THE EFFECTS
OF WAVES AND CURRENTS
ON SUBMERGED PIPELINES

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THE EFFECTS OF WAVES AND CURRENTS
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ABSTRACT

This report is a brief review and summary of selected literature pertaining to the effects of waves and currents on submerged pipe lines. It consists of five parts.

1. Summary or discussion section
2. Selected abstracts
3. Annotated bibliography
4. Translations
5. Bibliography
A survey of published literature was made as part of the coastal and ocean engineering program at Texas A&M University.

The report was primarily written by the senior author in partial fulfillment of the master of engineering degree requirement under the supervision of the junior author who was his major advisor.

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EFFECTS OF WAVES AND CURRENTS
ON SUBMERGED PIPELINES

I. INTRODUCTION

Offshore operators have long recognized the importance of a pipe line link between offshore production and onshore terminal facilities. Although pipe lines are costly at sea, they remain the only positive method available for transporting offshore products, as they are produced, to terminal facilities onshore.

Continuity of operation and the structural integrity of their pipe lines are of vital importance to the pipe line industry. Pipe line failures result in very costly repairs and in some cases the cost of repairs may be exceeded by the production losses.

It is the purpose of this paper to discuss the extreme environmental factors acting on pipe lines. Also, included is a brief review and summary of selected literature pertaining to steps now being taken to reduce the number of failures and to decrease installation cost.
II. THEORETICAL CONSIDERATIONS

The environmental forces acting on a submerged pipeline are considerably different for buried and unburied pipe lines. It is generally agreed that burial of the pipe line affords the best protection from the effects of waves and currents. However, burying pipe lines is expensive and in some locations difficult, if not impossible to perform, due to rocky bottom conditions or the presence of reefs. Because of these differences, the forces on buried and unburied pipe lines will be considered separately.

Forces on Buried Pipe Lines

Pipe lines that are adequately buried normally have no external mechanical forces exerted upon them. However, as Reid\(^{(43)}\) has pointed out, significant forces may act on the pipe while it is being buried.

Present techniques of pipe line burial at sea employ jets and jet-suction dredges to cut a trench beneath the pipe after the pipe has been laid. The trench is usually left open to be filled by natural sedimentation processes. As the sediment accumulates and increases in density, there may be a stage of this transition for which the sediment is still fluid enough to exert a buoyant force considerably greater than that of water.
This is illustrated in Figure 1 where $F_B$ is the buoyant force of the sediment laden water and $F_W$ is the total weight of the pipe including the coating and contents.

Figure 1. Forces on Pipe Surrounded by Sediment Laden Water
If adequate weight is not provided in the design to resist the increased buoyant force, sections of the pipe line may rise in the trench and become exposed as an unburied line. Sufficient weight must be provided to give the pipe an effective specific gravity which is at least equal to the specific gravity of the unconsolidated suspension within the trench. The extra weight may be provided by:

1. Increasing the pipe wall thickness.
2. Increasing the thickness of coating (usually concrete) around the pipe.
3. Weighting the pipe down with ballast.
4. Temporarily filling the pipe with water or some other liquid.
5. Combinations of the above.

**Scour.** Scour is another factor which in some cases may expose a buried pipe line. The expected depth of scour is usually taken into consideration in selecting the depth of burial and, therefore, is not too much of a problem except in local areas or in the vicinity of terminal structures.

Scour depths of 8 to 10 feet have been measured around platforms in the Gulf of Mexico according to Kreig(27). As a result, he recommends that the pipe riser and pipe line adjacent to a recently installed platform be buried about 8 feet deep to avoid exposure. If, however, the platform has been in place for sufficient time to permit the scouring action to reach an equilibrium state, the depth of burial
required should be considerably less.

**Forces on Unburied Pipe Lines**

The hazards encountered by a pipe line laid along the bottom are much more serious than those encountered by a buried pipe line. The principal external forces acting on an unburied line result from high particle velocities associated with waves and/or currents which may be tidal or storm driven. The resultant component forces are referred to as drag, lift and inertial.

Beckmann and Thibodeaux point out that inertial forces do not usually contribute to the maximum load situation for small structures, such as pipe lines located on the bottom, since they occur during acceleration periods and are out of phase with drag and lift phenomena. A typical flow pattern is shown in Figure 2 (a) with the resultant forces shown in Figure 2 (b) for the case of a pipe in contact with the bed. In Figure 2 (b), \( F_L \) is the force of lift, which is opposed by the effective weight of the pipe, \( F_{EW} \); \( F_D \) is the drag force which is resisted by a frictional force, \( F_R \) and the stiffness of the pipe.
In Figure 2 (a), the fluid velocity is increased in going over the pipe which, in accordance with Bernoulli's theorem, decreases the pressure over the pipe. This decrease in pressure above the pipe results in the lift force, $F_L$. In addition, there are pressure differences over the surface of the pipe resulting in the drag force $F_D$, exerted in the direction of fluid flow. It should be noted that the amount of the resisting force, $F_R$, that can be developed is dependent on $F_L$ and decreases as $F_L$ increases. The force of drag and lift can be considerable in magnitude.

According to Reid\(^{(43)}\) in a water depth of 30 feet a wave could attain a height of 23 feet and would have a current velocity, beneath the crest and just above the bottom, of 14 feet per second. This could result in a horizontal
force of approximately 200 lb. per sq. ft. of projected area if the pipe line is aligned perpendicularly to the wave approach. However, Reid points out that the mean drag force along a very long pipe line would be reduced to about 25 per cent of the maximum value or in the above case, a mean drag of approximately 50 lb. per sq. ft. would result. In addition to lift and drag forces exerted on a pipe line, severe scour and the related problems can also affect the pipe line.

Scour. The fluctuating bottom pressures and velocities can completely scour the sediment from underneath a pipe line, at least in a local area. Once a little of the sediment is removed from the bottom of the pipe the flow pattern changes and a downward force is exerted on the pipe. This force will tend to pull the pipe line back to the bed; however, as indicated by Blumberg (6), the development of the Karman vortex trail behind the cylinder will likely prevent any stable downward forces. Figure 3 (a) shows a typical flow pattern for this case with (b) and (c) showing the forces involved on transverse and longitudinal sections, respectively. In Figure 3 (b), $F_D$ is the drag force, $F_L$ is the lift force, $F_{EW}$ is the effective weight, and $F_{OSC}$ is an oscillating downward force.
Figure 3. Flow Pattern (a) and Forces on a Transverse Section (b), and a Longitudinal Section (c), of a Pipe where Scour has Removed the Support from Underneath.

It should be noted that in the reach where scour has removed the support from under the pipe, the drag force can be resisted only by the stiffness of the pipe.
Figure 4, by Blumberg (6), illustrates the alternate vortexes shed from each side of a cylinder with fluid flowing past.

**Figure 4. Fluid Flowing Past a Circular Cylinder Resulting in the Shedding of Vortexes.**

**Vibratory Forces.** The alternate vortexes shed from each side of the pipe can cause vertical oscillations of the pipe line. The frequency of the shedding of these alternate vortexes is dependent upon the pipe diameter and the velocity of flow around the pipe. If the frequency of shedding is near a resonant period for the pipe line, extreme vertical fluctuations can be induced which may damage or destroy the pipe line.

Vibratory forces resulting from the shedding of vortexes have been particularly significant in Cook Inlet, Alaska. Several failures have been known to occur. Two of these failures occurred adjacent to reefs where the lines could not be buried. Two other failures occurred at the platform and resulted from vibrations caused by the moving inlet tides.
III. REVIEW OF THE LITERATURE

A search of the literature produced results from only a few research projects that related directly to resolving the forces on submerged pipe lines. Of course, considerable research is being conducted by some of the major oil and pipe line companies, but in most cases such results are maintained as confidential. In general, the results of investigations may be classified as theory, laboratory studies and field experience.

Theory

Considerable theoretical information is available for the case of a circular cylinder remote from a wall or bed and subject to steady flow. However, little information is available for the case of a circular cylinder in contact with the bed under steady flow conditions and no information was found for a circular cylinder in contact with the bed and subject to reversible accelerating flow as produced by waves.

According to Beckmann and Thibodeaux\(^4\), in the potential flow theory the velocity field around an object that is in contact with the bed is solved by considering the bed as a mirror and introducing the image of an identical flow configuration on the other side of the wall. The bed becomes a streamline and is of no consequence to the flow pattern by the potential flow theory. However,
Beckmann and Thibodeaux\textsuperscript{(4)} indicate that such theoretical predictions are not in agreement with the actual flow pattern for such cases.

Wilson and Reid\textsuperscript{(55)} offer considerable doubt regarding the validity of applying a drag coefficient for steady flow to a situation involving unsteady conditions. One might conclude from the literature that a totally acceptable theoretical solution for resolving the forces on a submerged pipe line is not presently available.

**Laboratory Studies**

Results of some laboratory studies were found but they were of limited use. In most cases a steady flow condition was investigated.

Brown\textsuperscript{(17)} subjected a section of smooth pipe, laying on the bottom, to transverse horizontal currents and measured the differential pressure around the periphery of the pipe to determine the forces of drag and lift directly. He also investigated the influence of spoilers on the pipe to determine the effect of discontinuities on smooth pipe. The spoiler consisted of a metal strip, 3/4 inches in height, attached along the pipe. It was shown that spoilers could alter the drag and lift forces considerably, with the amount being influenced by their location.
Since the investigation by Brown was on 6 inch and 8 inch diameter pipes, they were considered to be prototype tests and he apparently did not adhere to any of the laws of modeling. This, probably makes any application of the results very limited since the water depth was only 18 inches.

Wilson and Reid\(^{(55)}\) presented the results of model and prototype studies by several investigators. These results consisted of drag and inertial coefficients and were primarily obtained from tests on vertical circular cylinders.

**Field Experience**

Field experience has been obtained principally from the investigation of failures in pipe lines and risers. In many cases of failure, the details are very closely guarded.

Blumberg \(^{(6, 9, 10, 11, 12)}\) reported on numerous failures involving drilling, production and pipe line facilities in the Gulf, offshore from Texas and Louisiana, during the hurricanes of 1961, 1964 and 1965. He reported that the losses sustained off the Louisiana coast alone during 1964 and 1965 exceeded $200 million. It was noted by Blumberg\(^{(6)}\), that companies which expended a major effort in developing sound environmental design criteria suffered fewer failures.
Throughout such reports the probable causes of failure were listed but no attempt was made to ascertain the magnitude of forces involved. As Brown points out, design based on past experience has proven costly, with money wasted either because of underdesign and failure or excessive expenditures for overdesign, and has not been sufficient to form a definite conclusion on the hydrodynamic forces involved.
IV. DESIGN CRITERIA AND DESIGN METHODS

The design criteria and design methods for submerged pipe lines are very flexible and could by no means be considered as standardized. This is somewhat understandable when one considers the rapid expansion of the industry to new areas and greater depths with both presenting many new challenges.

Considerable progress has been made in offshore design. Certain aspects of design will be covered in more detail, including route selection, laying methods, pipe line burial and pipe risers.

**Route Selection**

The best route for an offshore pipe line can be ascertained only after a study of the oceanographic and hydrographic conditions in the area of interest. Records should be obtained relating to early shore positions and previous contours of the ocean bottom along the expected route of the pipe line (42).

If soils data for the area are not available, core samples should be obtained along the proposed route. Soils information is necessary to evaluate the expected buoyancy forces, the depth of burial required (if applicable), and to indicate areas unfavorable from the standpoint of vertical stability.
According to Reid\(^{43}\), the long range erosional-depositional equilibrium can be analyzed through historical charts of the region, if available, or by analysis of the geological structure of the recent sediments. Areas of fairly rapid erosion or deposition may be disclosed and given proper consideration.

**Laying Methods**

Several methods of installing offshore pipe lines have been successfully used. Of these, the lay barge method has become the most versatile.

The lay barge method uses a large barge specially designed and equipped, as shown in Figure 5\(^{(52)}\), for lowering pipe to the sea bottom after it has been assembled on deck. To avoid overstressing the pipe during installation, the lay barge has a stinger attached, which provides support to the pipe line as it is lowered to the ocean floor.

![Figure 5. Typical Pipe Lay Barge.](image-url)
Dimensions of the stinger required are related to the size and weight of pipe and to the depth of water. According to O'Donnell (35), stinger lengths up to 807 feet in length are in use. A stinger of such a length could be used only under ideal conditions because of the lateral forces involved making it difficult to control. O'Donnell (35) estimates that a stinger of over 900 feet in length would be required to lay pipe by this method at a depth of 300 feet.

Since the stinger length appears to be limited and the need for lines at greater depths has increased, new methods have been sought. One method that has been used successfully at depths greater than 300 feet is the catenary method.

The catenary method involves supporting the pipe from the lay barge to the sea floor by applying longitudinal tension to the pipe on the deck of the lay barge. The additional support reduces the bending, resulting in a reduction of the flexural stress. Tests have proven that this method can be used to lay small diameter pipe in waters as deep as 500 feet.

**Pipe Line Burial**

Around the coastline of the United States the burial of pipe lines is required out to a water depth of 15 feet. Beyond this point, the burial of a pipe line will depend on such things as the exposure of the line to waves and currents, probable scour, vessel activity, fishing, shrimping
and oyster grounds.

Many operators once thought that it would not be necessary to bury pipe lines in the Gulf of Mexico in water depths over 100 feet. However, as noted by Blumberg (6), after the movement of unburied flow lines in about 240 feet of water during the passage of hurricane Betsy in 1965, it was recognized that significant ocean wave and current effects extended at least to this depth. Other areas, such as the North Sea where wandering sand dunes may cause changes in the bottom elevation of over 7 feet, require deep burial at all water depths.

Short (48) reports that a mechanical anchoring device has been developed that will pin the pipe to the ocean floor to eliminate movement problems. This method, utilizing a helix anchor, would anchor the pipe to the ocean floor and eliminate the need for pipe line burial in some cases. In other cases where pipe burial is necessary, the pipe could be anchored in the ditch which could result in a considerable reduction in weight of pipe required for stability. Such a reduction in weight would result in a considerable savings in materials as well as a reduction in stresses when laying the pipe line. This method has a lot of potential and according to O'Donnell (34) is growing in acceptance. Its use, however, should be preceded by a careful soil evaluation.
Pipe Risers

The pipe riser and the adjoining section of pipe are probably the most vulnerable portion of a pipe line. For sometime, the need for an increased depth of burial adjacent to the riser has been recognized.

One set of specifications reviewed which is probably typical for offshore from the Louisiana coast, required that the pipe at the riser base and the adjacent 300 feet be provided with at least 6 feet of cover while only 3 feet of cover were required for the remainder of the pipe line. In addition, a pipe wall thickness of 0.500 inches was required for the riser and adjacent 300 feet while a wall thickness of only 0.375 inches was required for the remaining portion of the line.

The incidence of failure at the pipe riser should be reduced by recently developed improvements. Probably the most significant of these is the use of a platform leg as a sleeve for the riser. The pipe line enters one of the platform legs at the mud line and then rises inside the leg to the platform deck. According to O'Donnell (37) this practice developed in Cook Inlet to protect the riser from vibratory forces. In addition it would protect the riser from inertial and drag forces as well as the crushing forces of ice.
V. SUGGESTIONS FOR RESEARCH

The Committee on Research, Pipeline Division of the American Society of Civil Engineers, has prepared a report on "Research Needs in Pipeline Engineering for the Decade 1966-1975."(46) It was the recommendation of the committee that 29 man-years and an expenditure in excess of $1,000,000 be devoted to research in the area of submarine pipe lines during the decade.

The committee's report includes a rather comprehensive list of the research needs of the pipe line industry along with detailed information to define each. The complete list follows, with detailed information included only for the first section since this section directly pertains to the present paper.

1. Hydrodynamic Forces on Submarine Pipe Line.

In this section the effort is concentrated on determining the hydrodynamic forces as related to the type of current, boundary layer effect and shape of pipe systems for pipe lines suspended and for pipe lines resting on the marine bottom.

A. Steady State Flow. - Research in this area is to be devoted primarily to the conditions that are found in areas of tidal and gravitational water flows similar to that found
in rivers and tidal estuaries. The purpose of the research is to determine the effect of shape, boundary layer, diameter and bottom conditions on the magnitude of the hydrodynamic forces.

B. Oscillatory Wave Currents. - The primary interest here is to determine the magnitude of the inertial forces accompanying a wave.

C. Various Shapes and Cross Sections. - The object is to determine the optimum in stabilization through the evaluation of various coating shapes. Pipe line bundling into various configurations should also be investigated for dual or multiple pipe line systems.

D. Spoiler Arrays. - The use of spoiler configurations for modifying the hydrodynamic forces have some beneficial applications provided the proper balance is achieved between the drag and lift forces. With this in mind, various spoiler configurations should be evaluated.

E. Suspended Pipe Lines. - Research is needed for submarine pipe lines exposed to flow
around the top and bottom. Considerable work has been devoted to the study of Von Karman vortex formation and collapse effect. Additional work is required in this area on the non-excited frequencies, plus new research associated with the sympathetic vortex collapse frequencies which can be induced in a suspended vibrating pipe line.

2. **Deep Water Pipelines**
   
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   B. Review Present Pipe Lay-Methods
   
   C. Adapting Present Lay Methods to Deep Water
   
   D. Proposed New Deep Water Lay Methods
   
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   F. Methods of Buoyancy Control Present and Future
   
   G. Starting and Stopping Lay Pipe
   
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   K. Submersibles for Survey, Repair Inspections, Connections
L. Divers-Use, Limitations, for Deep Water Pipe Lay

3. Requirements for Design Submarine Pipelines
   A. Requirements of Regulatory Bodies
   B. Survey Horizontal Control Systems
   C. Properties of the Ocean Environment
   D. Soil Survey Sampling and Acoustic Sounding
   E. Establishment of Design Criteria for Submarine Pipelines

As evidenced by the list above, considerable research is needed to improve the design criteria and construction techniques for submarine pipe lines.
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1. Arriens, J. L.
PROGRESS IN OFFSHORE PIPELINES,
Pipes & Pipelines International, Vol. II, No. 7,
pp. 28-33, July 1966

Outstanding achievements since the early days of offshore development in survey, design, construction, maintenance, repairs and operation of larger submarine pipelines laid in the waters of continental shelves are discussed.

Today, many accurate sea maps are available and pipeline engineers can choose from an array of modern, shipmounted instruments to survey and determine the best, safest and most economical route. Sonic scanning devices can detect the sea bottom contours, changes in soil density down to some 50 feet in the sea bed and can be used for locating metal objects.

In modern design methods intricate formulas, model tests, research and computers have replaced much of the old guesswork and information gained by experience.

Various improvements in construction include such methods as: pulling the pipeline over the sea bottom from shore to the proper location; pulling a number of floating pipeline sections of one mile length each to the proper site, joining them together and sinking the pipeline to the bottom; using lay barges with stingers; and the use of a large reel mounted on a seagoing barge. Additional improvements include the use of strain gages which allows the laying supervisor to avoid overstressing the pipeline during lay operations and improved techniques of pipe burial (explosive charges, jetting, etc.).
2. Beckmann, H., and Thibodeaux, M. H.
WAVE FORCE COEFFICIENTS FOR OFFSHORE PIPELINES,
Journal of the Waterways and Harbors Division,
American Society of Civil Engineers, Vol. 88,
No. WW2, Part 1, May 1962

Drag and lift force coefficients due to wave action on pipelines with circular and trapezoidal cross sections are evaluated for the case in which they are in contact with a smooth, hard-surfaced ocean floor. Inertial forces are also considered.

When a pipeline is placed over a rocky ocean floor it is fully exposed to the wave action if the floor is smooth. In this case, the hydro-dynamic forces have to be absorbed by Coulomb friction and the structural strength of the pipe itself. The allowable horizontal force, \( F \), may be determined by:

\[
F = f w = f \cdot \text{Volume} \cdot g \cdot (\rho_{\text{pipe}} - \rho_{\text{water}})
\]

where

- \( f \) = friction factor, depending on materials encountered, usually close to unity
- \( \rho \) = average density
- \( w \) = weight of pipe underwater
- \( g \) = acceleration due to gravity

If referred to unit length for a circular cross section the equation becomes:

\[
F_1 = f K h^2 g \rho_{\text{water}} (\gamma - 1)
\]

where

- \( K = \pi/4 \)
- \( h \) = height of structure (diameter)
- \( \gamma \) = average specific gravity of the pipe referred to ocean water.
The force on the pipe due to drag, \( F_2 \) may be represented as:

\[ F_2 = h \, C_D \, q \]  
where,

\[ C_D = \text{drag force coefficient} \]

\[ q = \frac{1}{2} \, \rho \, u^2 \]

\[ u = \text{the velocity component normal to the pipe} \]

The lift force on the pipe may be described as:

\[ F_3 = h \, C_L \, q \, f \]  
where,

\[ C_L = \text{lift force coefficient} \]

The hydrodynamic force (drag), \( F_2 \), must not be larger than the frictional force \( F_1 \), reduced by the hydrodynamic lift, \( F_3 \), or:

\[ f \left[ K \, h^2 \, g \, \rho_{\text{water}} \, (\gamma - 1) - C_L \, h \, q \right] \geq C_D \, h \, q \]

and,

\[ h \geq \frac{u^2}{2g} \, \frac{1}{K \, (\gamma - 1)} \, \frac{f \, (C_D + C_L)}{f} \]

The parameters on the right side of the resulting equation are determined by the contour of the pipe, the material used, and the flow conditions in the field. This equation gives the impression that an increase in the size of the structure would provide a greater margin of safety. This is true only to a limited extent since the flow velocity component, \( u \), increases with distance from the ocean floor.
A local inspection of the field conditions must be made to determine whether an "overdesign" of the pipe actually increases or decreases the margin of safety. For instance, the surface roughness of the ocean floor becomes important if its "grain size" is of the same order of magnitude as the pipe diameter, so that an increase in diameter would expose large portions of the pipe to the relatively undisturbed flow while a small diameter pipe would be hidden in the disturbed boundary layer.

The author includes a literature survey of experimental data for the purpose of evaluating the dynamic force coefficients $C_D$ and $C_L$ which are to be expected on a structure that is in contact with the ocean floor. Recommended values for rough surfaced pipes (concrete) should be in the neighborhood of $C_D = 0.5$, $C_L = 0.5$ for the case of contact with the ground. A drag coefficient in the range between $C_D = 0.35$ and $C_D = 0.40$ is suggested for a smooth pipe of circular cross section regardless of whether it is in contact with the ground or not. Surface roughness causes an increase in drag but a decrease in lift so that the combined effect is practically constant.

For a trapezoidal cross section, the author states that the drag and lift coefficients may be taken as $C_D = 0.7$ and $C_L = 0$. These values remain essentially unchanged for a freely suspended pipe and surface roughness would have a small effect on the drag coefficient.

The inertial-mass coefficient, $C_M$, for either cross section is estimated to be in the range of $C_M = 1$ to $C_M = 2$. This coefficient accounts for additional mass that is accumulated in the wake and for additional local acceleration in the flow field around the pipe.
The writer questions the validity of the simplifications made by the authors. The following comments are included:

1. An offshore pipe may be subjected to forces in shallow or deep water, giving two extremely distinct results.

2. The energy transferred by a single wave to cylindrical obstacles is quite different from energy transferred by a group of waves.

3. Even if the simple model of an irrotational flow with free boundaries around a flat plate, as envisaged by the authors, is considered, the results are not as simple as shown.

The writer notes that there are more advanced methods of approach that are more complex, and require considerable advanced hydrodynamics, but that yield results that are more reliable than those obtained by experience.
Wilson, B. W. and Reid, R. O.
Discussion on WAVE FORCE COEFFICIENTS FOR
OFFSHORE PIPELINES, by Beckmann, H. and
Thibodeaux, M. H.,
Journal of Waterways and Harbors Division,
American Society of Civil Engineers, Vol. 89,
No. WW1, pp. 61-65, February 1963

The writers present the essential results of
several model and prototype experiments on circular
cylinders covering accelerating flows of oscillatory
and non-oscillatory type. These results are shown in
Table 1 and Fig. 9 shown below. Concluding from these
results presented and the fact that all exposed submarine
pipelines sooner or later tend to acquire a coating of
marine fauna or flora and therefore lose their initial
smoothness, the writers advise adopting drag coefficients,
\( C_D \) of \( C_D \geq 1.0 \) for pipelines in contact with the sea
bed.

<table>
<thead>
<tr>
<th>Authority and Date</th>
<th>Nature of Experiments</th>
<th>Cylinder Diameter, in inches</th>
<th>Coefficient Value</th>
<th>Type of Flow (Remark)</th>
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Table 1.—DRAG AND INERTIAL COEFFICIENT VALUES FOR CIRCULAR
CYLINDERS IN ACCELERATING FLOWS
The writers also believe that the values recommended for the lift coefficient, $C_L$ and the inertial coefficient, $C_M$ are low and for design purposes, suggest that $C_L \geq 1.0$ and $C_M \approx 2.5$ be adopted.
Beckmann, H. and Thibodeaux, M. H.
Closure to WAVE FORCE COEFFICIENTS FOR OFFSHORE PIPELINES,
Journal of Waterways and Harbors Division,
American Society of Civil Engineers, Vol. 89,
No. WW3, pp. 53-55, August 1963

The authors agree with Altermann that wave forces on obstacles such as pipes are largely dependent on the type of waves encountered. For this reason, the subject was purposely restricted to force coefficients, and omitted actual velocities and accelerations of the fluid developed by these waves.

The writers indicate that they do not understand how more complex, advanced methods of theoretical approach, which do not consider the engineering problem encountered, can possibly yield results that are more reliable than those obtained by experience.

The authors indicate that they are not very confident in the test data (Table 1 and Fig. 9), presented by Wilson and Reid because of the scatter of data points. The authors are of the opinion that this scatter should be attributed to the uncertainty of the velocity distribution and not to the scatter of the drag coefficient. It is indicated that this error in wave velocity distribution prediction is considerably reduced on the sea floor, where only the longer waves are effective, and where interference by shorter wave lengths is negligible.

The authors agree that if marine fauna or flora will settle on the pipe alone, and not on the sea floor, a drag coefficient of $C_D \geq 1$ may well be experienced with time as suggested by Wilson and Reid. On the other hand, they indicate that if marine growth is experienced on both the pipe and the sea floor, low energy boundary layer flows, along these surfaces, come into
play. The result is a reduction of drag coefficient on the pipe, referred to the free stream velocities. It is the authors' opinion that the latter condition is most likely to occur in the field.

The authors conclude that offshore field experience as well as extrapolation from potential flow theories does not justify such conservatism as that proposed by Wilson and Reid.
The author states that the more than $200 million loss to drilling, production, and pipeline facilities sustained off the Louisiana coast during the prior two years from two major hurricanes attests to the importance of environmental factors in offshore petroleum operations.

Attention is called to the fact that not all offshore operators suffered the same losses in areas where they have had similar installations. Those companies which expended a major effort in developing sound environmental design criteria suffered far less than did companies satisfied with preliminary contractor's design.

According to the author, operator experience of the past 15 years in the Gulf indicates that the best protection for a submarine pipe line from the high bottom scour velocities associated with hurricanes is line burial with a minimum of 3 feet of cover. Buried pipelines have no external mechanical forces exerted upon them. When a pipeline is unburied or only partially buried there may be significant drag and lift forces acting on the line as a result of the particle motion around the line.

Fig. 1, reproduced from the article, shows the effects of particle motion on a partially buried flow line for several conditions of exposure.
Fig. 1 (a) shows the approximate streamline pattern of water particles over the partially buried flow line. Fluid velocity is increased in going over the mound created by the partially buried pipe. The pressure reduction associated with the speeding up of the water particles produces a lift force on the cylinder. The direct pressure on the fore side of the pipe and the pressure reduction on the rear side result in a drag force in direction of the fluid flow. As the streamlines close behind the pipe, turbulence is generated at the contact of the fluid and the bottom with a resultant scouring of some of the sediment.

Fig. 1 (b) shows the further effects of erosion both in front of and behind the pipe line as the turbulent flow occurs on the fore section of the line. Lift forces are still generated; however, drag forces increase as a result of the increase in projected area and an increase in drag coefficient associated with the shape.
If the pipe is not sufficiently weighted, the lift forces may pull it up from the sediment.

If erosion around the pipe line continues, there will be complete scouring of sediment from underneath the line, at least in a local area. The pipe line with complete scouring of sediment from under it is shown in Fig. 1 (c). With only a little of the sediment eroded from under the bottom of the pipe line, there will be a downward or "reverse lift" force developed. This force will tend to pull the pipe line back to the sediment; however, the development of the Karman vortex trail behind the cylinder will likely prevent any stable downward forces.

The author states that laboratory experiments have been conducted to measure lift and drag forces on pipelines at or near the bottom of a wave tank. However, he points out that the interaction between a soft erosible bottom and the pipe line is much different from the response of a pipe line in the vicinity of a hard flat steel bottom such as is used in wave tank experiments.

The shedding of vortexes behind a circular cylinder and the resulting vibratory forces are discussed. It is stated that fluid flowing past a circular cylinder is characterized by the development of alternate vortexes shed from each side of the cylinder. The frequency of the shedding of these alternate vortexes is dependent upon the cylinder diameter and the velocity of fluid flow. The frequency for a given cylinder and flow condition may be determined from the appropriate Strouhal number \( \frac{nD}{V} \), plotted as a function of Reynolds number \( \frac{VD}{\nu} \), where:

\[
n = \text{frequency of oscillations}
\]
\[ V = \text{velocity of fluid normal to the pipe} \]
\[ D = \text{pipe diameter} \]
\[ \nu = \text{kinematic viscosity} \]

If the high frequency transverse forces occur at or near a resonant period for the pipe line, excessive oscillations and possible destruction of the line may follow.
According to the author, probably the largest monetary losses sustained by offshore operators from hurricane Carla were associated with the movement or failure of underwater pipe lines and pipe line risers. One operator experienced an estimated $300,000 flow line damage in the South Pass, Block 27 area alone. This operator had 30 flow lines fail although waves were estimated at not over 15 to 20 feet in height in the area. All 30 of the breaks occurred in water depths greater than 30 feet, with 22 occurring in the center portion of the lines at distances over 200 feet away from the platforms being connected.

In an effort to explain the types of failure which occurred, 10 of the lines were brought to the surface for inspection of the failed sections. Five of these 10 lines were found to have failures which could be attributed to welding defects; however, during repair of several of the lines it was observed that rather abrupt changes occurred in the profile of the Gulf bottom near the flow line breaks. Divers noted a much stiffer clay in the immediate vicinity of the breaks in some instances.

South Pass is an area notorious for its soft and unstable bottom. Upper sediments are frequently referred to as "fluid mud." In some areas, driving piles to a solid foundation becomes extremely difficult
if not impossible. This area is also characterized by soil slides or slumping of the upper surface layers on the soft clay slopes of the area. Such slumping tendencies can be easily augmented by hurricane driven bottom currents to produce a substantial amount of instability in the upper soil layers. When these upper sediments slump, they frequently expose more consolidated clays which produce abrupt changes in the bottom characteristics. The irregular bottom profile produced this way can result in high axial, flexural, and shear stresses in the pipe lines.

To further complicate the foundation engineering problems of the South Pass area, mudlumps are formed when stiff, viscous silt and clay layers some distance below the ocean bottom fold and mushroom up through the soft surface silts and clays in a manner quite similar to that of salt domes. These mudlump areas vary from small submarine pinnacles to islands up to 30 acres in size as they penetrate the Gulf Bottom and are forced up toward the surface. Mudlump occurrence is extremely rare except in the Mississippi River Delta Area.

In the area of Block 27, the bottom slope is seen to be quite steep for water depths greater than 30 feet where the pipe line breaks occurred. In the water depth range from 30 to 60 feet the bottom slope varies from about 13 feet per mile to about 25 feet per mile seaward of the shore. These steep slopes (with sediments so soft that most pipe lines settle several feet into the mud shortly after they are laid) make lines vulnerable to high hurricane wave current forces.

The same operator who experienced the 30 pipe line breaks in the Block 27 area had no breaks on
lines which had been mechanically buried prior to hurricane Carla. Some of the methods suggested to limit failure of this type include the following:

1. Development of an economical burying method for deeper water conditions.
2. Burying the lines by jetting into the mud at least near the line terminals at risers going up the platform.
3. Increasing wall thickness of the pipeline for greater strength.
4. Utilizing weight coating to aid the lines in deeper settlement into the mud.

Development of sound design criteria for this area requires a careful analysis of the sediment characteristics in the area of interest. Careful evaluation of all factors in the pipeline system provides the best basis for efficient, economical installations.

The author reports on a failure in another area where a 7.35 mile long, 12-inch high pressure natural gas pipe line had been installed but not yet buried prior to hurricane Carla. It sustained considerable damage both to the risers and the pipe line itself. A diver's inspection of the condition of this riser revealed that the welded fitting was flattened out with the line off of the bottom for approximately 200 feet from the platform. At a distance of approximately 245 feet from the platform the pipe was on the bottom but moving vertically up and down with displacements between three and four feet. At a distance of 330 feet from the platform, the line was approximately 1/2 covered but still moving.

In the pipe line survey it was consistently noted that soft bottom material tended to partially bury
the pipe whereas the harder bottom clays encouraged
banking of sand on one side of pipe and scouring of holes
on the opposite side.
5. Blumberg, Randolph

**HURRICANE WINDS, WAVES AND CURRENTS TEST MARINE PIPE LINE DESIGN, Part 4: "Damage reports of pipe line and related facilities in Bay Marchand, South Timbalier, South Pelto, Ship Shoal, Eugene Island, and South Marsh Island Areas,"


According to the author one major offshore operator who experienced about 50 breaks in unburied pipe lines had much of this damage occur in the Bay Marchand Area. Severe bottom scouring was noted near many of these pipe line breaks.

Longshore drift currents, coming predominantly from the east-north-east, combined with wave driven currents from the east-southeast, to produce strong ocean currents from the water surface to the Gulf bottom. This was particularly true in water depths shallower than about 50 feet mean water level. The combined action of high currents and waves approaching the limiting breaking height in the shallower water depths (under about 30 feet mean water level) were the prime factors in Bay Marchand damage pattern.

A large scale ocean wave and wave force measuring installation is located in the South Timbalier area in about 100-foot water depth. Most storm data obtained during hurricane Carla at this installation has been maintained as confidential by the three major companies participating in this project; however, the maximum wave height measured was 40 feet.

In the adjacent Ship Shoal Area, a major pipe line network suffered damages as a result of Carla's passage. Figure 18, reproduced from the article, shows the original orientation of this pipe line network and
the location after the storm of the two sections that had been moved.

One line section was located in approximately 40 feet of water while the other installation was in approximately 25 feet of water. These two line sections comprised approximately 37 miles of small diameter pipe ranging in size from 3 inches to 8 inches with installation being completed at the time hurricane Carla formed. All of this line had been placed in position, but it had not been finally installed.
The operator had not planned to bury these pipe lines except adjacent to the pipe risers at the platform terminations and for a distance of 500 feet from the platforms. These riser sections had not yet been buried at the time hurricane Carla shut down operations; nor had the actual clamps which were to secure the pipe line risers to the structures been installed because the pipe had not yet been jetted to grade. Lines were secured only temporarily by field means consisting of cable and cable clamps and in some instances rope attachments to the platform.

A section of 8-inch pipe line installed one year prior to hurricane Carla and having only the riser sections buried (but having stable pipe riser clamps) was found not to have moved during Carla.

Movement of these two pipe line sections was approximately in the same direction as the approach of maximum hurricane Carla waves for this part of the Ship Shoal Area. The recurvatures of the shoreline in the Isle Dernieres Region apparently allowed substantial relief from strong longshore drift currents. The spread out longshore drift currents moved from a westerly direction to a northwesterly direction which was more nearly the direction of final pipe line movement.

The operator of this major pipe line system had other pipe lines installed in the South Pelto, Ship Shoal, Eugene Island, Vermilion, West Cameron, and High Island Areas with no other major damage to lines. In some instances, however, it was noted that the Gulf bottom before and after hurricane Carla had quite different profiles. This was particularly true in the Vermilion
Area. This operator makes it a common practice to bury all pipe lines 10 inches or larger and no movement was found in any of the buried pipe lines installed prior to Carla.

In the process of examining facilities, this operator did determine that scour existed around most lines or platforms. This scour was either of a broad or general nature or sometimes specifically in and around structures only. This bottom erosion in some instances measured to be as much as 8 feet.
6. Blumberg, Randolph


The author indicates that perhaps the greatest losses to offshore operators as a result of hurricanes and associated waves and currents were damage or loss of flow lines and gathering facilities. Movement of an unburied flow line in water depths greater than 90 feet during hurricane Hilda established that waves and currents are able to exert significant forces on flow lines even at these depths.

As quick analysis of the types of platforms which failed during the passage of hurricane Hilda reveals that many were designed to "one in 25 year hurricane" design criteria. It appears obvious that utilization of such design criteria for permanent or semi-permanent structures is inadequate when compared to the usually slight additional cost of developing a somewhat sturdier platform.

The entire damage pattern for hurricanes Carla and Hilda indicates that an additional investment in competent engineering design is justified to prevent or minimize failures of the type experienced.

There are still a great many unknowns in offshore design. However, much has been learned during the past decade from extensive experimental and theoretical studies. Some of the factors that deserve special attention in designing offshore pipe line facilities include the following:

1. Vulnerability of the proposed area to severe hurricanes should be determined.
2. Approach direction for severe winds, waves, and currents should be evaluated for a carefully selected design hurricane.

3. Character of the bottom sediment should be evaluated to determine its ability to support pipe lines under dynamic hurricane loading.

4. Connection of lines and risers to platforms should be given careful consideration.

5. Orientation of pipe lines with respect to normal longshore littoral drift, or expected maximum hurricane driven currents should be carefully evaluated.

6. Substructures of platforms should be kept as "clean" as possible to avoid projections which will absorb wave forces.

In every major petroleum gathering system to be installed offshore, careful consideration should be given to the total system. While each factor can be studied separately, the interaction of all of these effects, both the bottom and expected waves, tides, and currents, and the anticipated useful life of the structure must be taken into account. The total cost or total rate of return for a given gathering system will be a function of how carefully each of the factors are taken into account in arriving at the over-all design.

The most important lesson that can be learned from hurricanes Carla and Hilda and their resulting damage to offshore properties is that a wealth of data on the actual performance of offshore production and gathering facilities is now available. Many facets of these data can now be properly and accurately interpreted for improving the design of offshore installations.
7. Brown, R. J. 
HYDRODYNAMIC FORCES ON A SUBMARINE PIPELINE
American Society of Civil Engineers, Journal of the Pipeline Division, Vol. 93, No. PLI, March 1967

This paper examines the magnitudes of the hydrodynamic forces on a submarine pipeline laying on the bottom, by measuring the lateral current-induced differential pressure distribution around the periphery of the pipe. The paper describes the obtaining of experimental data from full scale tests, and the interweaving of theoretical and experience factors for resolution of these forces. All of the experiments of this series were conducted on a smooth surface pipe. To determine the effect of discontinuities on this smooth pipe, several spoiler arrangements were prepared and tested, two of which are shown in Fig. 8.

Test equipment consisted of a steel tank 3 feet high by 4 feet wide by 40 feet long, with pumps circulating the water through the tank at velocities up to 6 ft./sec. (Froude Number = 0.8). A venturi was built at the test specimen for obtaining velocity sufficient to simulate actual conditions. Six and ten inch diameter pipe specimens were prepared by installing pressure sensing stations around the periphery of the pipes which were connected to manometers outside the tank.
The water pressure differential readings obtained were used for determining the drag and lift for a 1-foot long increment of pipe. The data is illustrated in a pressure differential diagram as shown in Fig. 7, drawn to scale, around a 3-in. diameter circle. These differentials were divided into forces acting on 30° sectors, as indicated in the circle at the lower left corner of Fig. 7. The forces were then resolved into their horizontal and vertical components, as shown in the circle in the lower right corner of Fig. 7. The summation of these into drag and lift components is shown in the upper right corner of Fig. 7.

FIG. 7
The author draws the following conclusions:

1. Pipe on the marine bottom exhibits properties of lift and drag similar to that of a wing on an airplane.

2. In the range of Reynolds numbers tested, the coefficient of drag with the pipe on the bottom is less than that of the pipe suspended with flow around both top and bottom.

3. The coefficient of lift exceeds the coefficient of drag by some 50 percent in the range of Reynolds numbers tested.

4. Spoilers on the pipe alter considerably the hydrodynamic forces and their coefficients.

5. Varying the location of spoilers on the pipe causes considerable difference in the magnitude of drag and lift.

6. The coefficient of drag for the 6-in. and 10-in. pipe, within the Reynolds number range of $0.6 \times 10^5$ to $3.0 \times 10^5$, varied from 0.90 to 0.55.

7. The coefficient of lift for the same pipe sizes and Reynolds numbers varied from 1.3 to 0.8.

In this article, the author defines Froude Number as $\frac{V^2}{gd}$ and Reynolds Number as $\frac{SV}{\nu}$, where

- $V$ = velocity normal to the pipe
- $S$ = pipe diameter
- $g$ = acceleration due to gravity
- $d$ = water depth
- $\nu$ = kinematic viscosity
Larock, J. B., et. al.
Discussion on HYDRODYNAMIC FORCES ON A SUBMARINE PIPELINE, by R. J. Brown,
Journal of the Pipeline Division, American Society of Civil Engineers, Vol. 93, PL3,
November 1967

Larock states the physical situation is very much dependent on the effects of viscosity and turbulence. Also, if shallow submergence is to be studied, then the Froude number is an appropriate index to use in flow description, but if deep submergence is the prototype to be studied, the Reynolds number becomes far more important than the Froude number.

Another author issues a word of caution in relation to the use of steady state drag and lift coefficients in estimating forces caused by oscillatory flows.
An example was cited where exaggerated storm tides and wave agitation transported sand to the flota-
tion ditch depositing it in a fluid state, specific
gravity of about 2.0. The resultant buoyant forces on
the pipe lifted it to the surface.

To prevent the above, the author's two rec-
ommendations follow:

1. If during construction the pipe had been
filled with water, jetted, backfilled,
then time allowed for rehealing of the
soil before removing the water the vertical
displacement could have been eliminated.

2. Considerably heavier pipe used initially
and backfilled would have the same result.

In some areas the bottom is of such solution
that during periods of moderate wave action the mud
assumes the same profile as the waves. If in such an
area the wave action approaches the resonance of the
pipe, extreme vertical fluctuations can be induced in
the pipe.

The author states that difficulty in analyzing
hurricane waves is due to the number of elements involved
in modifying a given wave type moving over an arbitrary
path. The author presented Figures 1, 2, and 3, which
resulted from a study made by Gulf Consultants, of Houston
for the Tennessee Gas Transmission Company.
WAVE PROFILE, h (FT)

WATER VELOCITY AT BOTTOM, \( u \) (FPS)

PRESSURE ANOMALY AT BOTTOM, \( \Delta P \) (PSF)

Profile, bottom water velocity, and bottom pressure anomaly are plotted for 50-year maximum design hurricane in 40-ft. water. Fig. 1.
SHALLOW-WATER WAVE traverse is shown off Grand Chenier, La. Fig. 2.
The author shows sample calculations for specific gravity (natural state), and water content along with a direct double shearing ring apparatus used to determine shearing strength. These parameters are used in a method originated by Robert O. Reid to analyze the stresses induced in a pipe by uneven settling. Results of such calculations, for various size pipes, are shown as evidence to the hypothesis that larger-diameter pipes are more critical and therefore need more attention in design.
According to the author, the optimum route of the offshore pipeline laid in shallow water is perpendicular to the shore line where incoming waves will travel longitudinally with the pipe and where the least lateral area of pipeline will be exposed to wave forces. The seasonal wave heights should be determined from past records and the bottom scour limits evaluated. If scour or bottom current is indicated in other than hurricane conditions, the negative buoyancy or pipe burial, or both, in varying degrees should be adjusted to compensate for line instability. Allowance for hurricane conditions should definitely be considered; however, this will involve an economic evaluation relating to the size, utility, expected life or replacement cost of the pipeline.

Before determining the negative weight desired for a given pipeline, it is necessary to decide whether the pipe should be buried or not. The burial of pipelines beneath the bottom of navigable waters where the pipelines would impair navigation is required by the U.S. Army Corps of Engineers. Also, burial is required where the water depth is less than about 15 feet (may vary, depending on the Engineer District). Seaward of this point, the burial of a pipeline will depend on factors such as orientation, probable scour, as well as shipping or barge activity, fishing, shrimping and oyster grounds.

It is important to investigate the soil for bearing properties to prevent possible downward movement of the pipeline. Although movement of this type is possible, it is not probable in most areas. Equally and
probably more important are the sediments that will fill the ditch. It is these sediments (together with current) that could cause the pipe to float out of the submarine ditch. Assuming normal silting conditions and not storm conditions, the ditch backfill material should follow closely the existing sediments in the area. An exception to this would be after a major hurricane where large areas of ocean bottom have been disturbed. The scour of the pipeline in 0 ft. to 10 ft. of water should be considered as probable in a plane area devoid of shoals. If shoals exist, the current can expose a pipeline in deeper water and if the line is not buried, can cause lateral movement if the route of the pipeline is broadside of the seas.

The need to determine, by coring, the exact type of soil to use in evaluating depth of burial (if line is to be buried) and buoyancy control is dependent on the route itself and the degree of information already known concerning the area. Nautical charts differentiate between soft and hard soils and have been found to be accurate when compared with the soil surveys conducted on pipeline. Soil samples are not always required. The necessity for soil samples can best be determined by a study of the area using navigation charts, state geological data, experience of local boatmen, etc. Ocean bottom conditions vary widely in different areas; therefore, each area should be evaluated independently of previous experience in other areas.

The following is one of two examples included by the author and is a route evaluation for a 16-in. gas pipeline extending 17 miles offshore perpendicular to the shore line. A study of navigation charts and the shore line indicated a soft mud bottom. Proximity to bayous and passes made organic deposition a good possibility. Local information pointed to an area of semi-fluid
mud. A visual investigation and random probing located the semi-fluid mud approximately 2 miles from the proposed line; however, to supplement this inspection a coring survey was made to determine the depth of the soft mud because a depth recorder showed soft material at least 10 feet below the bottom. Soil samples were taken at a spacing of three per mile and electronic surveying was used because the offshore platform had not been installed at the time of survey. Core samples were inspected on the boat and notations made, and samples at depths where burial of the pipeline was being considered were preserved for laboratory analysis.

Based on soil analysis data, visual inspection, and estimated ease of obtaining trench in soft sediments, 8 feet of cover was specified seaward from the shore line for three miles, 4 feet for the next mile, and 3 feet of cover seaward of this point.

The pipeline, as constructed, was jetted down, and the 8 feet of cover was obtained with two "passes" of the jet. Most of the 3 feet and 4 feet cover was obtained with one "pass" of the jet. It should be noted at this point that 8 feet of cover was specified at the shore line and inland for 2,000 feet because of a retreating shore line. The specific gravity of the pipe installed was 1.33 as related to sea water.

The author indicates that particular attention should be given to the materials used in the riser itself, as well as the pipeline and adjacent to the riser. The pipe riser and adjoining section of pipeline (500 feet to 1,000 feet) represents a small part of a pipeline or any length in total cost, yet has a high incidence of failure. Better and more expensive materials and proper depth of burial at the riser can reduce the risk of failure at
this point with a small increase in over-all cost. This will not prevent riser failure on an unburied pipeline. If a long section of pipeline is unburied and is moved by a storm surge, the forces caused by the movement of the long section of line can exceed the strength of the pipe, and it is likely to part at the base of the riser, "rip" the riser clamps from the platform, or if welded, pull metal from members to which the riser is attached. An allowance for at least 3 ft. of scour should be made when terminating a pipeline at a platform recently installed. If a pipeline is terminated at a platform that has been installed for several years, scouring occasioned by the platform (as an obstacle) should have already occurred. This can be determined by diver inspection, measuring from the horizontal brace normally found at the mud line. Several inspections made at platforms prior to installation of a riser have shown as much as 8 ft. to 10 ft. of scour in and around the platform. A pipe riser suspended off the Gulf bottom is a natural target for storm or hurricane forces.
10. Kreig, Joe L.

HURRICANE RISKS AS THEY RELATE TO OFFSHORE PIPELINES,
Publication No. 1, American Society for Oceanography, Hurricane Symposium, Houston, Texas, October 10-11, 1966

According to the author the major risks to a pipeline during a hurricane are: (1) lateral movement of the pipeline along and over the bottom and (2) the loss of a platform where the pipeline either begins or ends.

The author indicates that a pipeline is likely to move when it lies on the bottom in water depths where either orbital wave particles or broad currents can act against a substantial length of the line.

The structural failure of a pipeline due to movement is a function of the orientation of the pipeline with respect to the hurricane, the length of the line, the diameter, wall thickness, net weight, and possibly the type of bottom has an influence. Of these factors, the length of the line and orientation seem to be the major factors. The pipeline movement observed by the writer has indicated the movement causes a stretching of the pipeline which results in a structural failure at either one or both ends of the pipeline which are attached to a platform. It is possible to design and attach the pipeline and riser to the platform so that the connection will withstand the total longitudinal tensile force that the line can withstand without failure. While this is possible, the risk of platform damage will be less if the riser connection to the platform is designed to fail unless such lateral forces were considered as a part of the platform design, and the member or members strengthened accordingly.
As much as 2-1/2 miles of lateral movement during a hurricane has been noted. A misalignment of this magnitude can cause the abandonment of the whole line due to its misalignment, or at the least a substantial re-connecting job.

The loss of a platform at either end is generally more serious than the damage to the pipeline because the platform is likely to take a longer period of time to replace. If producing wells are present, their productivity can be diminished or destroyed. The end product of the action of a hurricane on a gas line is to curtail the producing and transmission of natural gas to its ultimate market. Even though the offshore facilities do not sustain any physical damage, there is the associated loss of gas due to the lack of production during the hurricane period and evacuation prior to the hurricane and a period of restaffing platforms after the hurricane.

The author notes that depths where pipelines need not be buried for stability reasons is still unknown. Prior to hurricane Carla it was thought that lines in water depths of 100 ft. would be safe. They have now moved in depths deeper than 100 ft.

Figure III and Figure IV are reproduced from the article to show the results of lateral movement.
Pipeline movement in 80 feet of water during hurricane "Hilda" of 6" unburied line showing direction and magnitude of movement of one 6" line and non-movement of a parallel buried line. (500 feet adjacent to each platform was buried).

Figure III represents movement of a 6-in. pipeline approximately 2 miles in length in an 80 ft. water depth during hurricane Hilda. The pipe riser at the western end of the pipeline was severed dramatically from the platform. The eastern end of the pipeline when viewed above water showed no damage, but when examined underwater was severely strained and bent. The pipeline at and adjacent to the platform had been buried, but the main extent of pipeline lying between the two risers was not buried. The magnitude and direction of movement is indicated.
Figure IV

Diagram depicting underwater damage to pipe riser due to unburied 6" line shown in Figure III.

Figure IV illustrates the damage to the pipe riser at the eastern end of the line.
Continuity of operation is even more important offshore than onshore. Costly facilities are idled by an offshore shutdown, costly equipment must be moved in for repairs and costly labor must be employed to make them.

According to the author, a number of steps are being taken with regard to pipelines, to reduce the possibility of shutdowns and to lessen installation cost. These are some of them:

1. Using mechanical anchors to secure pipelines and to reduce the amount of concrete weight-
ing.
2. Burying pipelines in greater depths.
3. Including more valve connections during initial installation.
4. Using in-line generators to produce power for cathodic protection.
5. Going to higher yield-strength as well as heavywall line pipe.
6. Laying dual lines simultaneously.
7. Incorporating risers in platform structures.

The following comparison for a 30-in. pipeline with and without anchors is presented:

<table>
<thead>
<tr>
<th>Weight</th>
<th>Without Anchors</th>
<th>With Anchors</th>
</tr>
</thead>
<tbody>
<tr>
<td>lb/ft</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pipe</td>
<td>157.53</td>
<td>157.53</td>
</tr>
<tr>
<td>Somastic</td>
<td>56.38</td>
<td>56.38</td>
</tr>
<tr>
<td>Concrete</td>
<td>543.00</td>
<td>258.00</td>
</tr>
<tr>
<td>Total in Air</td>
<td>756.91</td>
<td>471.91</td>
</tr>
<tr>
<td>Total in Seawater</td>
<td>298.20</td>
<td>132.00</td>
</tr>
</tbody>
</table>
The author estimates the cost of burying a pipeline to be from one-third to one-half the lay cost excluding material. In a particular instance, 3 ft. of cover was provided for a 16-in. line at a cost of $3/ft.
This article reviews the causes of pipeline flotation, reasons for weighting, weighting criteria, and methods of pipeline weighting. It also suggests methods for more efficient and economical pipeline anchoring and recommends research to establish a technical procedure for the design of pipeline anchoring.

The article presents Table 1 to show how buoyancy is a function of pipe diameter for gas pipelines.

### Table 1—Relation of Bulk Specific Gravity of Pipe to Diameter

<table>
<thead>
<tr>
<th>Diameter, in inches</th>
<th>O. D., in inches</th>
<th>Wall thickness, in inches</th>
<th>Weight in lb per foot</th>
<th>Volume in cu ft per foot</th>
<th>Density in lb per cu ft</th>
<th>Specific Gravity</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>2.375</td>
<td>0.218</td>
<td>5.02</td>
<td>0.0307</td>
<td>163</td>
<td>2.61</td>
</tr>
<tr>
<td>3</td>
<td>3.5</td>
<td>0.216</td>
<td>7.87</td>
<td>0.0668</td>
<td>113.3</td>
<td>1.61</td>
</tr>
<tr>
<td>4</td>
<td>4.5</td>
<td>0.237</td>
<td>10.79</td>
<td>0.110</td>
<td>98.1</td>
<td>1.57</td>
</tr>
<tr>
<td>6</td>
<td>6.525</td>
<td>0.250</td>
<td>18.97</td>
<td>0.229</td>
<td>73.4</td>
<td>1.27</td>
</tr>
<tr>
<td>8</td>
<td>8.525</td>
<td>0.322</td>
<td>28.55</td>
<td>0.406</td>
<td>70.3</td>
<td>1.13</td>
</tr>
<tr>
<td>10</td>
<td>10.75</td>
<td>0.365</td>
<td>40.59</td>
<td>0.650</td>
<td>64.3</td>
<td>1.03</td>
</tr>
<tr>
<td>12</td>
<td>12.75</td>
<td>0.375</td>
<td>49.6</td>
<td>0.887</td>
<td>55.9</td>
<td>0.94</td>
</tr>
<tr>
<td>14</td>
<td>14</td>
<td>0.375</td>
<td>55</td>
<td>1.07</td>
<td>51.4</td>
<td>0.83</td>
</tr>
<tr>
<td>16</td>
<td>16</td>
<td>0.375</td>
<td>63</td>
<td>1.40</td>
<td>45</td>
<td>0.75</td>
</tr>
<tr>
<td>18</td>
<td>18</td>
<td>0.375</td>
<td>71</td>
<td>1.77</td>
<td>40.1</td>
<td>0.64</td>
</tr>
<tr>
<td>24</td>
<td>24</td>
<td>0.375</td>
<td>95</td>
<td>3.14</td>
<td>30.3</td>
<td>0.45</td>
</tr>
<tr>
<td>30</td>
<td>30</td>
<td>0.375</td>
<td>119</td>
<td>4.91</td>
<td>24.2</td>
<td>0.37</td>
</tr>
<tr>
<td>36</td>
<td>36</td>
<td>0.375</td>
<td>143</td>
<td>7.07</td>
<td>20.2</td>
<td>0.32</td>
</tr>
<tr>
<td>42</td>
<td>42</td>
<td>0.375</td>
<td>187</td>
<td>9.63</td>
<td>17.4</td>
<td>0.29</td>
</tr>
</tbody>
</table>

*Referred to water at 62.4 °F.

As can be seen from the table, gas pipelines in the larger diameters must be weighted very heavily to resist floating. Together with buoyancy, there are other water forces that must be resisted. Most common of these are current and scour associated with rivers and offshore locations. In river and offshore installations in which lines are laid without trenching, weight
must be applied to the pipe to make it resist the resultant of buoyancy and current forces. For buried lines, for which a ditch is dug and the pipe is pulled in place, but not mechanically backfilled, the pipe must be weighted sufficiently for the pipeline to stay in place during the backfilling period. The pipe must resist movement in the face of sediment-laden suspensions, currents, and scour, while it is being covered by the process of natural sedimentation.

The most common method of weighting is a continuous coating of concrete or asphaltic material uniformly applied to each joint of pipe. Data from eight offshore gas pipelines were reported as a result of questionnaires. These lines were all off the coast of Louisiana and are summarized in Table 2.

<table>
<thead>
<tr>
<th>Item</th>
<th>Miles</th>
<th>Cover, inches</th>
<th>Soil Description</th>
<th>Pipe Size, in inches</th>
<th>Protective Coating, in inches</th>
<th>Weighting</th>
<th>Specific Gravity</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Gas Lines—Offshore Louisiana</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>East Cameron (2 yr)</td>
<td>25.3</td>
<td>6 (No BF)</td>
<td>2.0-2.06 Sand, 1.35-1.72 Clay &amp; Silt, 1.7-2.2 Beaumont Clay</td>
<td>26 x 0.500</td>
<td>6/32 CT</td>
<td>3-5/8 in. (190 lb) CC</td>
<td>1.51</td>
</tr>
<tr>
<td>Eugene Island (9 yr)</td>
<td>15</td>
<td>3 (No BF)</td>
<td>1.3-1.7 (Average 1.43) Soft gelatinous mud near shore; silty clay</td>
<td>20.5 x 0.750</td>
<td>5/8 Sealing</td>
<td>1 in. (140 lb) CC</td>
<td>1.34</td>
</tr>
<tr>
<td>Chandeleur Sound (5 yr)</td>
<td>10</td>
<td>- (No BF)</td>
<td>Soft org, clay to silty clay</td>
<td>20 x 0.344</td>
<td>3/16 CTW</td>
<td>2-1/2 in. (165 lb) CC</td>
<td>1.28</td>
</tr>
<tr>
<td>West Cameron (1 yr)</td>
<td>23</td>
<td>3 to 10 (No BF)</td>
<td>3 ml.; sand, 10 ml.; medium clay, 10 ml.; Beaumont clay</td>
<td>16 x 0.375</td>
<td>4/-32 Asph. W</td>
<td>2-1/2 in. (140 lb) CC</td>
<td>1.36</td>
</tr>
<tr>
<td>Vermilion (1 month)</td>
<td>5</td>
<td>3 (No BF)</td>
<td>Soft clay, sand, and shell</td>
<td>14 x 0.500</td>
<td>4/32 CTF</td>
<td>1-3/16 in. (140 lb) CC</td>
<td>1.38</td>
</tr>
<tr>
<td>Main Pass (2 yr)</td>
<td>3.5</td>
<td>3 (No BF)</td>
<td>3 ft cover for protection out to 12 ft depth, then laid on bottom</td>
<td>12.75 x 0.375</td>
<td>3/16 CTW</td>
<td>1-3/4 in. (165 lb) CC</td>
<td>1.55</td>
</tr>
<tr>
<td>Breton Island Pass (6 yr)</td>
<td>10</td>
<td>3</td>
<td>3 ft cover for protection out to 12 ft depth, then laid on bottom</td>
<td>12.75 x 0.375</td>
<td>3/16 CTW</td>
<td>1-1/2 in. (165 lb) CC</td>
<td>1.48</td>
</tr>
<tr>
<td>West Delta (5 yr)</td>
<td>7.8</td>
<td>3</td>
<td>1.54 sp. gr., very soft gray clay</td>
<td>10.75 x 0.366</td>
<td>4/32 CTW</td>
<td>1 in (146 lb) CC</td>
<td>1.38</td>
</tr>
</tbody>
</table>

(a) Offshore Installations
It was pointed out that weighting materials were used on all these lines and the bulk specific gravities ranged from 1.28 to 1.55 relative to sea water at 64 pcf. This variation in weighting practice is considerably less than that for river crossings where specific gravities of gas lines ranged from 1.06 to 2.0 relative to fresh water.

Except for mechanically anchored pipelines, the stabilizing force is chiefly gravity. When lines are buried with suitable soil for cover, cohesion and friction in the soil are important forces acting to hold the pipe in place. Gravity acting on the metal of the pipe, the contents of the pipe, the overburden over buried lines, and any weight additives placed on the pipe provides a downward pull. Contents of the line in liquid service makes a significant contribution to holding the line in place.

A principal disturbing force that acts on pipelines in inundated areas is the buoyancy of water which in some cases is increased due to high-density suspension. Additional disturbing forces are caused by scour and currents that occur during river floods and offshore storms. In many instances these unpredictable forces are the principal causes for weighting pipelines. This is the chief reason for the arbitrary weighting criteria used in the pipeline industry. Because the inaccessible condition of submerged pipelines and the difficulties in handling them, the addition of weighting materials is the most expedient and practical method of anchoring underwater lines.

Plans for any offshore pipeline should be preceded by a study of oceanographic and hydrographic conditions in the area of interest. All factors that
may affect the life or continuous operation of the pipeline during its planned period of service should be evaluated. Records should be obtained relating to early shore positions and previous contours of the ocean bottom along the expected route of the pipeline.

A survey or study for any location is required to develop specific oceanographic factors at that particular site. When operating conditions permit, study of hydrographic charts, maps, aerial plots, and other records together with ground studies should determine the selection of the route. If it is possible to do so, an accreting shore should be selected for the pipeline to traverse the surf. In many cases, little can be done to alter the route of a proposed pipeline. But if positioning is possible, the orientation of the line should be based on oceanographic conditions of the area. Erosion and scour are principal hazards to pipelines traversing surf and beach. Active or less desirable bottoms are indicated by the presence of sand, shell, or sand and shell. Hard mud bottoms usually indicate little or no surf activity.

For offshore lines that are to be buried, enough weight must be supplied for the line to stay in place during backfilling. One method for laying offshore lines is to dig the trench along the pipeline route, place the pipeline in the trench, and depend on natural sedimentation to supply backfill to cover the pipe. The pipeline must resist buoyancy of water and soft fluid overburden materials to stay in its position and be covered. In addition to buoyancy for the pipe to remain covered, other unpredictable water forces must be resisted such as currents, scour, and water movements caused by waves.
So far, the only practical method of anchoring that has been developed for offshore pipelines is deadweighting. Weight is supplied to hold the pipe in place on the bottom of the trench. Optimum weighting criteria are needed for offshore pipelines so that safe anchoring can be obtained at the lowest possible cost.

A method of deadweighting that has been used successfully in getting offshore lines buried is to develop a volume density in the pipeline equivalent to the density of the ocean bottom material into which the pipeline is to be laid. Because of disturbance and dilution during the trenching and sedimentation processes, the density of the fluid backfill will certainly be less than the mean density of the sediments removed.

Correct weighting for unburied lines will depend directly on oceanographic conditions in the area of the pipeline. To prevent movement and to provide maximum protection for pipelines against interruption of service, offshore lines should be buried.

The authors make the following recommendations:

A. To place the design of pipeline anchorage on a more technical basis, a research program should be instigated to perform the following functions:

1. Develop a field and laboratory testing procedure to determine whether specific backfill materials can become fluid.
2. Determine what mechanical agitation or natural phenomena are required to make low strength materials become fluid.
3. Perform additional testing on many different types of soil to validate the proposed liquid limit method of determining the critical density of low shear strength material which can act as a fluid.
4. Investigate other procedures for finding critical density.

5. Develop a scale for measuring the relative time required for different thoroughly disturbed backfill oils to reconsolidate and regain shear strength to be stable overburden materials.

6. Explore the function of shear strength properties of backfill such as cohesion and friction in the pipeline flotation problem.


B. To provide the greatest security and to provide maximum protection against interruption of service during storms and floods, offshore pipelines should be buried.

C. To install unburied submerged lines the anchorage or pipeline weighting criteria should be based on a hydrodynamic study of the proposed location and water forces to which the pipeline may be exposed.
Some of the hazards which exist in marine pipeline construction are: (1) possible destruction or damage of the pipe risers or the structure itself by wave and wind action, especially during hurricanes; (2) possible destruction or damage of the pipeline due to shifting by action of waves and currents; (3) possible damage of the pipeline due to downward sag into very soft sediments; (4) possible damage by dragging of ship anchors or dredges in regions where the buried portion of the line has become exposed by scour; (5) possible damage to line and equipment due to storms occurring during the laying operations; (6) operational hazards due to the presence of reefs; (7) damage of protective coating by bacterial action; (8) corrosion by chemical or galvanic action; and (9) possible damage by excessive thermal stresses.

According to the author, to minimize these hazards requires special considerations in the design and coating of the pipe, in the planning of the line route, in the entrenchment or support of pipe, and in the time allowed for installation operations.

To evaluate properly and provide solutions to these problems, information about the wind and wave climatology of the region is needed. Information concerning local convergences of wave energy is especially important. Up-to-date knowledge of the bottom topography and of the physical, geological, and bio-chemical character is needed.
Erosion of significant amounts extending many miles offshore is known to occur in some areas of the Gulf Coast region. The pipe should be entrenched well below the expected depth of scour to be sure of a better means of support.

The long range erosional-depositional equilibrium can be analyzed through historical charts of the region, if available, or by analysis of the geological structure of the recent sediments. Areas of fairly rapid erosion or deposition may be disclosed, and taken into account in the planned depth of entrenchment.

The short term scour brought about by bottom currents related to waves and tides may likewise be of importance. Severe scour action by currents in restricted submarine channels might temporarily expose the pipe to the direct action of the current. Such areas can be tested for short term scour by repeated depth sounding traverses, especially directly preceding and following the occurrence of a heavy storm.

Whether the pipe is laid upon the bottom or entrenched within the sediment, it is necessary to gain some idea of the load bearing capacity of the sediments at several different depths. Certain isolated areas of the Gulf Coast are known to contain pockets of sediment which have virtually no shear strength. On the basis of considerations of the pipe as a uniformly weighted beam with fixed ends (fixed in the firmer sediments adjacent to the pocket of weak sediment), it can be shown that the maximum local stress approaches the elastic limit in a 20-in. diameter gas-filled pipe of 3/4 in. thickness if the length and depth of the weak sediment pocket are greater than 440 ft. and 7 ft., respectively.
A 10-in. gas-filled pipe with 1/2-in. walls, on the other hand, would be stressed to the elastic limit if it spanned a weak sediment pocket of only 170 ft. in length and but 2 ft. in depth.

If the line is jetted into the sediments, the material taken out will be so finely dispersed that the settlement of material will occur quite gradually. As the sediment accumulates and increases in density, there may be a stage of this transition for which the sediment is still fluid enough to exert a significant buoyant force on the pipe. The middle of the line may rise to the surface, exposing itself to the action of waves and other hazards of on-the-bottom line laying. To eliminate this risk the pipe can be weighted down with enough ballast to yield an effective specific gravity of the pipe which is at least equal to the specific gravity of the expected unconsolidated suspension within the trench. This can be done, however, only if the bearing capacity of the underlying sediment is great enough to withstand the added load.

The hazards encountered in a pipeline laid along the bottom are much more serious than those encountered in the case of entrenchment. Not only does the question of load bearing capacity come up, but also the question of the safety of pipe which is exposed to wave action arises.

In regions where the shear strength of the material appears exceedingly low, direct tests within the sediment as it exists in situ, should be devised. From the shear strength thus obtained the load bearing capacity of the material can be computed. This data will yield information on the allowable unit weight of the pipe.
The principal danger of the exposed bottom-laid pipe, if it is firmly supported at the surface of the sediment, is through the action of waves. Surface waves in shallow water have associated with them an oscillatory motion of the water particles. The horizontal component of this motion is significant at all levels beneath the wave. In a breaker the maximum possible water velocity at the bottom, just beneath the crest, will be about one third the wave velocity, the latter depending upon the depth of water only. Waves of breaking height can occur at certain times of the year in the Gulf Shelf regions, even out to depths of 30 ft. Waves in this depth of water could attain a height of 23 ft. before breaking. Such waves would have a propagational velocity of 41 ft. per second and the current velocity beneath the crest just above the bottom, would be 14 ft. per second.

If a pipeline happens to be so aligned along the bottom that it receives the wave broadside, then a force of roughly 200 lb. per sq. ft. of projected area can be developed as the crest passes over it. This extreme normal drag pressure, will not be developed throughout the entire length of a 10-mile pipe at a given instant, since the slightest deviation of the crest from a straight line or the slightest angle between the crest and pipe axis will mean that at the instant the crest passes the pipe at one point, a trough may be passing the pipe at some other point along the pipe. The mean drag force along a very long pipeline then is reduced to about 25 percent of the maximum value above. However, even a drag of 50 lb. per sq. ft. would cause considerable deflection of a pipeline whose unit weight is less than this, unless it is securely anchored or held down by concrete ballast.
The tidal currents and semipermanent circulation in the region would add their effect to the drag force. However, currents greater than about 5 ft. per second probably do not occur except in very isolated channels, and these currents will be reduced considerably at the depth of the pipe. The major consideration, as far as currents are concerned lies definitely with the periodic water motion associated with high waves.
According to the author, the last two major hurricanes in the Gulf of Mexico have damaged $200 million in capital equipment of drilling, production, and pipeline facilities off the Louisiana coast. If lost production is considered, these repairs can be more costly than the original installation. The author believes that underwater anchoring systems can prevent these losses to pipelines by pinning it to the ocean floor and preventing movement.

Observations and tests have indicated substantial lift and drag forces, similar to those acting on an airplane wing during flight, are acting on offshore pipelines. Wave, tide, current and storm forces effect the pipeline in varying degrees and can create excessive oscillations of the pipe that will damage or destroy both the pipe and/or coatings. Fluid flow over the pipe on the ocean bottom creates forces that lift the pipe off the bottom, changing the shape factor and flow pattern. The eddy currents change the magnitude of the forces and gravity pulls the pipe to the bottom to repeat the cycle. When these oscillations are at the critical rhythm of the pipe, it is not difficult to see how limited forces could destroy the pipe.

The mechanical anchoring system eliminates these movement problems by pinning the pipe to the ocean floor at fixed intervals capable of counteracting the lift and drag forces as shown in Figure 3. The spacing of the anchor sets, also limits the length of pipe acting as a single beam to the section between any two anchor sets and
with a much shorter beam length the pipe deflection will be much less with the same force.

Any reasonable design holding capacity can be developed for an anchor set with the proper combination of helixes and rod lengths for the given bottom soil conditions. This allows the design engineer to develop and economical anchoring system for any specified force he may expect on the pipeline.

The author states that evaluation of many underwater anchor pull tests in various areas where reliable soil boring laboratory analysis information was available allows correlation of our land test information of the last 50 years to the holding capacity in the ocean bottoms. Also installation torque requirements act as a double check to insure the anchor set will give adequate holding capacity based on design standards. However, the author indicates that regardless of the theory or formula used, experience and practical soil knowledge are necessary to determine the correct constants if reliable values are to be calculated for anchor holding capacities.

Anchoring of offshore pipelines offers the pipeline design engineer more flexibility than in the
past along with many engineering and construction advantages not previously available.

Concrete weight coating can be eliminated or reduced to a sufficient amount to overcome the positive buoyancy of the pipe in sea water. This can provide substantial savings in coating cost along with a drastic weight reduction on the stinger. For example, a 100 pound per foot reduction in weight coating would result in 20,000 pounds or 10 tons less weight on a 200-ft. long stinger. The elimination of this weight would substantially reduce the stress on the pipe during the laying operation and possibly allow reduction in the pipe wall thickness if this was the design criteria.

The savings from jetting costs often exceed the cost of anchoring and the possibility of the pipeline being uncovered at a later date would be eliminated. It is also possible to anchor jetted lines in the trench to make sure they are not jacked out of the trench during backfilling. Since the pipe is pinned to the ocean floor against movement, it is easy to chart for future location.

Elimination of expensive concrete field joints and reduced weight to be handled on both the lay and supply barges are possible additional savings. The use of anchors could result in pipe lines designed for greater storms at only a slight additional cost.
The author states that submarine pipeline construction is a relatively new field. Its entire development has taken place during the last two decades. As the necessity for submarine pipelines developed, equipment and techniques were adapted from that used in installing lines onshore, in the marshes, and across inland bays and rivers.

Several methods of installing offshore pipelines have evolved. These methods have been called; the "bottom tow" or "pull" method, the "flotation" method, the "reel" or "spool" method, and the "lay-barge" method. All of these methods of installation have been successfully used for installing underwater pipelines.

The lay barge method has become the most versatile method for installing offshore pipelines. This method uses a large lay barge specially designed and equipped for handling pipe from 3 inches to 48 inches in diameter and for lowering this pipe, after it has been assembled, to the sea bottom. The barge is moved along the pipeline route by a system of anchors. Tugs are used to shuttle the anchors as needed. Pipe and supplies are delivered from convenient shore bases by barge and tug or work boats. Crew boats are used to run men and supplies to and from the shore base. The lay barge has quarters and supporting facilities to provide for two shifts of the complete pipeline spread. The lay barge method offers better control of lay stresses and can maintain better alignment of completed lines which is essential for efficient burial operations.
One of the basic requirements of any of the construction methods is to provide safe continuous control of the pipeline to prevent overstressing during the installation. With the lay barge method, this requirement has led to the development of the pontoon stinger which provides support to the pipeline as it is lowered from the barge to the sea bottom.

The author presented the table, shown below, as a guide to the present range of capabilities in terms of water depth.

<table>
<thead>
<tr>
<th>Present Range of Offshore Pipeline Capabilities</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nominal Pipe Size</td>
</tr>
<tr>
<td>-------------------</td>
</tr>
<tr>
<td>8-in.</td>
</tr>
<tr>
<td>12-in.</td>
</tr>
<tr>
<td>16-in.</td>
</tr>
<tr>
<td>18-in.</td>
</tr>
<tr>
<td>24-in.</td>
</tr>
<tr>
<td>30-in.</td>
</tr>
<tr>
<td>40-in.</td>
</tr>
</tbody>
</table>

According to the author, the capability of laying pipe by this method may be extended to deeper water and larger diameter pipe by increasing the length of the stinger, by applying longitudinal tension, by selecting higher yield stress pipe or by reducing the specific gravity of the pipe.

Increasing the length of the stinger reduces the angle of inclination through which the pipe must bend and thereby reduces the bending stress. The practical limit on the length of the pontoon is approximately 700-ft. and this is dependent on relatively calm water with low velocity cross currents.
By applying longitudinal tension to the pipe on the deck of the lay barge, additional support is provided to the submerged pipe which conforms to a modified catenary curve. This additional support reduces the bending on the sag-bend near the sea floor which in turn reduces the critical flexural stress.
APPENDIX II

ANNOTATIONS
1. Blumberg, Randolph
HURRICANE WINDS, WAVES AND CURRENTS TEST MARINE PIPE LINE DESIGN, Part 1: "Climate, sun spot activity and hurricane path probability."

The author correlates climate, sun spot activity and hurricane path probability and indicates that during the rest of the century severe Gulf hurricanes will occur nearly twice as often as they occurred during the period 1920-1960.

2. Blumberg, Randolph
HURRICANE WINDS, WAVES AND CURRENTS TEST MARINE PIPE LINE DESIGN, Part 2: "Carla gave first significant data on a major hurricane in 18 years of Gulf of Mexico oil and gas operations."

The author presents a summary of damage reports, a storm tide contour plot and wave heights experienced during Carla.

3. Blumberg, Randolph
HURRICANE WINDS, WAVES AND CURRENTS TEST MARINE PIPE LINE DESIGN, Part 5: "Reports of details on Carla's effects......"

The author discusses damage to pipelines and gathering facilities in the Vermilion, Cameron and High Island Areas. He states that although platforms offshore from Texas were sparse, significant information on storm tide and wave heights were obtained from platform damage.

4. Blumberg, Randolph

The author discusses laying techniques and stinger designs.
EFFECTS OF VARIATIONS IN LAYING SYSTEM AND  
PIPE WALL THICKNESS ON RED SNAPPER PIPELINE,  
The authors present the results of a study,  
utilizing a computer program for analyzing  
pipe stress during offshore laying operations  
and supplemented with a scale model.  

6. Gard, W. S.  
ODD-SHAPED CONCRETE COATING SOLVES PROBLEM IN  
THE ARABIAN GULF,  
Pipe Line Industry, Vol. 18, No. 3, pp. 43-49,  
March 1963.  
The author discusses procedures in laying offshore  
lines and the use of a coffin-shaped concrete  
coating.  

7. Gard, W. S. and Bauerschlag, W. H.  
LATEST TECHNIQUES USED IN NORTH SEA PIPELAYING,  
Discusses advances in new pipelaying barges  
being built for Brown & Root. Also, a new burying  
barge with three jet pumps delivering 200 gpm at  
1000 psi to cut the ditch and one high capacity  
suction pump (24-inch suction, 20-inch discharge)  
used to lift the slurry.  

8. HIGH STRENGTH PIPE USED IN COOK INLET,  
Oil companies laying submarine pipelines on the  
rockstrewn bottom of Alaska's Cook Inlet are  
turning to high strength heavy-wall pipe.  
The new lines are expected to withstand constant  
pounding and vibration caused by rapidly moving  
currents and sweeping tides. Typical of the new  
lines is an 8-5/8 inch O.D. pipe which meets  
minimums of 75,000 psi tensile strength, 60,000  
psi yield strength has a wall thickness of 0.719 in.  
It weighs 60.69 lbs/ft.
9. Lee, G. C. and Bankston, C. L.
   PIPELINING OFFSHORE,
   Offshore, Vol. 27, No. 6, pp. 36-45, June 2, 1967.

   The authors discuss activities involved in laying offshore lines and describe the lay barge, stinger and other equipment used.

10. O'Donnell, J. P.
    JUST AHEAD: GREATER DEPTHS, LARGER PIPE; ARE TODAY'S LAY BARGES BIG ENOUGH,
    The Oil & Gas Journal, Vol. 64, No. 50, pp. 70-73, December 12, 1966.

    Studies of lateral forces show that pipe on a marine floor exhibits properties of lift and drag similar to those of an airplane. The coefficient of drag is less when the pipe is on bottom than when it is suspended with flow around both top and bottom. This article presents diagrams by R. J. Brown, Bechtel Corporation, showing drag and lift forces.

11. O'Donnell, J. P.
    MOVE IS ON TO NEW AREAS, GREATER DEPTHS, LARGER SIZES,
    The Oil & Gas Journal, Vol. 64, No. 50, pp. 67-69, December 12, 1966.

    Offshore pipelining, once pretty much confined to Venezuela's Lake Maracaibo, the Louisiana Gulf Coast and the Persian Gulf has moved to such new areas as Nigeria, Alaska's Cook Inlet, off the coast of California, the Texas Gulf Coast, and the North Sea. Peculiar conditions found in each new area are described.

12. O'Donnell, J. P.
    THE RISER: OFFSHORE PIPELINE'S MOST VULNERABLE MEMBER,
    The Oil & Gas Journal, Vol. 64, No. 50, pp. 98-104, December 12, 1966.

    One of the most vulnerable areas of the riser installation is the bottom bend. All companies
now bury lines deeper at this point so that the last part of the pipeline and much of the bend are buried.

13. O'Donnell, J. P.
THREE NEW, KING-SIZE BARGES INDICATE FASTER OFFSHORE PACE,
The Oil & Gas Journal, Vol. 64, No. 50, pp. 74-85, December 12, 1966.

The author discusses the significant features of offshore lay barges under construction. Two of them will have tensioning apparatus that will enable laying by the "catenary" method. McDermott will have the newest and largest burying barge ready for service early in 1967. Its pumps will be capable of delivering 8000 gpm at 1000 psi through a series of multiple jets.

14. PIPELINE ANCHORING SYSTEM--NEW TECHNIQUE "PINS" LINES TO OCEAN FLOOR,

The author reports that the A. R. Chance Company has developed a new anchoring system for underwater pipelines in water depths up to 200 feet. The new system pins the pipeline to the ocean floor to resist lift and drag forces created by tides, currents and storms.

One of the advantages to "pinning" the line to the bottom is elimination of excessive pipe weight, reducing transport problems and simplifying barge handling operations. Also there is less weight on pipe and stinger, reducing stresses on pipe during laying operation.

15. Reid, R. O.
SOME OCEANOGRAPHIC AND ENGINEERING CONSIDERATIONS IN MARINE PIPELINE CONSTRUCTION,
A & M College of Texas, Contribution in Oceanography and Meteorology, Contribution Number 14, 1950-1954.

This paper presents a very thorough treatment of the problem of vertical stability of offshore pipe lines.
16. Tesson, P. A.
LAYING PIPE FROM A REEL,

This article discussed the laying of 2-1/2 to 6 inch lines from a reel at a savings of up to $2.00 per foot.

17. Toebes, G. H. and Ramamurthy, A. S.
FLUIDELASTIC FORCES ON CIRCULAR CYLINDERS,
Journal of the Engineering Mechanics Division,

This article discusses experimental work performed and uses the data obtained to show a direct proof of the fluidelastic nature of the fluid-dynamic force acting on oscillating cylinders.

18. TWIN TWENTY MILE OIL LINES LAID IN ROUGH COOK INLET WATERS,

This article states that twin pipelines have been laid in what could very well be the world's roughest environment for line pipe - Cook Inlet, Alaska. The system consists of twin 20-mile lines connecting two platforms in the Granite Point offshore field.

Environmental conditions in the Inlet are:

1. Tides as much as 30 feet.
2. Currents of 6 to 8 knots, as swift as those of a good-sized river at flood stage, but changing direction every 6 hours.
3. Water depths 15 to 135 feet at mean low low water.
4. Bottom conditions including both soft sand and hard smooth rock, large boulders and deep crevices.

Besides these year-round conditions, in winter the Inlet is clogged with ice pans and flows which move back and forth with the tides. Thickness frequently exceeds 24 inches in severe weather.
19. Wilson, B. W.

FOUNDATION STABILITY FOR A SUBMARINE LIQUID SULPHUR PIPELINE,
Journal of the Soil Mechanics and Foundations
American Society of Civil Engineers, Vol. 87,
No. SM 4, August 1961.

This paper examines the vertical and lateral stability of a 7-mile long hot sulphur pipeline, entrenched in the sediments between Grand Isle, Louisiana, and the offshore sulphur mine in the Gulf of Mexico. Hot water under pressure in a jacket pipe maintains fluid sulphur flow in an inner line at an average temperature of 300°F. It is shown that the pipeline will be stable against buckling, but may suffer limited subsidence from thermal osmosis in the clay and convection currents in the sand.
APPENDIX III
TRANSLATIONS
The design and construction of underwater oil-gas pipeline in an open deep-water sea basin is a complicated engineering task. When building such pipelines, counter-storm measures are provided for, which, as a rule, includes quickly sinking the pipeline being laid to the bottom of the sea.

It is generally supposed that the pipeline, during a storm, is under the influence of slow currents in comparison with the currents in upper layers. The stable position of pipes on the seabottom is calculated, proceeding from the maximum speed of the ground current observed in a given region.

However, wave action initiated on the surface of the sea acts on a layer of water mass down to a depth equal to the length of the wave, that is, some tens or even hundreds of meters. Consequently, even at great depths, the pipeline experiences the influence of waves on the seabottom.

The coastal zone of a deep-water basin with constantly decreasing depths may be divided into four zones. In the first zone, the bottom has practically no influence on the form and dimension of the wave (Fig. 1). In the second zone, where the depth is less than half the length of the wave, but greater than the critical depth \( H_{KP} \), the bottom influences the wave (\( H_{KP} \) is the depth at which disintegration
of the wave occurs). The third zone is where the wave breaks. In the fourth zone final disintegration of the wave on the coastal slope takes place.

Presently, pipelines are laid in the second, i.e., shallow zone, at a depth of 30-40 meters. In conformity with such a depth, it is necessary to solve the problem of securing the pipeline in a stable position on the bottom. For solving this problem, there has to be known beforehand the nature and magnitude of wave effect on pipeline. The free, progressive, two-dimensional waves of the swell are more fully studied from all the varieties of gravity wave motion. They are characterized by movement of the particles of the liquid along flat closed orbits, the centers of which move constantly in the direction of wave propagation (Figure 2. a.).

All particles of the liquid revolve about elliptical orbits in planes, the parallel planes xz. The centers of the orbits are a little higher than when the still water level is in a quiet state.

The particles of liquid, having centers of orbits in a vertical plane, move synchronously along the whole depth, i.e. at any moment they are moving to one and the same angle and have, consequently, a common period of rotation about their orbits 2T. The phases of the particles depend on the distances $a_1$, $a_2$, $a_3$, ..., $a_n$, that is, the distances from the centers of the orbits to the radius oz.
The ellipse forming radii \( r \) and \( r_1 \) (Figure 2. b.) move with a constant angular speed, but the liquid particles move about the orbit with a varying linear speed. Each profile is formed by particles of liquid in a quiet state on one horizontal line.

The radii of the elliptical orbits are expressed by the formulas:

horizontal

\[
    r = h \frac{Ch \frac{\pi Z_0}{L}}{Sh \frac{\pi H}{L}}
\]  \hspace{1cm} (1)

vertical

\[
    r_1 = h \frac{Sh \frac{\pi Z_0}{L}}{Sh \frac{\pi H}{L}}
\]  \hspace{1cm} (1a)

where \( h \) is half of the wave height; \( H \) is the depth of the sea; \( L \) is the length of the half wave; \( Z_0 \) is the distance from the bottom to the particles of liquid in a state of rest; \( Ch \) is the hyperbolic cosine; \( Sh \) is the hyperbolic sine.

Since the hyperbolic cosine is greater than the hyperbolic sine, the horizontal axis of the elliptical orbit is always greater than the vertical.

With an increase in depth of propagation of the liquid particles, i.e. with a decrease of \( Z \), the elliptical orbits decrease in absolute dimensions. With this, the ellipses stretch more and more in a horizontal direction and on the bottom when \( Z_0 = 0 \), they change into a straight line, parallel to the bottom of the sea.
The radius becomes equal
\[ r = h = \frac{l}{Sh \frac{\pi H}{L}} \]  
(2)

This shows that ground speeds are not equal to zero.

The wave profiles have a form close to an elliptical trochoid, appearing as the projection of a point moving about an ellipse which, in turn, is moving horizontally at a rate equal to the speed of the wave. In a system of coordinates, shown in Fig. 2. b., the movement of the imaginary point describing the wave profile is expressed by the following equations:
\[ x = x_0 + r \sin \theta \]  
(3)
\[ z = z_0 + r_0 - r_1 \cos \theta \]  
(4)

The horizontal and vertical components of speed of the imaginary point are determined as derivatives of profile coordinates with respect to time.
\[ v_x' = \frac{dx}{dt} = \frac{d(x_0 + r \sin \theta)}{dt} \]  
(5)
\[ v_z = \frac{dz}{dt} = \frac{d(z_0 + r_0 - r_1 \cos \theta)}{dt} \]  
(6)

In connection with the fact that the ellipse forming radii move at a constant angular speed and have a common period of rotation 2T, the angular speed of the radii is equal to
\[ \omega = \frac{2\pi}{2T} \]

and their angular movement for time, T, makes
\[ \theta = - \frac{\pi}{T} t \]
In the reduced expression, the sign "-" is used because particles move in direction opposite to the direction of the positive (readings) of \( \theta \) angles.

The phase, \( \theta \) of the particle is a function of the distances \( a_1, a_2, \ldots, a_n \). Two particles, located one from the other at a distance (along the horizontal), equal to the length of the wave, have phases differing by \( 2\pi \).

Supposing that \( \theta \) changes proportionately to the distances \( a_1, a_2, \ldots, a_n \), we can write

\[
-\frac{\theta}{2\pi} = \frac{v_o}{2L}
\]

from whence \( x_o = -\frac{\theta}{\pi} L \)

Substituting the values of \( \theta \) and \( x_o \) into expressions (5) and (6) and differentiating, we get

\[
V_x = \frac{L}{T} - r \frac{T}{T} \cos \theta \quad (7)
\]

\[
V_z = -r \frac{T}{T} \sin \theta \quad (8)
\]

In expression (7), \( \frac{T}{T} \) is the rate of wave propagation;

\(-r \frac{T}{T} \cos \theta\) is the horizontal component of the speed of orbital movement.

Thus, the horizontal component speed of orbital movement

\[
V_x = -r \frac{T}{T} \cos \theta \quad (9)
\]

and the vertical component determined expression (8)

\[
V_z = -r \frac{T}{T} \sin \theta
\]
The horizontal and vertical component of the accelerations of orbital movement are the derivatives of speeds

\[ W_x = \frac{dV_x}{dt} = -r \left( \frac{\pi}{f} \right)^2 \sin \Theta \]  
\[ W_z = \frac{dV_z}{dt} = r_1 \left( \frac{\pi}{f} \right)^2 \cos \Theta \]  

(10)  
(11)

For determining speed and acceleration which the liquid particle possesses at a point with coordinates \( x \) and \( z \) (see Fig. 2. b.), it is necessary to know the phase angle \( \Theta \) and the initial coordinate \( Z_0 \).

Having substituted in expressions (5), (9), (10), and (11), the values of \( r, r_1 \), and \( T = \sqrt{\frac{mL}{g}} \), \( \text{cth} \frac{\pi H}{L} \)

and having completed several transformations, we get

\[ V_x = -\pi g h \frac{Ch \frac{\pi Z_0}{L}}{Sh \frac{\pi H}{L} \sqrt{\pi g L \text{cth} \frac{\pi H}{L}}} \cos \Theta \]  
\[ V_z = -\pi g h \frac{Sh \frac{\pi Z_0}{L}}{Sh \frac{\pi H}{L} \sqrt{\pi g L \text{cth} \frac{\pi H}{L}}} \sin \Theta \]  
\[ W_x = -\frac{H}{L} \pi g \frac{Ch \frac{\pi Z_0}{L}}{Sh \frac{\pi H}{L} \text{cth} \frac{\pi H}{L}} \sin \Theta \]  
\[ W_z = \frac{H}{L} \pi g \frac{Sh \frac{\pi Z_0}{L}}{Sh \frac{\pi H}{L} \text{cth} \frac{\pi H}{L}} \cos \Theta \]  

(12)  
(13)  
(14)  
(15)

The speed of wave flow may be determined by Stokes' approximate formula

\[ U = Ch \frac{L^2}{L^2} \frac{Ch \left( \frac{2\pi}{L} Z_0 \right)}{2 Sh^2 \frac{\pi H}{L}} \]  

(16)
where \( c \) is the rate of wave propagation.

In Table 1, are listed the maximum values of the orbital speeds (in meters/sec.) and the orbital accelerations (in meters/sec.\(^2\)), and also values of the wave flow at a level 0.25 meters from the bottom at various depths and wave parameters.

In Fig. 3 are presented the diagrams of the orbital speeds, accelerations and wave flow along the wave profile on a level 0.25 meters from the bottom when the depth of the sea \( H = 30 \) meters, (original article said \( H = 3 \) meters, translator believes \( H = 30 \) meters was intended) height of wave \( 2h = 8 \) meters and the length of the wave \( 2L = 80 \) meters.

From the diagrams it follows that the pipeline lying on the sea bottom may be considered as an endless cylindrical body around which flows unsteady currents during a storm.
РАСЧЕТ ПОДВОДНЫХ ТРУБОПРОВОДОВ НА ДЕЙСТВИЕ МОРСКИХ ВОЛН

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ПРОЕКТИРОВАНИЕ строительство подводных нефтегазо-
прово дов на открытом глубоководном морском анкерных
представляют собой сложную инженерную задачу.

При сооружении подводных трубопроводов предусматривают
протиоветровые мероприятия, которые, как правило, сводятся к быстрому погружению прокладываемого трубопро-
вода на две длины.

Обычно сообщением, что трубопровод в период шторма нахо-
дится под воздействием высоких течений со смещением с тече-
ниями в вышележащих слоях. Устойчивое положение труб на две длины рассчитывают исходя на максимальную скорость
донного течения, наблюдающегося в данном районе.

Однако волнение, возникающее на поверхности моря, вызывает
source водной массы глубинной, радиации длине

воды, т. е. всем со всейких сторон и даже смотрих метро.

Волна в длине и при больших глубинах трубопровод испы-
тывает волнение воздействиями на две длины.

Приблизируя полосу глубоководного подвода, представляю-
ные измерений глубины можно разделить на четыре зоны:

В первой зоне дно практически не уступает на форму и размер
воды (рис. 1). Во второй зоне, где глубина меньше половины
длины волны, но больше волна длины глубины \( H_w \), дно оказывает влияние на волну (пос \( H_w \) номинальна глубина, при которой происходит разрушение волны). Третья зона, как волна прибойной волны. В четвертой зоне прои-
ходит окончательное разрушение волны на берегу откосе.

В настоящее время трубопроводы прокладывают в волной,
т. е. мелководной зоне, на глубине 30—60 м. Применительно
tакой глубине и необходиму решать задачу обеспечения
устойчивого положения трубопровода на две длины.

Для решения задачи прежде всего надо знать характер
влияния волнового воздействия на трубопровод. Из всех
факторов, влияющих на движение жидкости более всего
влияют гидродинамические и гидростатические, а именно
волны. Они характеризуются длиной волны и высотой
волны, которая влияет на трубопровод, и на двух зонах
рис. 2, а).

Все частицы жидкости вращаются по эллиптическим орби-
там в плоскости, параллельной плоскости \( xz \). Центр орби-
ты находит несколько выше частиц воды в условиях
глубоководного состояния.

Частицы жидкости, имеющие центры орбит на общей вер-
тикали, в общем движутся неравно, т. е. в любой мо-
мент переходят из одной и той же точки в данную \( x \) и имеют,
следовательно, один период обращения по своим орби-
там \( 2T \). Валов частиц зависят от расстояния \( a_1, a_2, a_3, ..., a_n \), т. е. расстояний от центра орбит до оси \( z \).

Эллипсоидальные обводы \( r_1 \) (рис. 2, б) перекрываются в ско-

dостаточной плотности, а частицы жидкости дви-
жутся по орбите в неравномерной линейной скоростью. Изви-
дный профиль образует частицы жидкости, находящиеся в
состоянии погружения на одной горизонтальной линии.

Полуоси (разбушки) эллиптической орбиты выражаются
формулами:

горизонтальная

\[ r = h \quad \frac{\pi z_0}{L} \]  \( 1 \)

вертикальная

\[ r = h = \frac{\pi z_0}{L} \]  \( 1 \)

Здесь \( h \) — половина высоты волны; \( H \) — глубина моря; \( L \) — длина волну; \( z_0 \) — расстояние от дида частицы жидкости, находящейся в состоянии волны; \( Ch \) — гиперболический коэффициент; \( Sh \) — гиперболический синус.

Поскольку гиперболический коэффициент больше гиперболи-
ческого синуса того же аргумента, то горизонтальная ось
edлины обводов всегда больше вертикальной.

С увеличением глубины расположение частицы жидкости,
т. е. с увеличением, элиптические орбиты уменьшаются в абсолютных размерах. При этом эллипсы все более вытяги-
ваются в горизонтальном направлении и на дно при \( z_0 = 0 \)
приводятся в прямую линию, параллельную линии диа.

Полуось становится равной

\[ r = h = \frac{1}{\frac{\pi z_0}{L}} \]  \( 1 \)

Это указывает на то, что днища скорости не

равны нулю.

Волновые профили имеют форму, близкую к эллиптической трохоиде, представляющей собой проекцию точки, движущейся по эллипсу, который в своей очередь находится в равномерном горизонтальном движении со скоростью
волны.

В системе координат, показанной на рис. 2, б,

действие воображаемой точки, описывающей

волновой профиль, выражается следующими

уравнениями:

\[ x = z_0 + r \sin \theta, \]  \( 3 \)

\[ x = z_0 + r - r \cos \theta. \]  \( 4 \)

Горизонтальная и вертикальная составляющие скорости
действия воображаемой точки определяются как производные
от геометрического координат по времени

\[ v_x' = \frac{dx}{dt} = \frac{d (z_0 + r \sin \theta)}{dt}, \]  \( 5 \)

\[ v_x' = \frac{dx}{dt} = \frac{d (z_0 + r - r \cos \theta)}{dt}. \]  \( 6 \)

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В связи с тем, что эллипсообразующие радусы движутся с постоянной угловой скоростью и имеют общий период обращения $2T$, угловая скорость радиусов равнется

$$\omega = \frac{2\pi}{2T},$$

а их угловое перемещение за время $t$ составляет

$$\theta = \frac{\pi}{T} t.$$

В приведенных выражениях знак $-\pi$ принят потому, что частица движется в сторону, противоположную направлению положительных отсчетов угла \( \theta \).

Фаза частиц \( \theta \) является функцией расстояний \( a_1, a_2, ..., a_n \). Два частицы, находящиеся друг от друга на расстоянии (по горизонтали), равном длине волны, имеют фазы, отличающиеся на $2\pi$.

Подставив, что \( \theta \) изменяется пропорционально расстояниям \( a_1, a_2, ..., a_n \), можно записать

$$\frac{\theta}{2\pi} = \frac{x}{2L},$$

откуда

$$x_0 = -\frac{\theta}{\pi} L.$$

Подставляя значения \( \theta \) и \( x_0 \) в выражения (5), (6) и дифференцируя, получим

$$v_x = \frac{L}{T} - r - \frac{\pi}{T} \cos \theta, \quad (7)$$

$$v_z = -r_1 - \frac{\pi}{T} \sin \theta. \quad (8)$$

В выражения (7) \( \frac{L}{T} \) — скорость распространения волны;

$$-r - \frac{\pi}{T} \cos \theta$$ — горизонтальная составляющая скорости орбитального движения.

Таким образом, горизонтальная составляющая скорости орбитального движения

$$v_x = -r - \frac{\pi}{T} \cos \theta, \quad (9)$$

а вертикальная составляющая определяется выражением (8)

$$v_z = -r_1 - \frac{\pi}{T} \sin \theta.$$

Горизонтальная и вертикальная составляющие скорости орбитального движения являются произведениями скоростей по времени:

$$w_x = \frac{dv_x}{dt} = -r \left( \frac{\pi}{T} \right)^2 \sin \theta, \quad (10)$$

$$w_z = \frac{dv_z}{dt} = r_1 \left( \frac{\pi}{T} \right)^2 \cos \theta. \quad (11)$$

Для определения скорости и ускорения, которыми обладает частица жидкости в точке с координатами \( x \) и \( z \) (см. рис. 2, 6), необходимо знать фазовый угол \( \theta \) и первоначальную координату \( x_0 \).

Подставив в выражения (5), (6), (10) и (11) вместо \( r, r_1 \) и \( T = \sqrt{\frac{\pi L}{\delta} \cosh \frac{\pi H}{L}} \), их значения и выполнив некоторые преобразования, получим

$$v_x = -\pi gh \frac{C_h \pi_0}{L} \frac{\delta \cosh \frac{\pi H}{L}}{Sh \frac{\pi H}{L}} \sqrt{\frac{\pi g L}{c L} \frac{\delta \cosh \frac{\pi H}{L}}{L}}, \quad (12)$$
Iskenderov, I. A. and Kantor, A. G.
AN EVALUATION OF SUBMERGED PIPELINE ON THE ACTION
OF SEA WAVES (Conclusion)
Pipeline Construction, Vol. 6, No. 8, August 1961.

When a current of liquid flows around a body, an
interaction force arises, the component of which is along
the direction of current flow, and is called the drag
resistance. This force, when there is a moving current
and a motionless body, is formed from the drag resistance
(vortex resistance), inertial resistance, and the friction
on the surface of the body.

Friction resistance depends on the viscosity of the
liquid and is determined by the projection of the resultants
of all forces touching the surface of the body in the direction
of movement. In view of the low viscosity of water and the
insignificant surface friction, the friction resistance may
be disregarded.

Drag resistance arises basically as a result of vortex
formation behind the body and is determined as a projection
of the resultants of normal forces in the direction of
movement. The drag resistance on a unit length of a cylin-
drical element (pipeline) under a current perpendicular to
the axis of this element is expressed by the formula:

\[ P_\phi = C_\phi \rho D \frac{V^2}{2} \]

where,

\[ C_\phi = \text{the coefficient of resistance} \]
\[ \rho = \text{the density of the liquid in kilograms seconds}^2/\text{meters}^4 \]
\[ v = \text{velocity of the current in meters/second} \]

\( D_p \) is the projection of one linear meter of pipeline covered with insulation and lining of bricks (or concrete) to an area perpendicular to the direction of the current, in meters.

The coefficient of resistance, \( C_\phi \), depends on the shape and composition of the surface of the body, the viscosity of the liquid and the velocity of the current. The combination of these factors determines the rate of flow around the body which is characterized by the magnitude of Reynolds Number \( R_e \). When there is flow around a body, the \( R_e \) is determined by the same formula as when there is movement of a liquid through a pipe

\[ R_e = \frac{VD_p}{\nu} \]

where \( \nu \) is the coefficient of kinematic viscosity.

For a pipe with a rough surface covered with concrete or bricks, when \( R_e \) is \( 5 \times 10^5 \) to \( 1 \times 10^6 \), the coefficient of resistance increases almost 2.5 times (Figure 4). (Translator comment)

However, variation of \( C_\phi \) with \( R_e \) shown in Figure 4, applies to a moving current flowing perpendicularly to the axis of the cylinder. When a wave flows around a submerged pipeline, periodical movement results. Particles of the liquid flow around the pipeline at a variable velocity which changes in
time, but also are affected by the diameter of the pipe. For such movement of the liquid, the variation of $C_d$ with $Re$ is not established and may be obtained only from experimental research.

In addition to the drag resistance in the horizontal direction, the vertical lifting force $P^z$ also acts upon the submerged pipeline. This force is the result of an asymmetrical flow of a current of liquid around the pipeline, vortices arising from this bottom current and the presence of pressure from below because of water flowing under the pipeline. Experiments by M. A. Dementiev established that the lifting force is as follows:

$$P^z = K_o \rho V^2 D_p$$

(19)

where $K_o$ is the experimental coefficient (equal to 0.3 - 0.4 according to M. A. Dementiev); $V$ is the bottom velocity of the current in meters/second.

Thus, the horizontal component of the velocity of the orbital motion of the liquid particles ($V_x$), in addition to the drag resistance directed horizontally, also gives rise to the vertically directed lifting force.

The force of inertial resistance on a unit of length of a cylinder when there is a flat plate potential current may be expressed by the formula

$$P_i = \rho \frac{T D_p^2}{4} W$$

(20)
where $D$ is the diameter of the pipeline in meters; $w$ is the acceleration of the current in meters/second$^2$.

Because of the viscosity of the liquid, the variation of velocity in a real current rarely differs from the propagation of velocities in a potential current. As shown by experimental research, for real conditions, the force of inertia resistance may be determined by the formula

$$P_i = C_i \rho \frac{\pi D_p^2}{4} w$$

(21)

where $C_i$ is the experimental coefficient, depending on the nature of the current and the shape of the body.

Several authors have recommended a value of $1.3 - 2.0$ for the experimental coefficient $C_i$. The horizontal components of the velocity $V_x$ and accelerations $w_x$ of the orbital motion of the particles of liquid, and also the velocity of wave flow, $u$, depends on wave pressure on the pipeline in a horizontal direction $P_{v_x} + w$ and $P_{w_x}$.

The vertical components of the velocity $v_z$ and the accelerations $w_z$ depend on wave pressure on the pipeline in a vertical direction $P_{v_z}$ and $P_{w_z}$. Besides, as was noted above, the lifting force $P_\phi$ determined by the horizontal component of the velocity of orbital motion acts on the pipeline.

For securing the pipeline in a stable position at the bottom when there are vertical and horizontal stresses, it is necessary to observe the following condition:
\[ K = f \frac{C_B + C_b - \Sigma P_z}{\Sigma P_x} \quad (22) \]

where \( K \) is the coefficient of the stability of the pipeline; \( f \) is the coefficient of friction of the pipeline on the bottom; \( C_B \) is the weight of the pipeline in water (negative buoyancy); \( C_b \) is the weight of the ballast; \( \Sigma P_z \) is the resultant pressure on the pipeline, directed vertically upwards; \( \Sigma P_x \) is the resultant pressure on the pipeline, directed horizontally.

In Figures 5 and 6 are shown the curves of the horizontal and vertical wave pressures (along the wave profile) on a pipeline with a rough surface and outside diameter \( D_H + 0.5 \) m, \( H = 30 \) m, \( 2h = 8 \) m, and \( 2L = 80 \) m. Since the value \( Pw_z \) is very small, it is not included in the calculations.

When constructing the curves, wave pressures were determined from the following expressions:

\[ P_V = C_\phi D_H j \frac{V^2}{2g} \quad (23a) \]

\[ P_{V+u} = C_\phi D_H j \frac{(V+u)^2}{2g} \quad (23b) \]

\[ P_{V+u+u_T} = C_\phi + D_H j \frac{(V+u+u_T)^2}{2g} \quad (23c) \]
The given expressions are obtained from formulas (17), (19), and (21) on the basis of the equality

\[ \rho = \frac{J}{g} \]

where \( J \) if the unit weight of water in KG/m\(^3\); \( g \) is the acceleration of the force of gravity in m/second\(^2\).

The coefficient of the kinematic viscosity \( \nu \) in the given case is 0.015 centimeters\(^2\)/second (temperature of the water + 4\(^o\) C.); the unit weight of sea water \( i = 1025 \) kilograms/meter\(^3\); the coefficient of the added mass \( C_1 = 1.75 \); the experimental coefficient \( K_o = 0.4 \).

In Figures 7 and 8 are presented the curves of the horizontal and vertical wave pressures (along the wave profile) on the same pipeline, which were calculated for the bottom velocity, \( u_t = 0.3 \) m/sec., in the direction of the wave propagation. On the curves are also listed the pressures which depend only on the ground flow.
From the curves, it is apparent that, at a given depth, wave parameters, pipeline diameter, velocity and direction of bottom flow and the maximum horizontal wave pressures on the submerged pipeline, (neglecting the bottom flow) are approximately 8 times greater than the pressure exerted on the pipeline by the bottom flow. Also they are 10 - 11 times greater if the horizontal wave due to the bottom flow is not neglected (sentence modified by the translator). The vertical wave loads are correspondingly 5 and 10 times greater.

With a change in the magnitudes stated above, the correlation of the wave loads and pressures on the pipeline from the bottom flow may change. However, calculating the bottom flow, the wave pressure on the pipeline will always be greater than the pressure exerted by the bottom flow above.

The maximum values of the horizontal and vertical components of the wave pressure on the submerged pipeline relatively displace one another along the wave profile. Therefore, when there is an established coefficient of stability $K$ for determining the weight of the ballast insuring stability of the pipeline on the ocean bottom, the values $\mathbf{\Xi}P_x$ and $P_x$ should be substituted in formula (22) when the pipeline is in the most adverse position relative to the wave profile.

With an increase in wave length, the pressure on the submerged pipeline increases. In Figure 9 are given the curves
of the resultant horizontal wave pressures on a pipeline along the wave profile \(D_H = 0.5 \text{ m}, H = 30 \text{ m}, 2h = 8 \text{ m}, \) and \(2L = 120 \text{ m}\) without calculating the bottom flow. From these curves, it is apparent that the maximum horizontal wave pressure reaches 38 Kg./linear meter. Including the bottom flow, the maximum horizontal pressure is \(\approx 55 \text{ Kg./linear meter}.\)

The resultant vertical wave pressures, without calculating the bottom flow, are equal to 32 Kg./linear meter. However, these values are excessive and formula (19) is not completely accurate.

The given data allows one to estimate the nature and magnitude of wave loads on a submerged pipeline.

All curves of wave pressures considered were computed for a wave propagation normal to the axis of the pipeline. When wave propagation is at an angle to the pipeline greater or lesser than 90°, the pressure on the pipe along its axis is found to vary in magnitude and direction, and will change in time as well as along the pipeline.

Under the horizontal components of the velocity and acceleration of the orbital movement of the particles of liquid, i.e., the projection of the horizontal components of speed \(v_x\) and acceleration \(w_x\) to the normal towards the pipeline axis, should be taken into account.
The curves of wave pressures for sections of a pipeline between two neighboring wave troughs (or crests), will be repeated along the pipeline axis.

The effect of sea waves on a submerged pipeline is not limited only to wave pressure varying in magnitude and direction.

As was already noted, drag resistance arises basically as a result of vortex formation. When particles of a liquid flow around a cylindrical body, owing to the two-dimensional symmetry of the cylinder, the initial break of flow from its limits is revealed as a formation of two symmetrical vortices directly behind the pipe profile. With an increase in $R_e$, these vortices grow and finally, alternately break from the surface of the cylinder (the pipeline). However, such vortex formation changes with a further increase of the value $R_e$ and the symmetrical formation and subsequent shedding of the vortices are affected. The shedding of the vortices changes the distribution of pressure on the cylinder surface. With an unsymmetrical development of the vortices, the side pressure, which constantly changes in direction, increases. This situation, especially if the period of the pipeline's own vibrations are in resonance with the frequency of the vortex formation, causes a vibration of the pipeline in a direction perpendicular to the current.

With an unsteady current, a steady vortex formation is apparently, impossible. Consequently, the passage of a wave will cause a vibration of the pipeline. Pipeline lying on
the bottom of the sea under storm conditions will be under the influence of varying loads, which are the most frequent reason of failures of submerged pipelines as a result of "fatigue" of the metal.

The given considerations for determining wave loads on submerged pipelines lead to conditions of unsteadiness, that is, to the free wave (translators comment). In practice, storm waves influenced by wind force are more often encountered. In this instance, liquid particles move not in closed ellipses, but they describe more complex curves. Besides, the principal difficulty is that the inherent characteristic theories of trochoidal waves require that correction factors be introduced into the calculation. The coefficients of the drag resistance \( C_d \) and mass \( C_l \) for wave movement of a liquid have not yet been determined. Also unknown is the effect of depth on the wave pressure near the bottom.

For specifying the quality and quantity estimates of wave effects on pipeline lying on the ocean bottom, it is necessary to conduct experimental research in laboratories and under natural conditions.

However, we may now ascertain that determining a stable position of a pipeline on the ocean floor only on the basis of the magnitude of the maximum bottom velocity without taking into account the wave pressure (as this was done on several projects) is not permissible.
Расчет подводных трубопроводов на действие морских волн

(Окончание; начало см. «Строительство трубопроводов» № 7, 1961)

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Таким образом, горизонтальная составляющая скорости орбитального движения частиц жидкости (u_0), позиционивает сопротивление формы, направленной горизонтально, вызывает к вертикальному направленному подъемной силе.

Сила инерционного сопротивления на единицу длины цилиндра при искомом потенциалном потоке может быть выражена формулой

$$ P_n = q \frac{D^2}{2} \frac{D}{u}, $$

где $D_n$ — расчетный диаметр трубопровода в м; $u$ — ускорение потока в м/сек.

Вследствие вязкости жидкости распределение скорости в реальном потоке резко отличается от распространения скоростей в потоке потенциальном. Как показывают эксперименты,

$$ P_n = C_n \rho \frac{D^2}{2} \frac{D}{u}, $$

где $C_n$ — опытный коэффициент, зависящий от характера потока и формы тела.

Ряд авторов рекомендует значение опытного коэффициента $C_n$ в пределах 1,3—2,0.

Горизонтальные составляющие скорости $u_x$ и ускорения $u_{xx}$ орбитального движения частиц жидкости, а также скорость волнового течения $u_w$ обусловливают волновое давление на трубопровод в горизонтальном направлении $P_{xx} + \rho u_w^2$. Вертикальные составляющие скорости $u_z$ и ускорения $u_{zz}$ обусловливают волновое давление на трубопровод в вертикальном направлении $P_{zz} + \rho u_z^2$. Кроме того, как отмечалось выше, на трубопровод действует подъемная сила $P_x$, определяемая
Строительство трубопроводов

Стр. 10

Рис. 5. Эпиоры горизонтального волнового давления на трубопровод вдоль профиля волны.

Pipeline Along the Wave Profile.

Horizontally oriented drag force

На рис. 5 и 6 приведены эпиоры горизонтального и вертикального волнового давления (вдоль профиля волны) на трубопровод с гладкой поверхностью на длине 2L = 0,5 м. H = 30 м, t = 8 м и 2t = 80 м. Поскольку значение $P_u$ весьма мало, в расчетах его можно не учитывать.

При построении эпиор волнового давления определялись из следующих выражений:

$$ P_v = C_F D_n \frac{\rho u^2}{2g} \quad (23a) $$

$$ P_{v+u+u_e} = C_F + D_n \frac{(v+u+u_e)^2}{2g} \quad (23b) $$

$$ P_{v+u+u_e} = k_0 D_n \frac{(v+u+u_e)^2}{2} \quad (23c) $$

Используемые выражения получены из формул (17), (19) и (21) на основании равенства

$$ q = \frac{F}{A} \quad (24) $$

где $F$ — общий вес воды в $kg/m^2$, $g$ — ускорение силы тяжести в $m/s^2$.

Коэффициент кинематической вязкости $\nu$ в данном случае принят 0,015 $cm^2/sec$ (температура воды $+4^oC$); объемный вес морской воды $t = 1025$, коэффициент присоединенной массы $C_F = 1,75$; опытный коэффициент $k_0 = 0,4$.

На рис. 7 и 8 представлены эпиоры горизонтальных и вертикальных волновых давлений (вдоль профиля волны) на трубопровод с учетом донного течения $u_d = 0,3$ м/сек, направленного в сторону распространения волны. На эпиорах приведены также давления, зависящие только от донного течения.

На эпиоры видно, что при принятых нами габаритах, параметрах волн, диаметре трубопровода, скорости и направленности донного течения максимальное горизонтальное волновое напряжение на подводный трубопровод без учета донного течения приближается к 10—11 раз больше, а с учетом донного течения в 25—29 раз больше, чем давление, оказываемое на трубопровод донным течением. Вертикальные волновые напряжения больше в 3—6 раз.

С изменением указанных выше величин соотношения волновых напряжений и давлений на трубопровод от донного течения может изменяться. Однако с учетом донного течения волновое давление на трубопровод всегда будет меньше давления, оказываемого только одним донным течением.

Максимальные значения горизонтальной и вертикальной составляющих волнового давления на подводный трубопровод относительно друг друга вдоль профиля волны. Поэтому при принятом коэффициенте устойчивости $k$ для определения всей балки, обеспечивающей неподвижность трубопровода на дне моря, в формулу (22) следует подставить значения $P_{v+u+u_e}$ и $P_{v+u+u_e}$ при наиболее неплохом положении трубопровода относительно профиля волны.

С увеличением длины волнового давления на подводный трубопровод возрастает. На рис. 9 приведены эпиоры суммарных горизонтальных волновых давлений на трубопровод вдоль...
Рис. 8. Зависимость вертикальных волновых давлений на трубопровод вдоль профиля воды с учетом донного профиля волн (Dd = 0,5 м, H = 30 м, 2h = 8 м и 2l = 120 м) без учета донного течения. На этих зонах видно, что максимальное горизонтальное волновое давление достигает 38 kPa. С учетом донного течения максимальное горизонтальное давление составляет ~55 kPa. С учетом донного течения волновые давления в этом случае уменьшаются до 32 kPa. Однако, эти зависимости, как и формула (10), не совсем точны.

Приведенные данные позволяют в первом приближении (в случае с однородной и большей мощностью) судить о характере и величине волновых нагрузок на подводный трубопровод.

Все рассмотренные эталоны волновых давлений используются для случая распространения волнения на кромке трубохода. При распространении волнения под углом к трубопроводу большей или меньшей 90°, ввод пульсации на трубопровод в этом случае не происходит по всем параметрам и направлению и будет изменяться как во времени, так и по длине трубопровода.

Под горизонтальными условиями симметрии и ускорения орбитального движения частицы жидкости в этом случае следует принять проекции горизонтальных составляющих скорости u и ускорения k на нормаль к оси трубопровода. Зависимости волновых давлений для участков трубопровода, расположенных между двумя соседними шпангоутами (или вершинами) волн, будут повторяться для каждой волны.

Влияние морских волн на подводный трубопровод может быть определено только волновыми давлениями, возникающими по волне и направлению.

Как уже отмечалось, сопротивление тела возникает в основном вследствие вихреобразования. При обтекании цилиндрического тела частицы жидкости благода размерности и мерзлой среды цилиндра, вертикального отражения от его поверхности наблюдается образование двух симметричных вихрей, непосредственно появляющихся на волне. С увеличением вибрации вихрях структура, одновременно, смещается, и происходит от поверхности цилиндра (трубопровода). Однако, такое вихреобразование носится неустойчивым. При дальнейшем развитии вихря, происходит его устойчивое образование и одновременно отражение вихрей на цилиндре. Отражение вихрей изменяет распределение давления по поверхности цилиндра (трубопровода). При несимметричном распределении вихрей наблюдается более сильное давление, которое непрерывно меняется по направлению. Это обстоятельство, особенно при периодическом колебании трубопровода, должно быть учтено в расчетах с учетом вихреобразования.
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