitably pressure tested before and after launching either hydraulically or with air, but more usually with air according to anticipated operating pressures.

D. Pipe-laying methods:

1. Launching-pulling technique:

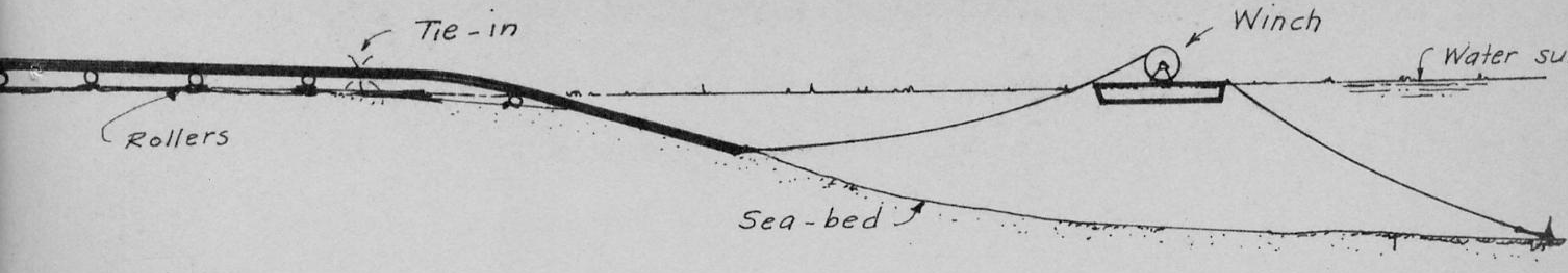
This method is normally applicable on lines up to ten kilometers in length and often used for the medium distance lines.

Lengths of pipe in sub-multiples of the total length of the line are constructed in their entirety on land: for instance a two kilometer outfall may be divided into three sections. The number and hence the length of sections depends upon the construction site and the area and length available. It is not necessary to build the pipes in a straight line on the projected line of pull. Fig. 4 shows six sections built up on land prior to launching.

a. Pipe conveyers:

For launching, the first section of pipe is placed on specially designed self-balancing pneumatic conveyors, the pipe resting on very heavy duty pneumateic tyres with further sets of the same type of tyre which can be adjusted to embrace either side of the pipe. These adjustable tyres enable various diameters of pipes to be contained. The conveyors, being self-balancing, with a "see-saw" action enable undulations on the construction site to be taken up. The adjustable side wheels enable transverse support to be given when curving the pipe on land.

The first pipe is pulled into the sea, along the sea bed by a winch mounted on a barge and anchored offshore some distance away from the final outfall position. The second section is then joined to the first and is pulled in by the same method. The procedure is then repeated



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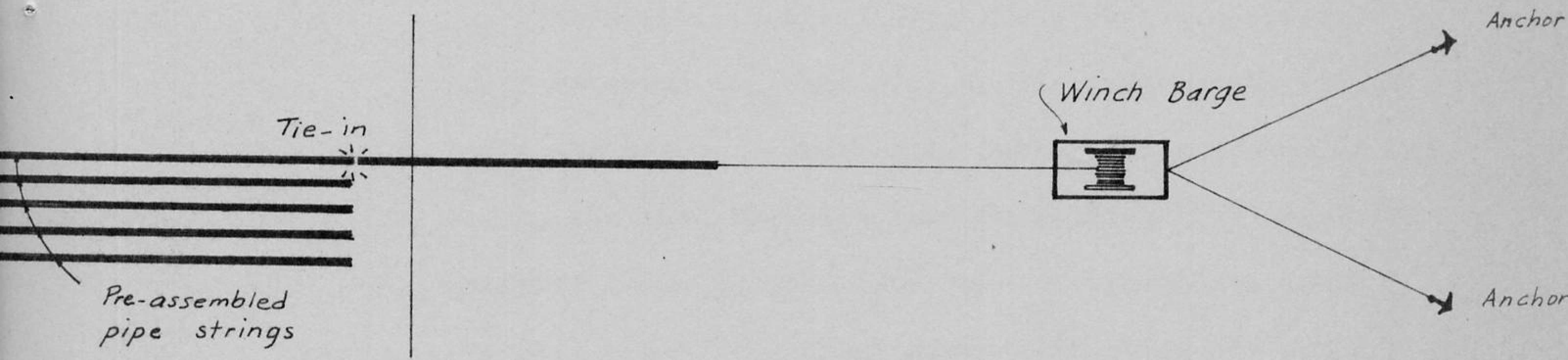

PLAN

Fig. 4 Launching - pulling technique

for all subsequent section. These winches are also specially designed for submarine pipeline projects, and are usually within the range of 75 tons direct pull of the drum to 200 tons direct pull at speeds of up to 18 meters per minute.

The barge should be anchored carefully since this may mean the success or failure of the whole project. The anchors not only have to withstand pulls of sometimes up to 200 tons, but also enable the barge to be moored exactly in currents of anything up to 12 knots and in addition be sufficiently strong to enable the barge to ride out any storms which may occur.

b. Buoyancy tanks:

The weight of the steel pipe plus the reinforced concrete means that the pipe would be too heavy to pull and would become overstressed during the process. To reduce the pull the pipe must be lightened by attaching buoyancy tanks at precalculated centers. These tanks are strapped to the steel pipes with steel strapping which passes through two guillotine attachments on the top of each buoyancy tank. A float and wire are attached to the arm of the guillotine. When the pipe is in its final position the float is retrieved and by pulling on the wire the guillotine cuts through the steel strappings and the buoyancy tanks are released. In certain circumstances, particularly where severe transverse forces can be applied to the pipe by the speed of currents in the area, it is necessary to release the buoys simultaneously and to achieve this a common trip wire is laid prior to launching.

c. Trenching:

Depending upon transverse current speeds and wave action it may be necessary to bury the pipeline in a trench. Underwater pipelines are buried by dredging, ploughing or jetting methods or a combination of these methods.

Some of the dredging equipment types are:

i. Trailer dredger.

The trailer dredger self-propelled trailing suction hopper dredger is an excellent tool for preparing a pipeline trench. Dredgers are kept very accurately on line by electronic equipment (e.g Decca hi-fix) which is independent of visibility conditions. The trailer dredger will deal with all sands and soft clays and silts-in fact all materials that can be jetted and sucked.

ii. Ploughing:

Ploughing techniques have been developed recently in Britain by the land and Marine Contractors Company (3). The plough is pulled across the sea bed by a winch which is either situated on a barge anchored at sea, or on the opposite bank in the case of an estuary crossing. In some cases the pipeline is attached immediately behind the plough and pulled into the trench as it is ploughed. While ploughing is a most economical form of underwater trenching, it is applicable to only non-hard types of soils. Soft to medium clays, cohesive silts, are ideal ploughing materials. Underwater trenches have also been ploughed in quite firm clays and soft chalk. The underwater plough, however, is unsuitable as a primary trenching tool in sands and gravels though it can have usage as a secondary tool in these materials in conjunction with other trenching equipment.

iii. Jetting method:

When the bottom consists of softer sediments, the pipeline can be buried using a submarine trencher rides along the pipe and undercuts it by the use of high pressure water jets fed from a pump and compressor on a floating service barge. By repeated passes of the jet trencher, the pipe is gradually lowered to grade.

After dredging it is advisable to take sweep soundings to ensure that there are no projecting humps.

The side slopes at which a trench will stand, and the rates at which the trench may silt up, are dependent upon the nature of subsoil, tidal currents, soil transportation in the area, etc.-factors which require very careful study at the outset of a project. It is normal practice to provide for an overdredged depth of about half a meter to allow for trench siltation, depending on the time elapsing between excavation and final installation of pipe.

d. Tie-in joints:

Having applied the lining to individual pipe lengths on land, it is still necessary to join these sections together by welding. This would of course, burn off the lining and so stainless steel tubes are welded to the end of each long section, and the lining cut back out of the range of the heat weld. The stainless steel tube section are welded together with stainless steel electrodes. Therefore continuity of protection is achieved without continuity of lining. There is bound to be, of course, a certain leakage from the cathodic protection system at the stainless steel sections, but this is negligible and is allowed for in the original design.

2. Launching - floating technique:

The floating technique, as the name implies, involves fabricating the whole line, and floating it over its final position, then lowering it to the trench as illustrated by Fig. 5 . This is a method frequently used for river crossings. The sequence of operations is as follows:

a. The assembly is built up parallel with the water's edge and slightly above high spring tides, slipways are prepared and depending upon the slope available these can either be "dead" (e.g. timbers) or , if the slope is shallow, the pipes may be placed on bogies at precaluculated intervals, the bogies resting on rail tracks running at right angles to the water's edge.

On the first available spring tide after construction has been completed, the whole assembly is launched sidways into the water. It is arranged that the assembly will come to rest in sufficienttly deep water at high water spring tides to enable specially constructed pontoons to be floated over the top of the pipe.

At low water the pipe assembly is inspected for damages, if any, and on the next high water the pontoons are floated over and secured to the pipe.

The remaining period of the rising tide is utilized to lift the whole of the pipe assembly off the bottom on the pontoons and the pontoons are then floated out into deeper water and are lowered to the bottom to a temporary position until the next period of neap tides.

b. The next sequence will be to utilize the neap tides to swing the pipe over its permanent position. Neap tides are chosen because the currents will not be as strong as on spring tides. The temporary location of the pipe will be arranged so that one and of the assembly is very close to the permanent landward end of the outfall and the pipe will lay on the bottom so that its line will be as near

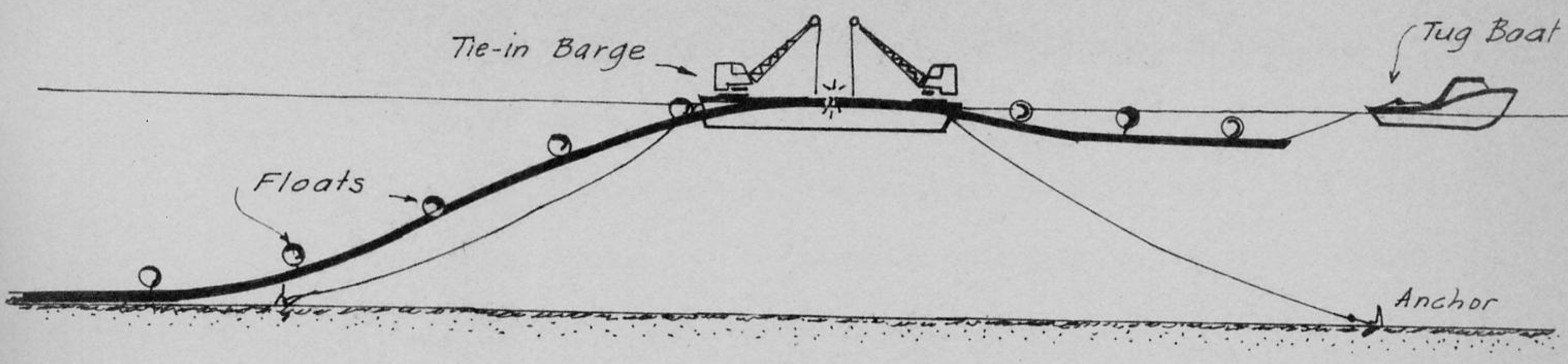

Fig. 5 Floating-launching technique

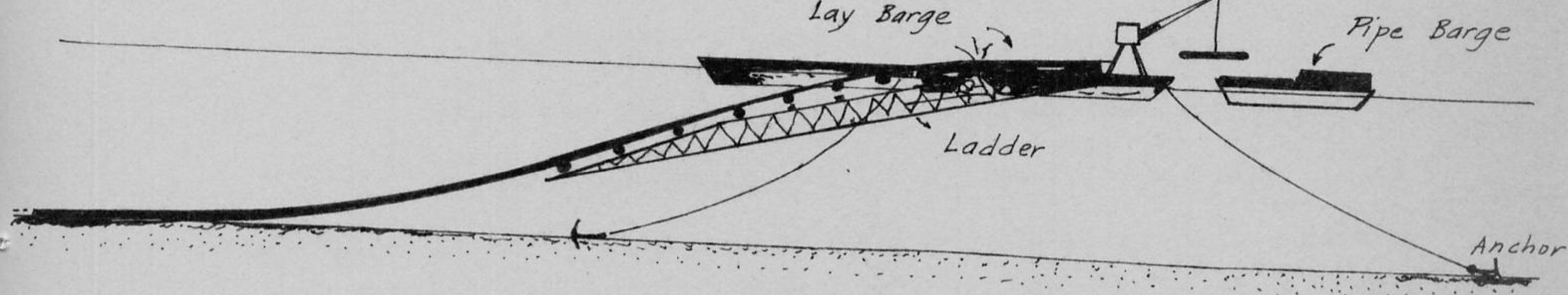

Fig. 6 Lay-barge method

as possible parrallel to the prevailing currents.

A neap tide with the least range is chosen to swing the pipe into position and an accurate forecast of the speed and direction. etc., are carried out over periods from two to three month's duration. The object is to rely entirely upon the currents themselves to swing the pipe into position and therefore the assembly must be released at precisely the right time before high water so that the pipes arrive over their final position exactly at dead slack water. It is during slack water that the pipes are lowered into their final position on the sea bed.

3. Variations on the pulling and floating techniques:

Very occasionally a combination of both techniques or slight variations of them can be used to advantage. These are briefly described in the following sections:

a. Combined float and pull:

Where currents are very weak, a typical example being, say, on a lake, and a construction area is available for the pulling technique and at the same time is large enough to build the pipe in its entire length on land, then the pipe can be pulled out with a high - speed winch or by special tugs with strain gauges on the pulling hawser, the pipe being supported by buoyancy tanks in much the same way as in the pulling techniques described, but in this instance the tanks are at closer centers to give complete buoyancy.

When the whole line is over position; the tanks are released by a common trip wire and the pipe is allowed to drop on to the bed of the sea or lake. This technique can be used where the pipe has to be built some distance away from its final resting position and is then actually towed into the shore and not away from it. The number of occasions when the conditions are such that this system can be used are limited. These conditions are occasionally met at calm seas where the tidal range is negligible and currents almost non-existent.

b. The lay-barge technique:

This method is generally used for long sea lines to offshore wells and for gathering lines between wells.

A barge is constructed on which an angled "ladder" is supported. The "ladder" is adjustable in that the portions of its length in and out of the water can be varied, together with its angle relative to the barge. The pipeline is then laid from the shore along the sea bed, up the ladder and on to the barge, which at this stage is anchored as close to the shore as possible, and the pipe falling in a catenary from the ladder on to the sea bed as shown in Fig. 6. Extra lengths of pipe are placed on the portion of ladder projecting out of the water above the deck of the barge. Jointing is then carried on the ladder, and when complete the barge is moved farther out to sea, the pipe sliding down the ladder on roller bearings. In this way it is theoritically possible to lay any length and diameter of pipe. However, there are severe practical limitations.

The system is obviously very vulnerable to weather conditions, and of course, as the diameter of pipe increases the resistance to transverse forces of tide and current becomes critical. There is no direct control on the stressing of the pipes which can be seriously influenced by rise and fall of tide, wave action, transverse forces of current, and bending in the catenary.

c. Platform with " spuds ".

This system consists of a floating platform with columns or " spuds " which can be lowered on to sea bed. The platform can then be jacked up on the columns or spuds until it is above the surface of the water, if necessary, well above the highest tide. Such platforms are very rigid and stand up to extreme weather conditions.

They can be used in the same way as the lay-barge technique described above, the ladder being supported over the side of the platform, and the platform itself being used to stock the pipes.

Another method using this equipment is to float out long sections of pipe to the platfoem, which then lowers them to the sea bed where divers join up to the previously laid pipeline anything up to 60 meters at a time being handed in this way.

The platform technique can be used in this way to lay pipes of very large diameter, where the pulling technique would be out of the question.

The platform can also be used as a base for the winch in the pulling method. They are not so vulnerable to weather as a barge, and on long lines they can be moved away from the shore stage by stage as the pipeline is built up on land, although of course, the same procedure is carried out with a barge which is moved to previuosly laid moorings along the route of the pipe.

The platform with " spuds " is a very useful piece of equipment, and is used extensively for offshore drilling and underwater marine works in general.

E. Alinement and relocation of the pipeline:

In as much as marking of the line is either impracticable or very expensive, a great deal of effort is needed to lay the line in such a manner that it can be relecated with ease. There are probably several methods of control; the most accurate one is to use a transitman to maintain direction. The most practicable is to set buoys on a transit line at close intervals along the route and check with the transit at certain times as the line is laid. For shallow depths a temporary trestle or survey tower is usually erected as a control point. The accurate location of the pipeline is basically important at two different times:

1. When a structure is to be built in the vicinity of a pipeline.
2. When a tie-in is to be made on the pipeline.

Being unable to locate the pipeline immediately when desired can result in the expenditure of a great deal of money.

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CHAPTER THREE

OCEANOGRAPHY

Tides, currents and waves are three allied physical phenomena having direct effects on the design and laying techniques of underwater pipelines.

In this chapter, the causes and actions of these three agencies are discussed.

A. Tides:

The term "tide" refers to the periodic rising and falling of the water level that results from the gravitational attraction of the moon and the sun acting on the rotating earth. Although the accompanying horizontal movement of the water resulting from the cause is sometimes called the tide also, this is usually designated as "tidal current" and will be covered briefly in the following section.

1. Astronomical tides:

The major forces causing the tides are the gravitational forces of the moon and the sun, the centrifugal force due to the movement of the earth in its orbit and the Coriolis force due to the earth rotating about its axis. The gravitational forces of the other planets have been found to be negibigible. Although the moon's mass is much less than the mass of the sun, the moon is much closer to the earth than the sun that its gravitational field is a greater tide generating force than the sun.

Tides are distinguished as "springs" or "neaps". Spring tides are the highest tides of the month, and they occur when the moon is new or full, or nearly so; in technical terms, when the moon is in conjunction with or in opposition to the sun, and the tides raised by the two bodies are exactly in concord, so that their joint effect is a maximum. At or about the time of the equinoxes, the sun and the moon are actually or nearly vertically over the Equator. Their influences are then most closely coincident and direct, and equinoctial spring tides are therefore exceptionally high.

Neap tides are the lowest tides of the month, and they occur about the times when the moon is in its quarters.

Shortly before and after new moon, the sun and the moon occupy positions such that their resultant attraction on the surface of the earth is directed towards a point somewhere between them. This causes high water to occur a little before or a little after its usual time, and the tide is said to "prime" or to "lag". The tide is late before new moon and early ater it. Similar action takes place just before and after full moon. The average interval between corresponding tides on successive days is 24 hours 51 minutes, but the priming and lagging each makes difference of from about 20 to 40 minutes in the tidal interval, so that it ranges between 24 hours 32 minutes and 25 hours 32 minutes.

Tides have been recorded in many areas for a great number of years and records have been analyzed to obtain the amplitude and the phase of the tidal components. With the aid of such data, collected and correlated, it is possible to predict astronomical tides on advance. The importance of predicting the time of spring and neap tides is felt in some methods of laying as was mentioned previously in Chapter Two.

2. Meteorological tides:

In addition to the tides caused by the forces just mentioned there are some changes in the sea level due to meteorological conditions which are called meteorological tides, storm surges, storm floods, etc. Part of the change in water level is due to a coupled wave system caused by a low or high-pressure region moving over relatively shallow water, and part of it is due to the piling up of water along the coast because of the stress of the wind on the water surface.

B. Currents:

The waters of the oceans have a circulation produced by thermal differences, by the rotation and revolution of the earth, and by the motion of the earth's atmosphere. The circulation may occur at the surface of the water or below it.

The effect of oceanic currents may be far-reaching, being largely responsible for certain climatic conditions of many portions of the world. The small currents are of considerable importance in regard to the movement of sewage discharged from a sewer outfall, or the movement of oil slicks.

1. Classification of currents:

There are several main classes of currents in the oceans, these currents may be classified as follows:

a. Wind drift currents of relatively short duration: When wind blows over the surface of water, a shear stress develops at the interface and the air drags water along; because of viscosity and turbulent mixing, this current gradually deepens.

b. Interial currents: Once the driving force (the wind) has ceased blowing, the current generated by it will be under the influence of its inertia, fluid resistance at its boundaries, and the deflecting coriolis force (which acts on a moving body due to the earth's rotation), and it will be a damped inertial current. Its path is elliptical and its period in hours varies from twelve hours at the North Pole to an infinite value at the Equator (1).

c. Wave - induced currents: Wind blowing over the water surface transmits energy to the water, part of this energy is in the form of surface currents and part in the form of surface waves, in the region where currents and waves (called wind waves or a sea) are generated, the motion of the water particles have a complex form. After the wind has died down, or the currents and waves have left the generating area, they become free (currents are now called intertial currents and waves are called swell).

Along the shore the amount of water transported shoreward by the waves must be compensated for by a hydraulic current flowing seaward. Because of the mass transport of water, and because the wave front breaking along a beach, becomes a wave of translation, a certain amount of water piles up against the coast. A hydraulic head is thus established and then water must return seaward. Sometimes it returns directly seaward, but in other places it flows along the beach as an alongshore current inside the breaker zone.

Along many beaches there are irregularities that cause the water to flow seaward as a narrow rip. The general process of seashore circulation is illustrated in Fig. 7. As indicated in the sketch, a rip current consists of three main parts: the feeder current, which flows parallel to the shore inside the breakers: the rip current proper which flows through the breakers in a narrow band; and the seaward current, where the current windens and slackens. Once rip currents have formed, they cut through in the sand and remain fairly stable until the wave conditions change.

Although waves tend to become parallel with the coast as a result of refraction, they usually break at a slight angle to the shore, with the result that a "littoral current" is induced and is effective in moving a mass of water along the coast in the surf zone.

One of the more important effects of littoral currents is the movement of sand along a coast known as "littoral drift ". The littoral current, combined with the agitating action of the breaking waves, is the primary factor in causing such movement. The greatest volume of littoral material is transport in the immediate vicinity of the breaker zone where there is sufficient turbulance to maintain the sediment in motion or suspension. Deposition occurs at the point of change in the transporting capacity of the waves.

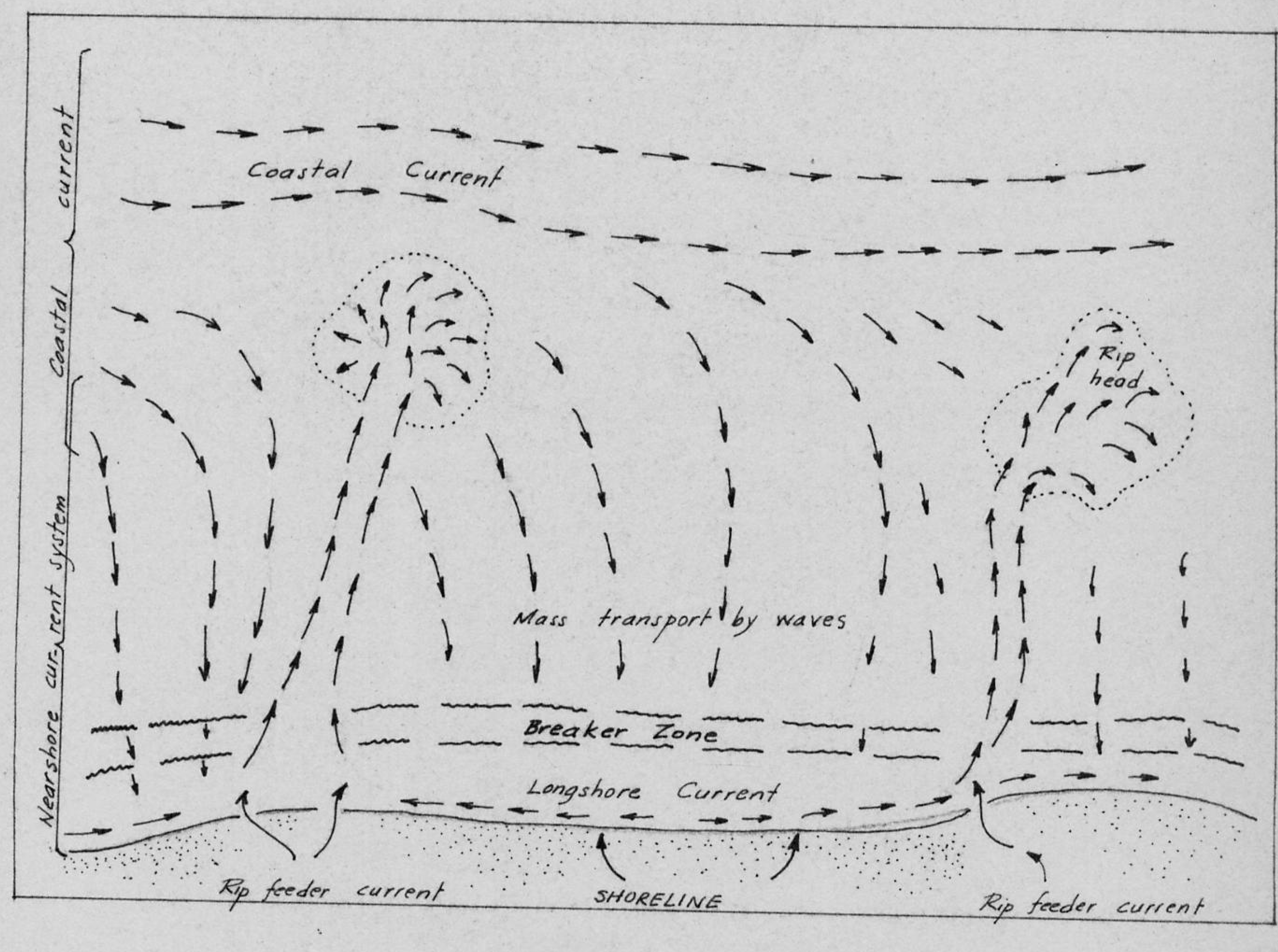

Fig. 7 Schematic diagram of two inter-related current systems nearshore

d. Tidal currents: The astronomical forces of the moon and the sun cause tides in the ocean which have both vertical and horizontal motions. These tidal motions, combined with the topographical features, give rise to three types of tidal currents, all of which are periodic (2).

i- The rotary type, usually occuring in the open sea or along the coast. The rotary motion is due to the effect of the Coriolis force; i.e., from hour to hour the currents change both in direction and speed.

ii- The rectilinear or reversing type, illustrated by currents in most inland bodies of water. An example is the current formed by a salt-water wedge moving through fresh water, occuring between the flood and ebb flows.

iii- The hydraulic type, illüstrated by the currents in the straits connecting two independant tidal bodies of water. The latter are reversing currents which are due primarily to a temporary difference in head between two bodies of water brought about by tidal action, rather than by the action of a progressive or a stationary type of wave.

e. Major ocean currents:

Far more is known about the surface currents of the ocean than about the deeper currents. Charts illustrating the directions of currents over the seasons are available for most of the oceans.

The circulation of the water of the ocean is caused by the heating at the tropics and cooling at the poles, which induces a general surface motion poleward and a motion in the depth toward the Equator which is more or less modified by the continental masses and irregularities and by the difference in the velocity of the rotation of the earth at the Equator and the poles.

The ocean currents have an indirect effect on the temperature and rainfall of the various land areas eastward of the courses in which they flow. The warm water currents on their poleward flow increase the temperature of the superincumbent atmosphere which is drifting eastward, warms the northwestern shores of the continents.

There have been many attempts to predict the main features of the circulations of the oceans, from a consideration of the wind field over the oceans and the Coriolis deflecting force; some of these attempts have included the effect of lateral stresses.

2. Investigation of currents:

The purpose of the investigation is to determine the direction and velocity of currents, and to observe their relation to the configuration of the coastline.

a. Float surveys: This method is extensively used in engineering surveys. The float is constructed in such a manner as to minimize wind drag and to represent the motion at the desired depth.

The limitations to the use of floats is due to the fact that current velocity and direction do not remain the same but fluctuate over a period of time. Thus the velocity and direction indicated by a 6-hours float travel can be extrapolated for an 18-hour travel, unless a statistical variance correction factor is employed.

Some of the float types usually employed in the determination of the direction and velocity of currents are:

i- A circular or square pole with a wooden cylinder or prism at its lower end weighted so as to flow vertically, affords a suitable form of instrument. Such an indicator is shown in Fig. 8.

ii- A combined drag and float consisting of a skeleton box frame covered with canvas. The drag is usually weighted with detachable weights and suspended from the float by means of a steel wire, the length of which could be adjusted to suit the depth. At the top is a flag fixed within a socket. This type is ordinarily used to measure subsurface currents, and is shown in Fig. 9.

iii- Cross current float, consisting of a light frame 90cm. deep cruciform in section, with a flat top about 23cms. square on which is mounted a small flag. The frame is weighted at the underside to an extent just sufficient to bring the flat top level with the surface of the water, or rather just to submerge it, so that any light breeze might have no effect upon it. A schematic diagram of this type of surface float is shown in Fig. 10.

iv- A drum float consisting of an empty oil-drum sunk to almost complete immersion and ballasted by a basket of stone attached below it is another arrangement shown in Fig. 11. On top of the drum is fixed a sighting mark composed of a semaphore, flag or disc.

v- For purposes to which no great accuracy is essential any convenient buoyant object, such as any empty keg or barrle, may serve to indicate the direction and general speed of the current, and in positions where a small object can be easily seen; small pieces of cork, wood, flourescein dye, aluminum powder wetted with koresene, or oranges will do satisfactorily.

b. Current meters: Observations of sub-surface currents may also be made by anchoring a boat and using some type of current meters.

i- Carruthers current meter: This instrument, as shown in Fig. 12 is a simple device for measuring the speed and direction of a current at any depth in the sea. The meter consists of a cone made of metal mesh which is connected to a rod, the rod being attached to a

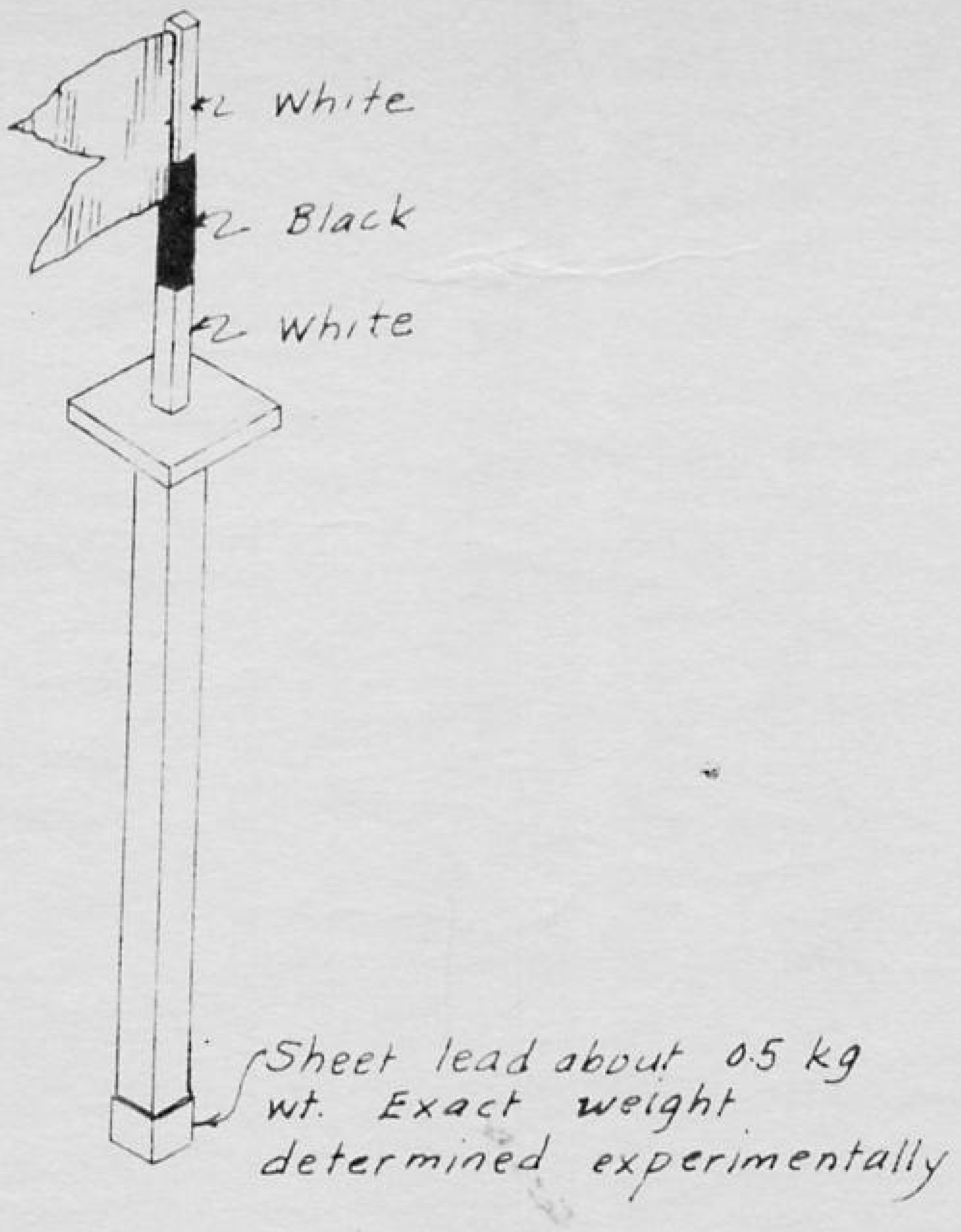

Fig. 8 Float for determining direction of current

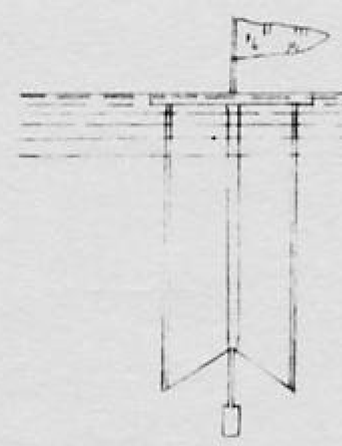

Fig. 10 Surface float

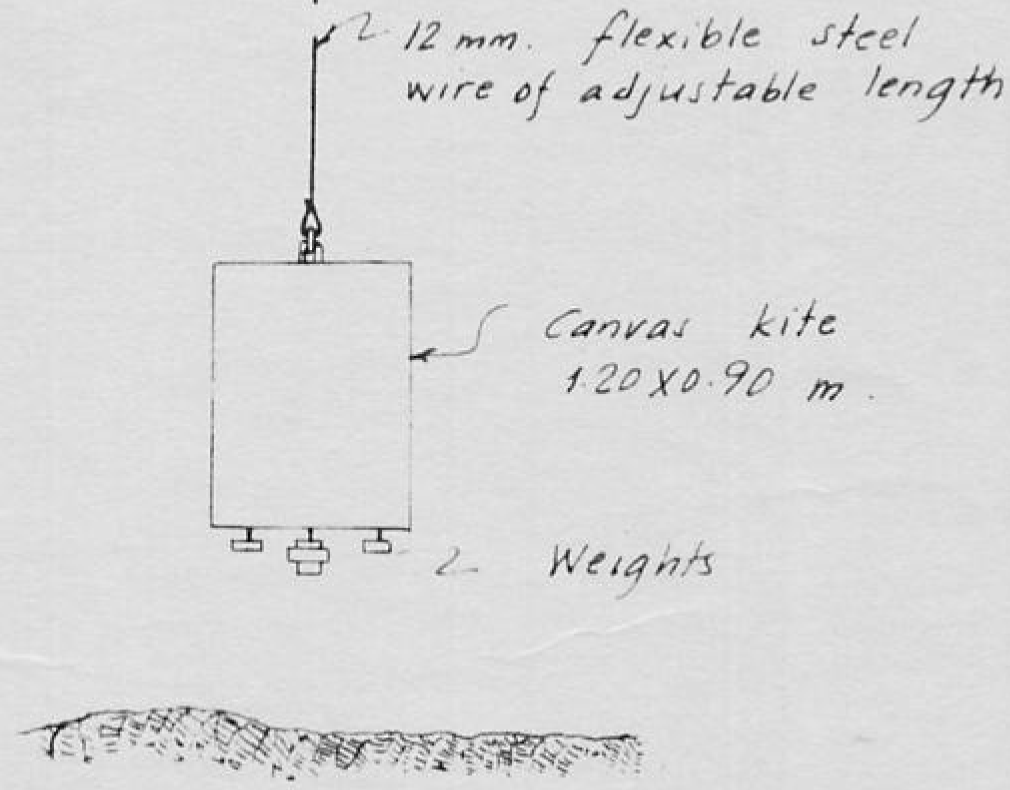

Fig. 9 Float for deep water

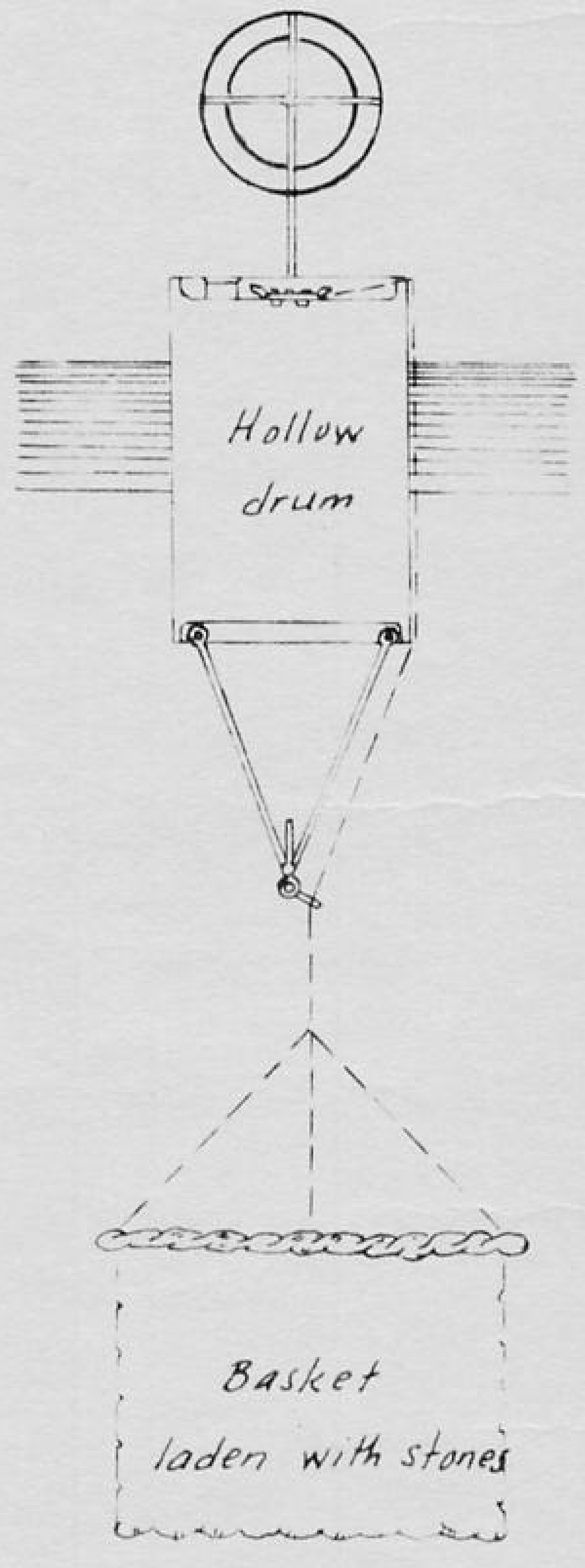

Fig. 11 Drum float

wire or rope by a corkscrew twist at either end, and fixed in position by a split bobbin (3). The meter cone contains a vertical pendulum and compass which is free to swing when the cone is sent down. A locking device clamps the indicating mechanism in position after a pre-determined interval.

The cone takes up a slope which depends on the speed of the current and the speed is read on a scale showing the value in knots. Morever, as the cone always heads into the current, the compass reading gives the direction required.

ii- Roberts radio current meter: This instrument represents a great advance in current observation and technique. It is designed to measure the direction and speed of currents and at the same time to transmit radio signals to a receiving station where the signals are recorded and translated into actual values of velocity and direction.

The meter operates satisfactorily from 0.25 to 10 knots, though the results from currents less than 0.5 knot need some care in interpretation. This type of meter is very helpful when measuring currents due to vertical density variations.

c. Other studies may be required for eddy diffusion and total transport caused by currents along the shoreline.

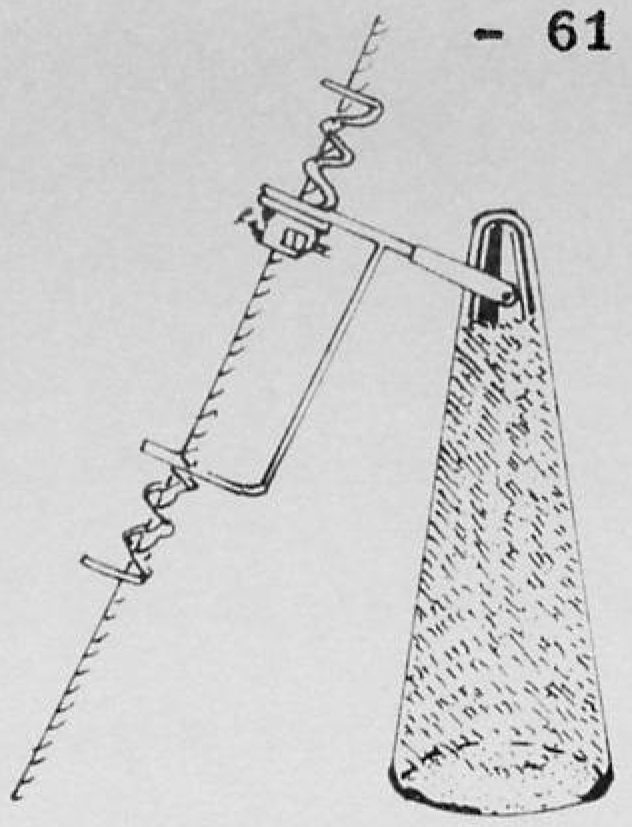

Fig. 12 Carruthers current meter

Fig. 13 Typical current diagram

The figures at the arrows indicate the average speeds, in knots.

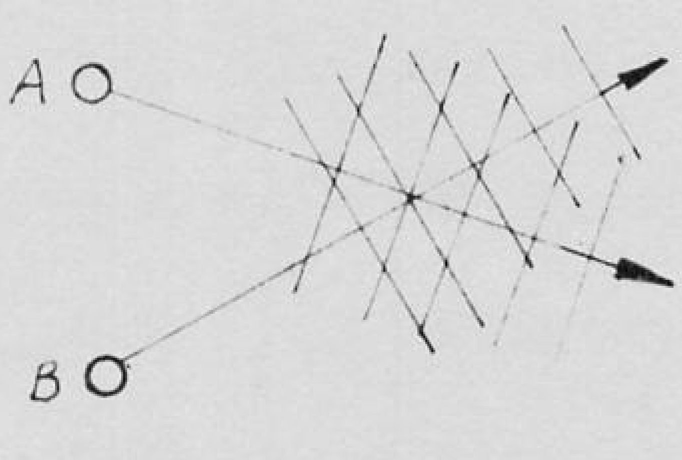

Fig. 14 Intersection of waves

d. Representation of investigations: A simple method of representing the direction and speed of surface currents is to draw arrows in the direction of flow, and to indicate the speed in knots in numbers along the directional arrows. An example is shown in Fig. 13.

For subsusface currents, different overlays of the same type of representation can be made, or contours of currents at different depths may also be plotted.

C. Waves and Swell:

Waves are fluctuations in the surface level of a fluid. Most waves are generated by the action of the wind, and the manner in which this is accomplished appears to be as follows: before the wind begins blowing, the water may be perfectly still, but as soon as the wind commences a change quickly takes place.

Wind velocity is never constant, but acts in puffs and gusts, causing varying pressures to be exerted upon the surface of the water, which, under this action, soon becomes ruffled, presenting undulations upon which the wind can act directly. The elevated portions of the undulations thus formed are fully exposed to the wind, while the depressions are partially or entirely screened by the crests in the rear. These undulations or waves are continually changing in form and increasing in size under this action until limited by the "fetch" and the velocity of the wind.

When the velocity of the wind is considerably greater than that of the waves, it acts both by friction and by direct pressure upon the crest of the latter, exerting a force tending to push forward the top too strongly for continuous undulation. The crest is then carried forward too far, and the wave breaks as is seen in the case of "whitecaps" and of large waves which break during storms in water of great depth.

Waves may be produced by other causes than wind action, as far example, by vessels underway, submarine explosions, barometric fluctuations, the attraction of the sun and moon, earthquakes, volcanic eruptions, etc.,

1. Classification of waves:

Waves have been variously classified by different writers as follows (4):

a. With respect to the continuance or noncontinuance of the generating force, as forced or free.

   Forced wave: The wave upon which the force causing it continues to act, as in the case of a wave during a storm.

   Free wave: The wave upon which the force causing it no longer acts, as when swells roll in after a storm.

b. With respect to periodicity, as solitary or successive.

c. As regards the position of the wave with respect to the water surface as ordinary, positive, and negative.

   A positive wave has an advancing elevation.

unaccompanied by a depression.

A negative wave is one which lies wholly below the general water level.

d. In respect to the orbital motion of the particles, as oscillatory waves or waves of translation.

Oscillatory waves are waves in which each particle describes a closed orbit around its position of rest, without advancing in the direction of wave travel; usually applied to the waves in deep water.

Translatory waves: waves which after their passage, leave the particles shifted in the direction of wave travel.

e. In regard to size as tidal waves, seiches , storm waves, and ripples.

Tidal wave: the wave of the tide; also applied to waves of unusual size and of rare occurence, as those produced by underwater seismic disturbances. A species of tidal wave is the "bore" a high crested, roaring wave caused by the rushing of flood tide up a river or estuary, or by the meeting of tides.

Seiche: a wave of very great length, found in large fresh water lakes or land-locked seas, and attributed to sudden local variations in barometric pressure.

Ripple: the smallest class of waves and one in which the force of restoration of the particles is chiefly the surface tension of the water.

f. As to appearance, as ordinary waves, white caps, swells and breakers.

Swell: a long unbroken wave which rolls in after a storm or as the result of a distant storm.

Breaker: a wave which breaks in shallow water or upon reaching shore.

These classifications, however, are largely arbitrary, and do not serve to define the wave completely, for example, a wave may at the same time be free, solitary, positive, and a wave of translation.

Storm waves, swells, and breakers comprise the principal classes of waves which act against engineering structures.

The tides, seiches, and ripples develop no injurious wave action proper against these structures, and any damage caused by the two former will be due either to the change in water level or to the resulting currents.

2. Characteristics of waves:

The formation of sea waves takes place in the open sea and their inception is due to the wind, their height at any point depends primarily upon the velocity and duration of the wind and the distance across open water from the windward shore "the fetch", but is modified by the configuration of the adjacent shore lines, and by the depth of water in which waves travel.

The waves themselves are described by the length from the crest to crest (L), the height from trough to crest (H), and the period (T), the latter being the time interval between the arrival of successive crests at a stationary point.

It is now possible to estimate the height, length, and period of sea waves and swell which will be produced by any given wind force, and the maximum sizes of waves that can be generated in any locality due to sea area, geography and other limitations.

Not only is the period of waves limited by considerations of nature but a limit is also set to the height of sea-waves. Usually this will not exceed one- fifteenth and rarely one-tenth of the wave length. When the heights are one-seventh of the wave length or less, the mechanics of gravitational waves cause them to break and dissipate a considerable part of their energy. Also the depth of the water limits the maximum possible height which waves can attain without breaking and dissipating their energy.

Waves do not travel from a storm area in an endless procession of waves of uniform characteristics, but rather in groups or trains of waves of different dimensions, some high, some low, and of varying lengths. In deep water, the longer waves travel the faster, and overtake and move through the shorter waves, dying out and being compensated for by the formation of another wave in the rear of the group.

When two lines of wave fronts proceeding from two storm centers A and B as shown in Fig. 14., cross each other, the characters of orbital motion of each are unimpaired and the procession of waves from A will pass through those from B and vice versa (5). Sometimes there is a local disturbance noticed when the individual waves of each group come into phase and cause an ejection of broken water at the crest due to the release of excess energy.

It will be realized that the estimation of the largest waves likely to assail marine structures is a task of some difficulty having regard to the varying nature of the factors governing wave generation, together with the modifying effects of the sea bed, topography of the coast, and meteorological conditions. However this is a task of great importance, and it is usually simplified by direct observation and records of waves and storms. Over-estimation can be most costly while under estimation will, more often than not, lead to disaster.

a. Form of waves:

The outline assumed by waves is extremely variable, depending both upon the length of the undulation and its period, the crest or summit of a wave is sometimes rounded, sometimes acute, and in either case it attains a height above mean sea level greater than the depth of the trough below it.

In a swell in the open sea, the profile of a wave perhaps most nearly resembles a sinosoidal curve, the slopedirectly exposed to wind action, however, being more gradual and less steep than the leeward side.

b. Surface profile of waves: <u>Trochoidal theory</u>:

The Trochoidal wave theory for deep water was developed by Gerstner (1802), Froude (1862) and Rankine (1863). The theory has been used extensively by naval architects and civil engineers in their studies. One advantage of the theory is its exactness. Rankine's researches showed that a theory of rolling waves could be deduced from that of the position assumed by a mass of water revolving in a vertical plane about a horizontal axis. He also proved that in a mass of a gravitating liquid whose particles revolve uinformly in a circle, a wave surface of trochoidal profile fulfils the condition of uniformity of pressure. Such a trochoidal profile is generated by rolling, on the under-side of a staight line a circle whose radius is equal to the conical pendulum that revolves in the same period with the particles of the liquid moving in the path of some point on the perimeter of a concentric circle of lesser diameter (6)- as, say, some fixed point on a spoke of a wheel inside the rim as illustrated in Figs. 15 and 16.

Let L = Wave length from crest to crest.

H = Wave height from crest to trough.

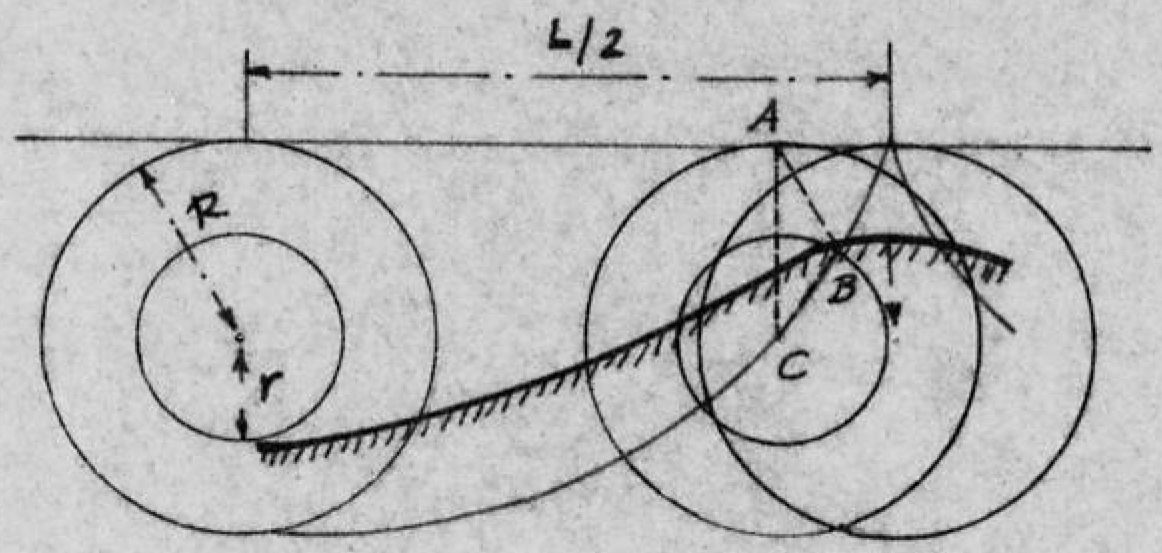

Fig. 15 Trochoidal wave profile on Rankine's theory

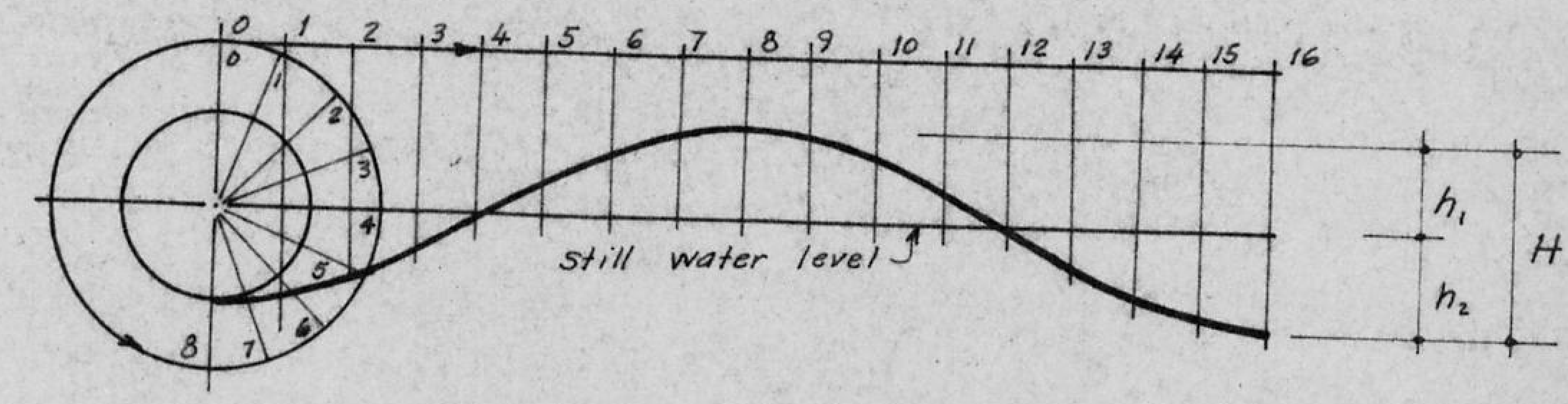

Fig. 16 Construction of Trochoidal wave curve

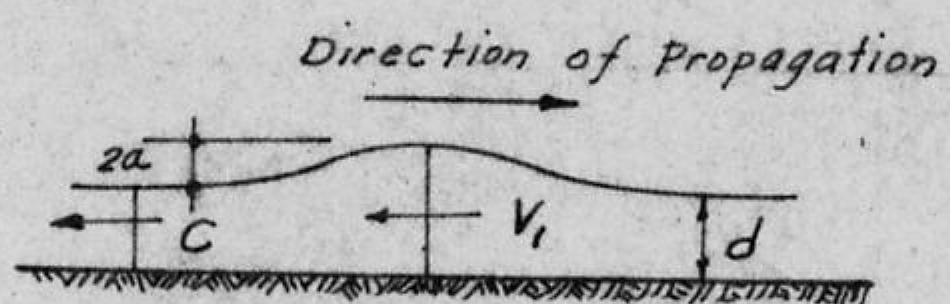

Fig. 17 Derivation of wave characteristics

T = The period in seconds.

g = The acceleration of gravity.

C = Celerity i.e., velocity of propagation of wave.

Assuming that the total energy at the surface remains constant, the increase of potential energy between the crest and the trough must be equal to the decrease of kinetic energy between the two positions. Further assuming that the wave travels along a channel of unit width, and is brought to rest by imparting a velocity C (equal to the wave velocity) to the mass of water in a direction contrary to the wave travel.

Let v = Velocity of a drop of water in its orbit = $2\pi a/T$

where (2a) is the wave height,

then the velocity of a drop of water at the trough = $-C-v$

and the velocity of a drop of water at the crest. = $-C+v$

Then the decrease of kinetic energy between the crest and the trough of unit mass of water =

$$\frac{1}{2g} \quad (-C-v)^2 - (-C+v)^2$$

and the increase of potential energy between the crest and trough considering a unit mass of water = 1 X 2a.

Substituting and equating the two expressions

$$C = \sqrt{\frac{gL}{2\pi}} \quad \text{and} \quad L = \frac{gT^2}{2\pi}$$

which shows that in deep water, the velocity of propagation of a wave is a function of wave length.

As (T), the period, is L/C, the velocity can be expressed as

$$C = \frac{gT}{2\pi}$$

A variation of the above method of treatment is as follows: Assume that a wave is propagated in the channel at such a velocity against the natural flow of the stream that the wave remains stationary with reference to the sides of the channel. Then as the water continues to flow through each cross section (Fig. 17,) :

$$V_1 (d + 2a) = Cd$$

By Bernoulli's theorem

$$d + 2a + V_1^2 / 2g = d + C^2 / 2g$$

Hence

$$2a = \left[1 - \left(\frac{d}{d + 2a} \right)^2 \right] \frac{C^2}{2g}$$

And

$$C^2 = 2g \frac{(d + 2a)^2}{2d + 2a} = \frac{2g (d+2a)}{1 + \left(\frac{d}{d + 2a} \right)}$$

when 2a is small compared with the water depth d, the last expression may be written

$$C = \sqrt{g (d + 2a)}$$

and if the height of the wave trough to crest (2a), as defined in Fig. 17 is very small

$$C = \sqrt{gd}$$

That is, the velocity is a function of the depth of water. This demostration is usually applied to shallow water surface waves.

Referring to Fig. 15, if B revolves in a vertical circle and if CA:CB = g: centrifugal force, the resultant is AB. A curve normal to AB will be the surface of equal pressure. Such a surface is a trochoid traced by B, which is carried by a circle of radius AC rolling on the underside of a horizontal straight line.

The height of the wave is 2r and L (length of wave) is $2\pi R$.

Rankine has also established that the crests of waves rise higher above the level of still water than the troughs fall below it. The height of the center of the orbit of the wave above still water is $\left(\frac{\pi r 2}{L}\right)$, that is, height due to the velocity of the particles.

Therefore the mechanical energy of a wave is double that due to the motion of its particles only, there being an equal amount due to the mean elevation of the particles above the position where the water is still.

The height of the crest above still water level is

$$h_1 = \frac{H}{2} + \frac{\pi r^2}{L} = \frac{H}{2} + \frac{\pi}{L}\left(\frac{H}{2}\right)^2 = \frac{H}{2} + \frac{\pi}{4}\frac{H^2}{L} = \frac{H}{2} + 0.785\frac{H^2}{L}$$

and the depth of the trough below still water level will be

$$h_2 = \frac{H}{2} - \frac{\pi}{4}\frac{H^2}{L} = \frac{H}{2} - 0.785\frac{H^2}{L}$$

Still water level is the level of the water surface when undisturbed by waves.

For shallow water waves, the height of the crest above still water level is materially greater than that of deep water waves. According to Gaillard, the following formulas can be applied:

$$h_1 = \frac{H}{2} + 2.0\,\frac{H^2}{L}$$

$$h_2 = \frac{H}{2} - 2.0\,\frac{H^2}{L}$$

c. Orbital motion in waves:

It was confirmed by many researchers that there is an orbital movement in waves, each particle of which they are composed pursuing a path such that the surface profile of the sea generated by wind is more truly trochoidal than cycloidal.

The precise nature of the path depends upon local conditions. In deep waters, that is, where the depth is at least equal to the length of the wave from crest to crest, the motion of the particles of water is rotary along the circumference of a circle, as shown in Fig. 18. The wave is of the oscillatory type, and each particle completes a revolution, returning roughly to its initial position.

The paths described by water particles are circles with the radii decreasing exponentially with depth. Airy, Bertin and Boussinsq (7) established that in depths of water varying from the surface in arithmatical progression

0, $^r/2$, r, $\frac{3r}{2}$, 2r, etc., the radii of the generating circles vary from unit value in a geometrical progression of which the common ratio is $\frac{1}{\sqrt{e}}$, where e is the base of Napierian logarithms. That is, corresponding to the above depths, the radii are-

Algebraically - r, $r\frac{1}{\sqrt{e}}$, $r\left[\frac{1}{\sqrt{e}}\right]^2$, $r\left[\frac{1}{\sqrt{e}}\right]^3$, $r\left[\frac{1}{\sqrt{e}}\right]^4$, etc.

Numerically - r, 0.61 r, 0.37 r, 0.22 r, 0.14 r,

Thus it will be observed that the wave height decreases rapidly with depth.

In shallow waters of uniform depth, that is, in water the depth of which is less than the length of the wave, the orbit of the water particles, is approximatly elliptical, with the major axis horizontal, as shown in Fig. 19. The ellipses of movement become flatter as the distance below the surface increases, until finally at the bottom there is horizontal motion only. In water which has a depth of only one-tenth of the length of the wave, the ratio of the elliptical axes at the surface is about $^7/12$, and at nine-tenths of the depth it is $^1/16$ (8).

d. Height of waves:

The generation of waves being due to the wind, their development clearly depends upon the extent of surface acted upon. First, the impelling force of the wind is transmitted to the water surface. Ripples form which become undulations of longer and higher dimensions as the surface

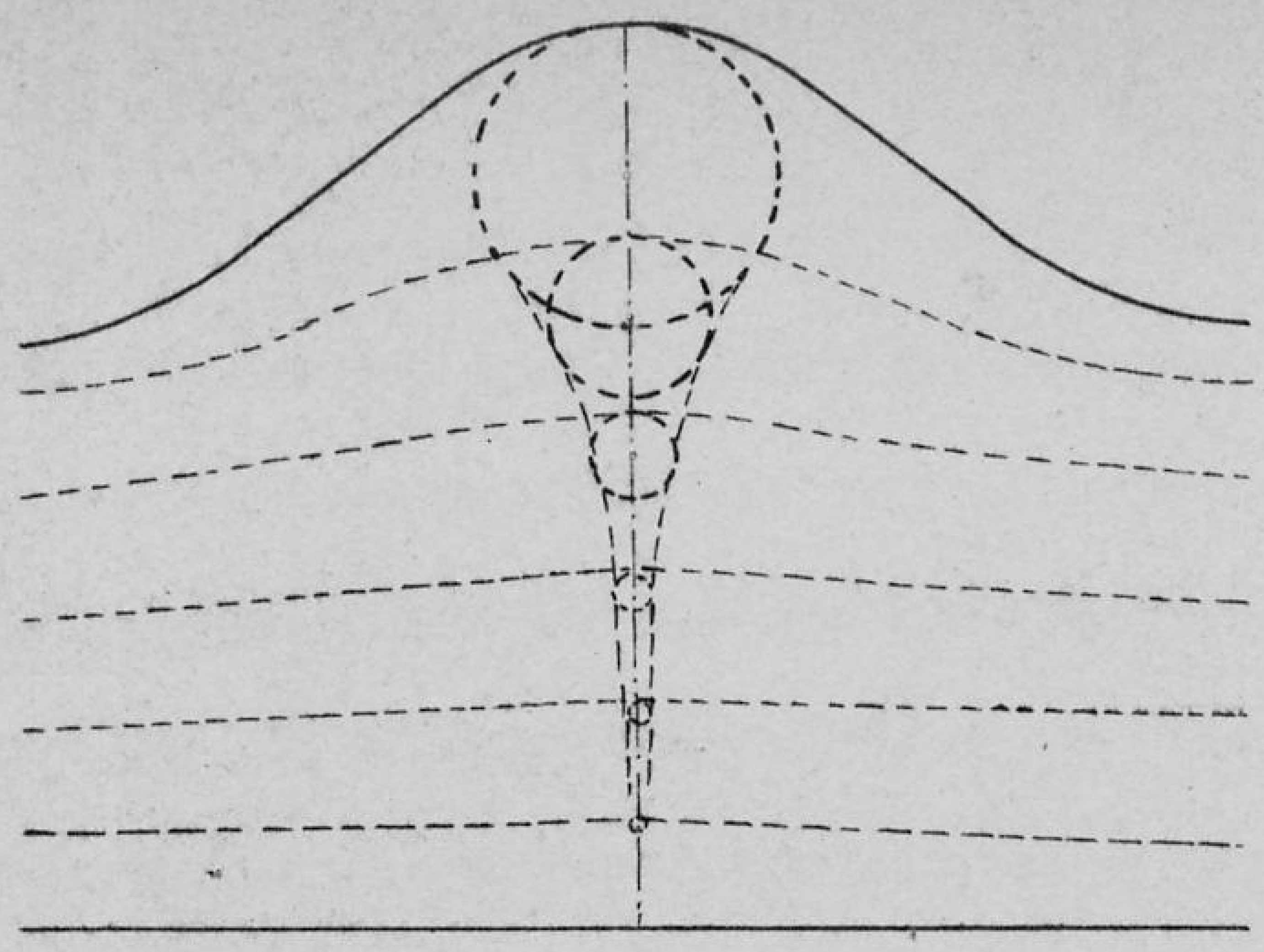

Fig. 18 Motion of wave particles in deep water on Trochoidal theory

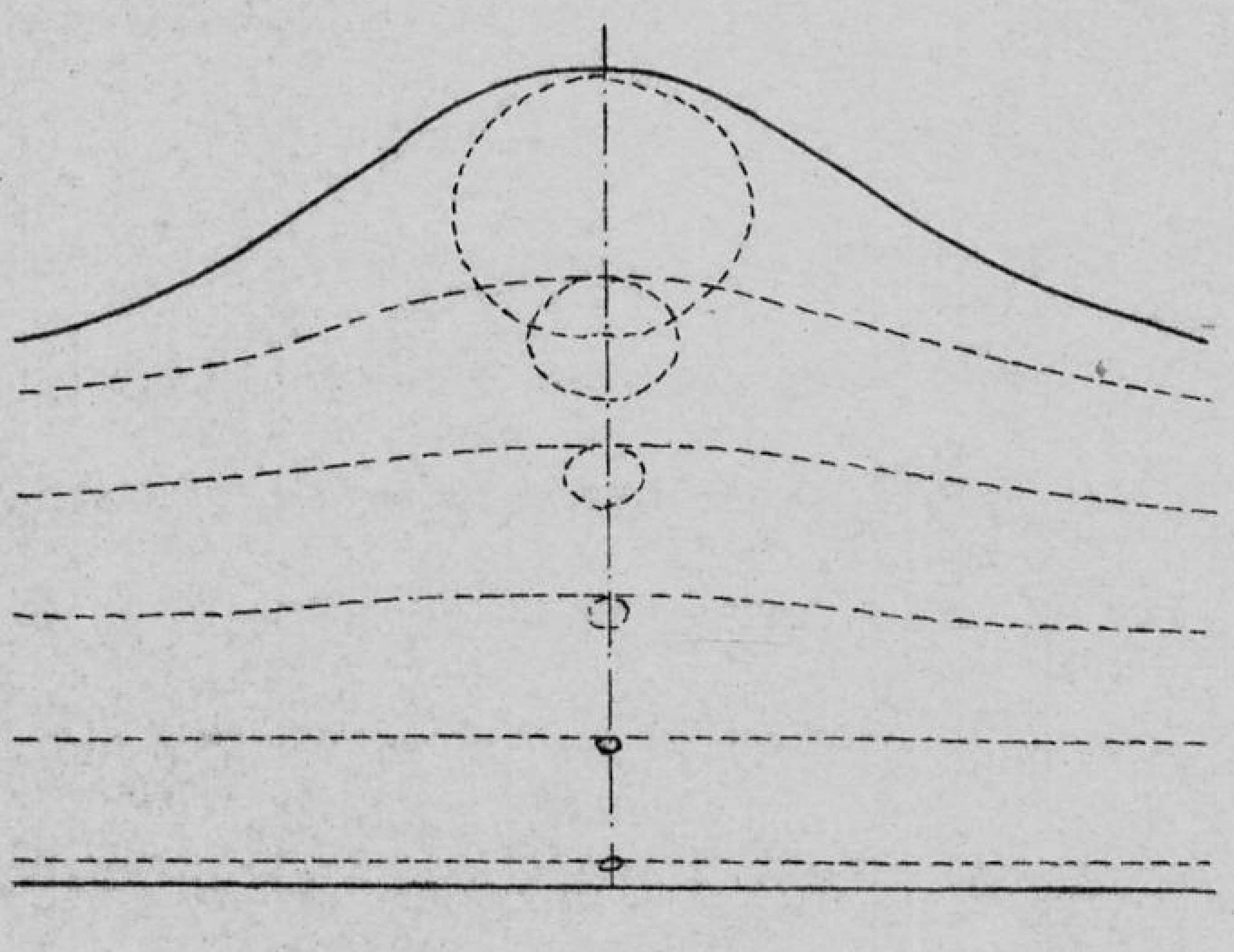

Fig. 19 Motion of wave particles in shallow water on Trochoidal theory

of the water slopes exposed to the moving air become longer and the frictional contact is increased, whereas the lee side of the undulations is relieved of wind pressure and subjected to suction by the passing wind. Thus the wind, if it persists and remains constant, builds up wave forms of dimensions proportionate to the strength of the wind and the duration through which the given wind acts upon them.

Table 1 (9), gives the approximate dimensions of waves generated in deep sea by winds of various forces, though it must be remembered that while the heights attained by wind waves have been found to bear a linear relationship to the velocity of the wind, the length of the fetch of waves and duration of the wind action are also factors that influence the height of waves.

During severe storms the highest waves often do not occur when the wind velocity is a maximum but are observed soon after the wind begins to subside. This is explained by the fact that when the wind velocity is greatest the tops of the waves are blown off, the waves becoming broken and irregular; but when the force of the wind diminishes the confused seas have the tendency to unite into a system of much larger and more regular waves than those previously existing. As the wind subsides the waves continue to travel by inertia with diminished velocity

Table 1

Dimensions of deep sea waves

Wind			Waves			
Description	Beaufort* No.	V k.p.h.	T sec.	L m.	H m.	L/H
Strong breeze	6	43	7.2	80	5.3	15
Strong wind	7	56	8.9	120	6.6	19
Fresh gale	8	67	10.6	175	7.9	22
Strong gale	9	80	12.6	250	9.4	26
Whole gale	10	95	15.2	360	11.4	32
Storm	11	108	18.3	520	13.6	38
Hurricane	$>$ 12	$>$125				

*Beaufort number is an arbitary system for classifying wind according to its strength, with -0- describing a calm wind to -12- describing a hurricane. The system is ascribed to Admiral Beaufort.

over long distances.

For many years wave height was estimated by a formula devised by Stevenson from observations of sea waves and later modified by Molitor to include wind speed as a factor.

This Stevenson-Molitor formula is:

$$H = 0.17\sqrt{V_wF} + 2.5 - \sqrt[4]{F}$$

where

H = Height of wave in feet.

F = Fetch in statute miles.

Statute mile = 5280 feet

Nautical mile= 6080 feet = 1.15 Statute miles.

V_w = Wind speed in statute miles per hour

Knot = Nautical mile per hour

If the fetch is greater than 20 miles, the last two terms are dropped from the equation.

After a study of observed wave forms, Sverdup and Munk developed charts showing the relations among wave height, wind speed, duration and fetch (Fig.20). An estimate of the wave height is determined from each chart, and the lower of the two values is used. If the lower value comes from the fetch chart (a), the waves run on a shoreline before they develop maximum height. If the lower value comes from the wind duration chart

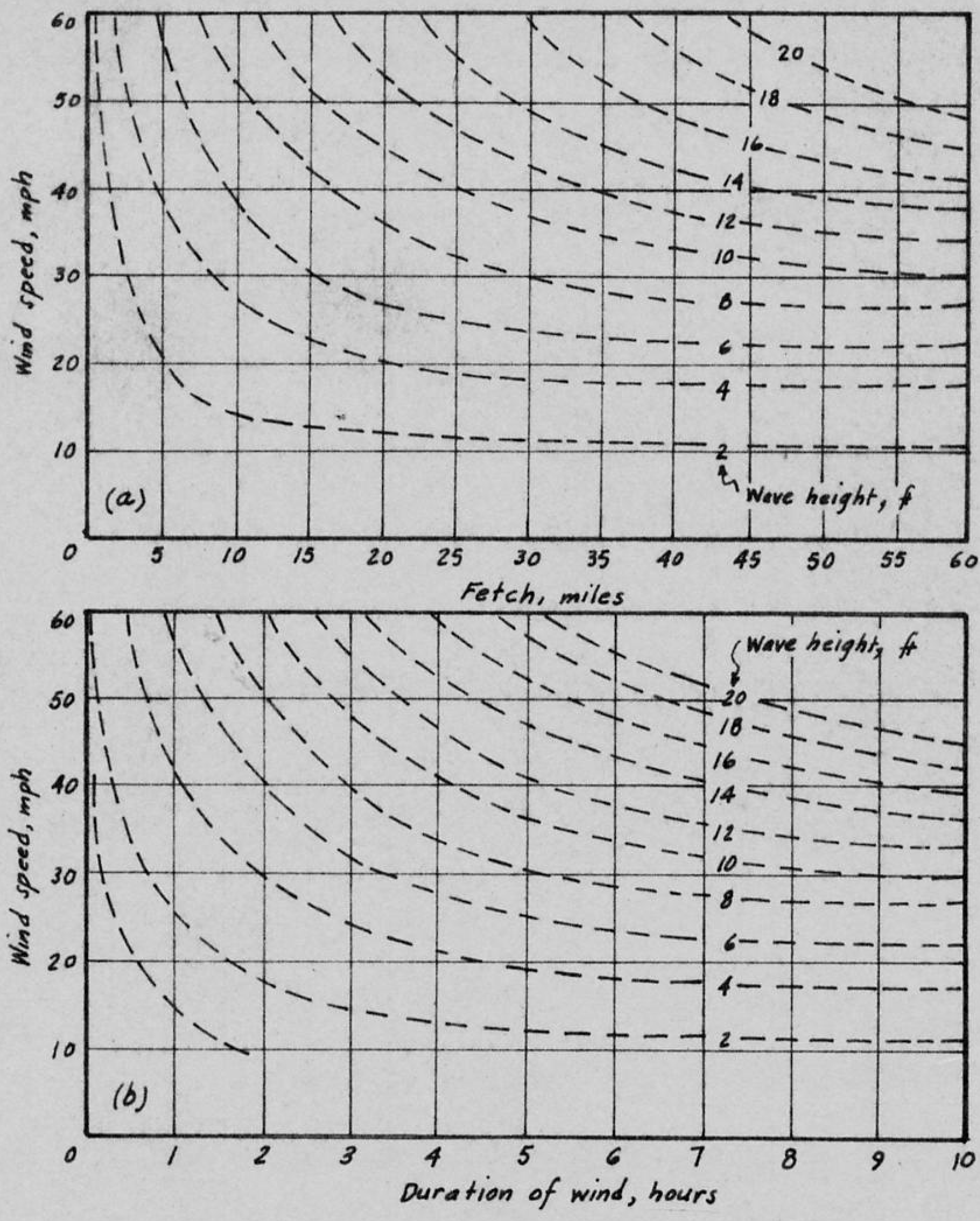

Fig. 20 Relations between wave height, wind speed and duration (After Sverdup & Munk)

Source : ASCE Trans., Vol. 118, p. 558, 1953.

(b), it indicates that the wind duration is too short to develop maximum height.

It is clear, however, that waves can not attain their full development where there is inadequate depth. No wave can have a height greater than the depth of water through which it passes. Consequently, the occurence of shoals in the path of a wave serves to limit its size. Reefs and sandbanks, even though entiraly submerged, materially reduce the range of undulation, and the length of fetch must be guaged accordingly. Thus the effective length of open sea may be much less than the apparent length of fetch.

As the height of wave seldom exceeds 45 feet, there is evidently a limit to the influence of the fetch, which in this case would correspond to 900 miles, while the width of the ocean may be several thousand miles. On the other hand, however, most ocean winds have a rotary direction, it is seldom that violent winds follow approximately a straight course of more than 900 miles.

From past records observations and analysis of data, the averages of recorded maximum wave heights in certain seas are as follows:

Mediterranean Sea	6 meters.
Black Sea	9 meters.
North Atlantic Ocean	12 meters.

South Atlantic Ocean	15 meters.
Pacific Ocean	22 meters.

e. Length of waves: The length of wave is a feature which seems to be independent of the height, though it is connected in some way with the amount of exposure to wind action, and it influences the force of the wave. In the Atlantic Ocean waves of from 150m to 180m. between crests have been observed, while in the Pacific they are stated to reach anything from 180m. to 300m. Numerous and widely distributed observations have indicated, according to Gaillard, that the ratio of length of wave to height $^{L}/H$, usually called steepness of wave, varies between the following limits:

For inland lakes $^{L}/H$ is between 9 and 15

For ocean waves $^{L}/H$ is between 17 and 33.

In both cases the more violent storm conditions are represented by the smaller ratio.

For ocean waves the following are given by G. Shott:

Moderate wind	: Beaufort scale 5	L = 33 H
Strong wind	: Beaufort scale 6-7	L = 20 H
Storm	: Beaufort scale 9	L = 17 H

The length of waves in the open sea is a difficult matter to determine owing to the absence of any reliable linear standard. Alongside jetties and piers, the obstacles in the way of exact measurements are not so great and serviceable computations may be made.

Observing the length of time in seconds, which elapses between the passage of the same point by two successive crests and calling this period (T), we have

$$L = \frac{gT^2}{2\pi}$$

2. Transformation of waves approaching shore:

Waves generated in deep water will be propagated almost indefinitely with little change in form until they approach shore. As they enter regions of decreasing depth, however, the wave characteristics change; the height increases, and the wave length and celerity decrease, until finally the wave breaks and terminates in an uprush of water on the shore. Mass conservation requires that the wave period remains essentially constant, and it is found also that energy dissipation is usually negligible **until** the actual point of breaking and uprush.

Furthermore, the wave may be affected by the phenomena of reflection, refraction and diffraction.

In order to determine the design pressures and forces on submarine pipelines, as well as beach erosion and deposition, the characteristics of waves must be determined as they approach the shore.

Consider first the case of a train of oscillating waves approaching a straight shoreline, with a uniformly sloping bottom. Assume also that the wave crests are paralled to the shoreline, so that their approach is at right angles to the shore and offshore bottom contour lines as sketched in Fig. 21.

In shallow water, the celerity will decrease as the depth decreases. Since the period is assumed constant, the wave-length ($L = CT$) must also decrease. If the rate of energy transmission is also constant, then it is reasonable to expect the wave height to increase.

For deep water oscillatory waves, Morris expresses the total energy in a wave as $E = \frac{1}{8}\ wL.\ H^2$, half of this energy is retained by the orbiting fluid particles and only one half is transmitted with the wave form (10). The energy being transmitted by the wave form is therefore $\frac{1}{16}\ wLH^2$.

The power, or rate of energy transmission, is thus:

$$P = \frac{1}{2}\ \frac{E}{T} = \frac{1/8\ wLH^2}{2L\ /\ C} = \frac{w}{16}\ H^2C$$

On the assumption of unit power transmission, with negligible frictional dissipation, the relation between deep water and shallow water wave heights is then

$$H = Ho\left[\frac{Co}{C}\right]^{1/2} = Ho\left[\frac{Lo}{L}\right]^{1/2}$$

where

Ho, Co = the height and celerity of wave in deep water.

H, C = the corresponding values in shallow water.

By such calculations, the wave characteristics can be

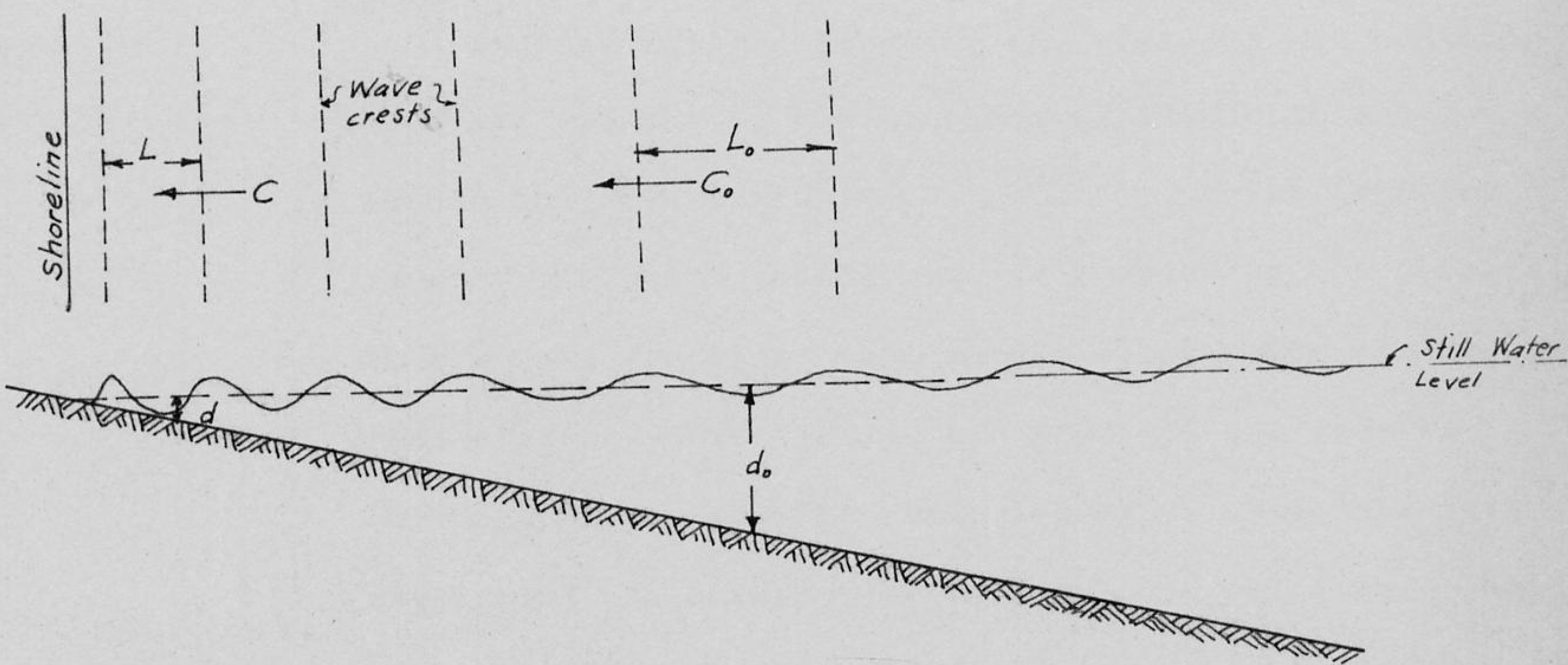

Fig. 21 Waves approaching a sloping shoreline

approximately calculated for any arbitrary location near the shore.

3. Breaking of waves:

A wave will break when the increasing velocity of the water at the wave crest exceeds the decreasing velocity of the wave form. The water then overtakes the wave form and the wave falls over and breaks.

Another factor affecting the collapse and breaking of waves relates to the decrease of volume of water within the wave form. As the particle orbits increase in size the water in the wave is reduced by the decrease in the wave length so that insufficient water remains to complete the orbit causing the front of the wave to become unsupported. The crest therefore collapses into the trough and the wave breaks. Breaking will occur when the depth of water is 4/3 of the wave height at the break-point. Gaillard found that if the breakers were due solely to a swell of the sea, the ratio of the depth of water to breaking H varied with the bottom of the sea; with a slope of 1% the depth is equal to H, but with a slope of 8% the depth is equal to 2H.

There are two types of breakers depending largely on the beach gradient and the wave steepness: these are plunging and spilling breakers as illustrated in Figs. 22 and 23. The former type in which the crest of the wave falls into the through enclosing a pocket of air and breaking into surf, normally occurs when a fairly low wave approaches a steep beach. A spilling breaker, on the other hand, advances at the correct speed for the depth with a foaming crest; the wave does not lose its identity, but gradually decreases in height until it becomes swash on the beach. Such waves are often fairly steep in deep water and advance over a gently sloping, usually sandy beach. It is these waves that produce several rows of breakers advancing shoreward simultaneously and they may be called surf waves.

The problem of maximum steepness (L/H) that a wave can attain without breaking was investigated by Stokes (1847), Mitchel (1895), and Havelock (1918). Their conclusions were much in agreement, and a crest of 120 degrees or a steepness ratio (L/H = 7) was found to be the theoritical limit.

4. A numerical example:

A wave train is generated in deep water in the Mediterranean ($\frac{L}{H}$ = 18) by a wind blowing in a direction perpendicular to shore, over a fetch of 100 miles at a speed of 40 knots. Offshore contours are parallel to the shoreline, with the bottom sloping at 1 vertical to 5 horizontal.

a. Calculate the probable wave height, length, period and celerity in deep water ?

b. Calculate the height of the crest of wave above still water level and the depth of the trough below it ?

c. Determine the velocity at a depth equal to half the wave length ?

d. How far offshore does the bottom begin to exert an appreciable effect on the waves ?

e. How far offshore will the waves break ?

f. Determine the characteristics of waves at this point of breaking ?

Solution:

a. Probable wave height by using the Stevenson-Molitor formula:

V_w = 40 knots = 40 X 6080 / 5280 = 46 statute mi./hr.

$$H = 0.17\sqrt{V_w F}$$

$$= 0.17\sqrt{46 \times 100} = 11.6 \text{ ft} = 3.5 \text{ m.}$$

$$L = 18\,H$$

$$= 18 \times 3.5 = 63 \text{ m.}$$

$$C = \sqrt{\frac{gL}{2\pi}}$$

$$= \sqrt{\frac{9.81 \times 63}{2 \times 3.14}} = 10 \text{ m./ sec.}$$

$$T = \frac{L}{C}$$

$$= \frac{63}{10} = 6.3 \text{ sec.}$$

b. The height of the crest above still water level

$$h_1 = \frac{H}{2} + 0.79\,\frac{H^2}{L} = \frac{3.5}{2} + 0.79 \times \frac{(3.5)^2}{63} = 1.90 \text{ m.}$$

The depth of the trough below still water level

$$h_2 = \frac{H}{2} - 0.79\frac{H^2}{L} = \frac{3.5}{2} - 0.79 X \frac{(3.5)^2}{63} = 1.60 \text{ m.}$$

c. $C = \frac{2\pi r}{T}$, so $r_o = \frac{CT}{2\pi} = \frac{L}{2\pi} = \frac{63}{6.28} = 10$ m.

At a depth of 31.5 m $= \frac{31.5}{10} = 3.15\ r_o$

the radius of the orbit becomes

$$10\ X\left(\frac{1}{\sqrt{e}}\right)^{2X3.15} = 10\ X\ (0.6)^{6.30} = 10\ X\ 0.04 = 0.40 \text{ m.}$$

$$V = \frac{2\pi r}{T} = \frac{2\ X\ 3.14\ X\ 0.40}{6.3} = 0.40 \text{ m/ sec.}$$

d. At a depth equal to the wave length, the bottom of the sea starts to exert an appreciable effect on the characteristics of the wave, that is at a depth of 63 m.

Distance offshore = 63.0 X 5 = 315 m.

e. To determine the depth at which the wave is expected to break in shallow water, we use the equation

$$H = H_o\left[\frac{Co}{C}\right]^{1/2}$$

H_o = 3.5 m.

C_o = 10 m./sec.

$C = \sqrt{gd}$

and H = d at breaking on a shore with a slope of 0.2

hence $d = 3.5\left(\frac{10}{gd}\right)^{1/2}$

$$d^{2.5} = \frac{122.5}{\sqrt{9.81}} = 39$$

and $d = H = 4.4$ m.

The distance offshore = 4.4 X 5 = 22 m.

f. Characteristics of wave at point of breaking in shallow water

$H = d = 4.4$ m.

$$C = \sqrt{gd}$$

$$= \sqrt{9.81 \times 4.4} = 6.6 \text{ m./sec.}$$

$$L = CT$$

$$= 6.6 \times 6.3 = 41.5 \text{ m.}$$

$$L/_H = 41.5/_{4.4} = 9.4$$

5. Wave forces:

Forces affecting underwater pipelines are of two sources: the first is of the non-breaking wave and the second and more severe is the breaking wave.

Non-Breaking waves:

The orbital velocities induced by passing waves exert a dynamic pressure against immersed objects proportional to the square of the velocity. A frictional drag is also exerted which, as in the case of a rough surface (such as that caused by barnacles), might be considerable. In cases of unburied pipes, a drag may also be caused by the turbulance of the wake in the lee of the object. The typical streamlines and differential pressure distribution about the periphery of a pipe are sketched in Fig 24.

Lateral forces acting on the pipe can be estimated by methods proposed by Wiegel (11):

$$F = 2\rho X a + \frac{1}{2} C_D \rho A U^2$$

F = force due to inertia, frictional drag and form drag.

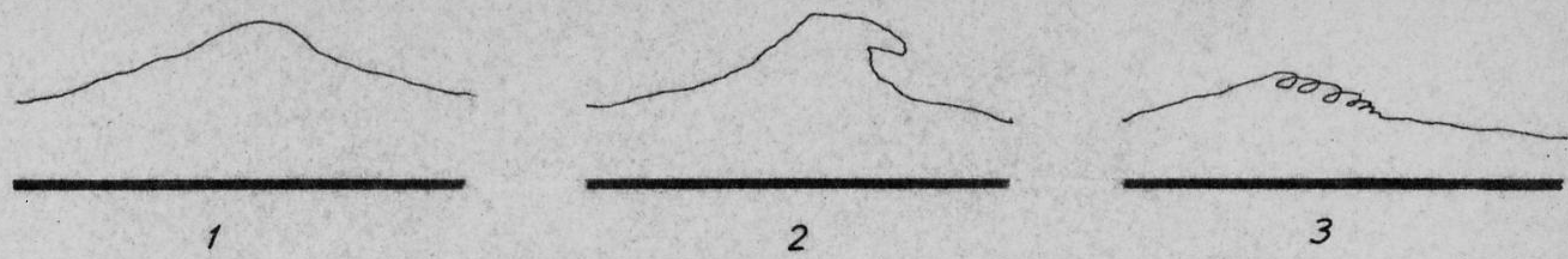

Fig. 22 Plunging breaker

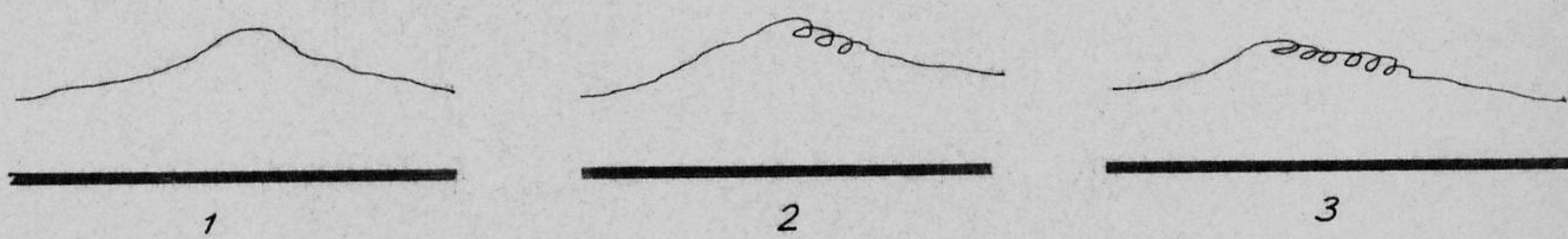

Fig. 23 Spilling breaker

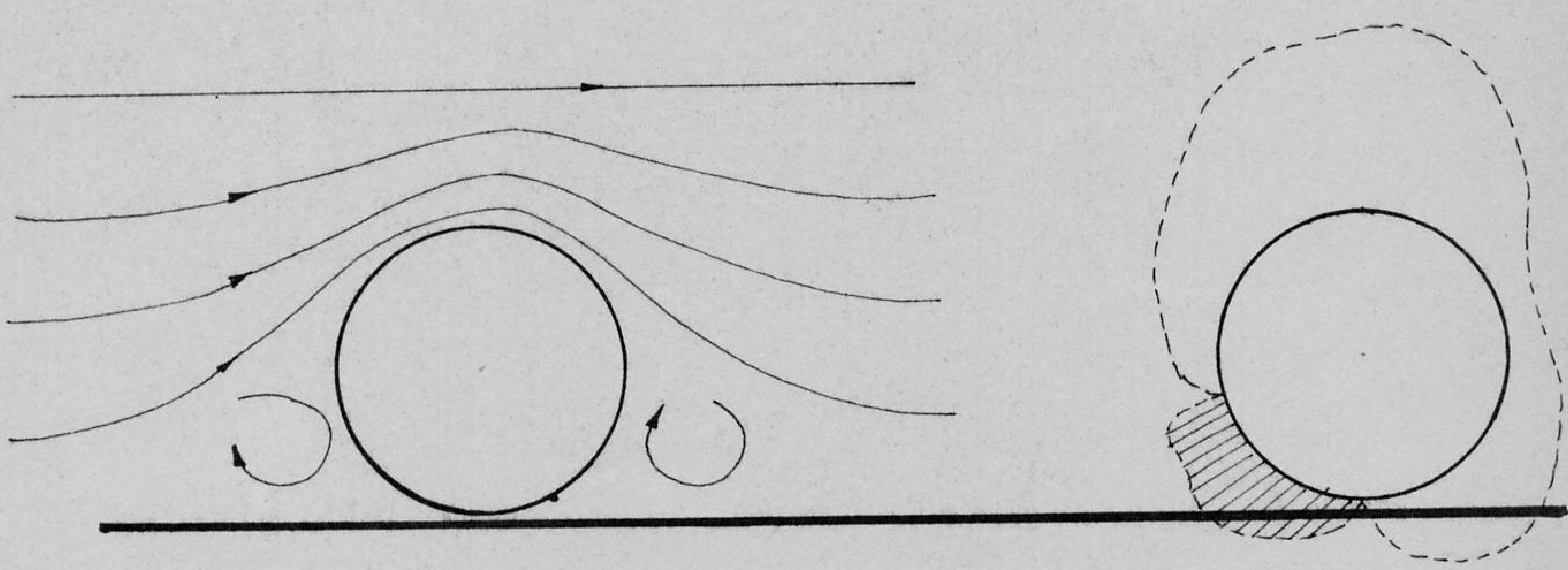

Fig. 24 Streamlines and Differential pressure diagram about the periphery of a pipe

ρ = mass density of fluid.

$\forall$ = Volume of fluid displaced by the body.

a = acceleration of the undisturbed fluid at the center of the body were the body not there.

C_D = Coefficient consisting of both form and friction drag depending upon Reynolds number, Diameter of pipe, roughness, turbulence of the flow, etc.

A = projected area perpendicular to stream velocity.

U = horizontal stream velocity.

A great deal more research is needed in this aspect of underwater pipeline design since the above equations are theoritical and since the coefficients are difficult to determine due to the different boundary effects of the bottom. The only means of determining these coefficients is to set up models and establish by experimental procedures the coefficients of the actual bottom and burial conditions.

Lift force can be determined by the following formula

$$F = \frac{1}{2} C_L \rho AU^2$$

where

C_L = coefficient of lift.

Values of the drag and lift coefficients for a submerged pipe were determined by the Hydraulics Research Station at Wallingford in the United Kingdom, to be:

$$C_D = 0.9$$

$$C_L = 0.5$$

According to Wiegel (12), the horizontal component of wave velocity (U) in a non-breaking wave passing over the pipeline can be estimated by:

$$U = \frac{\pi H}{T} \frac{\cosh \frac{2\pi}{L} (d-z)}{\sinh \frac{2\pi d}{L}} \cos \frac{2\pi t}{T}$$

where

d = still water depth

z = depth below still water at which the velocity is measured.

t = time measured as positive from the time the crest of the wave passes to the center of the object.

The horizontal velocity (U) is a maximum when $\frac{2\pi t}{T} = 0^{\circ}$ or 180°

Hence it follows that

$$U_{max} = \pm \frac{\pi H}{T}$$

This certain value can be established from the consideration of the Trochoidal theory as such

$$U = \frac{L}{T} = \frac{\pi H}{T}$$

The horizontal acceleration is readily obtained by differentating the expression of the horizontal velocity U with respect to time:

$$a = \frac{2\pi^2 H}{T^2} \frac{\cosh \frac{2\pi}{L} (d-z)}{\sinh \frac{2\pi d}{L}} \sin \frac{2\pi t}{T}$$

and the value of the acceleration is a maximum when

$$\frac{2\pi t}{T} = 90^\circ \quad \text{or} \quad 270^\circ$$

Hence

$$a_{max} = \pm \frac{2\pi U_{max}}{T}$$

Breaking waves:

The greatest energy of a wave is available and transmitted at the point of breaking, In addition to dynamic pressure and drag forces, there is a shock pressure induced by a breaking wave which may be considerable especially in the case of rigid structures. Theoritically the greatest shock pressure will result when the front of the wave is approximately in a vertical plane and contacts a vertical wall at that instant.

The difficulties arising in the determination of the precise efforts of a wave are due to several causes. In the first place, there is the incompressibility of water, combined with the extreme mobility of its particles. Arrested suddenly in the course of the motion, it produces all percussive effects of a solid body in an infinite number of directions. The effects of these shocks may be transmitted through joints, or cracks, first by hydraulic pressure, second by pneumatic pressure due to air entrapment, and third by vibrations of the material in the structure.

In the second place, the wave stroke is both abrupt and continuous; its first blow is sharp and decisive, and of high momentary intesity; succeeded by statical pressure during the small interval of time for the dispersal of the wave. Accordingly, there are two phases to be considered:

- The initial concussion
- The subsequent pressure

The question is dealt with sometimes as a matter of simple continuous impact, but it should be noted that wave action is far from being completely indentical with the unbroken impluse of a water jet.

According to the principles of dynamics, the reaction of a surface subjected to continuous impact is measured by the rate at which momentum is destroyed. if, therefore, (W) denotes the weight of a unit volume of water, (WV/g) is the mass which impinges on unit surface in unit time, and (WV^2/g) is the rate at which momentum is consumed. Hence, if (P) is the pressure on unit surface, we have $P = WV^2/g$. Accordingly, if it is possible to determine the velocity of impact, the pressure momentarily exerted may be deduced from this relationship.

Now in order to determine the velocity, we have to take into consideration the depth of water in which the wave is moving. In deep water, the speed is practically independent of the depth, and is almost exactly equal to the velocity acquired by a body falling freely through a height equal to one-half the radius of the circle, the circumference of which constitutes the length of the wave (8) that is

$$V = \sqrt{\frac{gL}{2\pi}}$$

On the assumption that the wave is trochoidal in form, the height of the wave bears to the length the ratio $L = \pi H$. Hence we can write

$$V = \sqrt{\frac{gH}{2}}$$

and substituting in the equation of pressure, we get

$$P = \frac{WV^2}{g} = \frac{WH}{2}$$

which leads to the conclusion that the pressure intensity of a wave in deep water is equal to the weight of a column of water one-half as high as the amount of free fall required to generate the specified velocity in particles of which the wave is composed.

As a result of experimentation performed by Gaillard (13), and allowing for an increase in velocity of the crest of wave, while breaking, he found that an average value could be adopted:

$$P = 1.7\ wh$$

The value assigned for pressure in the foregoing equation is based on the assumption that the line of action of the wave is perpendicular to the surface on which it impinges. When the line of incidence makes an angle (α) with the surface, the pressure, according to Lord Raleigh (14) becomes:

$$P = \frac{2g\ \pi \sin\alpha}{2+g \cdot \sin\alpha} \cdot WH$$

for the case (α) = 90° , P = 1.96 WH which is the formula proposed by the investigator for such a case.

Based on the forgoing studies, it is possible to construct a table of relative heights, velocities and pressures for breaking waves, as follows:-

Table 2

Pressures of breaking waves

Height m.	Velocity m./ sec.	Pressure of impact ton / m^2
5	8.0	7.0
10	12.5	14.0
15	15.0	22.0
20	17.0	30.0
25	19.0	35.0
30	21.0	45.0

D. Scour:

The passage of a wave over a pipe causes a pronounced pressure differential and current along the bottom, with the current travelling in one direction for the crest and in the opposite direction for the trough. This action causes loosening of the soil particles and consequently some particles are dislodged and eroded with the moving waves.

The variables significant to this problem are the wave height, length and period, the depth of water, the embankment slope and the number of waves impinging on it.

As to the depth of water, Professor Airy found that in the open sea when the depth is great in comparison with the length of the wave, the motion of water at considerable depth below surface decreases in geometrical progression and at depth below the surface equal to the length of the wave, is less than 0.02 percent of the surface movement. For practical purposes, it can be said that the effect of waves in deep water becomes negligible at a depth of half the wave length below water surface.

Additional variables to be considered are the specific gravity, porosity of the sand and the diameter of particles.

In the case of an unburied pipe, the situation will be more serious, since more eddies will be formed which tend to suck up particles from the bottom and carry them away.

The removal of the supporting soil is the danger, and the pipe has to be anchored in such a way so that the free span will not exceed the spanning capacity of the pipe.

A layer of riprap dumped around the pipe is beneficial sometimes and the whole case is better studied by setting up an experimental model.

E. Measures Against Oceanographic Agencies:

As part of the design process, the engineer must consider environmental factors which might tend to limit the useful life of the pipe. In Chapter Two, exterior protection and cathodic protection were discussed as to types and modes of application.

To safeguard against lateral forces, scour and floating, the following measures should be studied:

1. Pipe bedding:

The pipe must be designed for the types of bedding conditions expected along the length of the line. The bedding varies from pipe laid on a prepared bed in an open trench to that placed directly on the ocean bottom with no seperation.

In order to take care of minor variations of the bottom, the first condition of support is that of line leading. Although the bottom is rather soft and the pipe will undoubtedly sink sufficiently to spread out the load, it is not expected that this will happen uniformly along the full length of the pipe.

The second condition is the possibility that a full pipe section will receive absolutely no support from the ocean bottom. This then requires that the pipe section be capable of spanning its own length and that the joint be capable of transferring the resulting loads.

The third type of loading results from a condition in which the pipe receives the reaction of a section of an unsupported pipe. This will result in a concentration of load at point. In order to evaluate the effect of this loading, the pipe is assumed to settle a certain distance, and the resulting soil pressure is established and checked with the capability of the existing soil.

2. Pipe burial:

The pipeline must be protected from displacement due to waves, tides and currents. The most positive means of accomplishing this, is to bury the pipe below the bottom surface.

In the case of hard non-erodible sea beds, such as rock or hard boulder clay, a pipeline need be buried only to its own depth, i.e., so that its surface is just covered. With erodible sea beds, consideration must be given to burying the pipeline below known or foreseeable scour level.

Other justifications for burial of pipelines are:

a. To provide protection against corrosion and abrasion.

b. To provide protection against movement by breakers.

c. To provide protection against marine borers.

d. To provide some protection against dynamiting and hanging anchors.

e. To provide adequate support under the line to safeguerd against scour and consequently prevent spanning.

f. To prevent possible adverse effects of a leaky joint. If the opening occurs at the underside of the pipe, the leak may act as a jet scouring out the support from underneath the pipe. This in turn will result in more settlement of the pipe, thereby increasing the amount of leakage, and scouring out even more material at a faster rate. Eventually this may result in the joint completely opening up.

In backfilling against the laid pipe, firstly dirt or sand are hauled for filling in next to the pipe in the trench and above the crown of the pipe for the purpose of protecting the coating. Large sharp boulders should not be dumped against the coating.

3. Pipe anchorage:

The reasons for burying the pipe through the surf and nearshore areas were discussed in the preceding section. However, the requirements for anchorage of that part of the pipe resting directly on the ocean bottom are somewhat different for various types of pipes. The steel pipe will have no joints to open up as a result of local scour and should be quite capable of spanning across any small troughs or channels that can be predicted.

The main need for special anchorage can be divided into two seperate categories namely, forces due to currents and stability against thermal expansion.

Again, the pipe section should be checked for spanning horizontally across anchors, if they are installed. As to thermal expansion; if no anchors are installed the pipe is free to move longitudinally and no stresses are exerted, if it is assumed that temperature stresses will be relieved by permitting the pipe to lengthen, the pipe must stretch over several feet. If the ends of the pipe are restrained against movement, then the extra length must be obtained by buckling of the pipe laterally. It is obvious, therefore, that an investigation must be made concerning the restraining forces against such movements as will be explained in Chapter Four. If the pipe can be safely assumed to receive bottom support, an allowance for fiction can be made, and the frictional resistance can be computed on the basis of the coefficient of friction between the pipe material and sea-bed soil.

4. Buoyancy control:

As mentioned previously in Chapter Two, the control of the pipe weight is an extremely critical factor in construction. Inspection procedures must be very rigorous if tolerances are to be met. Coating and lining thicknesses are determined using the exact weights of materials applied based on laboratory tests. The amount of absorption of sea water into external coating should be determined on the basis of actual immersion tests. This is an important factor as the pipe can pick up about 2 lbs per foot of negative buoyancy from this source.

The important determination to be made is the bulk density of the pipe in relation to the fluid in which it is immersed. Obviously, if this is less than the specific gravity of the fluid, the pipe will float. This is important not only in the context under discussion, but perhaps even more so when considering the trenching and jetting methods of burying. In some sandy sea beds bulk densities in excess of 2 have been found to be necessary. Whilst in silts and muds bulk densities as low as 1.25 are used. The correct pipeline weight is also important to prevent it subsequently working its way us to the sea bed surface.

The design weight is normally provided by an appropriate thickness of concrete coating. In the case of most oil and water lines, where it is known that the pipelines will always remain full of fluid, the weight of the fluid content is included in the specific gravity calculations though it is important to consider the possibility of air pockets becoming entrained within the pipe. With effluent and gas pipelines, the necessary weight must be provided by the pipeline materials.

Pipelines are kept empty during construction and laying and their negative buoyancy (weight in water) is kept to minimum to ease handling and stress problems, consistant with having sufficient weight to resist stability in the selected wave and current conditions, during laying.

In certain types of unstable soils, it is necessary to avoid excessive vertical displacement. A recommended preventive measure is to fill the pipeline before backfilling, then to allow time for rehealing of the soil before removing the water (15).

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CHAPTER FOUR

STRUCTURAL DESIGN CRITERIA

Having established the point of connection or discharge, and in consequence obtained the length of pipeline involved, and the external forces that it will be subject to during the laying operation and in the final resting position, it then becomes purely a problem of structural engineering with due consideration to achieve maximum economy in materials, fabrication and construction operations. The basic objective of the design process is to place the pipeline without damage and hold it in place against oceanographical and hydraulic forces for the design life.

The forces that should be taken into consideration are:

A. Internal Pressures:

The internal pressure within a pipe is caused by hydrostatic pressure and water hammer. The hydrostatic pressure should allow for flushing pressures occasionally needed, and for the specified test pressure which is usually 1.5 times the design head.

1. Internal pressure causes circumferential tension in the walls of the pipe which may be derived as follows:

In Fig. 25 is shown a half section of a thin cylindrical shell of length (l), thickness (t), and internal radius (r), subjected to internal pressure (p), and held by the circumferential tension forces (F). If the unit tensile stress in the circumferential direction is denoted by (s), them F = slt. The force (df) exerted by the internal pressure upon a differential surface area $lrd\theta$ is $plrd\theta$. The component of this force in the x - direction is $R.lr\cos\theta\, d\theta$. By integrating the latter expression over the entire surface, the total force in the x-direction resulting from internal pressure becomes:

$$\int_{-\frac{\pi}{2}}^{\frac{\pi}{2}} plr\cos\theta\, d\theta = 2plr$$

Since the circumferential tension is in equilibrium with the internal pressure

$$2\,s\,l\,t = 2\,p\,l\,r$$

$$\text{or} \qquad s = pr/t$$

It has been established that in a cylindrical shell,

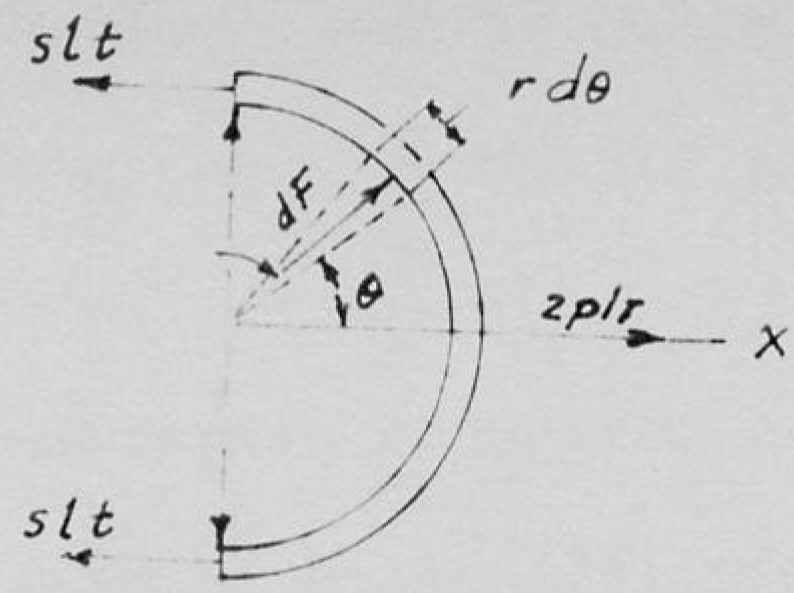


Fig 25-a Internal pressure

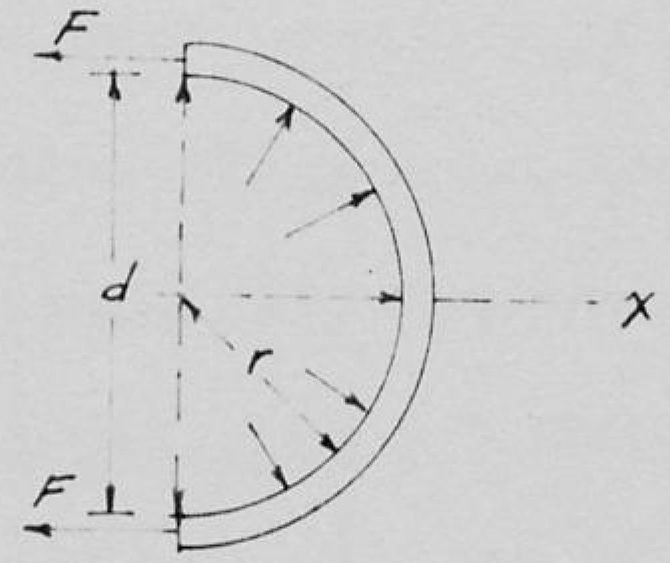


Fig. 25-b Hoop tension due to Internal pressure

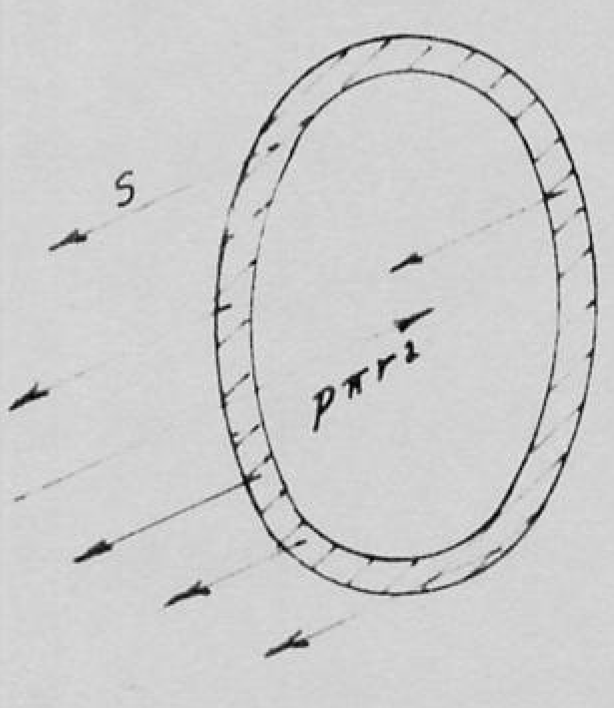


Fig. 26 Longitudinal stresses

internal pressure acting radially outward gives rise to a tensile stress in the cirumferential direction, called "hoop tension".

To allow for corrosion reduction in thickness and for tolerance in manufacture Miller (1) proposed the following formula for computing the thickness of pipe needed to resist pressure:

$$t = \left[\frac{pr}{s(1-A)} \right] + c$$

where

t = wall thickness in inches

p = internal pressure in psi

r = radius of pipe in inches

s = maximum allowable working stress

A = Underthickness tolerance of pipe expressed as a decimal part of (t)

c = wall thickness allowance for corrosion

In general, the intensity of hoop tension varies from a maximum at the inner surface to a minimum at the outer surface. When the thickness of a cylindrical shell is small relative to the diameter of the shell (about 1:50), the variation in the intensity of stress is small, and the stress may be considered uniformly distributed. For thick-walled pipes, error will be introduced if the same assumption, is made, and the following analysis, as derived by Timoshenko, is the conventional procedure (2):

If a circular cylinder of constant wall thickness is subjected to the action of uniformly distributed internal and external pressures, the deformation produced is symmetrical about the axis of the cylinder and does not change along its length. In Fig. 27 we consider a ring cut from the cylinder by two planes perpendicular to the axis and at unit distance apart. From the condition of symmetry it follows that there are no shearing stresses on the sides of an element mnn_1m_1 of this ring, Fig. 27 . The element mnn_1m_1 is bounded by two axial planes and two concentric cylindrical surfaces. Let σ_t denote the hoop stress acting normal to the sides mm_1 and nn_1 of the element, and σ_r the radial stress normal to the side mn. This stress varies with the radius r and changes by an amount $(d\sigma_r/dr)$ dr in the distance dr. The normal radial stress on the side m_1n_1 is consequently:

$$\sigma_r + \frac{d\,\sigma_r}{dr}\,dr$$

Summing up the forces on the element in the direction of the bisector of the angle $d\varphi$ gives us the following equation of equilibrium:

$$\sigma_r\, r\, d\varphi + \sigma_t\, dr\, d\varphi - \left(\sigma_r + \frac{d\sigma_r}{dr}\,dr\right)(r + dr)\, d\varphi = 0 \quad \text{-----}(a)$$

or, neglecting small quantities of higher order,

$$\sigma_t - \sigma_r - r\,\frac{d\sigma_r}{dr} = 0 \quad \text{-----}(b)$$

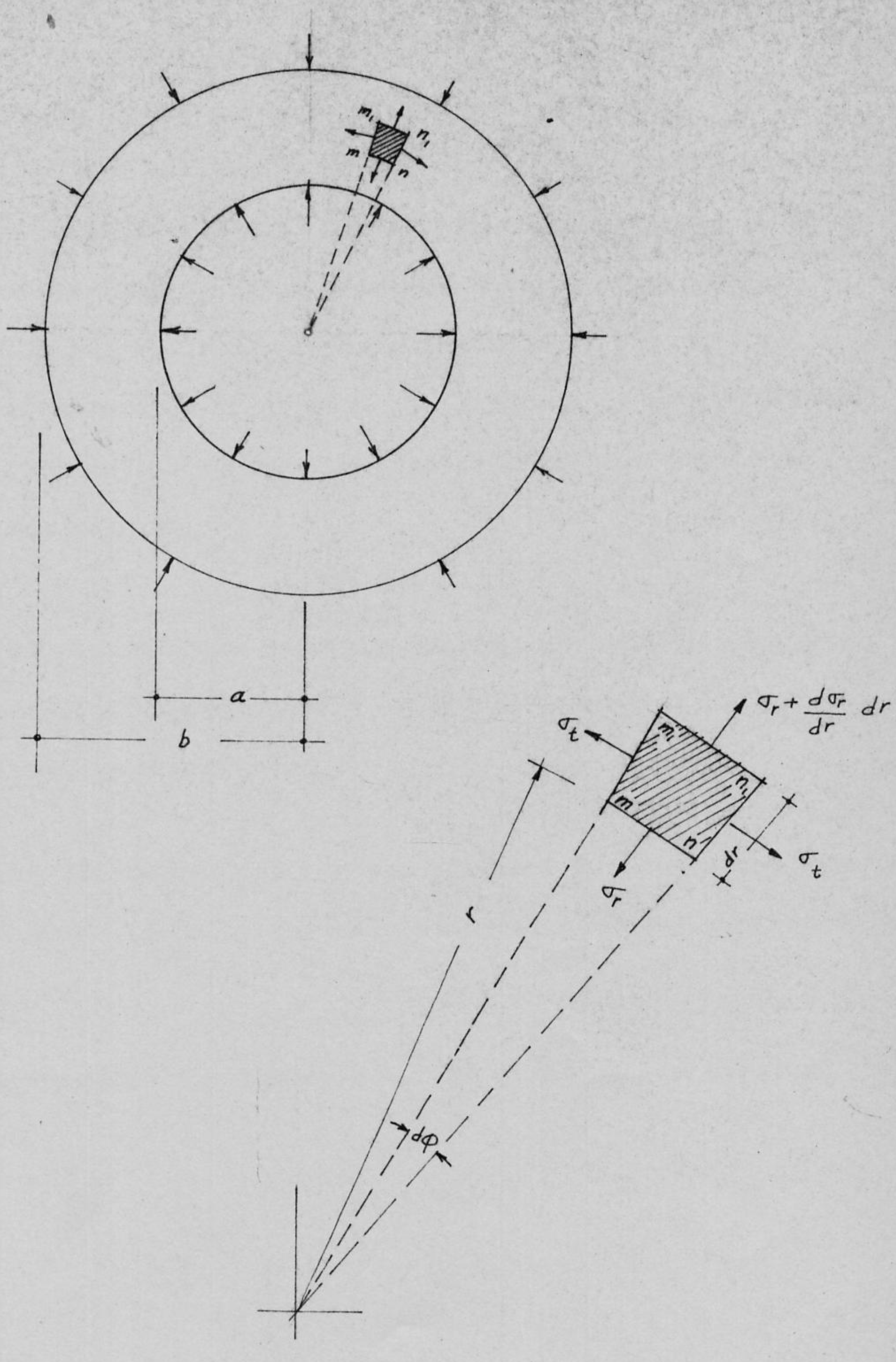


Fig. 27 Thick pipe analysis

This equation contains two unknowns, the stresses σ_t and σ_r The second equation necessary for the determination of these quantities is obtained from a consideration of the deformation of the cylinder.

The deformation of the cylinder is symmetrical with respect to the axis and consists of a radial displacement of all points in the wall of the cylinder. This displacement is constant in the circumferential direction but varies along the radius, i.e., it is a function of the radius only. If u denotes the radial displacement of a cylindrical surface of radius r, then the displacement for a surface of radius r+dr is

$$u + \frac{du}{dr}dr \qquad \text{(c)}$$

Hence an element such as mnn_1m_1 undergoes a total elongation in the radial direction of (du/dr)dr, and the unit elongation is therefore

$$\epsilon_r = \frac{du}{dr} \qquad \text{(d)}$$

The unit elongation of the same element in the circumferential direction is equal to the unit elongation of the corresponding radius, or

$$\epsilon_t = \frac{u}{r} \qquad \text{(e)}$$

The expressions for the stresses in terms of the strains are (3)

$$\begin{aligned} \sigma_r &= \frac{E}{1-\mu^2}\left(\frac{du}{dr} + \mu\frac{u}{r}\right) \\ \sigma_t &= \frac{E}{1-\mu^2}\left(\frac{u}{r} + \mu\frac{du}{dr}\right) \end{aligned} \qquad \text{(A)}$$

The normal stresses σ_r and σ_t are evidently not independent, since they can be expressed in terms of one function u. By substituting from eqs. (A) into eq. (b) we obtain the following equation for determining (u):

$$\frac{d^2u}{dr^2} + \frac{1}{r}\frac{du}{dr} - \frac{u}{r^2} = 0$$

The general solution of this equation is

$$u = C_1 r + \frac{C_2}{r}, \quad \text{---------------} (f)$$

which can be verified by substitution. The constants C1 and C2 are determined from the conditions at the inner and outer surfaces of the cylinder where the pressures, i.e., the normal stresses are known. Substituting from eq. (f) into eqs. (A), we obtain

$$\sigma_r = \frac{E}{1-\mu^2}\left[C_1(1+\mu) - C_2\frac{1-\mu}{r^2}\right] \text{-----} (g)$$

$$\sigma_t = \frac{E}{1-\mu^2}\left[C_1(1+\mu) + C_2\frac{1-\mu}{r^2}\right] \text{-----} (h)$$

If Pi and Po denote the internal and external pressures respectively, the conditions at the outer and inner surfaces of the cylinder are

$$(\sigma_r)_{r=b} = -P_o \quad \text{and} \quad (\sigma_r)_{r=a} = -P_i \quad \text{-------} (i)$$

The sign on the right-hand side of each equation is negative because the normal stress is taken as positive for tension. Substitution of the expression for σ_r, eq. (g) in eqs. (i) gives two equations for determining the constants C_1 and C_2, from which

$$C_1 = \frac{1-\mu}{E} \cdot \frac{a^2 P_i - b^2 P_o}{b^2 - a^2}; \qquad C_2 = \frac{1+\mu}{E} \cdot \frac{a^2 b^2 (P_i - P_o)}{b^2 - a^2}$$

With these values for the constants in eqs. (g) and (h) the general expressions for the normal stresses σ_r, and σ_t become

$$\sigma_r = \frac{a^2 P_i - b^2 P_o}{b^2 - a^2} - \frac{(P_i - P_o)\, a^2 b^2}{r^2 (b^2 - a^2)}$$
$$\sigma_t = \frac{a^2 P_i - b^2 P_o}{b^2 - a^2} + \frac{(P_i - P_o)\, a^2 b^2}{r^2 (b^2 - a^2)} \qquad \text{------ (B)}$$

It is important to note that the sum of these two stresses remains constant, so that the deformation of all elements in the direction of the axis of the cylinder is the same, and cross sections of the cylinder remain plane after deformation. This justifies our consideration of the problem as a two dimensional one.

Let us consider now the particular case when $P_o = 0$, i.e., the cylinder is subjected to internal pressure only. Then eqs. (B) become

$$\sigma_r = \frac{a^2 P_i}{b^2 - a^2}\left(1 - \frac{b^2}{r^2}\right)$$
$$\sigma_t = \frac{a^2 P_i}{b^2 - a^2}\left(1 + \frac{b^2}{r^2}\right)$$

These equations show that σ_r is always a compressive stress and σ_t a tensile stress. The latter is maximum at the inner surface of the cylinder, where

$$(\sigma_t)_{max} = \frac{P_i (a^2 + b^2)}{b^2 - a^2}$$

This equation shows that (σ_t) max is always numerically greater than internal pressure, and approaches this quantity as b increases. The minimum value of σ_t is at the outer surface of the cylinder. The ratio

$$\frac{(\sigma_t)\ max}{(\sigma_t)\ min} = \frac{a^2 + b^2}{2a^2}$$

increases with increase in the thickness of the wall of the cylinder. For a comparatively small thickness there is not a great difference between the maximum and minimum values of σt. Taking, for example, b=1.1a, (σt) max exceeds (σt) min by only 10 $\frac{1}{2}$ per cent. The error is therefore small if we assume the tensile stresses σt to be uniformly distributed over the thickness of the wall and use the equation

$$\sigma_t = \frac{P_i\ a}{b - a},$$

which coincides with the equation cited before for thin cylinders. The shearing stress is maximum at the inner surface of the cylinder where

$$\tau_{max} = \frac{\sigma_t - \sigma_r}{2} = \frac{1}{2}\ \frac{P_i\,(a^2+b^2)}{b^2-a^2} + \frac{P_i\,(b^2-a^2)}{b^2-a^2} = \frac{P_i\,b^2}{b^2-a^2}$$

When only an external pressure acts on the cylinder we have Pi=0 and eqs. (B) give

$$\sigma_r = -\frac{P_o\,b^2}{b^2-a^2}\left(1-\frac{a^2}{r^2}\right) \quad \ldots\ldots\ldots\ldots\ldots(c)$$

$$\sigma_t = -\frac{P\ b^2}{b^2-a^2}\left(1+\frac{a^2}{r^2}\right) \quad \ldots\ldots\ldots\ldots\ldots(D)$$

In this case σ_r and σ_t are both compressive, and σ_t is always numerically greater than σ_r. The maximum compressive stress is at the inner surface of the cylinder, where

$$(\sigma_t)_{r=a} = - \frac{2P_o b^2}{b^2 - a^2}$$

It is interesting to note that as the ratio b/a of the radii of the cylinder is increased, this maximum compressive stress approaches twice the value of the external pressure acting on the cylinder, namely, $- 2p_o$.

2. Water hammer.

When a fluid flowing in a pipeline suffers an abrupt change in velocity, dynamic energy is converted to elastic energy and a series of positive and negative pressure waves travel back and forth in the pipe until they are damped out by friction. This phenomenon is known as water hammer. At the instant the valve of Fig. 28 is closed, the element of water (x_1) just upstream from the valve will be compressed by the water flowing against it. This results in a pressure rise which causes a portion of the pipe surrounding the element to stretch. In the next instant, the forward motion of element (x_2) is stopped, and it, too, is compressed by the remaining water in the pipe, which still possesses forward motion. The process is repeated on successive elements until in a relatively short time the pressure wave has traveled back to the reservoir and all the water in the pipe is at rest.

A pressure in excess of hydrostatic cannot be maintained at the junction of pipe and reservoir, and the pressure at C drops to normal as some of the water in the pipe flows back into the reservoir.

As the reduction in pressure travels back down the pipe toward the valve, the pipe contracts and the water expands until normal pressures exist throughout the pipe. The inertia of the water flowing into the reservoir results in the discharge of more water than the excess originally stored in the pipe. This causes negative pressures, and a refraction wave travels back up the pipe from the valve to the reservoir. Since a presure less than hydrostatic cannot be maintained at C, a wave of normal pressure again travels back down the pipe as water flows from the reservoir into the pipe. This results in a return to the conditions which initiated the water hammer, and the pressure variation at several points along the pipe as a result of instantaneous valve closure is shown in Fig. 28.

The velocity (u) of a pressure wave in any medium is the same as the velocity of sound in that medium and is given by (4)

$$u = \sqrt{\frac{E}{\rho}}$$

where E = the modulus of elasticity of the medium

ρ = the density of the medium

For water under ordinary conditions $u = 1400$ m/sec.

The velocity of a pressure wave created by water hammer is less than that given by the aforementioned equation because of the elasticity of the pipe. If longitudinal extension of the pipe is prevented while circumferential stretching takes place freely, the velocity of a pressure wave u is given by (4):

$$u_p = \sqrt{\frac{E}{\rho}} \sqrt{\frac{1}{1+(ED+E_pt)}}$$

where E_p = the modulus of elasticity of the pipe walls

D = the pipe diameter

t = the pipe wall thickness

If the valve of Fig. 29 is closed instantaneously, a pressure wave travels up the pipe with the velocity .
In a short inter al of time (dt) an element of water of length (.dt) is brought to rest. Applying Newton's second law and neglecting friction,

$$Fdt = M\,dV$$

$$-A\,dp\,dt = \rho A\,u_p\,dt\,dV$$

$$dp = \rho\,u_p\,dV$$

Since velocity is reduced to zero, dv = - V and (dp) equals the pressure P_h caused by water hammer. Hence,

$$P_h = \rho\,u_p\,V$$

The total pressure at the valve immediately after closure is P_h + p, where P is the static pressure in the pipe.

If the length of the pipe is L, the wave travels from

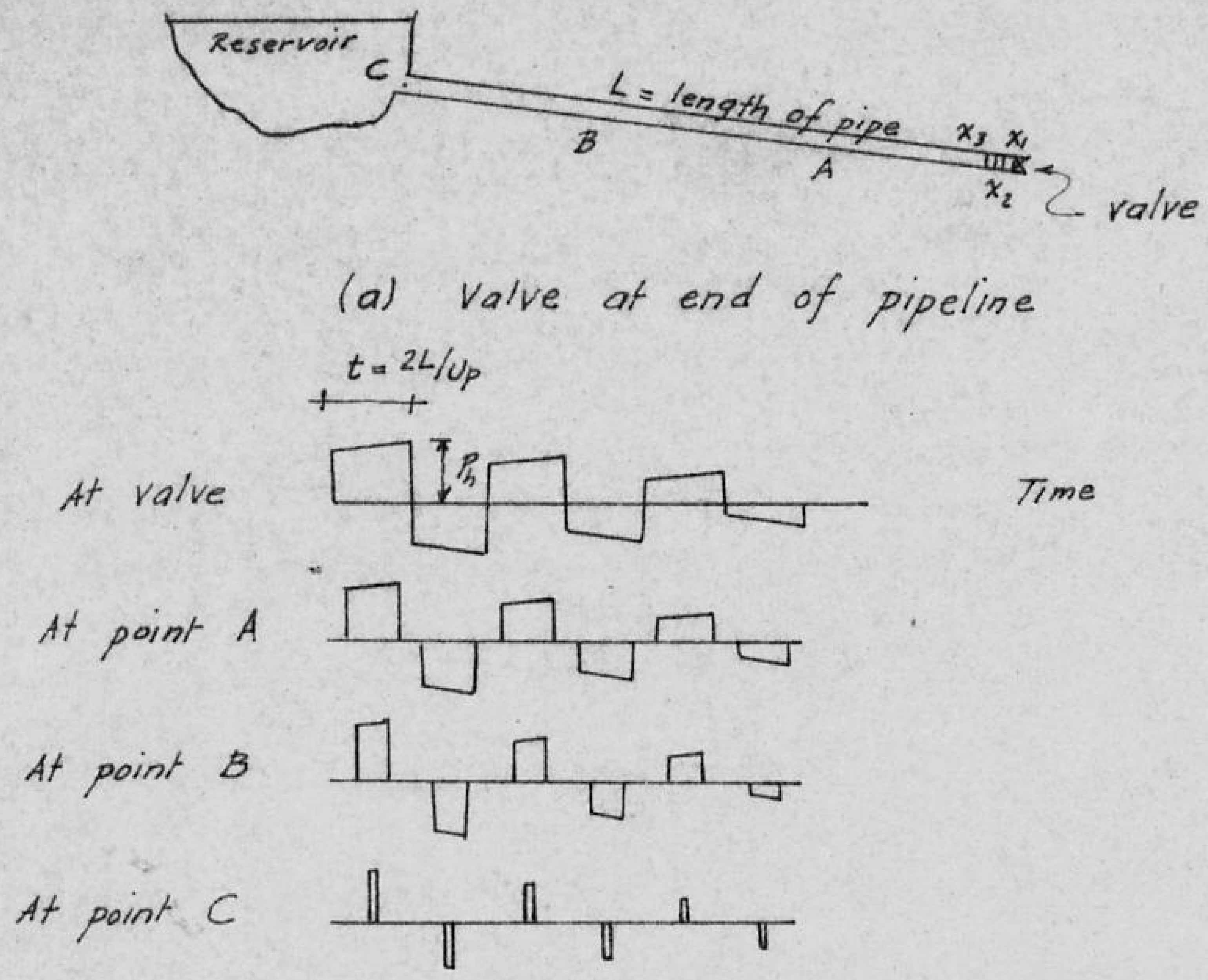


Fig. 28 Water hammer pressures at selected points for the case of abrupt valve closure

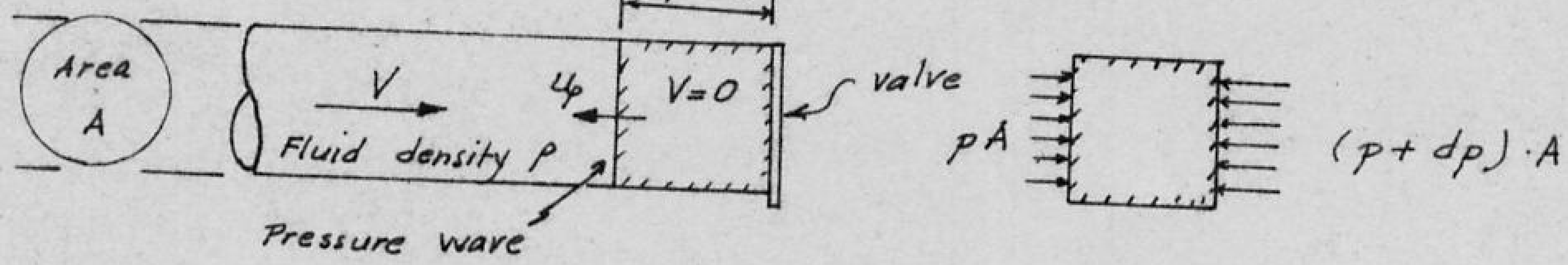


Fig. 29 Definition sketch for water hammer in pipes

valve to reservoir and returns in time $t=2L/u_p$. This is the time that a positive pressure will be maintained at the valve. If the valve is closed gradually, a series of small pressure waves is transmitted up the pipe. These waves are reflected at the reservoir and return down the pipe as waves of normal pressure. If the valve is completely closed before the reflected wave returns from the reservoir, the pressure increase is approximated by

$$P_h = \rho\, u_p V$$

If the closure time t_c is greater than t, negative pressure waves will be superimposed on the positive waves and the full pressure will not be realized. The water hammer pressure P_h developed by gradual closure of the valve when $t_c > t$ is given approximately by

$$P_h' \approx \frac{t}{t_c} P_h = \frac{2 L P_h}{u_p t_c} = \frac{2 L \rho V}{t_c}$$

In order to safeguard against the destructive effects of water hammer, pipes should wherever possible be equipped with slowclosing gates. Air chambers, surge tanks, and other devices placed near the outlets of pipes are in special cases employed to counteract partly the effects of water hammer.

B. <u>External Hydrostatic Pressure</u>:

In deep water, the hydostatic pressure on an empty pipe can be quite considerable, and on large diameter pipes of comparatively thin-walled sections this factor alone can be critical. However, during design, the stresses imposed by hydrostatic pressure are always checked even on small diameters, as no matter how small the stresses imposed, it is necessary to include them in the general summation of simultaneous stress applications. The same remarks apply to the buoyancy tanks, as these can vary in diameter from 75 cms. to 150 cms. However, they are also fitted with pressure valves to enable them to be "prestressed" by compressed air prior to launching. In order to give an example: an ordinary 40 gallon oil drum will collapse at a depth of 7.20m in water. One can imagine the catastrophic effects of buoy collapse on a long pull.

The formule for the buckling of a thin cylindrical tube as derived by Roark (5), is as follows:

$$p' = \frac{1}{4}\left(\frac{E}{1-\mu^2}\right)\left(\frac{t}{r}\right)^3$$

where

p' = critical pressure in psi

μ = poisson's ratio

C. Longitudinal Stresses:

1. Longitudinal tension due to plugging of one end of pipe:

In case the end of a pipe is plugged, the total force exerted upon the end of the pipe normal to the axis of the pipe is ($p \pi r^2$). This force is held in equilibrium by the longitudinal stress acting upon the cross section of the pipe normal to its axis as shown in Fig. 26. The area is $(2\pi rt)$, and if (f) denotes the intensity of the longitudinal tensile stress, the total tension on the section becomes $(2\pi rt.f)$.

Hence $2\pi rt.f = p\pi r^2$

$$f = pr/2t$$

It is seen that the intensity of stress in the longitudinal direction in a cylinder is one-half the intensity of stress in the circumferential direction.

2. Thermal stress:

Changes in temperature occur between time of installation and at different seasons of the year. Moreover, there is the variation due to the depth of pipe below water surface, and the variation due to the temperature of the pumped fluid.

Longitudinal stress of considerable magnitude may develop in pipes exposed to large changes in temperature. The change in length (δ) of a pipe of length (L) when subjected to a temperature change (ΔT) is

$$\delta = \alpha L.\ \Delta T$$

where (α) is the coefficient of thermal expansion of the pipe material. If this change in length is prevented, longitudinal stresses will develop. From mechanics of materials it is known that in the elastic range:

$$б = E\epsilon = E\delta/L$$

where ϵ = the unit strain

E = the modulus of elasticity

and σ = the resulting unit stress. Combining both equations gives:

$$\sigma = E\alpha \cdot \Delta T$$

This indicates the longitudinal stress that would result when a pipe with fixed ends is subjected to a temperature change.

3. Pulling stresses during construction:

The pulling force obviously depends upon the weight of the pipe in water, and the frictional coefficient of the material of sea bed. From the foregoing considerations it will be apparent that the maximum pulling force can not be calculated by merely taking the sectional area of pipe available and multiplying by the allowable working stress. The pulling force has to be very carefully balanced to provide adequate reserve for other stresses imposed on pipe. This is achieved by reducing the weight with buoyancy tanks; or to reduce weight of concrete coating to resist transverse currents by trenching to a depth equal to full or partial diameter of the pipe or a combination of both of these methods.

The pulling force can also be increased by increasing the wall thickness of the pipe itself, but one has to consider the question of anchorge of the barge, and relative economies in the extra costs of materials involved.

D. Flexural Stress Due to Spanning:

The design conditions for the pipe laid directly on bottom are quite different. It is possible due to sea bed undulations or bridging over boulders that a full section of pipe will receive absolutely no support. This requires that the pipe section be capable of spanning its own length. Again during the pulling operation, when the pipe is placed on the rolling conveyors, pipe sections are required to span certain center-to-center distances, though in this case only own weight of pipe is acting.

The hollow cylindrical section is an efficient section for overcoming bending. For our purposes bending moment is considered the only source of pipe injury. These following relationships hold for elastic members in bending. So far as moment, radius of curvature and stress are concerned:

$$M = fI/c = EI/R$$

from which,

$$f = \frac{Ec}{R}$$

where

M = moment at the section

E = modulus of elasticity

I = moment of inertia of pipe

R = radius of curvature

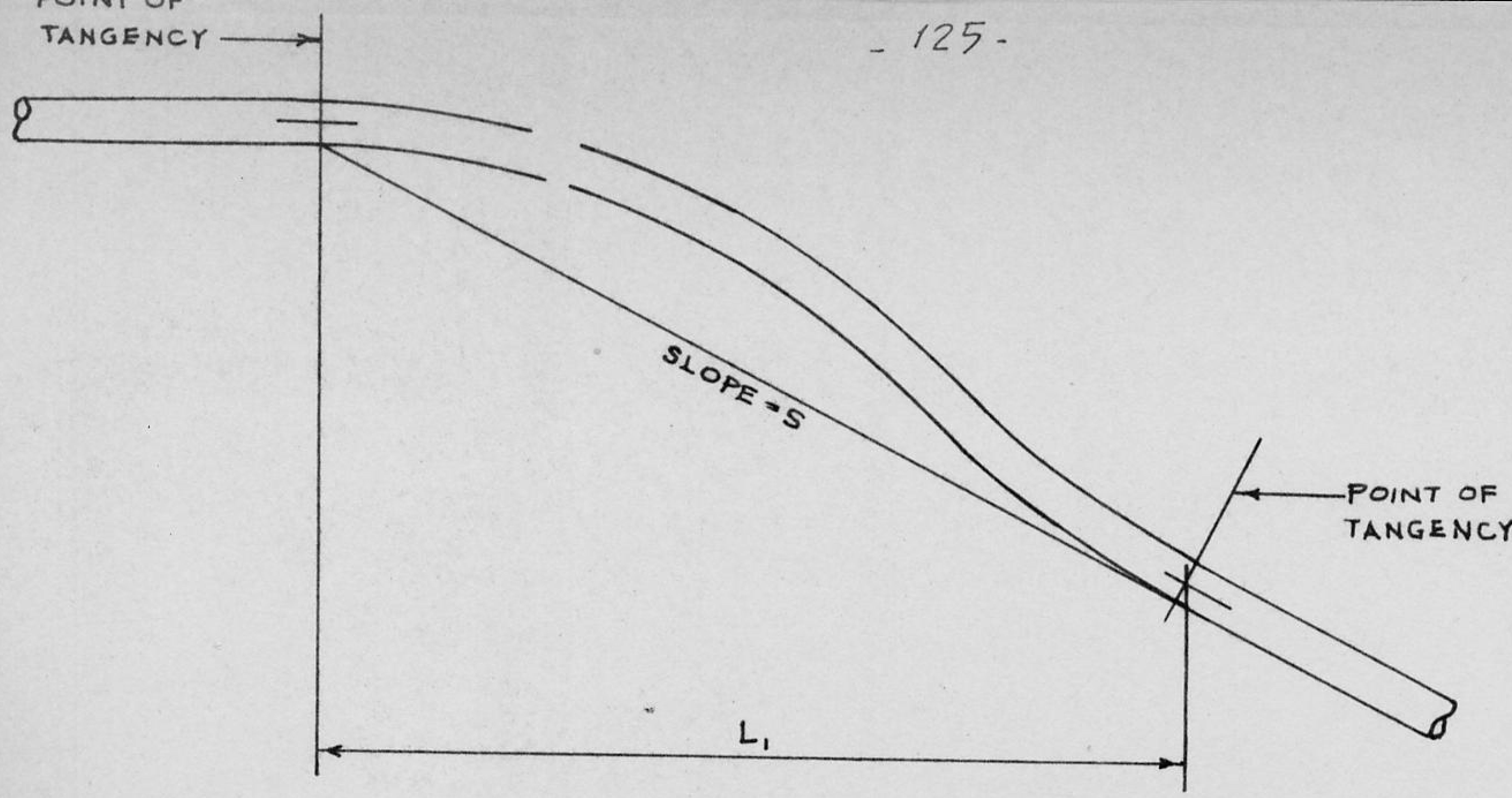
f = maximum bending stress in section

c = distance from neutral axis of section to outer fibers

E. Stress Through Curvature:

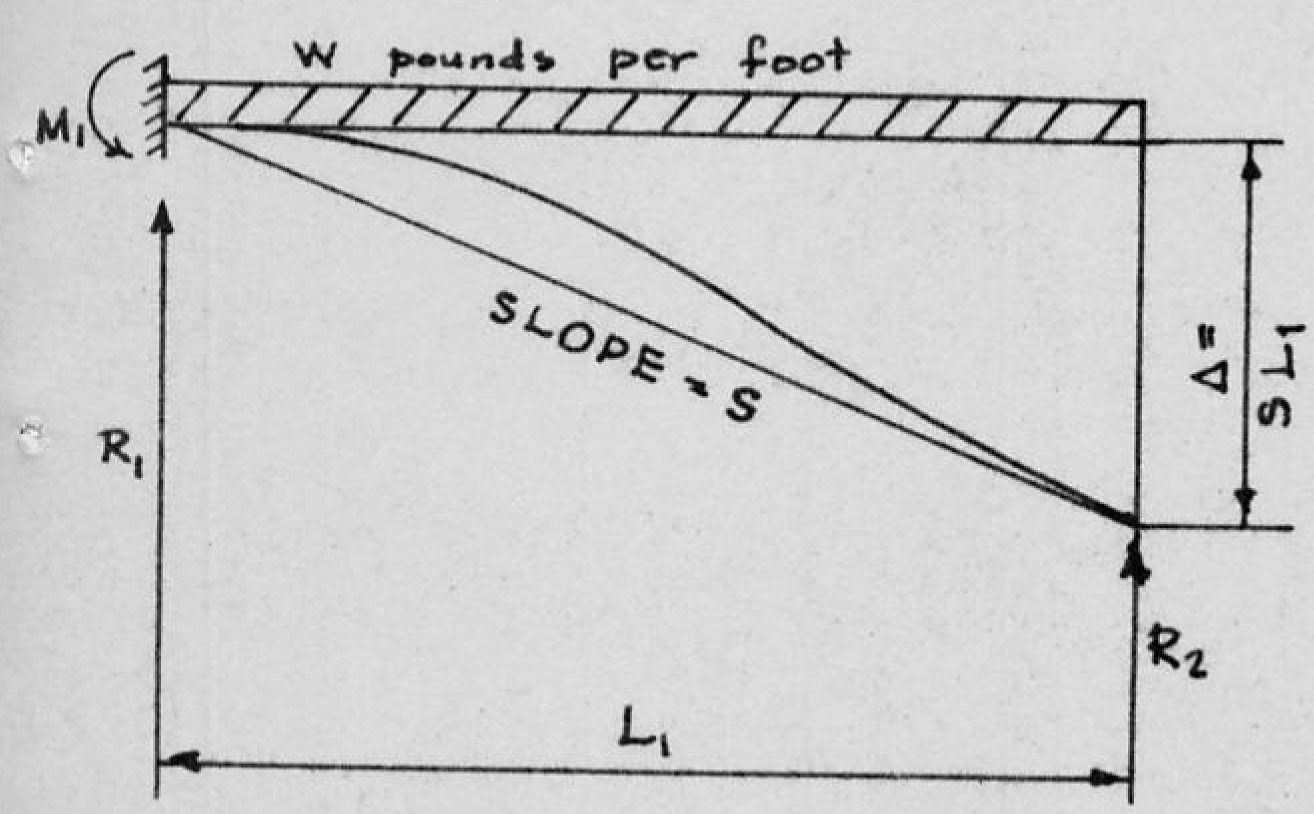
The sag of a pipe section, having a length of the order of 60m. or more, introduces a complex problem from the standpoint of computation of induced stresses. One is dealing with a beam which is so long that when vertical deformation occurs it is accompanied by a significant elongation. The firm supporting sediment will tend to restrain the movement of the pipe at ends of the sagging portion of pipe so that practically all of the elongation will accur in the sagging section. This can induce a net exial tension of considerable magnitude which must be added to flexural stresses induced by the bending of the pipe. In the case of a very long span, the bending effect can become so small that the sagging pipe can be considered as a flexible cable (6). If the span is very short, then the pipe can be designed as a continuous beam with stresses resulting only from bending plus any tension required for pulling the pipe during construction. In order to assure a safe design where sag is likely to occur , it is necessary to compute both the flexural and pure tensile stresses induced by the sag.

For preliminary design equations representing the above conditions, see Figs.30 and 31.



EQUIVALENT BEAM CONDITIONS

OPERATING STATE



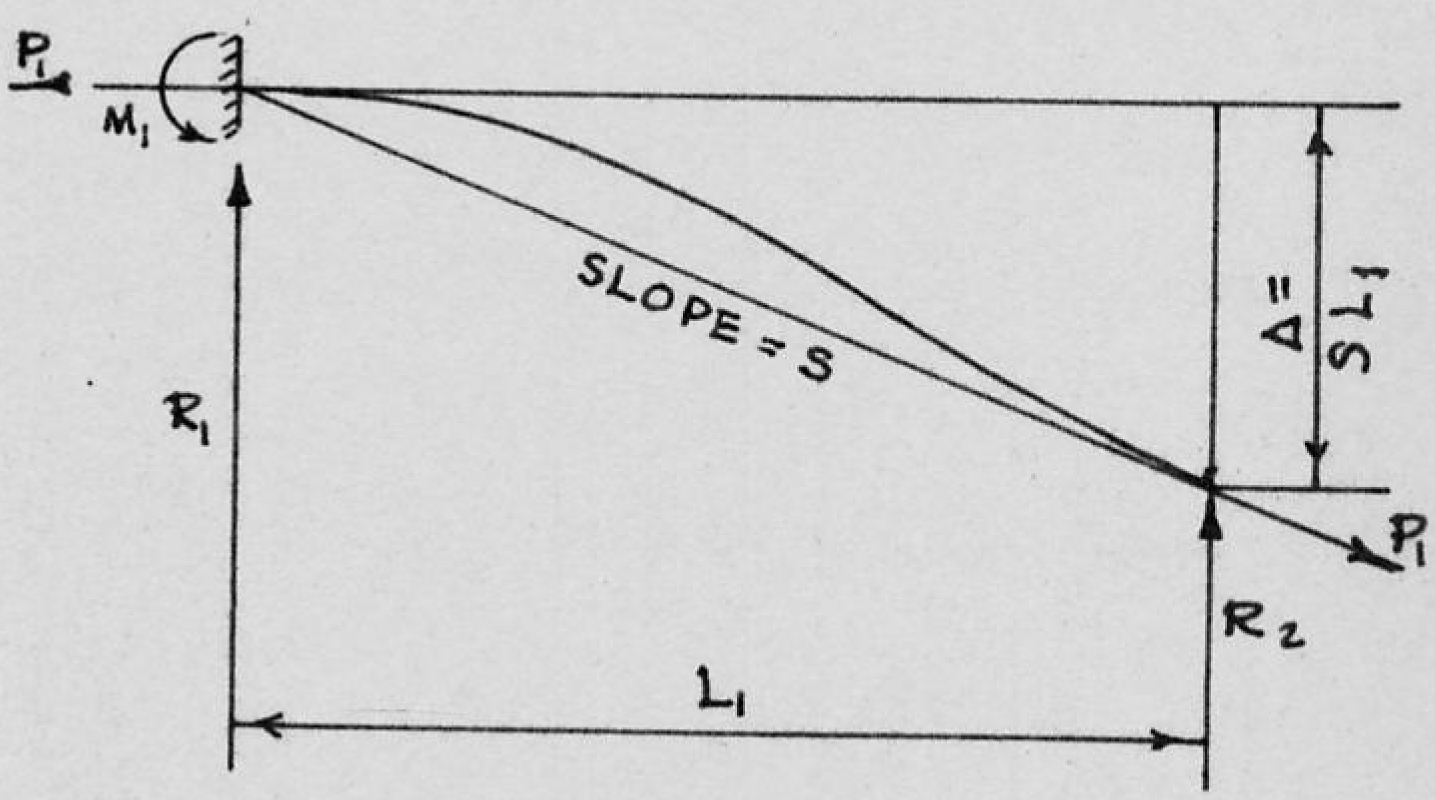
(1) $L_1^3 = \frac{24 S E I}{W}$

(2) $R_1 = \frac{3 W L_1}{4}$

(3) $R_2 = \frac{W L_1}{4}$

(4) $M_1 = -\frac{W L_1^2}{4}$

CONSTRUCTION STATE



(1) $W L_1^3 - 24 S \left(\frac{P_1 L_1^2}{6} - E I \right) = 0$

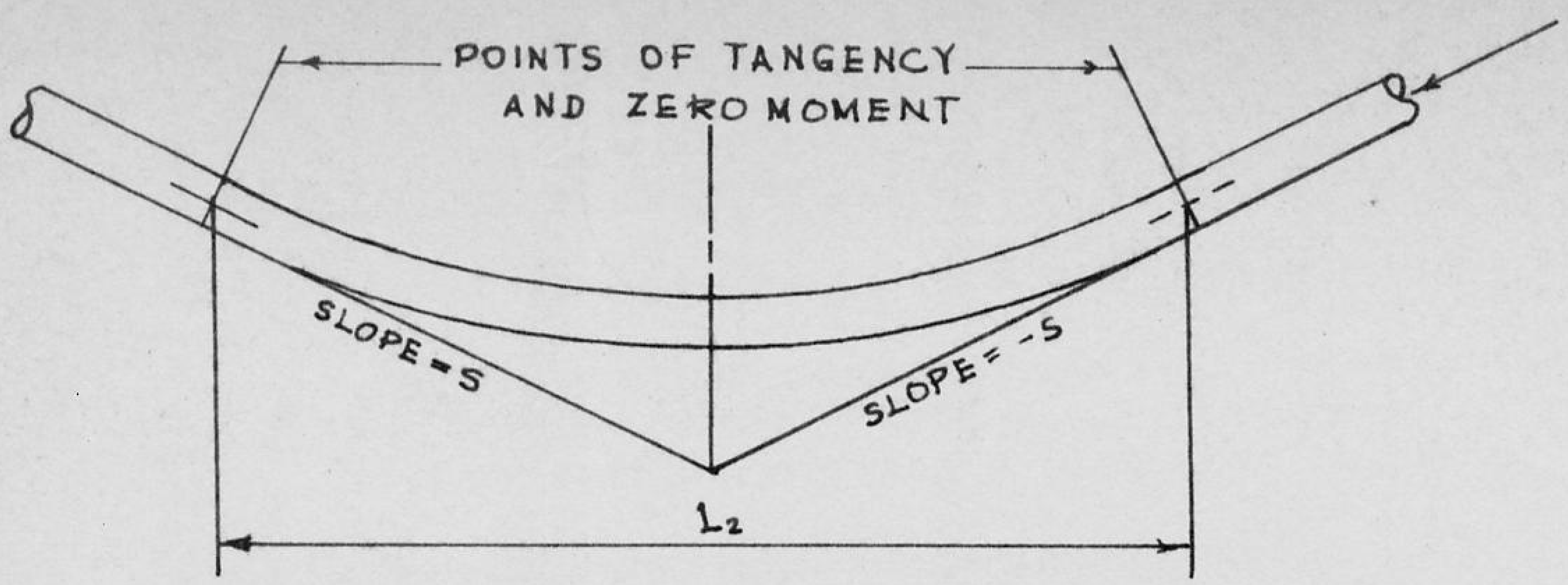
(2) $R_1 = \frac{3 W L_1}{4} + P_1 S$

(3) $R_2 = \frac{W L_1}{4} - P_1 S$

(4) $M_1 = -\frac{W L_1^2}{4}$

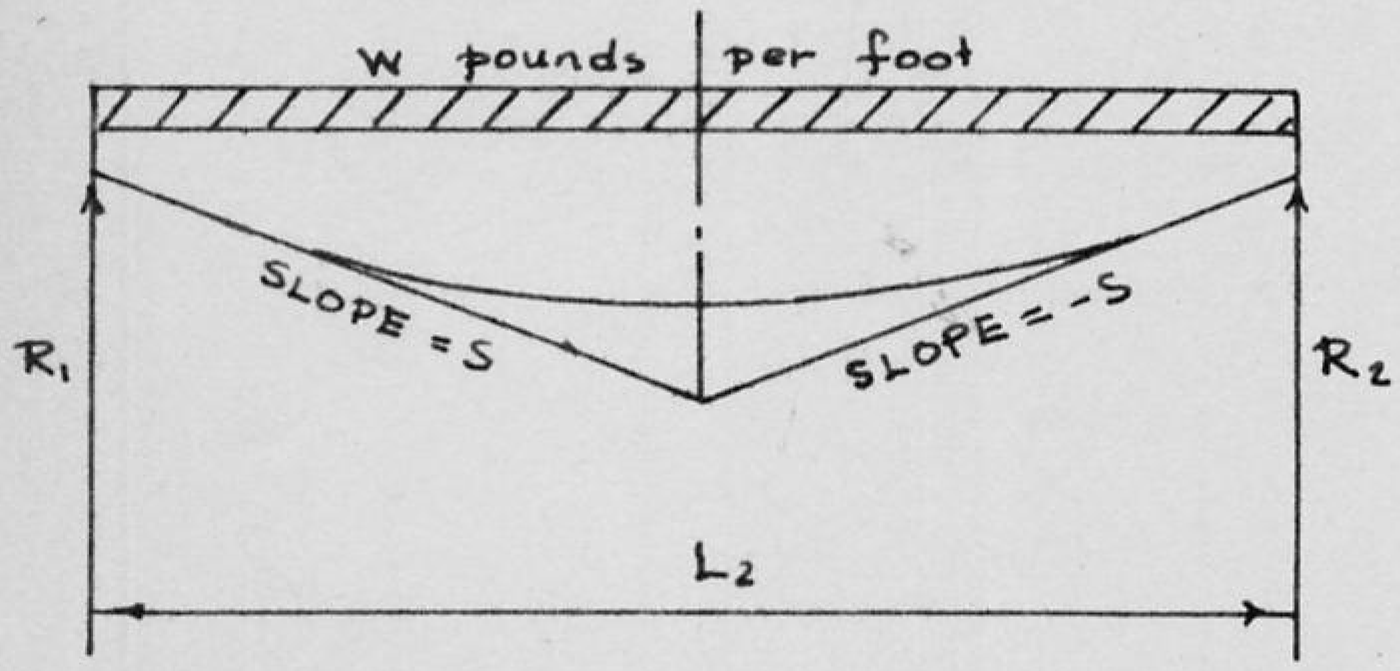
Fig. 30

Preliminary design equations

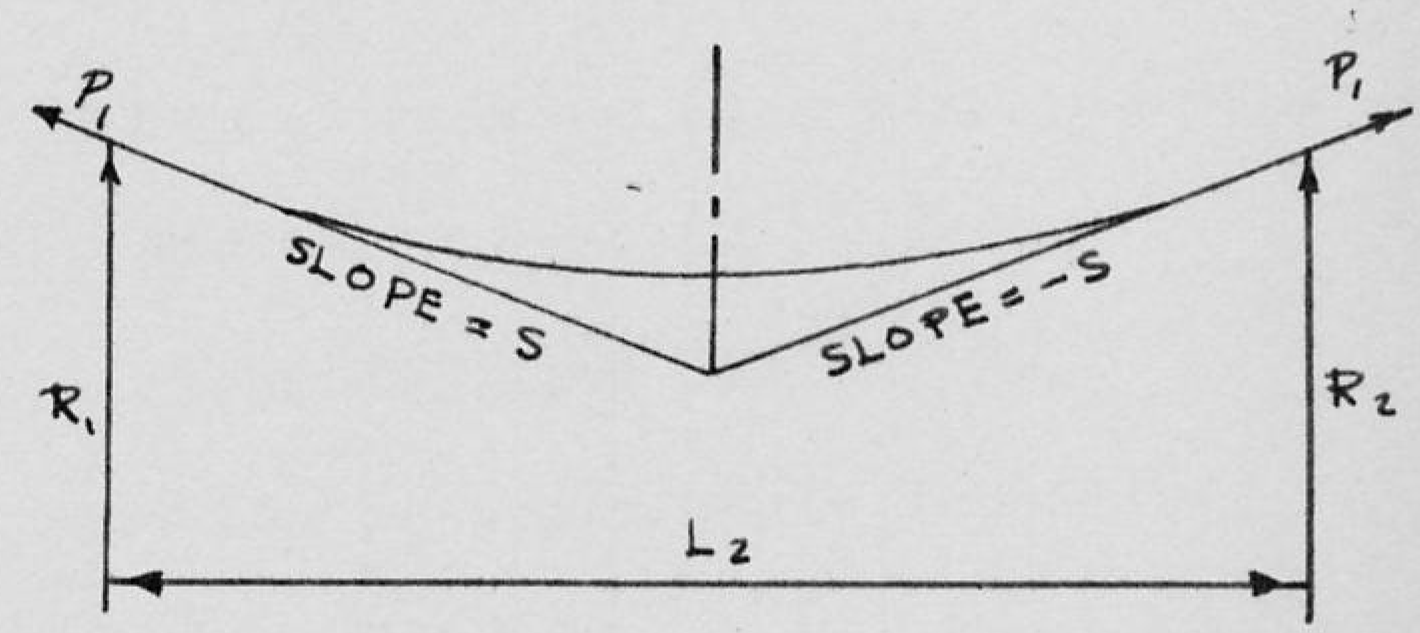


EQUIVALENT BEAM CONDITIONS

OPERATING STATE



CONSTRUCTION STATE



Operating state

① $L_2^3 = \dfrac{24\,SEI}{w}$

② $R_1 = R_2 = \dfrac{w L_2}{2}$

③ $M = \dfrac{w L_2^2}{8}$

Construction state

① $w L_2^3 - 3S(P_1 L_2^2 + 8EI) = 0$

② $R_1 = R_2 = \dfrac{w L_2}{2}$

③ $M = \dfrac{w L_2^2}{8} - \dfrac{P_1 S L_2}{2}$

Fig. 31

Preliminary design equations

F. Other Stress Considerations:

Detailed stress calculations should include the factors of bending, direct tension, hoop tension, yielding of supports and other secondary stresses induced at points of support.

1. Special construction considerations:

The shape assumed by a pipe when laid by the lay-barge technique approximates a catenary form. It closely conforms to a beam with uniform loading fixed on the upper end and infinite in length, the lower end resting on a level surface (7). At the point of rest, the moment and slope become zero. A schematic representation is shown in Fig. 32.

The general moment equation of a uniformly loaded beam is:

$$M = M_a + V_{ab}\,x - \qquad \text{but } M = 0$$

Therefore $M = V_{ab}\,x - \dfrac{wx^2}{2}$ ....-

Integrating eq. -1- $EI\theta = \dfrac{V_{ab}x^2}{2} - \dfrac{wx^3}{6} + C_1$ ......-2-

At $x = 0$. $\theta = 0$; therefore $C_1 = 0$

At $x = L$, $\theta = \theta_b$; therefore $EI\theta_b = \dfrac{V_{ab}L^2}{2} - \dfrac{wL^3}{6}$ ......-3-

Solving for V_{ab}; $V_{ab} = \dfrac{2EI\theta_b}{L^2} - \dfrac{wL}{3}$

V_{ab} is also the reaction at the bottom

Combining -1- and -3-

$$M = \frac{2EI\theta_b x}{L^2} + \frac{wLx}{3} - \frac{wx^2}{2} \qquad \text{......-4-}$$

Solving for $M = M_b$ at $x = L$

$$M_b = \frac{2EI\theta_b}{L} + \frac{wL^2}{3} - \frac{wL^2}{2} = \frac{2EI\theta_b}{L} - \frac{wL^2}{6} \qquad \text{......-5-}$$

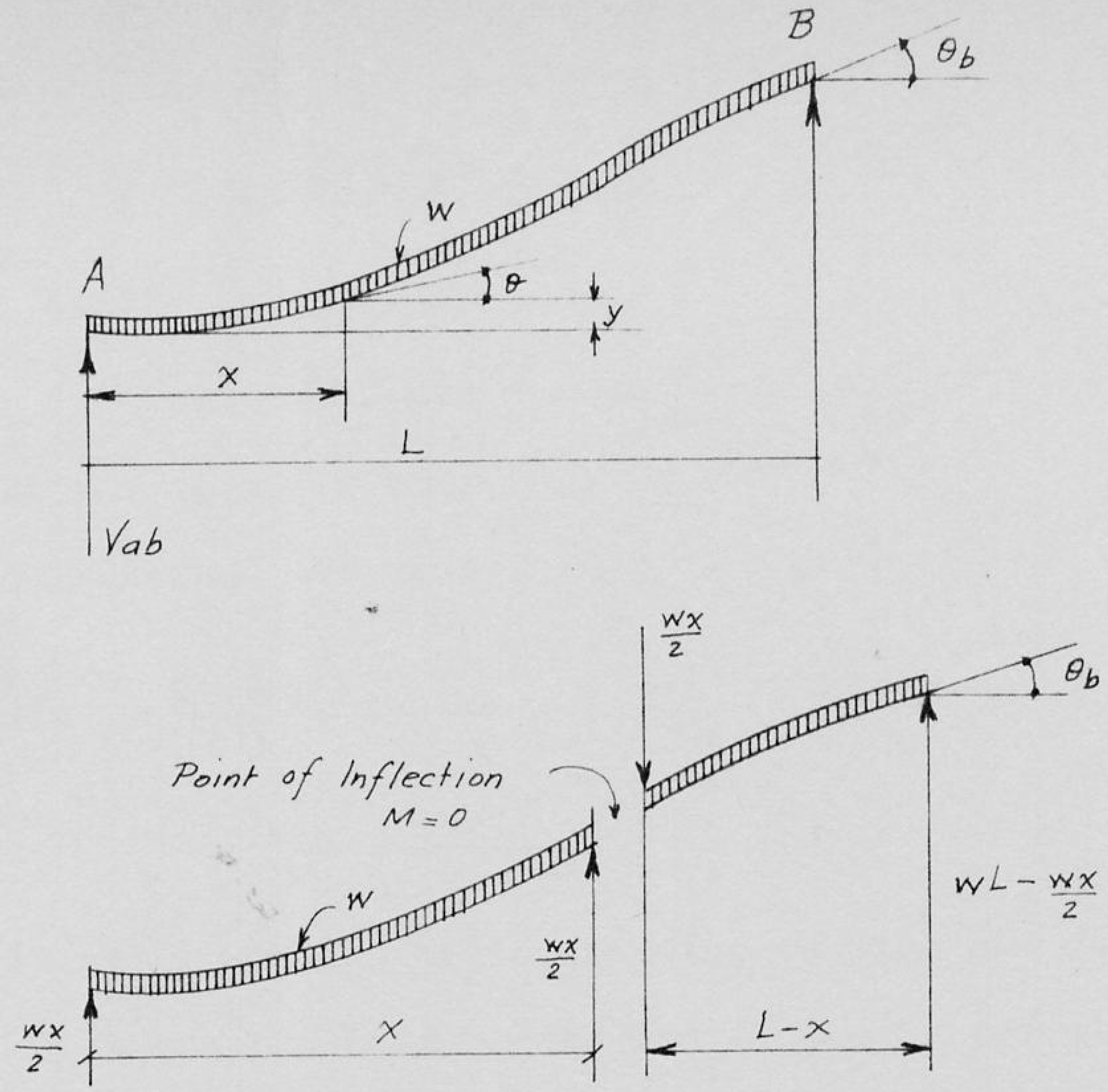


Fig. 32 Force diagrams for negative buoyancy method

Explanation of Symbols Used

- M Moment
- M_a Moment at point pipe touches bottom - O
- M_b Moment at point of last support
- M_c Moment at center of sag portions of pipe
- M_{fa} Fixed end moment at A if both ends are fixed
- M_{fb} Fixed end moment at B if both ends are fixed
- L Unsupported length of pipe
- x Horizontal distance from origin
- y Vertical distance from origin
- y_b Height of support above bottom
- θ Angular deflection from horizontal - radians
- θ_b Angle of pipe at last support - radians
- E Modulus of elasticity $= 30 \times 10^6$ psi for steel
- I Moment of inertia of pipe.
- R Radius of curvature
- S Maximum bending stress in section
- C Constant of integration
- w Net load on pipe
- Z Section modulus
- c Distance from neutral axis of section to outer fibers.
- V_{ab} Shear at A in direction at B

Combining -2- and -3-

$$EI\theta = \frac{2EI\theta_b x^2}{L^2} + \frac{wLx}{3} - \frac{wx^2}{2} \qquad \text{......-6-}$$

Integrating eq. -6-,

$$EIy = \frac{EI\theta_b x^3}{3L^2} + \frac{wLx^3}{18} - \frac{wx^4}{24} + C_2 \qquad \text{......-7-}$$

At $x = 0$, $y = 0$ therefore $C_2 = 0$

Solving eq. -7- at $x = vL$, $y = y_b$:

$$EIy_b = \frac{EI\theta_b L}{3} + \frac{wL^4}{18} - \frac{wL^4}{24} = \frac{EI\theta_b L}{3} + \frac{wL^4}{72} \qquad \text{......-8-}$$

or $$\left[y_b = \frac{\theta_b L}{3} + \frac{wL^4}{72EI} \right]$$

If the launching angle, loading on pipe and depth of water are known, the unsupported length can be calculated from eq. -8- This unsupported length can be sabstituted in eq. -5- to determine the moment at top. Solving eq. -8- for w

$$\frac{wL^4}{72EI} = y_b - \frac{\theta_b L}{3} \; ; \text{or} \quad \left[w = \frac{72EIy_b}{L^4} - \frac{24EI\theta_b}{L^3} \right] \qquad \text{......-9-}$$

The value of the radius of curvature is given by

$$M = \frac{EI}{R} \qquad \text{......-10-}$$

Combining eq. -5,9, and 10- M_b can be shown to be less than zero

$$M_b = \frac{-EI}{R} = \frac{2EI\theta_b}{L} - \frac{L^2}{6}\left(\frac{72EIy_b}{L^4} - \frac{24EI\theta_b}{L^3}\right)$$

$$\therefore \quad \frac{-EI}{R} = \frac{2EI\theta_b}{L} - \frac{12EIy_b}{L^2} + \frac{4EI\theta_b}{L}$$

Multiply -8- by $\frac{RL^2}{EI}$ $\therefore -L^2 = 6R\theta_b L - 12Ry_b$ ......-11-

By regrouping terms and completing squares, then taking square roots of both sides.

$$(L + 3R\theta_b) = (12Ry_b + 9R^2\theta_b^2)^{1/2}$$

$$L = (12Ry_b + 9R^2\theta_b^2)^{1/2} - 3R\theta_b \quad \text{......-12-}$$

If radius of curvature, launching angle and depth of water are specified eq. -12- can be used to determine the unsupported length. Then eq. -10- will give the loading to make the pipe conform to this shape.

So far, only the moment at the upper end has been consid this may not be the maximum moment.

The point of inflection can be determined (using eq. -4-) by setting M = 0

$$M = \frac{2EI\theta_b}{L^2} - \frac{wL}{3} - \frac{wx}{2} = 0$$

Solving for x

$$\therefore \frac{wx}{2} = \frac{2EI\theta_b}{L^2} + \frac{wL}{3} ; \quad x = \frac{4EI\theta_b}{wL^2} + \frac{2L}{3} \quad \text{......-13-}$$

$$\text{Then} \quad L - x = \frac{L}{3} - \frac{4EI\theta_b}{L^2} = \frac{-2}{wL}\left(\frac{3EI\theta_b}{L} - \frac{wL}{6}\right)$$

$$\therefore \quad L - x = \frac{-2M_b}{wL} \quad \text{......-14-}$$

The pipe can be considered as two separate portions. The sag as a supported beam, and the overbend as a cantilivor with a load on the end and a uniform load.

The maximum moment of the sag portion $= \frac{wx^2}{8} = M_c$ ......-15-

The maximum moment of the overbend is

$$\frac{-wx}{2}(L-x) - \frac{w(L-x)^2}{2} = M_b \quad \text{......-16-}$$

If both moments are equal, but opposite in sign,

$$\frac{wx^2}{8} = \frac{wLx}{2} - \frac{wx^2}{2} + \frac{wL^2}{2} - wLx - \frac{wx^2}{2}$$

From this it can be found that

$$x = 2(\sqrt{2}-1)L$$

$$L-x = (3-2\sqrt{2})L = (3-\sqrt{8})L = 0.17\,L \qquad \text{.......-17-}$$

Therefore, if $\frac{2M_b}{wL^2}$ is less than $(3-\sqrt{8})$, the maximum moment is in the sag.

This moment can be calculated from eq. -16-.

Another useful formula derived from the same conclusions is the depth that the pipe can be lifted with certain negative buoyancy.

$$Y = \frac{3EI}{wR^2} \qquad \text{.......-18-}$$

2. Checking stress in the concrete coating; The concrete coating develops hair cracks under high stress conditions imposed. The concrete is not normally taken into account in stress calculations, and the steel pipe is designed to withstand the full stress. The allowable stress in the concrete should however be kept below 170 kg/cm^2 to prevent spalling. This being so, the following radii or curvature will provide guide lines for limits to minimize cracking(8):

26 inches - 700 meter.

20 inches - 550 meter.

16 inches - 420 meter.

12 inches - 350 meter.

In a cement mortar lining, a stress 70 kg/cm2 is not allowed to be exceeded.

For large diameter pipelines, special consideration should be given to the concrete coating as a structural member, and the combined section acts as a thick cylindrical tube.

3. Simultaneous stresses:

Stresses acting simultaneously are summed up, and total temporary stresses up to 80% of yield point are frequently used. Almost invariably, the temporary stresses applied during construction are greater than the permenant working stresses, and in many cases the temporary stresses are the determining factor in selecting pipe quality and wall thickness.

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