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017 Scour Around Seafloor Structures

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SCOUR AROUND SEAFLOOR STRUCTURES

Keith Philpott Consulting Limited
in association with
Acres Consulting Services Limited

Scientific Advisor: P. Sabatini

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A major part of the reserch was devoted to questionnaires which were designed to obtain unpublished data. These were circulated to the industry, to consultants, and to research centres in Canada and abroad. See Appendix A.

The report was planned by K.L. Philpott and written by R.B. Nairn, K.L. Philpott, and C.A. Fleming and C.R. Beresford. Prototype case histories, summarized in the report, were prepared by C.R. Beresford. They are listed in Appendix B. Valuable information which has been incorporated in the report, where appropriate, was obtained from numerous others in industry, consulting and reseach centres in Canada and abroad. They are listed in Appendix C.

Comments and guidance were obtained from members of the ESRF commitee on scour and sedimentation who reviewed the work as it progressed.

1. SUMMARY

The objective of this study is to outline and assess the general state of knowledge and experience of scour around seafloor structures. Sources of information were: the technical literature; personal contacts with relevant representatives of the oil and gas and related service industries; and questionnaires. The questionnaires were intended to identify unpublished case studies suitable for analysis.

A thorough assessment of the literature revealed contradictory views concerning several aspects of scour. Many of these differences are only misunderstandings which result from the lack of a standardized approach to the description of scour. The most crucial parameter in the description of scour is 'ultimate maximum scour depth', which may only be an instantaneous condition reached during the peak of a storm.

The definition of scour adopted for this study is as follows: scour is erosion caused by modification of water movement due to the prescence of a natural obstacle or man-made structure.

Throughout the literature there is also confusion concerning the development of scour under the different hydrodynamic conditions of waves alone, and the combination of waves and currents. This confusion may in part be attributable to scale effects in physical model tests which obscure a clear representation of the prototype. For most structures in deepwater, (excluding those exposed to breaking waves), currents are the dominant cause of scour and wave effects, if they are significant, will be superimposed on those currents.

There is a definite lack of good prototype data, in particular scour measurements which can be related to environmental conditions and are essential to improve prediction techniques. There is a need to develop a technique to monitor scour during extreme storm conditions since little is known concerning the ultimate maximum scour depth. Surveys of scour holes

following major storm events cannot be expected to represent the ultimate maximum scour depth condition since infilling and restabilization of the bed will have likely occurred in the interim.

The most common scour protection technique is the use of rock, stone, or gravel layers. This technique has been proven successful and has a relatively low cost. An alternative method of protection is artificial seaweed; however, it remains relatively unproven.

There are a large number of scour prediction techniques to choose from, although most are empirical and based on physical model results. The influence of scale effect, inherent in mobile bed models, has not been systematically investigated in the literature and consequently results are questionable. There are no true analytical scour prediction techniques. Some success has been achieved with numerical models as a predictive method for scour around large gravity structures.

Prediction techniques in general are poor and can only be improved through i) the development of the hydrodynamic theory for flow around structures (particularly concerning turbulence and three-dimensional flow); ii) a systematic investigation of the influence of scale effects on physical model results; iii) through the improvement of field measurements and; iv) improvement of knowledge of boundary layer flow under waves and currents.

Scour design procedure should be inititated with collection of environmental data. A statistical evaluation of the wave climate is usually undertaken to define structural loading and operational conditions for the structure and this is usually sufficient for scour design purposes. Usually, not enough attention is given to determining current conditions for scour design purposes. An effort should be made to measure currents during storm events at various water depths, including at the seafloor. The present method of taking a few weeks of measurements to define tidal components should also be continued, again, at various depths in the water column. Bottom sediment surveys should always be conducted to determine grain size distributions.

Predictive techniques based on empirical formulae derived from physical model results cannot be relied upon for accuracy, (because of the problem with scale effects), however they should give some indication of whether or not scour protection is required. For large gravity structures, numerical models are available as a predictive technique. If appreciable scour is anticipated, scour protection should be considered. Riprap layers may be designed utilizing unidirectional flow techniques (i. e. the Shields curve), however the design of gravel layers at present can only be established through model experiments, field experimentation, or numerical models within the limits where theory is sufficient.

Pile supported jacket structures are favoured as production platforms for the Scotian Shelf. Pipeline installations will also be required for a connection to mainland Nova Scotia. The prevailing environmental conditions and previous experience in the area indicate that scour is definitely a potential problem. The complex bathymetrty of the Scotian Shelf is a special problem which will have to be addressed in predictive models of wave and current climates.

Opinions in the oil industry are that scour is not an important issue and this is apparent in the approach taken to the problem of scour. These opinions are based on experience and severe problems with scour are indeed not widespread. However, better indications of ultimate maximum scour depths during a design storm should be attained before further judgement is made on the extent of the problem.

2. RESUME

L'objectif de cette étude est de souligner et d'évaluer l'état général des connaissances et de l'expérience sur l'érosion, ou affouillement, qui se produit autour des structures du fond marin. Les sources de renseignments ont été: documentation technique, contacts personnels avec les représentants des industries du pétrole, du gaz et des services connexes, et questionnaires. Les questionnaires étaient destinés à identifier des études de cas inédites, qu'il convenait d'analyser.

Une évaluation approfondie de la documentation a révélé des points de vue contradictoires sur plusieurs aspects de l'érosion. Bon nombre de ces divergences ne sont que des malentendus dus à un manque de méthode standardisée pour la décrire. Dans cette description, le paramètre de base est "la profondeur maximum finale de l'érosion", qui pourrait n'être qu'une condition instantanée atteinte au plus fort d'une tempête.

La définition adoptée pour cette étude est la suivante: l'affouillement est une érosion provoquée par la modification du mouvement de l'eau du à la présence d'un obstacle naturel ou d'une structure construite par l'homme.

La lecture de la documentation apporte également des confusions sur l'évolution de l'affouillement, qui subit l'effet des différentes conditions hydrodynamiques des vagues seules, puis de la combinaison des vagues et des courants. On peut attribuer en partie cette confusion aux effets d'échelle employés pour les tests de modèles physiques, qui empêchent une bonne représentation des prototypes. Pour la plupart des structures en eaux profondes (à l'exception de celles qui sont exposées aux vagues déferlantes), les courants sont la principale cause d'érosion et les effets des vagues, s'ils sont importants, viennent s'ajouter à ceux des courants.

Il y a un manque de données sur les prototypes, en particulier sur les relevés mesures de l'affouillement, qui peuvent être liés aux conditions de l'environnement et sont essentiels à l'amélioration des techniques de prévision. Il faut développer une technique pour surveiller l'érosion dans les conditions d'une tempête de violence extrême, puisqu'on dispose de très peu de renseignements sur sa profondeur maximum finale. On ne peut s'attendre à ce que des études sur les trous d'érosion, provoqués par des tempêtes d'importance majeure, représentent une bonne condition d'évaluation de la profondeur maximum finale de cette érosion, puisque resédimentation et nouvelle stabilisation du fond marin seront probablement intervenues dans l'intervalle de temps.

La technique de protection de l'affouillement la plus courante est l'utilisation de couches de roches, de pierres ou de gravier. Cette technique, relativement peu coûteuse, s'est révélée très efficace. Une autre méthode de protection, qui n'a cependant pas encore beaucoup fait ses preuves, consiste à utiliser des algues artificielles.

Il est possible de faire un choix parmi les nombreuses techniques de prévision des érosions, bien qu'elles soient empiriques et fondées sur des résultats de modèles physiques. L'influence de l'effet d'échelle, propre aux modèles de fonds marins mobiles, n'a pas fait l'objet, dans la documentation, d'études systématiques; les résultats sont, par conséquent, contestables. Il n'existe pas de vraie technique analytique des prévisions d'érosion. Les modèles numériques ont remporté un certain succès en tant que méthode prévisionnelle d'étude de l'affouillement autour des structures de grande densité.

En général, les techniques de prévision sont faibles et ne peuvent être améliorées que par i) le développement de la théorie hydrodynamique du courant autour des installations (en particulier la théorie concernant la turbulence et le courant tri-dimensionnel); ii) une recherche systématique de l'influence de l'effet d'échelle sur les résultats des modèles physiques; iii) l'amélioration des relevés mesures en milieu natural et iv) celle des connaissances sur le déplacement de la couche limite sous l'effet des vagues et des courants.

La procédure d'étude de l'érosion devrait commencer par la cueillette de données sur l'environnement. D'ordinaire, on effectue une évaluation statistique du régime des vagues, afin de définir la charge structurelle et les conditions opérationnelles qui en résultent pour l'installation; cette évaluation suffit en général pour étudier l'affouillement. Habituellement, on ne tient pas assez compte des conditions du courant. Il faudrait déployer des efforts pour mesurer les courants pendant les tempêtes, à diverses profondeurs de l'eau, y compris dans le fond marin. Il faudrait aussi continuer d'utiliser la méthode actuelle, qui consiste à prendre des relevés, pendant plusieurs semaines, afin d'établir les composantes de la marée, également à diverses profondeurs. On devrait toujours effectuer des études sur les sédiments pour déterminer les conditions granulométriques.

Les techniques de prévision fondées sur des formules empiriques, issues des résultats de modèles physiques, ne sont pas assez précises pour être fiables (à cause du poblème des effets d'échelle); cependant, elles devraient indiquer si une protection de l'affouillement est nécessaire ou pas. En ce qui concerne les structures de grande densité, on dispose de modèles numériques comme technique de prévision. Si l'on prévoit une forte érosion, il faut envisager sa protection. On peut concevoir des couches d'enrochement en utilisant les techniques du courant unidirectionnel (c'est-à-dire la courbe de Shields); cependant, la conception actuelle des couches de gravier ne peut s'établir que grâce à des expériences effectuées sur les modèles, des essais réalisées en milieu naturel ou des modèles numériques, dans les limites où seules la théorie suffit.

On préfère avoir des structures de protection sur pilotis comme plate-forme de production pour le plateau continental Scotian. Il faudra également prévoir l'aménagement de pipelines pour assurer une liaison avec la terre de Nouvelle-Ecosse. Les conditions actuelles de l'environnement et les expériences précédentes réalisées dans la région indiquent que l'affouillement est précisément un problème spécial dont il faudra s'occuper dans les modèles de prévision des régimes de vagues et de courants.

On pense, dans l'industrie pétrolière, que l'affouillement n'est pas un sujet important, ce qui se reflète d'ailleurs dans la façon d'aborder la question. Ces opinions reposent sur l'expérience, et les graves problèmes occasionnés par l'érosion ne sont pas vraiment courants. Cependant, on devrait obtenir de meilleures indications sur les profondeurs maximum finales de l'affouillement pendant l'étude d'une tempête, avant de porter d'autres jugements sur l'étendue du problème.

2. INTRODUCTION

2.1 Purpose

Scour around seafloor structures has been the subject of a large number of studies and reports. There are many aspects to be studied considering the wide range of structure types and the variability of environmental conditions which exist. The purpose of this report is to produce a synthesis of all available information and knowledge of scour, provide guidance for all aspects of scour design.

1. to collate dispersed information resulting from international experience on scour around seafloor structures;

2. to identify successful design procedures and practical experience; and;

3. to outline design guidelines including recommendations for predesign investigations, data requirements and procedures for selecting effective scour prevention measures;

4. to identify and relate aspects of internatinal experience of scour that appear relevent to structures and conditions likely to be found on the Scotian Shelf.

The three principal sources of information were the literature, personal contacts, and a questionnaire survey. A literature search provided the greatest amount of information and the details are summarized in Section 2.3.1. The purpose of the questionnaire survey was to elicit unpublished information in the form of practical experience of the designers, owners, and operators of seafloor structures (further details are provided in Section 2.3.2).

Personal contacts in both face to face meetings and by telephone have almost always proved to be enlightening, even in the many instances when they did not result in the production of documented evidence. By this means, for example, it was possible to obtain an overview of geographical areas where scour has at one time or another been considered a matter for concern. These are listed below:

- jack-ups on the Scotian Shelf;
- scour around both gravity structures, and pile supported jacket structures in the south and central North Sea area;
- scour due to hurricane surge effects around pile supported production platforms in the entrance to Mobil Bay, Alabama (Gulf of Mexico area);
- scour due to tidal and storm induced currents in Cook Inlet, Alaska, around monopod and other structures;
- scour and spanning problems associated with pipelines in the southern North Sea;
- tidal current and storm induced scour around structures off the north coast of Australia.

Apart from the actual on-site experience with jack-up rigs in the Scotian Shelf itself, the most closely related area in terms of depths and other conditions appears to be the southern North Sea.

2.2 Scope

The report investigates many different aspects of scour for a variety of structure types. First, typical scour problems are investigated through a review of descriptions of scour phenomena in the literature and then through a quantitative outline of prototype scour experience. This forms Section 3 of the report. Scour protection methods are investigated in Section 4. Section 5 offers a review of measurement, estimation, and design practices including available scour prediction techniques. In Section 6 existing codes of practice are reviewed and a framework for guidelines to scour design procedures is developed. The findings of the report are related to the Scotian Shelf in Section 7 and discussions and conclusions follow in Section 8.

Scour phenomena are investigated for many types of seafloor structures including pile supported jacket platforms, jack-up rigs, pipelines, submerged bottom structures, large gravity structures, sacrificial islands, and caisson retained islands.

Whether in simple unidirectional flow or the more complex marine environment, scour is the manifestation of locally varying flow conditions which are induced by the presence of the structure itself. The development of a scour hole can be quite rapid but is generally thought to be limited to some equilibrium depth. Effective means of providing scour protection poses special problems as protective work can itself induce further scour.

Scour can occur around gravity based structures, pile-founded structures, and pipeline installations. However, each such case exhibits its own distinctive set of hydrodynamic conditions. There is considerable practical experience with dealing with such problems in the marine environment although the theoretical means of predicting scour and the effectiveness of scour protection are not yet well developed. Consequently, knowledge of actual examples of successful design or remedial works is extremely useful when considering further designs and possible improvements to existing works.

In view of the above, the aims and scope of this study are:

- to review all available literature concerning scour problems including available data, analytical techniques, design measures and remedial works;

- to identify a number of case studies that provide sufficient quantitative information on the hydrodynamic forces (principally waves and currents) and bottom material characteristics;

- to categorize and relate selected case studies to different situations on the Scotian Slope and Shelf;

- to obtain as much information as possible from oil companies, design engineers, hydraulic institutions, research organizations and the like, worldwide concerning design standards and guidelines, physical model results or prototype data relating to both scour prediction and scour prevention methods;

- to prepare design guidelines which will include recommendations for pre-design investigations, data requirements and procedures for selecting effective scour prevention measures.

2.3 Study Methodology

2.3.1 Literature Search

The published literature search has drawn upon three independent data base searches conducted for the Consultant plus literature identified by various contacts. In addition, a very extensive literature list was provided by Mobil Oil Canada. From several hundred titles obtained by these means, one hundred and five papers were identified as having very direct relevance and are referenced in Section 9 of this report. Also, papers which are directly related to the subject of scour around seafloor structures were abstracted and have also been archived by COGLA. The unpublished literature search has depended mainly on personal contacts. The results included have been referenced in Section 9.

2.3.2 Questionnaire Survey

The purpose of circulating questionnaires was to elicit quick sketches of cases of scour and scour protection applications from the practical experience of designers, owners, and operators of seafloor structures. A circulation list was generated covering oil companies, contractors, consultants, and laboratories worldwide. The circulation list and the standard questionnaire package can be found in Appendix 3.

The initial response to the survey was disappointing both in the percentage of responses and in the time scale for replies. However, through follow-up contacts, extending over the duration of the study, a fairly respectable return was eventually received. The list of questionnaires issued and responses have been archived by COGLA. Of the one hundred and thirty-five questionnaires circulated, fifty percent eventually replied and of these, thirty percent provided useful information of some description. Revelant data from the questionnaires is included in Section 4. The returned questionnaires have been archived by COGLA. Further information on the questionnaire survey and a summary of replies are included in Appendix A.

3. TYPICAL SCOUR PROBLEMS AND PROTOTYPE EXPERIENCES

3.1 TYPICAL SCOUR PROBLEMS

3.1.Ø Seafloor Scour Phenomena

For erosion to occur, the bottom stress must be sufficiently high to initiate movement of sediments. Currents may not be sufficient to move sediments by themselves but when combined with wave components the threshold of movement velocity may be exceeded. Wave initiated sediment movement results in very little transportation by itself because of its oscillatory nature. However, currents may transport sediments stirred up in combination with waves, the mean velocity required to transport material being lower than that to initiate movement. Sediment transport across an area results in erosion if there is an imbalance in the quantity of material entering and leaving the area. Individual extreme storms can have a great effect on the basic stability of the seabed overall, and on scour depths and patterns around a structure. The extent of these effects may not be apparent from scour depth measurements made some time after the storms due to partial refilling of scour cavities under calmer sea conditions.

In many areas, the seabed is in relatively stable equilibrium. The introduction of any obstruction to the natural patterns of water movement is likely to alter the basic equilibrium, causing increased local velocities and turbulence, initiating scour. Scour adjacent to an object on the seabed will continue until a new local equilibrium is achieved. A pattern of erosion and accretion will have developed which is dependent on the geometry of the object, water depth, the nature of the seabed material and the magnitude and direction characteristics of waves and currents. Scour can be of a local nature, at the corner of a platform base for example, or global, covering an area perhaps two or three times that of the structure or element.

Overall seabed levels may change over a much longer time scale, measured in years or decades as a result of migration of large sand bedforms. The investigation of these large scale seabed movements are not included within the terms of reference of this study which deals with effects caused by and adjacent to structures. However, bedforms are mentioned in connection with pipelines because it is so closely related to scour phenomena as to be indistinguishable from it in some cases.

3.1.1 Individual Piles

The total blockage of flow by a structural element such as a pile at the seafloor alters the flow field in two ways, by creating increased local flow velocities, and by increasing the level of turbulence. First, potential flow theory shows that the flow past the sides of the structure will be double the ambient velocity due to convectional acceleration. Second, the interaction of the approach flow boundary layer and the solid structure causes a secondary flow field to develop. In front of the structure, a vertical zone of relatively quiet water forms. Along this stagnation line a strong descending fluid jet (referred to as the primary vortex) is formed because of the dynamic pressure gradient resulting from the velocity distribution within the boundary layer. The approach flow can then be represented by filaments of turbulence which wrap themselves around the base of the structure creating the familiar horseshoe vortex configuration (see Figure 3.1). As the scour pit develops, a separation vortex forms at the upstream edge of the pit, similar to those found in the lee of ripples and a wake plume forms behind the structure (see Figure 3.2).

3.1.1.1 Scour Around Piles Due to Currents

The boundary layer in unidirectional flow is relatively thick and therefore turbulence will be the dominant cause of scour. The scour pit is formed as the primary vortex excavates material from the base of the obstruction which leads to the formation of an unstable slope. The unstable slope slumps into the pit until it reaches its angle of repose,

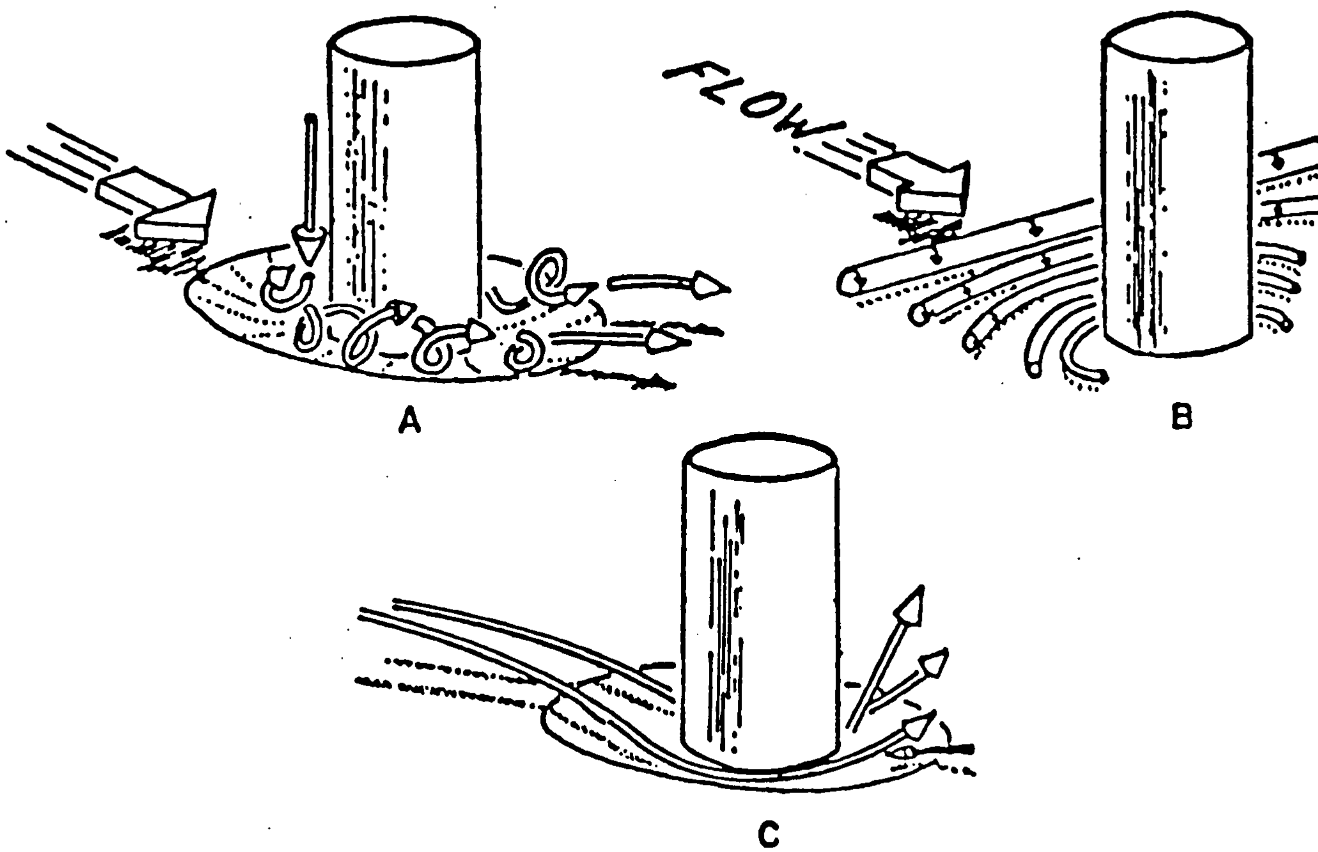


Figure 3.1 Turbulence Around a Pile (Palmer, 1970)

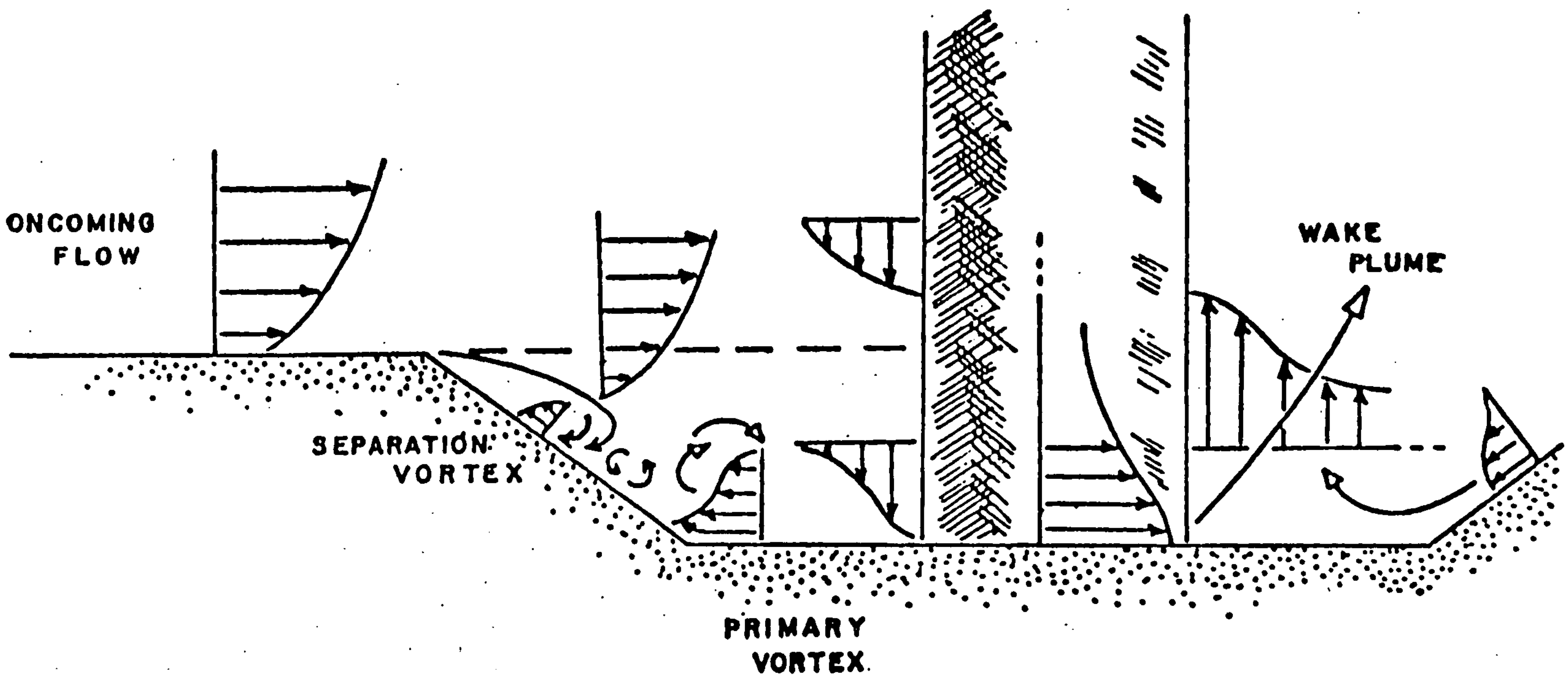


Figure 3.2 Schematic of Velocity Gradients Around a Cylindrical Obstruction (Palmer, 1970)

typically 35o for sand. The slumped material is lifted into suspension by the primary vortex and carried away by the accelerated flow surging past the sides of the obstruction.

The growth process as shown in Figure 3.3, continues until the primary vortex distance, X reaches a terminal value related to the diameter of the pile. Palmer (1970) suggests that for unidirectional flow the lateral extent of the pit L, is no greater than twice the primary vortex distance and any continued enlargement beyond this limit may be a unique feature of scour under oscillatory conditions. The weaker separation vortex formed at the leading edge of the pit initially works to reinforce the primary vortex and as the pit becomes larger it contributes to slope instability. Scour continues to occur until the sediment transport into the pit is equal to the rate of scour caused by the flow alteration. The unidirectional flow scour pit is radially asymmetric with tail and vortex ridges corresponding to the horseshoe flow configuration (see Figure 3.4).

3.1.1.2 Scour Around Piles Due to Waves

For waves acting alone, the flow field varies substantially from the case of unidirectional flow. The boundary layer associated with oscillatory flow is comparatively thin. The limited distance of fluid particle travel per half wave period is not sufficient to develop a comparable thickness of boundary layer to that which would accompany a similar unidirectional flow. Consequently, Niedoroda et al (1981), stated that the horseshoe vortex (which develops from interaction of the boundary layer with the obstruction) is insignificant for oscillatory flow. Instead, scour arises from the concentration of bottom shear stress caused by the acceleration of the primary flow as indicated by potential flow theory. Scour due to waves begins at the sides of the pile where velocity is greatest and initially some deposition will occur at the upstream and downstream faces of the pile (see Figure 3.5). The familiar circular scour pit which completely surrounds a pile will develop as a result of the development of the separation vortex at the edge of the pit slope (see Figure 3.6).

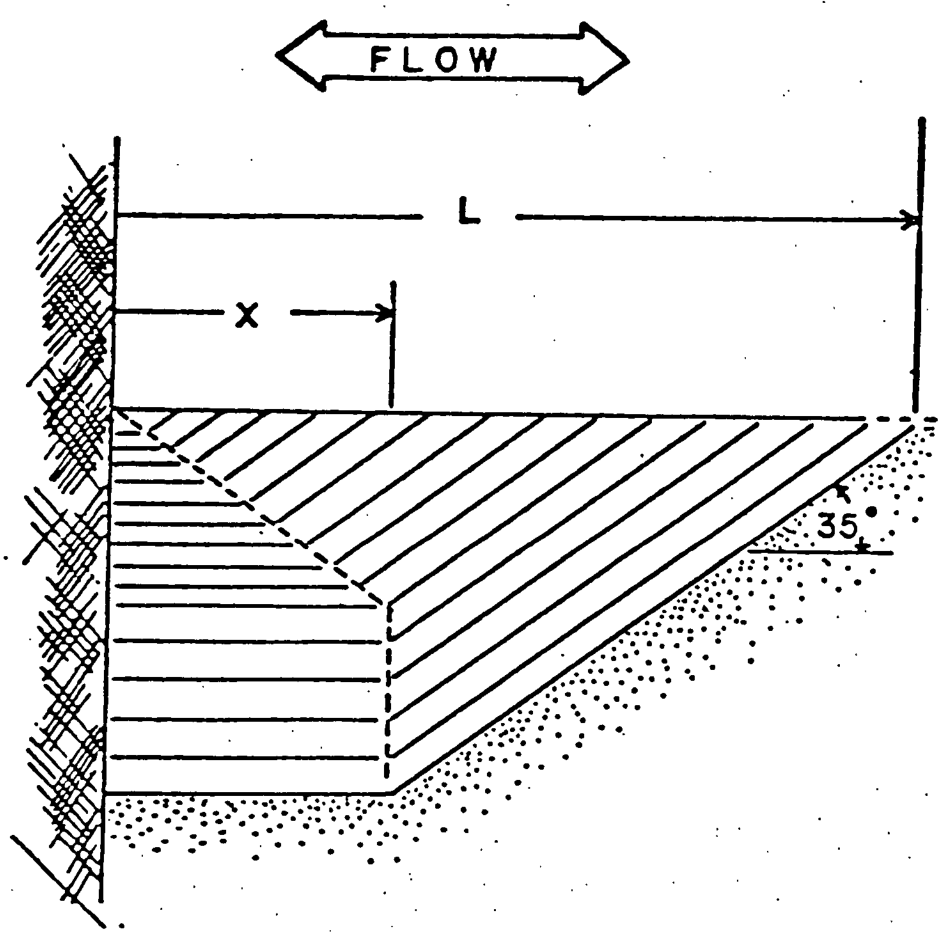


Figure 3.3 Growth of Pit to Terminal Scour Condition (Palmer, 1970)

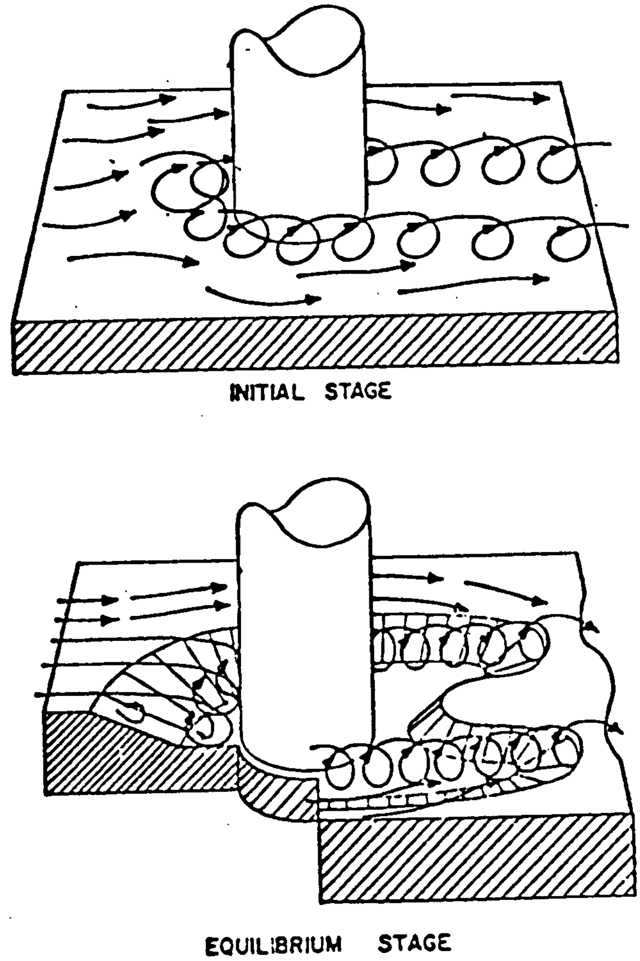


Figure 3.4 Flow and Scour Produced by a Steady Boundary Layer Flow (Niedoroda et al, 1981)

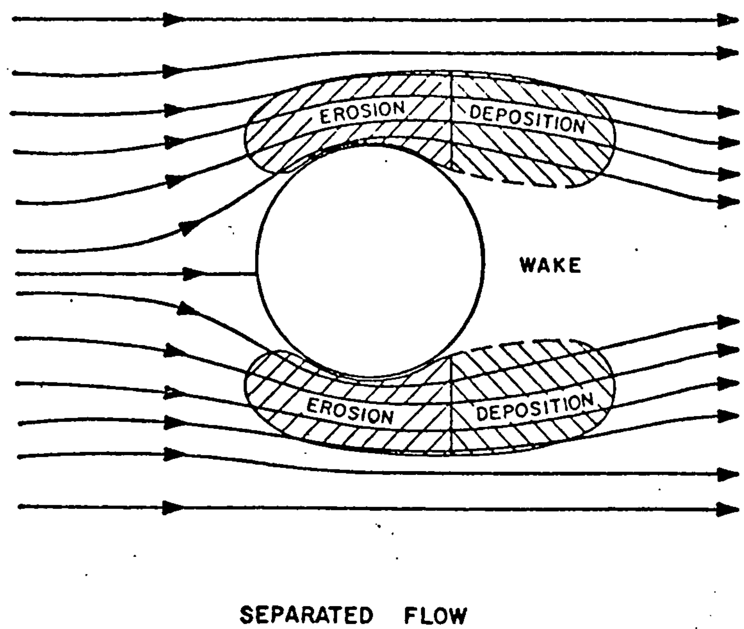


Figure 3.5 Instantaneous Flow and Scour Due to Oscillatory Flow (Niedoroda et al, 1981)

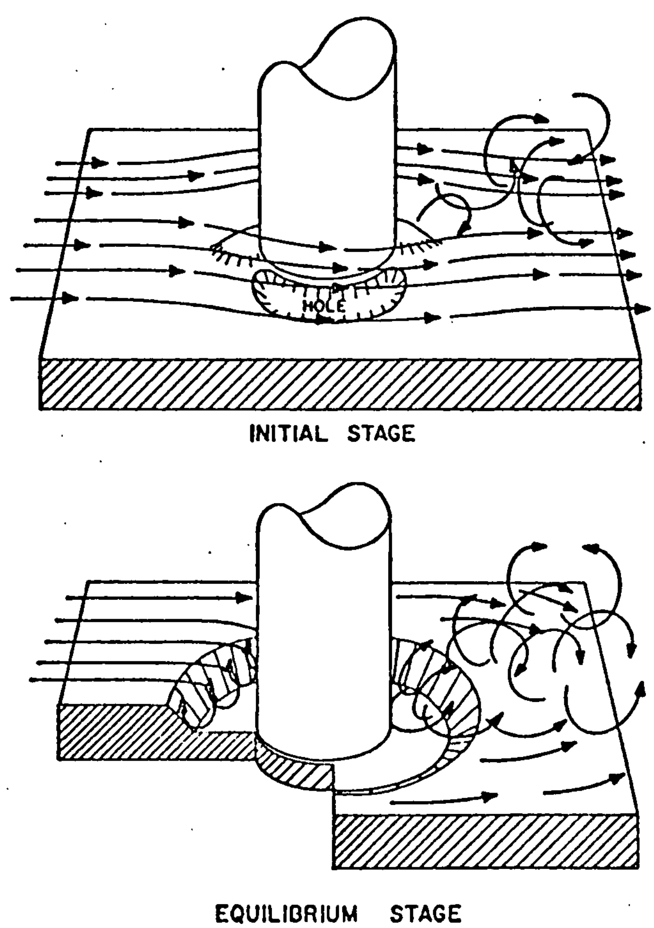


Figure 3.6 Stages of Wave Induced Scour Hole Formation (Niedoroda et al, 1981)

3.1.1.3 Scour Around Piles Due to Waves and Currents Combined

In most shallow marine environments, both waves and currents will contribute to the growth of scour pits. In model tests Ninomiya et al. (1972) found that the shape of the scour pit produced by the combined effect of currents and waves is similar to that which results from currents alone. They also found that the rate of erosion is much greater under the combined action of waves and currents. There seems to be general agreement on this point. In contrast,there is considerable disagreement on the ultimate depth of scour due to waves and currents combined as compared to the depth due to the action of waves alone. Wilson and Abel (1973), Breusers (1971) and Ninomiya et al. (1972) all suggest that the maximum depth of scour will be less for the combination of waves and currents than for currents alone, while Niedoroda (1981) and Herbich (1977) state that it will be greater. One possible reason for this apparent contradiction may be that the data used by the various authors for making the comparisons between the two cases (i.e. equating shear stress or velocity parameters) may have differed. Another reason may be that observations from model tests involve scale effects which might distort the actual prototype phenomenon.

The ultimate scour depth is influenced by pile diameter, wave, steepness, sediment grain diameter, flow velocity and water depth, as well as the nature of flow (oscillatory or unidirectional). The pile diameter has the greatest influence on the extent and depth of scour. Relationships for ultimate scour depth are based on the pile Reynolds number and are presented in Section 5.

In a compilation of results from model studies, Chow and Herbich (1978) find that, all other things remaining constant, the extent of wave induced scour increases as wave steepness increases and scour decreases as water depth increases. Palmer (1970) stated that scour depth is independent of grain size and flow velocity for the ranges he observed (0.120 to 0.630 mm and 24 to 61 cm/s respectively). However, dimensional analysis shows that these parameters will become increasingly significant as the critical stress for incipient motion is approached. Palmer also found that the shape and inclination of the structure do not affect ultimate scour depth.

3.1.2 Pile Groups

A group of piles will initially scour as individual obstructions although eventually the scour pits coalesce to form a circular pit around the group (Palmer, 1970). This group effect has been referred to as `field', `global' and `dishpan' scour in the literature and results in an overall lowering of the seabed (see Figure 3.7).

From prototype tests, Breusers et al. (1977), found that if two piles are separated by less than 6 diameters, scour depths will be greater than for an isolated pile. The scour around the group may eventually resemble that of a combined diameter equivalent to the sum of separation distances and the pile diameters. For separation distances exceeding six diameters there must be a transition towards the smaller scours associated with isolated piles.

In the case of piled jacket platforms local scour may also occur beneath horizontal braces close to the seabed resulting from an increase of water velocity around and underneath such members. Another consideration with jacket platforms which is often overlooked, yet may be critical in the extent of scour, is the clustering of conductor pipes. These are usually placed in the middle of a square platform or towards both ends of a rectangular base jacket. The spacing between conductor pipes is quite small and therefore the associated turbulence is greater.

As scour around piles increases the point of fixity lowers and the soil shear strength reduces. Unless remedial measures are taken to halt scour the piles could eventually be exposed to such an extent that instability and buckling occurs if not the loss of pullout resistance.

The principle cause for concern with leg supported rigs is exposure and

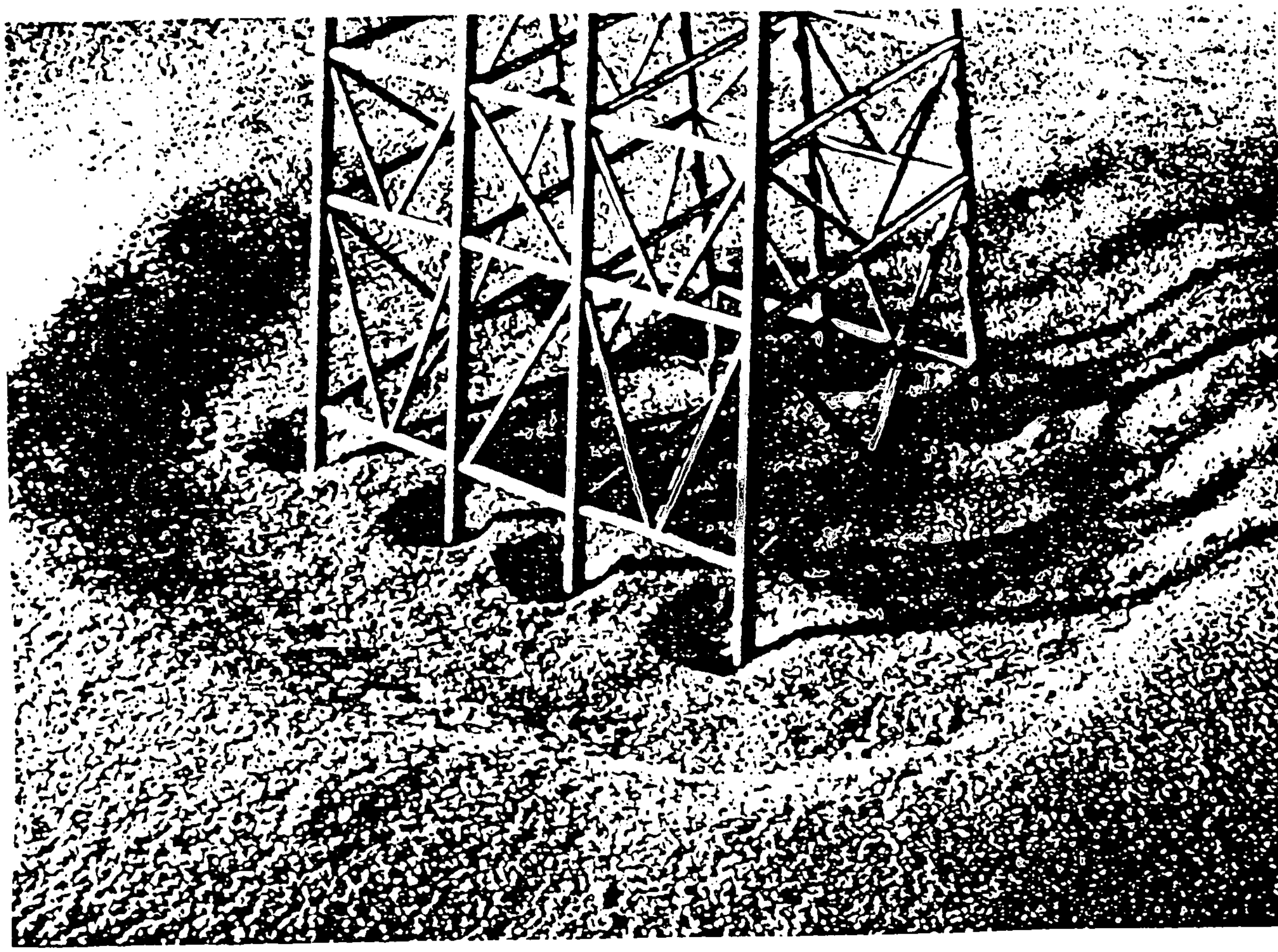


Figure 3.7 Photographic Representation of Platform Scour (Angus and Moore, 1982)

undermining of the spud cans of temporary jack-up facilities and of the caissons on more permanent structures, by scour processes at the legs. Caissons are typically sunk 20 m into the seabed whereas spud cans typically sink no more than 5 - 6 m before setting up. If undermining or exposure occurs the overall trim and stability of the jack-up structure suffers. Because of the absence of bracing members, jack-up rigs are more sensitive to a lowering of the point of fixity than piled structures. On the other hand, global scouring is likely to be less than for other large structures because of the relatively open nature of the rig.

Often it is necessary for a jack-up rig to perform drilling close to a jacket platform for `workovers', that is for the repair or redrill of wells. The introduction of the jack-up legs may destabilize the equilibrium scour pattern previously established around the jacket structure and cause scour to be reactivated leading to problems not previously present for the jacket structure alone.

3.1.3 Scour Around Pipelines

3.1.3.1 Scour Around Submerged Pipelines

Scour around offshore pipelines is also a problem because it poses a hazard where it results in the exposure of the pipeline to excessive loads leading to the eventual rupture of the pipeline with consequent and possibly far-reaching environmental damage and economic loss. However, while scour may lead to offshore pipeline failure, it is not the direct cause of such failure. To clarify this point, some "direct causes" of pipeline failure may be mentioned:

1. Rupture due to excessive horizontal forces which may in turn be due to one of the following:

    a. natural hydrodynamic forces;

b. mass movements of bottom soils by natural processes or triggered by works associated with the pipeline;

c. dragging of equipment such as anchors or trawl boards.

2. Rupture of the pipeline due to the imposition of excessive vertical forces due to one or more of the following causes:

   a. Sagging under its self-weight due to the removal of the support of the soil beneath the pipe due to:

      - natural sediment transport processes, such as sand wave migration, causing spanning;

      - mass movements of destabilized bottom sediments;

      - the presence of the pipeline itself; i.e. conventional scouring;

      - the mode of installation of the pipeline, such as absence of backfilling; or

      - a combination of the foregoing.

   b. Impacts due to equipment falling on the pipeline or hogging stresses due to the accidental lifting of the pipeline with an anchor or other equipment.

3. Rupture of pipelines due to the imposition of excessive longitudinal forces, usually in the vicinity of risers or connections to other seabed structures.

All failures may be exacerbated by the weakening of a pipeline due to other causes, in particular, due to corrosion. Thus, when a failed pipeline is recovered, it may not be easy to determine which of several contributory factors triggered the rupture. This may account for the fact that extensive pipeline failure data from U.S. sources cited by Herbich (1981) suggested that corrosion is the most common cause of failure (see Figure 3.8 and Tables 3.1, 3.2, and 3.3). Aside from this problem, it is clear from the foregoing summary of direct causes of failure that several modes of bottom sediment behaviour may contribute to pipeline failure, as follows:

1. Scouring around and beneath a partly exposed pipeline.

2. Migration of sand waves causing exposure of pipelines.

3. Mass movements of weak bottom soils triggered by wave action, earthquakes, or explosions. (See Figure 3.9)

4. Flotation and exposure of pipelines due to liquefaction of backfilling materials more dense than the pipelines. (See Figure 3.10)

5. Pipe exposure due to jacking of a pipe out of the shelter of an unfilled trench caused by inflow of sand beneath the pipe during storms.

6. Pipeline sinkage and sagging due to liquefaction of less dense soils beneath a heavy pipeline.

Herbich (1981) also suggests that scour and liquefaction may proceed together because the materials most susceptible to scour are the same as those most likely to liquefy. Thus, the exact process by which a pipeline becomes exposed may not always be clear.

Liquefaction occurs when pore pressure in loose, fine-grained backfill

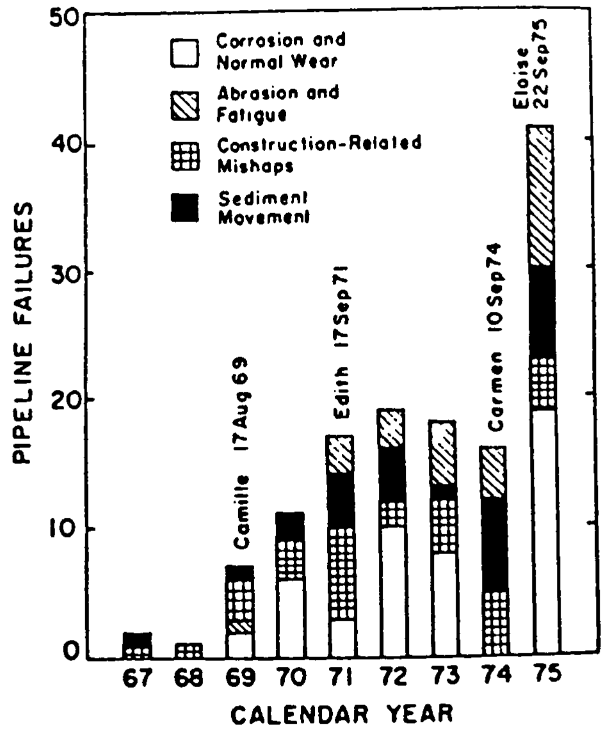


Figure 3.8 Histogram of Pipeline Failures 1967 to 1975 (Demars et al, 1977)

Table 3.1 Summary of Flowline and Transfer Line Breaks at Mississippi Delta (Arnold, 1967)

	1958-1965 (excluding hurricanes)	Carla (1961)	Hilda (1964)	Betsy (1965)	Total	
Corrosion	79				79	
Anchor or spud	23				23	
Leak in clamp	19		1	1	21	Due to Soil Motion or Currents
Rubbing	21	1	2	1	25	
Line goes into mud	10	5	5	5	25	
Line in tension	2	5	5	0	12	
Riser pulled	4	4	6	1	15	
Breaks above mean ground level	0	4	4	14	22	
Unknown mechanical breaks	22	11	6	10	49	
Totals:	100	30	29	32	271	

Table 3.2 Bureau of Land Management: Pipeline Failures, June 1976 to February 1977 (Herbich, 1981)

Date	Location	Type of pipeline	Cause
June 1976	Block 168 Ship Shoal area	20" Natural gas pipeline	Pinhole leak on flange connecting riser to submerged pipeline
July 1976	Block 28 Ship Shoal area	Oil pumping platform	Float failure on generator, causing oil spill
Aug. 1976	Block 63 East Cameron area	Natural gas pipeline	Pinhole leak in riser
Nov. 1976	Block 250 Eugene Island area	18" Natural gas pipeline	Anchor damage by tugboat
Dec. 1976	Block 297 Eugene Island area	14" Pipeline for crude oil, condensate, & liquid hydrocarbons	Fishing trawl pulling out nipple & ball valve at 200' depth
Jan. 1977	Block 320 East Cameron area	6" Oil pipeline	Valve left open after dewatering process, safety bleeder plug broken off, closure
Feb. 1977	Block 313A Eugene Island area	20" Oil pipeline	Connection pulled by unknown forces, causing the pipe to crack

Table 3.3 U.S. Geological Survey: Pipeline Failures, September 1974 to April 1977 (Herbich, 1981)

Cause of failure	Number of failures 1974	1975	1976	1977	Total
Corrosion (internal & external)		19	12		31
Unknown (information not available)		5	4	11	20
Passing vessels (collision, anchor damage)		7	2		9
Mechanical failures (valve leaks, defective gaskets, ruptured valves)		5			5
Physical damage due to unspecified causes (holes, cracks in welds & sleeve clamps, broken valves)			3	2	5
Mudslides	1	3			4
Wave damage		1			1
Construction-related mishaps		1			1
Operator error	—	1	—	—	1
Totals:	1	42	21	13	77

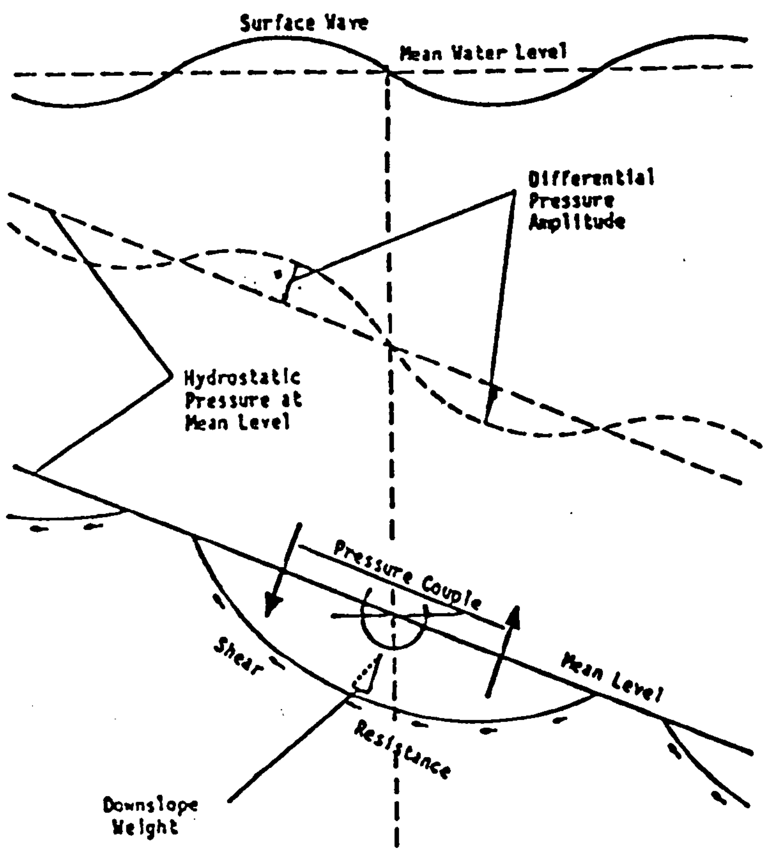


Figure 3.9 Slide Initiation Analysis Model (Bea, 1971)

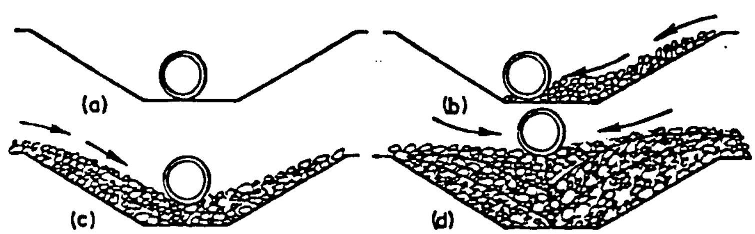


Figure 3.10

Jacking process. a) the pipe is placed in a dredged trench; b) current and/or wave action from right to left produces sediment transport which causes the pipe to move to the left and slightly upward; c) current and/or wave action from left to right produces sediment transport which moves the pipe to the right and upward; d) at a later date after several changes in current and wave action the pipe is found laying on top of the trench. (Herbich, 1981) induced by wave action equals overburden pressure, so that the effective (intergranular) shear stress in the soil mass is reduced to zero, whereupon the pipeline floats up if it is less dense than the resulting slurry, or sinks down if more dense. (See Figure 3.11)

Scour on the other hand, is the localized planing away of bottom sediment occuring where the sediment transporting capacity caused by the disturbance of the natural flow of water is increased by the presence of the pipeline. The process continues until the bottom contours adjacent to the pipe are modified to match the dynamic equilibrium condition of surrounding undisturbed bottom area under the given flow conditions.

The scour hole configuration may be adjusted as the flow conditions change, so that, as with other seabed structures, local observations of scour under calm conditions may not be representative of conditions when hydrodynamic loadings are greatest and failures most likely to occur at the height of a storm. Where spanning is due to sand wave migration, this is less of a problem, although in that case too, the length of a span may also be underestimated due to the fact that local scour will always be a contributory factor once an offshore pipeline has become exposed, no matter what the original cause of the exposure.

It would appear in some cases where excessive spanning occurs that failure is due to fatigue induced by resonant vibration associated with vortex shedding occasioned by flow around the pipeline. This is evidently a somewhat complex function of pipeline diameter, involving flow conditions as well as density, elasticity, the length of span and to a degree, the relative size of gap beneath the pipe.

Scour Around Pipelines due to Currents

According to Bijker (1983) the final depth and profile of scouring around pipelines due to currents depends on the proportion of the pipe diameter embedded below the original seabed level. The range of patterns observed is illustrated in Figure 3.12. See Figure 3.13 also, from Kjeldsen et al., (1973).

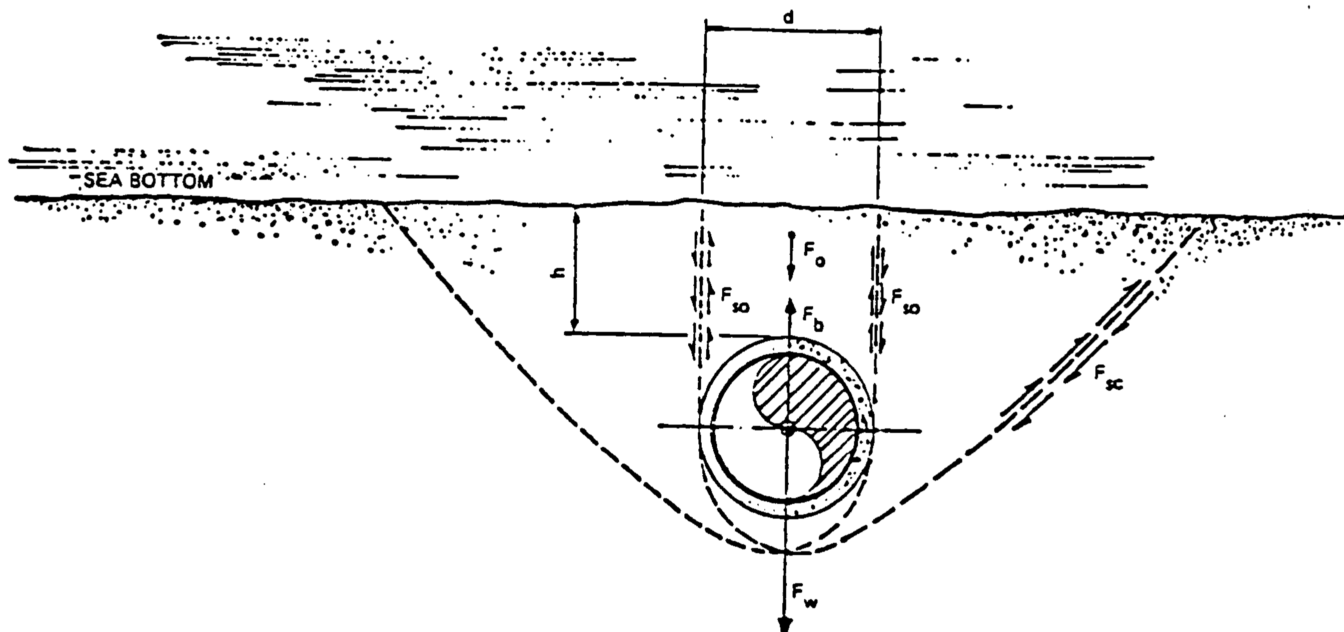


Figure 3.11

Forces on a buried pipeline: d = outside diameter of pipeline; h = depth of cover; F_w = weight of pipeline and contents, lb/ft; F_b = buoyancy of pipeline, lb/ft; F_o = weight of overburdening soil, lb/ft; F_{so} = apparent shear force of overburdening soil, lb/ft; F_{sc} = apparent shear force due to compression of the pipeline into the bottom soil, lb/ft. (Brown, 1971)

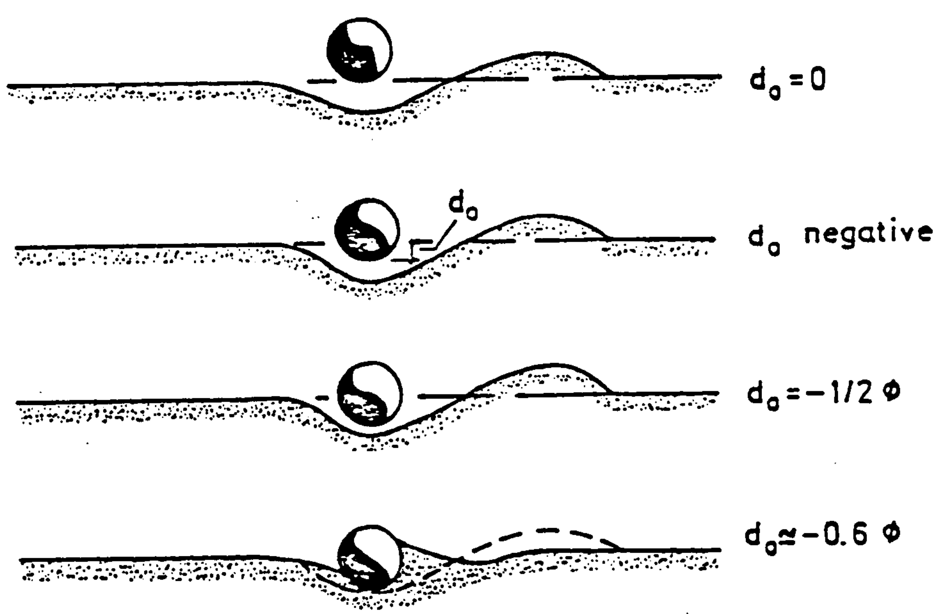


Figure 3.12 Scour Underneath a Pipeline at Different Heights Above the Seabed Due to Unidirectional Current (Bijker, 1983)

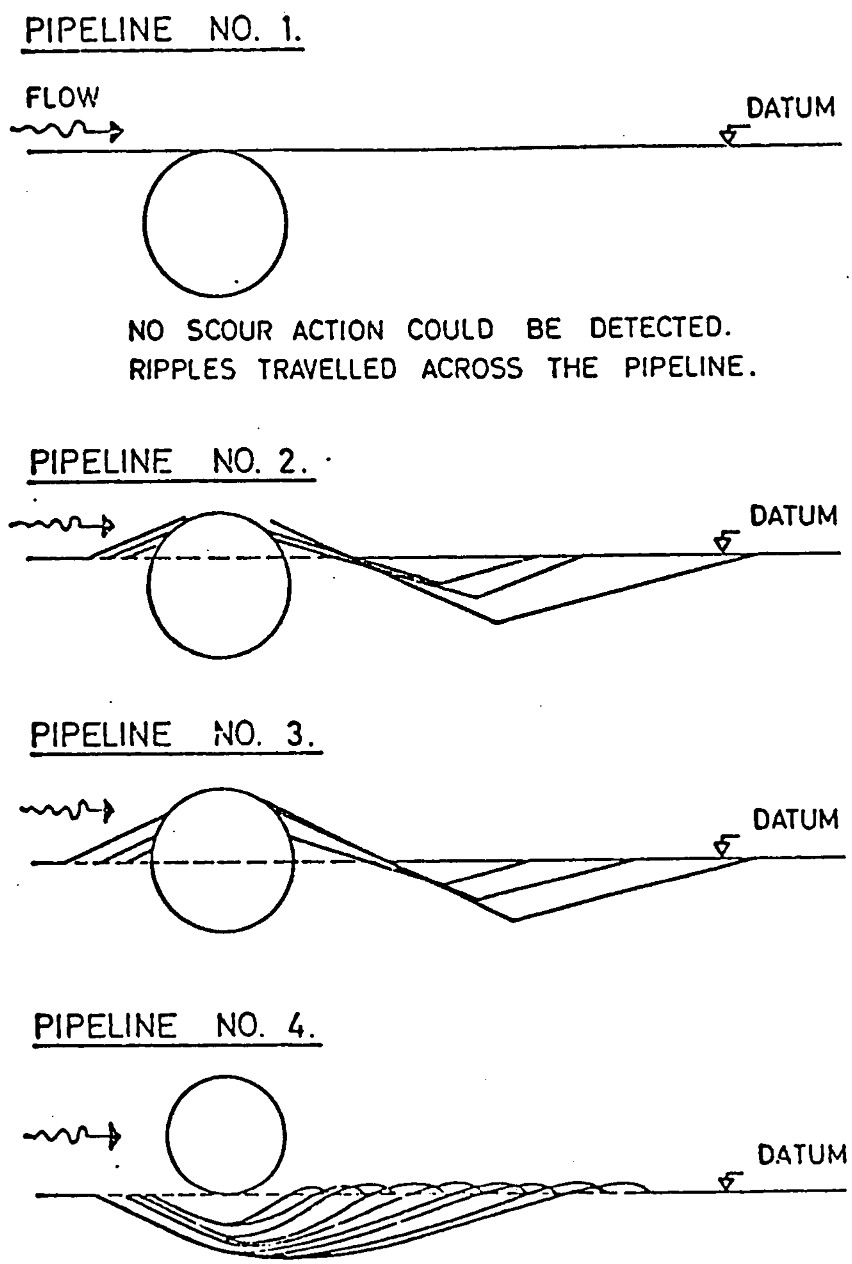


Figure 3.13 Scour Underneath a Pipeline at Different Heights Above the Seabed Due to Unidirectional Current (Kjeldsen et al, 1973)

Three distinct patterns of flow, scour and deposition about pipelines resting fully exposed on the bed due to currents alone have been described by Bijker op cit and are each illustrated in two of the sketches in Figure 3.14. The effects of these he refers to as "luff", "lee", and "tunnel" erosion depending on whether erosion is in front of, behind, or beneath the pipe. Luff erosion occurs where the pipe diameter is relatively large compared to the boundary layer and probably only applies to relatively low velocities. Luff erosion leads rapidly to tunnel erosion, while lee erosion, when it includes deposition against the downstream face of the pipe may retard the onset of tunnel erosion. Zdravkovich & Kirkham (1982) also classify three, presumably related, flow regimes which differ according to whether the pipe rests on the bed or is raised slightly or to a greater extent above the bed. (See Figure 3.15)

There is general agreement that the ultimate scour depths due to current action alone are more closely related to pipe diameter than to ambient velocity, other factors being equal. This makes the depth of unidirectional current induced scour around offshore pipelines relatively easy to predict. There are several empirical formulas available notably that of Kjeldsen et al. (1973), from which others have been derived. Unfortunately, though, actual cases of pipeline scour are, as pointed out above, usually due to combination of causes.

The three types of erosion identified by Bijker had also been noticed by others. See Figure 3.13, from Blumberg (1974) which also illustrates the principal forces involved.

Scour Around Pipelines due to Waves

Scour around pipelines due to waves alone is described and illustrated in Bijker's 1983 paper. The process is complex, involving more variables than scour due to currents alone. However, scour depths for pipelines are less for wave induced scour than for unidirectional currents with velocities comparable to the maximum orbital velocities of the waves.

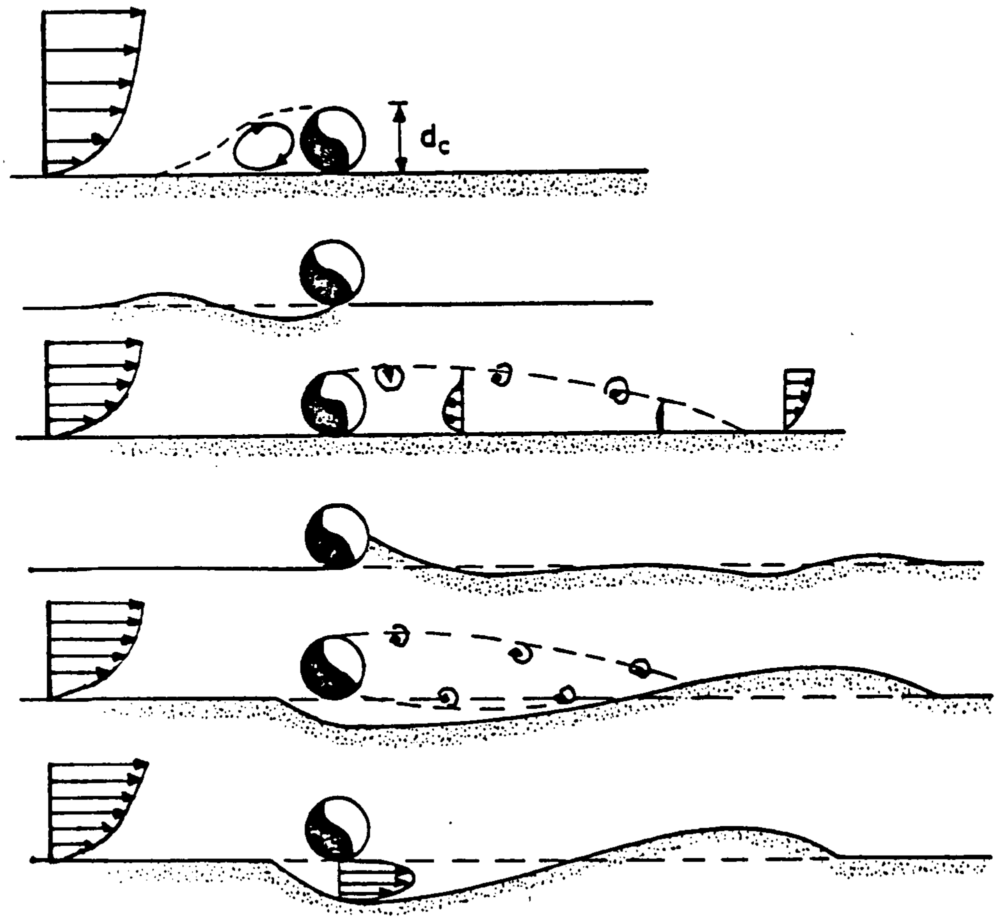


Figure 3.14 Scour Around Exposed Pipelines (Bijker, 1983)

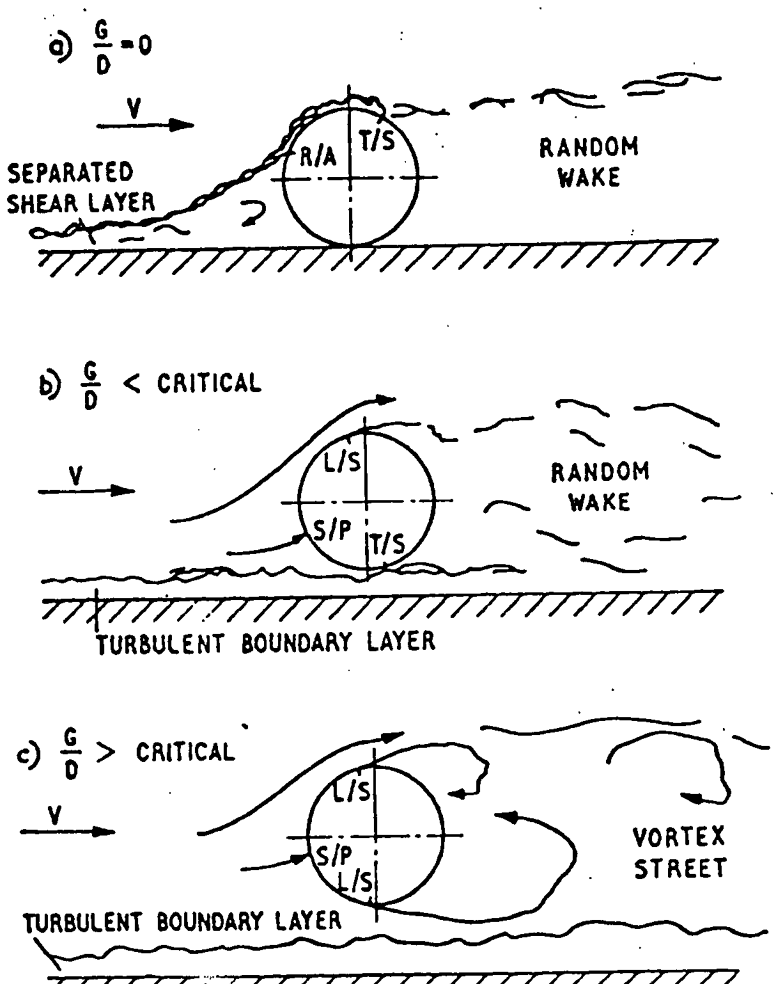


FIGURE 3.15

G = gap between cylinder and plane boundary
D = outer diameter of the cylinder

Classification of Interference Flow Regimes
(Zdravkovich and Kirkham, 1982)

Also, unlike currents, scour depths due to waves show little influence of pipe diameter although orbital excursion distance is of some importance. Apparently, the oscillatory nature of the process results in less material being removed from the scour hole than by unidirecitonal currents and shear stresses of similar magnitude because sediment particles are carried back and forth across the scour cavity with a significant probability of being repeatedly redeposited beneath the pipe. According to Zdravkovich and Kirkham (1982), this may lead to a double lobed scour cavity as shown in Figure 3.16, and in some cases producing definite negative scour or deposition (Figure 3.17). The nature of the process in terms of the zone of occurrence of super-critical shear stresses is show in Figure 3.18 from Bijker.

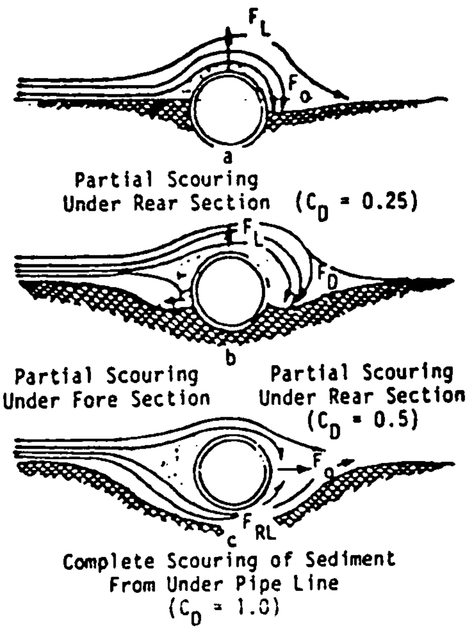


FIGURE 3.16

Effect of particle motion on a partially buried pipeline: F_L = lift force; F_d = drag force; F_{RL} = soil resistive force.

Note: The scour around partially buried pipeline causes increase in the relative drag coefficient C_D. Thus the force on the pipe due to current increases as the scouring action exposes the pipeline. (From Blumberg, 1974)

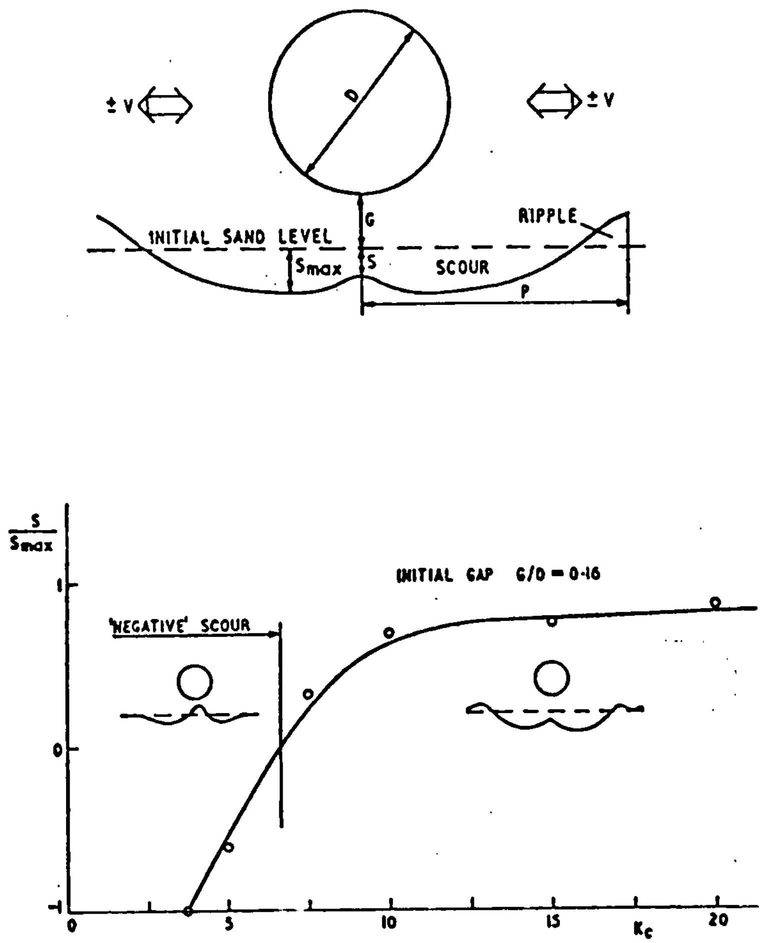


FIGURE 3.17 Variation of Relative Scour with Keulegan-Carpenter Number (Zdravkovich and Kirkham, 1982)

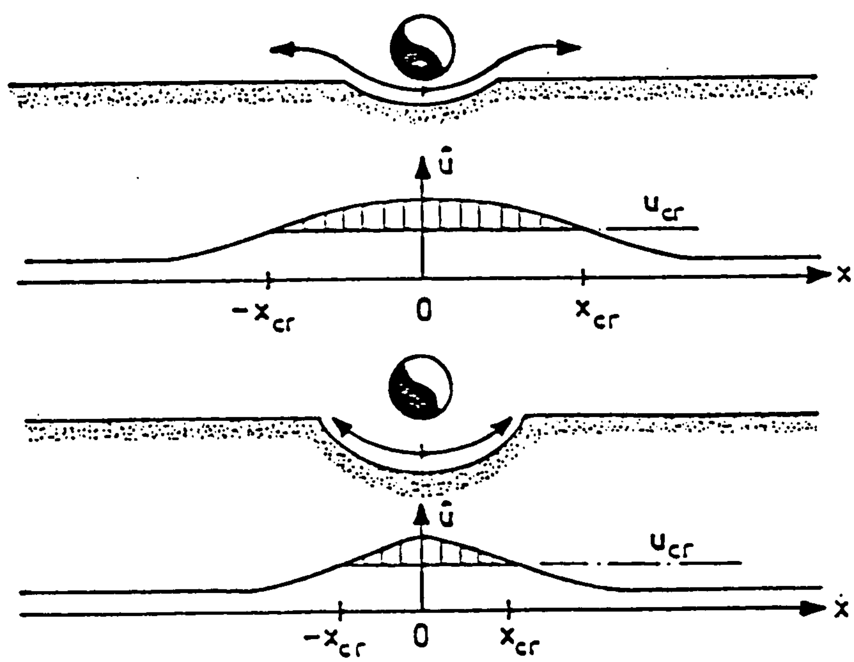


Figure 3.18 Scour Underneath Wave Exposed Pipelines (Bijker, 1983)

Scour Around Pipelines due to Waves and Currents Combined

Not much seems to be known about scour due to the combined effects of waves and currents as may be apparent from the following from Bijker (1983):

> "The most general conditions for full scale pipelines will include both current and wave action which each have their own angle of incidence relative to the pipeline direction. Experiments have been carried out at the Delft University of Technology which have shown a decreasing scour depth when waves were superimposed on a given constant current (van Ast, de Boer, 1973). Superiposition of waves on an existing constant current with associated sediment transport increases the time average resulting transport. A part of this sediment will settle in the scour hole and thus be responsible for the decrease of scour depth. According to these results, the scour depth under a combination of current and waves will lie between the upper limit valid for the current and a lower limit for the wave only (which is approximately 30% of the upper limit). Experiments in progress must show whether the limits stated above still hold when resulting transports are negligible."

It should be mentioned that no other reference corroborating this finding has been found in the literature and that if true, it is probably specific to pipelines. As notes above, for other structures such as piles the combination of waves and currents produces greater scour than that due to currents alone.

3.1.3.2 Scour at Pipeline Shore Crossing

Where pipelines enter shallow water and cross the sea/land interface, they pass through areas where hydrodynamic forces due to wave shoaling and breaking are generally far greater than in deeper waters. However, other than within, and close to, the breaker zone, the patterns of forces, flows and scouring are indeed much the same although stronger than those in deeper water.

Where waves actually shoal and break, and in particular, when beaches are involved, - as they usually are - the principal scour related phenomenon is due to the sometimes startling changes in the underwater beach profile that can occur. These profile changes are cyclic in character related to alternating periods of stormy and calmer water. Even though pipeline burial is mandatory under these conditions, substantial lengths of pipe may frequently be uncovered by storm wave action (see Section 3.2.2.3 on prototype experience).

The most general scour related phenomena affecting the security of pipelines at shore crossings is due to periodic changes in beach profiles. This topic has been examined in some detail by Herbich (1981) who reported on both two-dimensional and three-dimensional model tests on beach profile changes and the effects of model pipeline crossings under laboratory conditions.

A large volume of beach and nearshore bottom profile measurements have been collected in the USA and many of these were analyzed by DeWall and Christenson (1979). See Figures 3.19, and 3.20, and Table 3.4. They show that the maximum range of fluctuation of elevation of a beach profile is closely related to large storm wave heights and to the "exposure" of the site. They define exposure in this context as the range of directions from which significant wave attack is possible, and thus to the sector width at longer fetches. The maximum scour depth due to profile changes was found by them to be 4.6 metres in the worst case.

The underlying phenomena of beach profile variation is very well-known and described in all coastal engineering and many coastal science texts including the Shore Protection Manual (1984), Komar (1976) King (1972), Muir-Wood and Fleming (1981), and so forth. Under storm conditions, the face of a typical "summer" profile beach is cut away, usually becoming steepened. Often a breaker bar is formed from the eroded material some distance out from the eroded beach face and a deep trough forms between the bar and the face of the beach. This is the typical so called winter

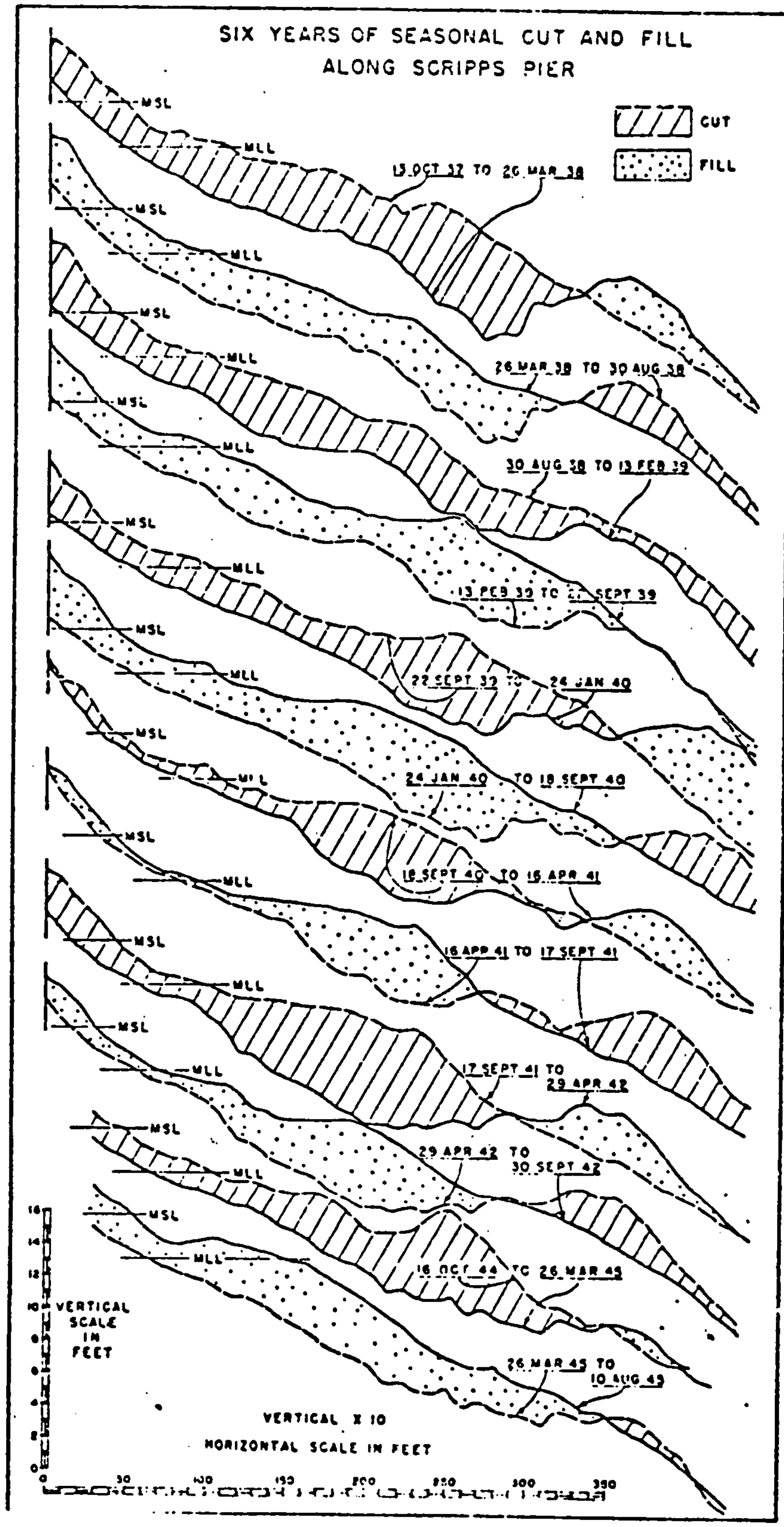


Figure 3.19 Seasonal Nearshore Profile Changes at Scripps Pier (DeWall and Christensen, 1979)

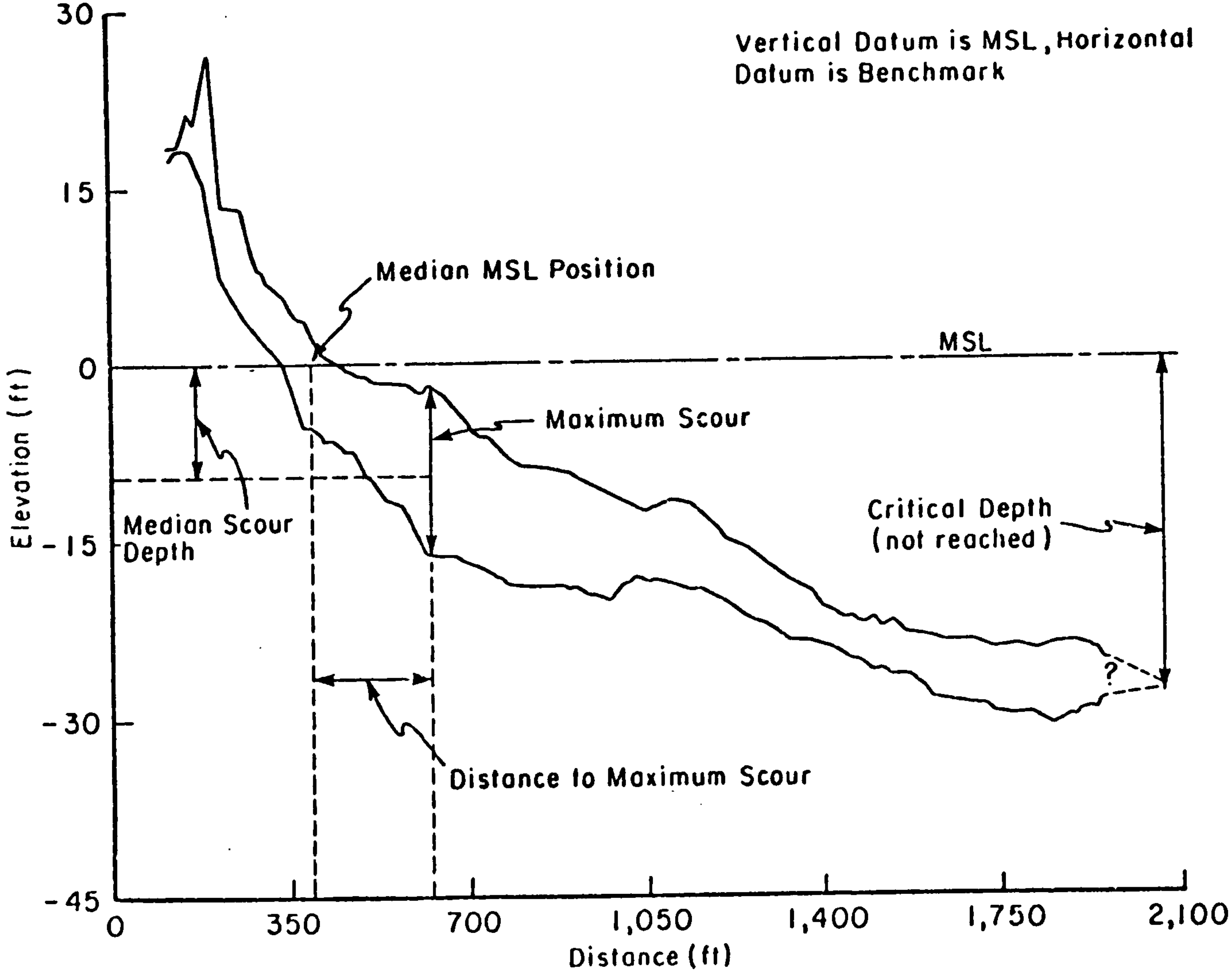


FIGURE 3.20 Envelope of 80 Weekly Profile Surveys from the Coastal Engineering Research Centre, July 1977 to January 1979 (DeWall and Christenson, 1979)

Table 3.4 Characteristics of Profiles and Profile Envelopes (DeWall and Christenson, 1979)

PROFILE SET	Foreshore Slope	Nearshore Slope	Berm Elevation (Ft)	Berm Distance (Ft)	Bar Height (Ft)	Bar Distance (Ft)	Maximum Scour (Ft)	Scour Distance (Ft)	Median Scour Depth (Ft)	Critical Depth (Ft)	Number Profiles	Months *
Virginia Beach, VA												
Timber Pier												
9/63-6/64	0.05	0.02	5.3	-76	1.3	354	5.1	370	-12.1	<-20	6	10
8/64-5/65	0.04	0.02	5.6	-66	1.2	200	7.1	70	-4.3	-15	6	9
7/65-6/66	0.04	0.02	7.1	-135	1.1	714	6.1	140	-5.1	-19	6	11
8/66-5/67	0.06	0.02	5.2	-82	1.8	200	6.4	130	-5.8	-19	6	10
7/72-3/73	0.05	0.02	5.8	-85	0.6	415	5.4	-75	+ 3.1	-19	6	8
8/73-5/74	0.05	0.02	6.7	-115	1.7	308	5.4	480	-13.6	<-16	6	8
Steel Pier												
9/63-6/64	0.10	0.03	7.2	-63	1.2	341	6.5	5	+ 1.5	<-16	6	10
8/64-5/65	0.04	0.01	9.8	-148	1.7	380	7.7	385	-10.7	<-15	6	9
7/65-6/66	0.05	0.02	10.0	-145	1.3	445	5.3	240	-6.7	-12	6	11
8/66-5/67	0.05	0.02	9.1	-102	1.5	393	6.0	-8	+ 0.1	<-12	6	10
7/72-3/73	0.09	0.01	8.5	-44	1.5	300	7.7	-47	+ 1.1	<-12	6	8
8/73-5/74	0.05	0.01	8.7	-143	1.6	220	6.5	-47	+ 2.4	<-10	6	8
Sea Sled (R18)	0.09	0.02	6.4	-61	1.2	135	3.9	148	-5.1	-11	4	36
Bodie Island, NC												
CERC Pier (N)	0.05	0.01	8.2	-86	5.2	245	12.9	294	-10.1	<-28	58	17
CERC Pier (S)	0.05	0.01	7.8	-68	7.8	765	14.7	238	-9.3	<-28	80	17
Kitty Hawk Pier	0.07	0.02	-	-	4.9	425	10.6	300	-10.7	<-20	109	38
Avalon Pier	0.08	0.01	-	-	5.2	425	12.2	405	-14.5	<-20	109	38
Nags Head Pier	0.07	0.01	-	-	6.9	705	10.2	303	-10.2	<-20	109	38
Jennette's Pier	0.07	0.01	-	-	4.9	790	11.8	243	-10.7	<-20	109	38
Outer Banks Pier	0.07	0.01	-	-	5.7	595	13.0	318	-12.7	<-20	109	38
Boca Raton, FL	0.20	0.02	8.5	-77	3.5	225	6.8	73	-6.3	<-12	232	12
Florida Panhandle												
St. Andrew Pier	0.05	0.03	4.0	-63	5.5	190	5.9	180	-4.8	<-12	8	8
Avondale Pier	0.09	0.03	5.4	-57	2.9	220	5.3	380	-9.3	-15	8	8
Crystal Pier	0.10	0.03	4.7	-38	2.5	226	4.0	190	-4.6	<-16	8	8
Beasley Pier	0.09	0.03	6.2	-39	3.7	139	5.4	80	-2.3	<-10	8	8
Navarre Pier	0.09	0.03	6.1	-39	2.5	239	6.5	5	+ 0.2	<-13	8	8
Corpus Christi, TX	0.03	0.01	4.8	-170	4.7	940	5.8	640	-7.6	-12	15	41
Mission Beach, CA	0.05	0.01	8.7	-140	0.8	280	4.1	480	-9.8	-23	8	11
Torrey Pines, CA												
North	0.05	0.02	8.8	-250	-	-	6.4	435	-8.4	-23	28	23
Indian Canyon	0.04	0.02	8.2	-180	-	-	6.0	-170	+ 5.6	-24	28	23
South	0.03	0.02	8.2	-165	-	-	5.2	-120	+ 4.8	-25	28	23
Scripps Pier	0.05	0.02	-	-	3.0	325	6.2	160	-7.5	<-21	14	94
E. Lake Michigan	0.10	0.01	-	-	6.8	585	7.7	602	-11.8	-23	6	25

*Number of months covered by survey data analyzed. Changes are listed for 24 months or less.

profile. Under subsequent more prolonged periods of moderate wave action, the bar moves inshore, the trough is largely filled in and the original less steep summer beach profile re-established.

Herbich (1981) relates the amplitude of beach profile variation to beach slope on the basis of model tests, without indicating how these results may be correlated with wave climate or beach material (Figures 3.21 and 3.22). He also shows examples of beach trough erosion in relation to buried pipes. These are similar to natural wave-breaker trough formations. They do not suggest that there is a very great influence on the natural beach process due to the presence of the pipe (Figure 3.23).

3.1.4 Submerged Bottom Structures

3.1.4.1 Small Structures

Scour around small submerged structures such as footings can cause undermining leading to tilting or even failure of supported structures. Due to the diverse nature of footing shapes a detailed theoretical description of the flow disruption is not possible. It suffices to say that the obstruction, though often not tall enough to create the primary vortex observed with piles, does create large scale turbulence in the form of vortices.

In field studies by DeWall (1981) on experimental footings and by Palmer (1970) on artificial and natural objects on the seafloor, it was found that the shape of the obstruction had little effect on the final extent of the scour pit. Also, the shape of the scour pit was not directly related to the nature of the flow. Oscillatory flow often produced asymetrical scour holes instead of the familiar concentric pit. Often test footings were quickly buried, not through local deposition but by excavation due to scour processes.

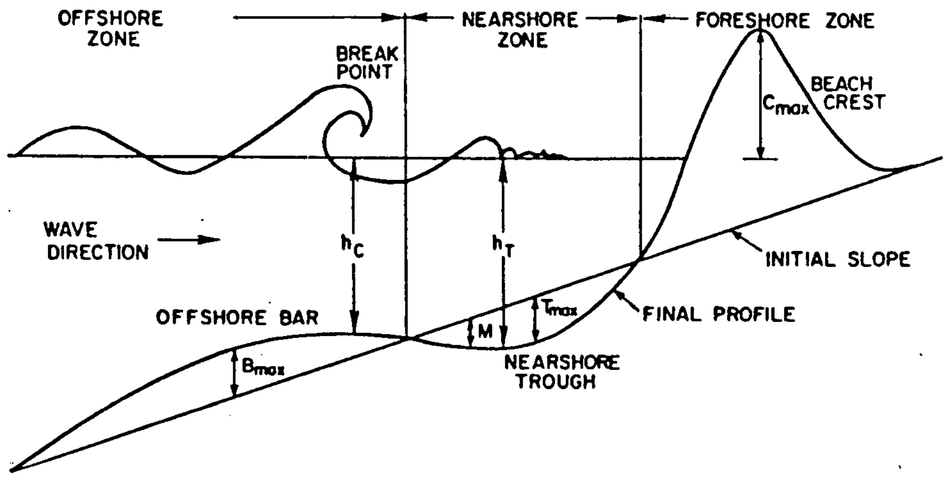


Figure 3.21

Definition sketch for T_{max}, B_{max}, h_C, and h_T; B = maximum height of the offshore bar; T_{max} = maximum depth at the nearshore trough; h_T = vertical distance from water level to the nearshore trough; h_C = vertical distance from water level to the offshore bar. (From Herbich, 1970.)

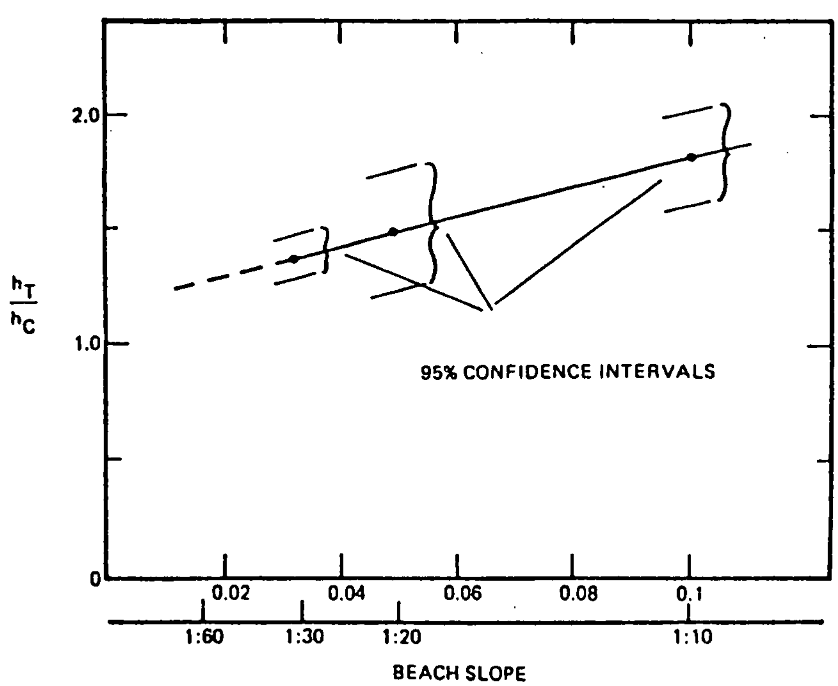
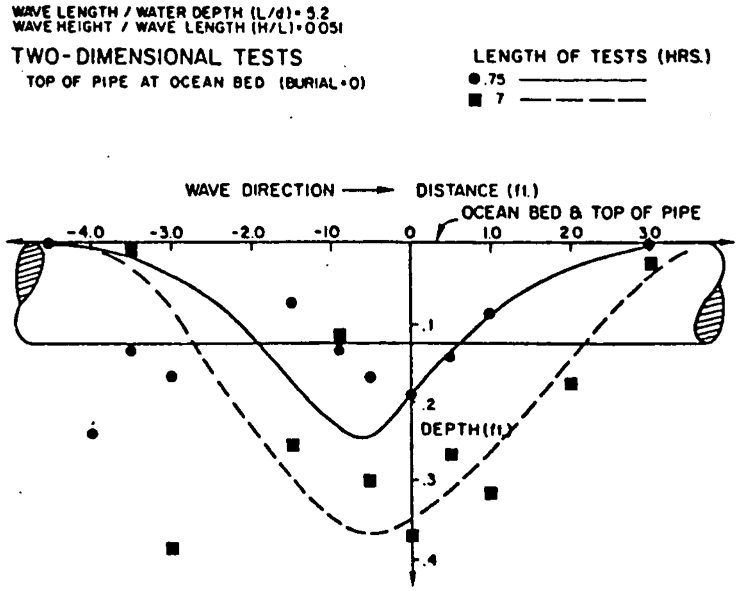


Figure 3.22 Ratio of h_T/H_C as a Function of Beach Slope (Herbich, 1970)



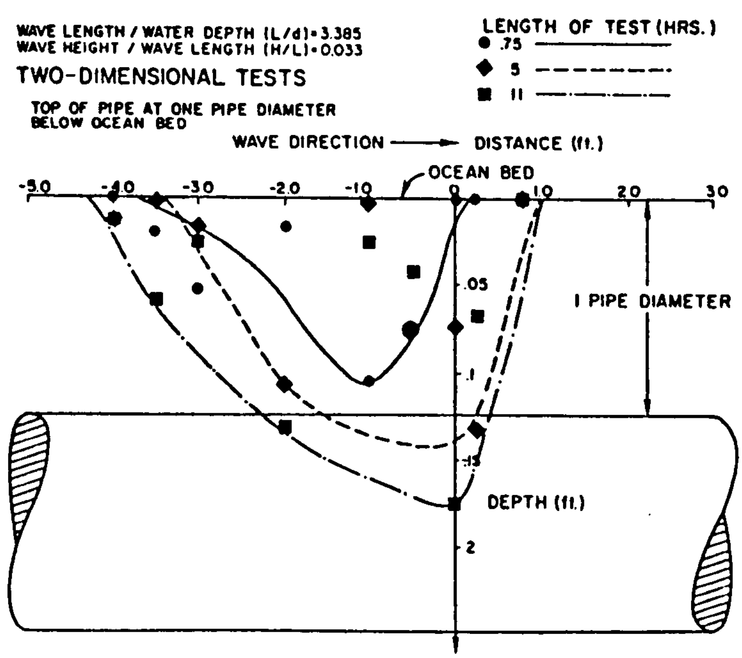


Figure 3.23 Sample Result of Scour; Two-Dimensional Tests (Herbich, 1981)

3.1.4.2 Large Structures

The scour around large submerged structures may resemble the scour mechanism around piles where a primary vortex develops upstream of the obstruction providing the structure is sufficiently tall. However, if the object diameter is large compared to the orbital excursion of water particles for oscillatory flow, there may be insufficient time for full development of primary vortex and wake plume turbulence.

The Danish Hydraulic Institute (1983) has performed a model test on a rectangular valve chamber with a relatively large equivalent diameter compared to the orbital excursion of water particles during oscillatory flow. The shape (specifically the upstream inclined face) and limited height of the chamber precluded the development of a primary vortex (See Figure 3.24). The leeward inclined face limits the scour potential of a horizontal lee eddy created by flow over the top of the structure. For a unidirectional current scour holes developed around the upstream corners and around the downstream corners extending some distance downstream. The scour pattern differs for oscillatory flow since the vortex pattern created around the structure changes with each change in flow direction and the primary vortex is not formed. Therefore, for such structures with relatively large effective diameters, oscillatory flow will create a less critical scour condition than unidirectional flow at an equivalent shear stress. From their model tests, DHI (1983) also concluded that the depth of scour holes for unidirectional flow depends on current strength, whereas the extent of the erosion depends on the shape of the object.

Scouring about few prototype bottom structures have been reported in the literature. Tesaker (1980) attempted to use wrecks on the ocean floor to evaluate long-term scour. However, there was no apparent relationship between either the width and shape of wrecks and the depth of scour. This does not contradict the findings of DHI where scour depth was found to be dependent on flow velocity and not the shape of the object.

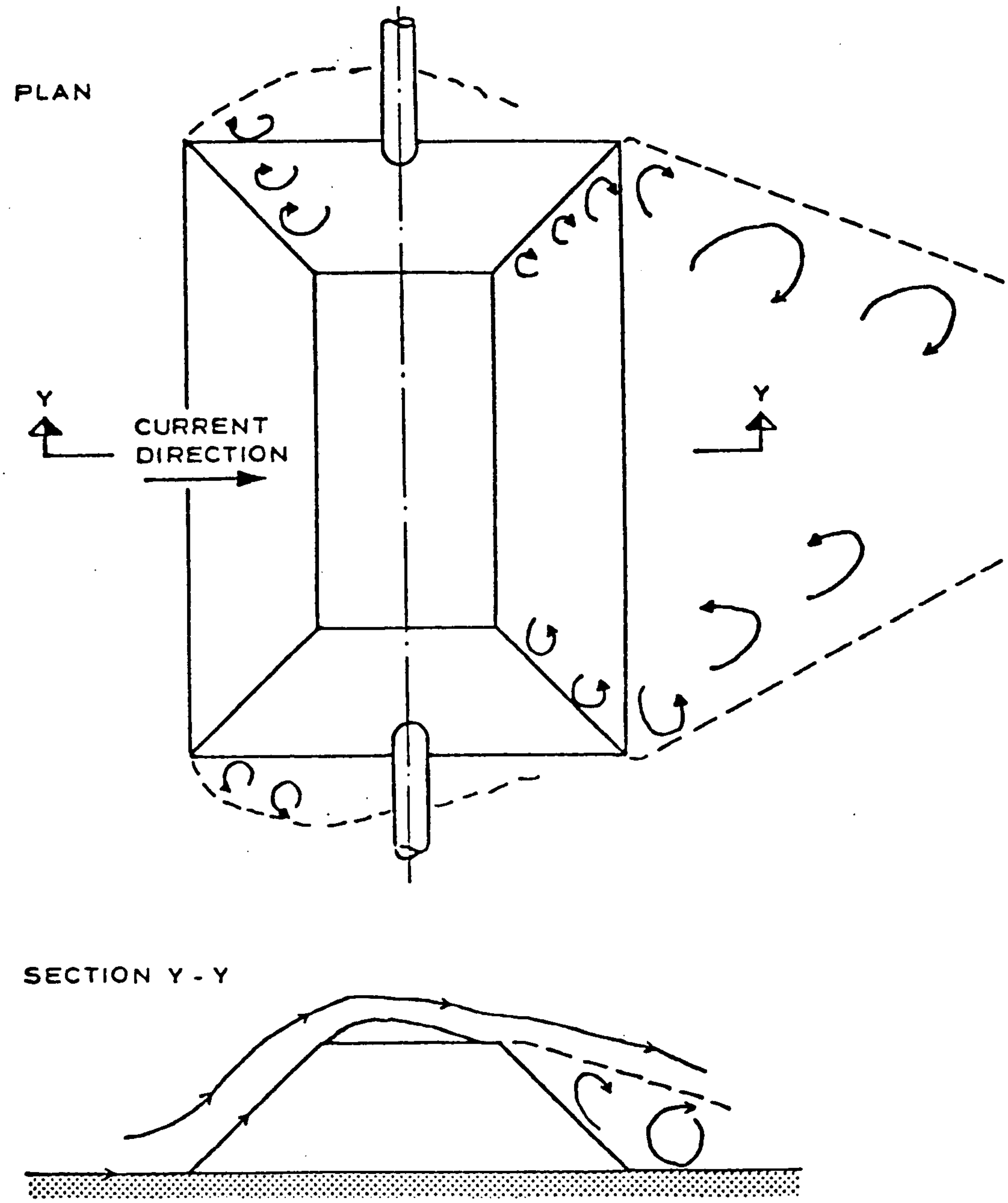


Figure 3.24 Changes in Current Pattern due to Presence of a Valve Chamber (DHI, 1983)

3.1.5 Large Gravity Structures

The most familiar type of structure in this category is the typical concrete gravity base oil production platform used in the North Sea. Few scour problems have been encountered in the North Sea, and Hoeg (1983) suggests that of all the platforms installed only one developed significant scour, the platform in that case having a square base. However, when large gravity structures are located in shallow water scour problems are of greater concern.

The North Sea platforms can be characterized as deep water surface penetrating structures (most are in depths greater than 75 m). At these depths oscillatory motions due to waves have little effect on scour. However, Toerum et al (1974) suggest that scour has been observed to occur at depths of 100 m and it must therefore be a result of accelerated unidirectional currents. Potential flow theory indicates that ambient velocity will be increased by a factor of two around circular structures and by a factor of three around hexagonal structures (Rance, 1980). Corners on the structures also induce turbulence. There tends to be an interaction between local and global scour. The equilibrium position of the faster generating local scour is continually being altered by the slow global scour. (See Figure 3.25). If scour causes undermining of the platform (where foundation skirts are not included in the design) rocking may result which will accelerate the scour process through the addition of erosion associated with the resulting "pumping action", and the related excess pore water pressures, leading eventually to overall instability.

For large gravity structures in shallower water, the main mechanism of scour can be associated with the complex effects of wave diffration around and reflections from the structure (Halcrow, 1981; Rance, 1978; Apelt and MacKnight, 1976). These alterations to the local wave pattern will only occur if the equivalent diameter of the structure is larger compared with the wave length. Diffraction and reflection cause

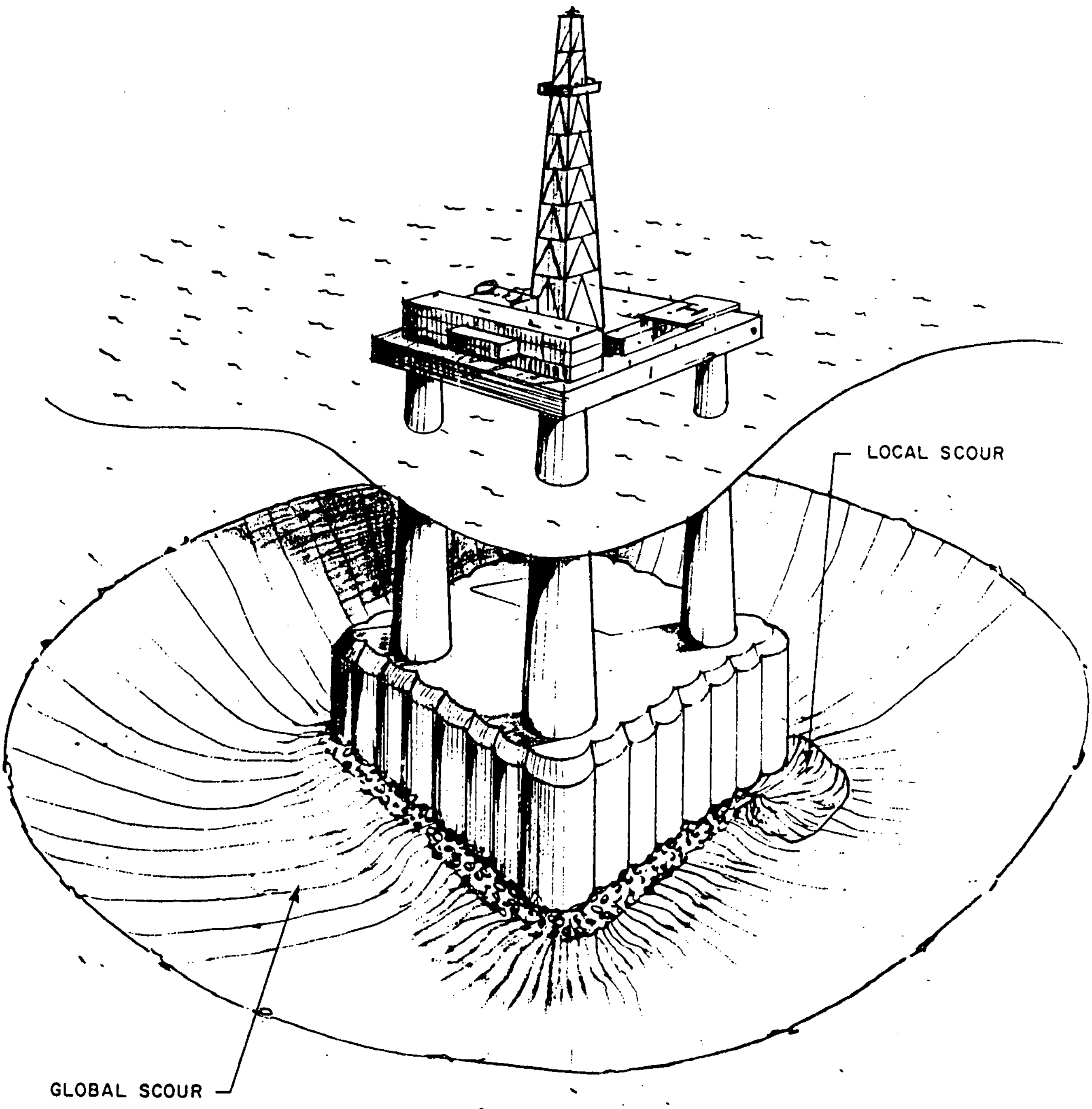


FIGURE 3.25 Idealized Local and Global Scour - Gravity Structure

increased wave heights and greater orbital velocities. Along with local and global scour the alteration of the wave pattern associated with shallow water structures may cause large detached depressions typically half a principle wave length from the structure (See Figure 3.26).

Square and hexagonal structures show the greatest erosion with large scour holes occurring around upstream corners, scour around circular structures is not nearly as pronounced. If currents are imposed on the wave action, the scour pattern is similar but scour holes are much deeper. Rance (1983) has reported on model tests performed at the Hydraulic Research Station on square, hexagonal, and circular structures in shallow water with waves and currents. Figures 3.27, 3.28, and 3.29 show typical scour patterns associated with these large gravity structures.

3.1.6 Sacrificial Islands

Sacrificial beach islands consisting in most cases of sand only have been used in the Beaufort Sea for a number of years. The Beaufort Sea is frozen 75% of the year and for the balance of the year open water fetches are limited by the distance of the margin to the permanent ice sheet. During the ice season, the artificial islands are initially subject to ice scour until they become surrounded by shorefast ice. The islands have been found to be most vulnerable during open water season. Until recently, the only material available to build the islands was fine sand dredged from the sea floor. Rock protection is often unavailable and other methods such as sand bags have only shown limited effectiveness. Consequently, scour of the islands progresses very rapidly and is of serious concern.

Extensive model studies have been performed with sacrificial beach islands constructed of sand only and exposed to regular waves, and a description of the morphological development is provided by Kamphuis and Nairn (1984). Such islands are exposed to breaking waves which are

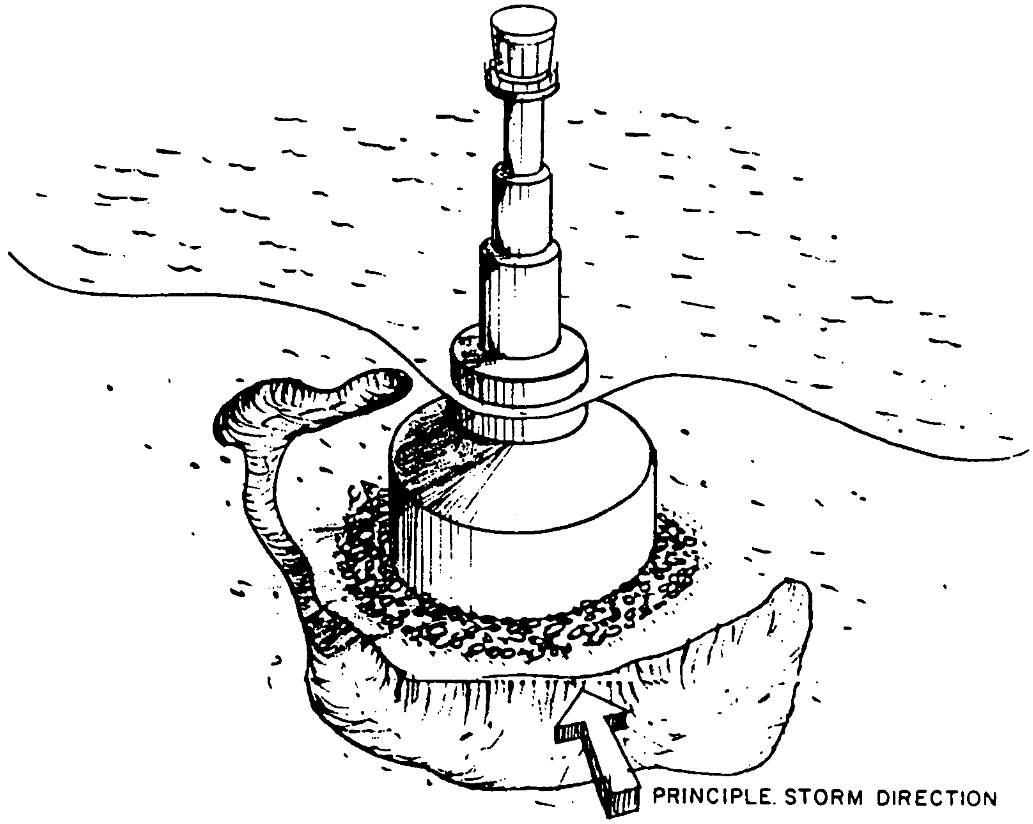
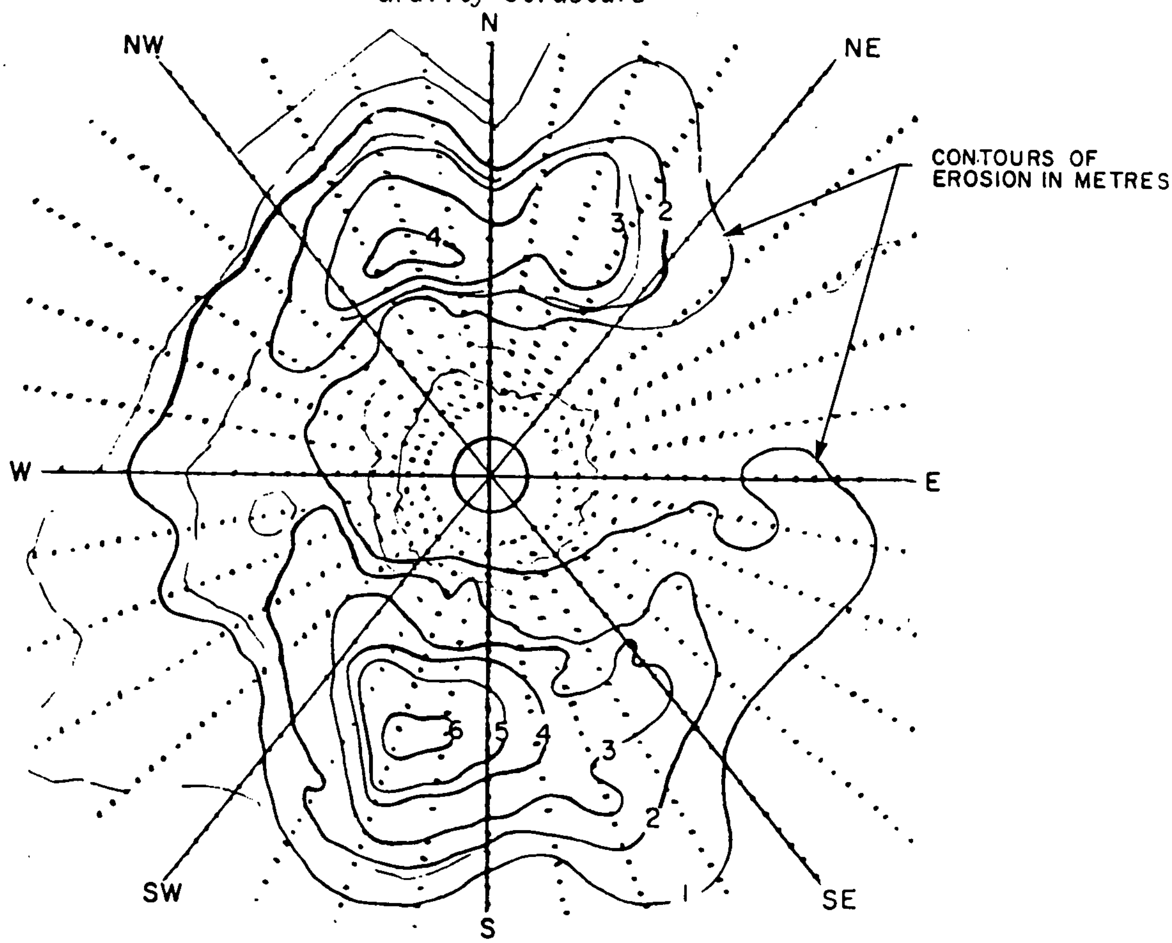


FIGURE 3.26 Idealized Reflected Wave Scour - Gravity Structure



Plot Prototype Soundings of Scour

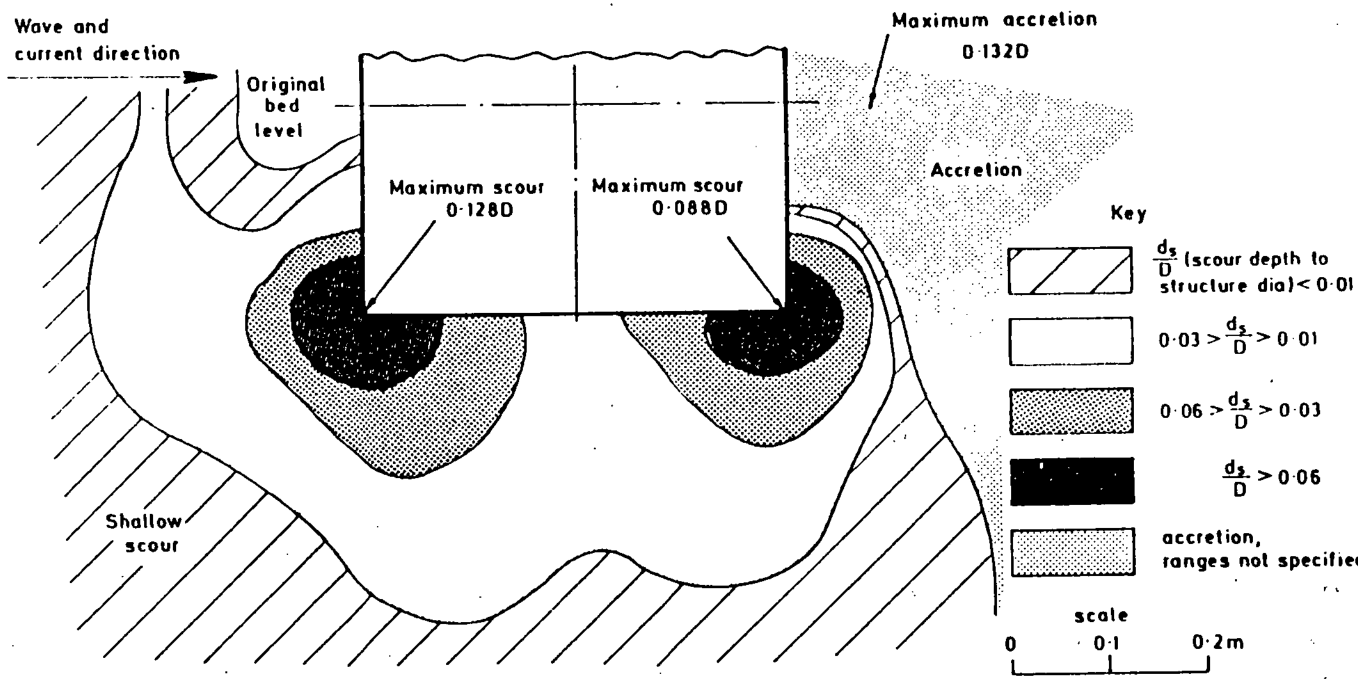


Figure 3.27 Physical Model of a Square Gravity Structure with Leading Face. Bed Topography Under Wave and Current Action (Rance, 1980)

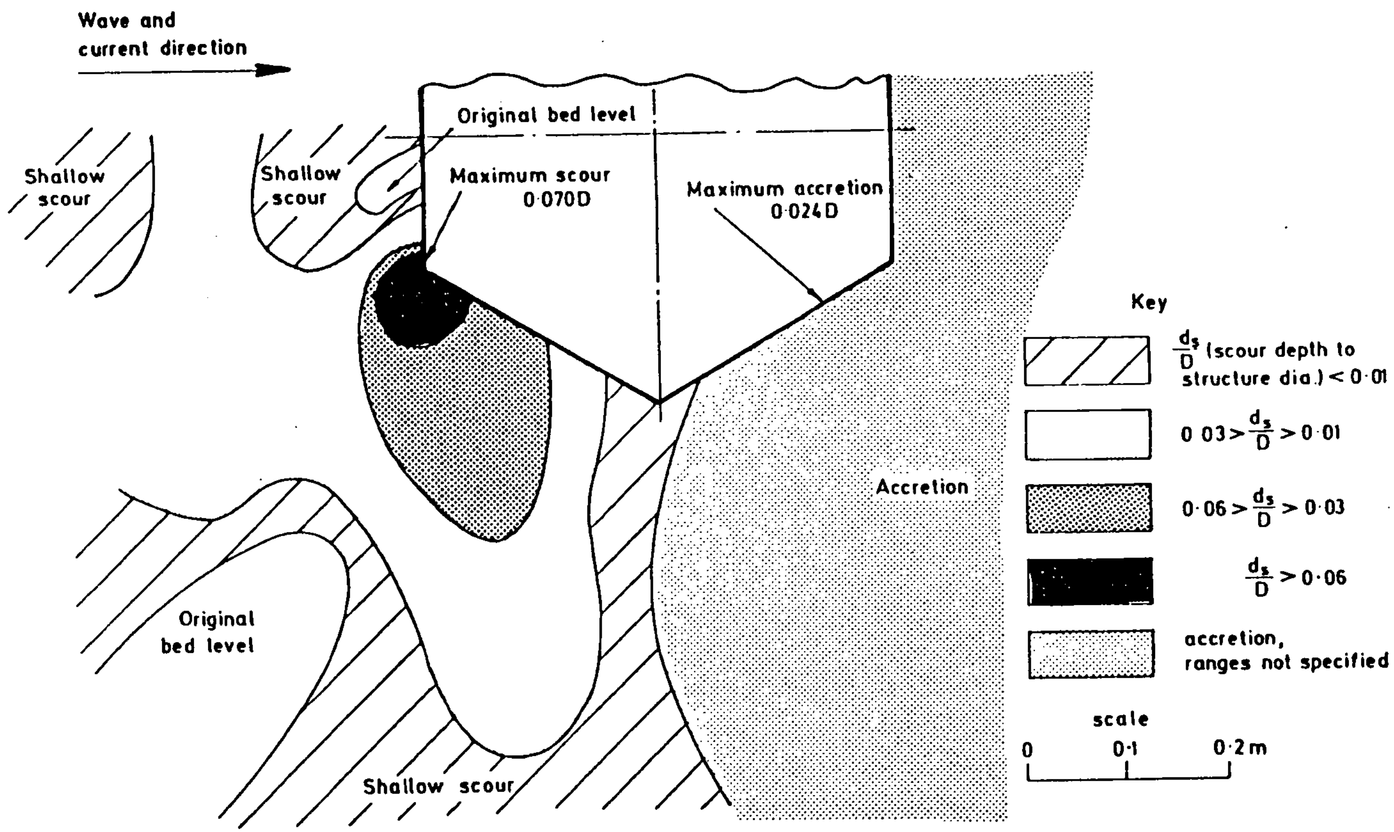


Figure 3.28 Physical Model of a Hexagonal Gravity Structure with Leading Face. Bed Topography Under Wave and Current Action (Rance, 1980)

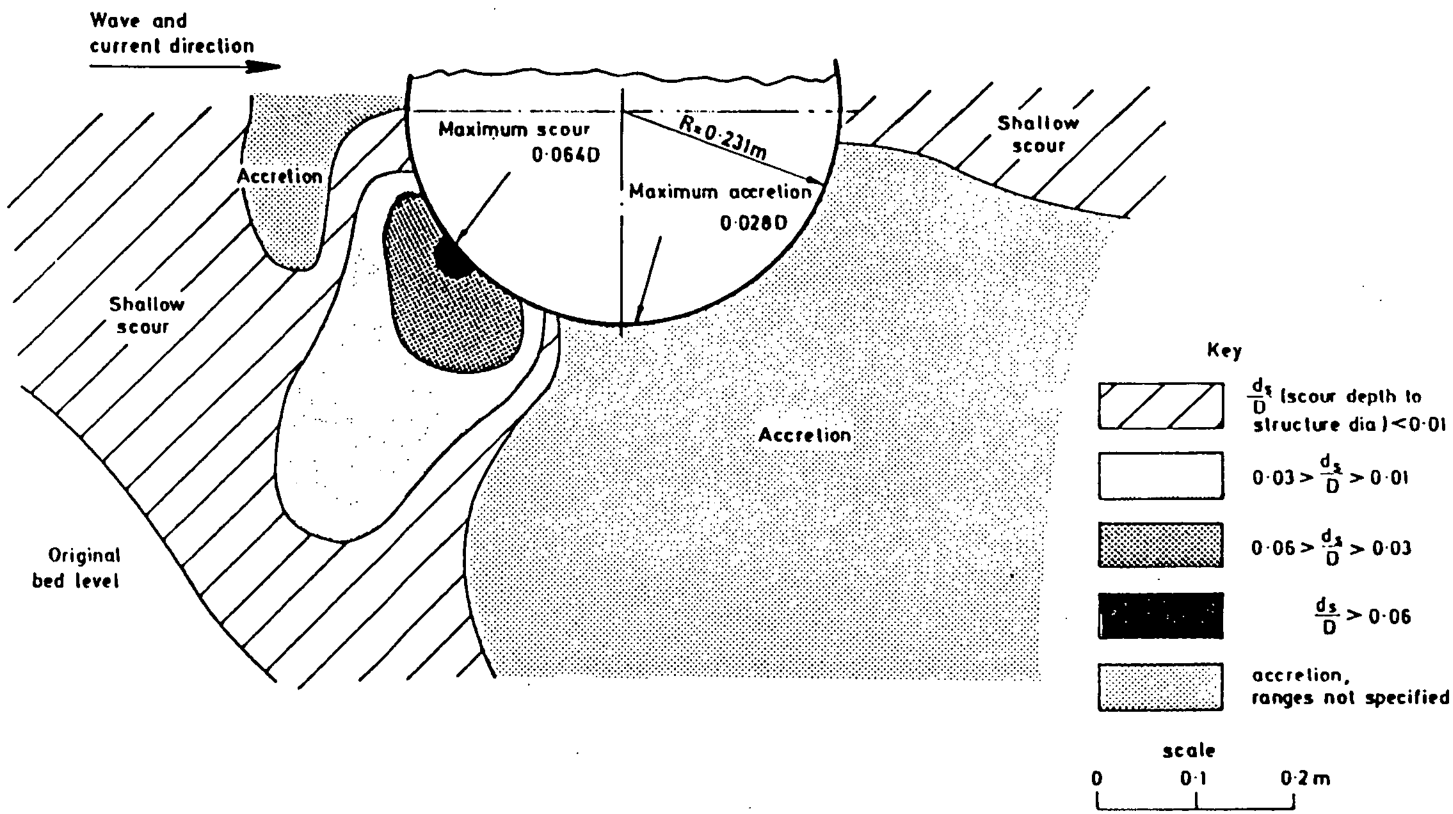


FIGURE 3.29 Physical Model of a Large Circular Gravity Structure.
Bed Topography Under Wave and Current Action (Rance, 1980)

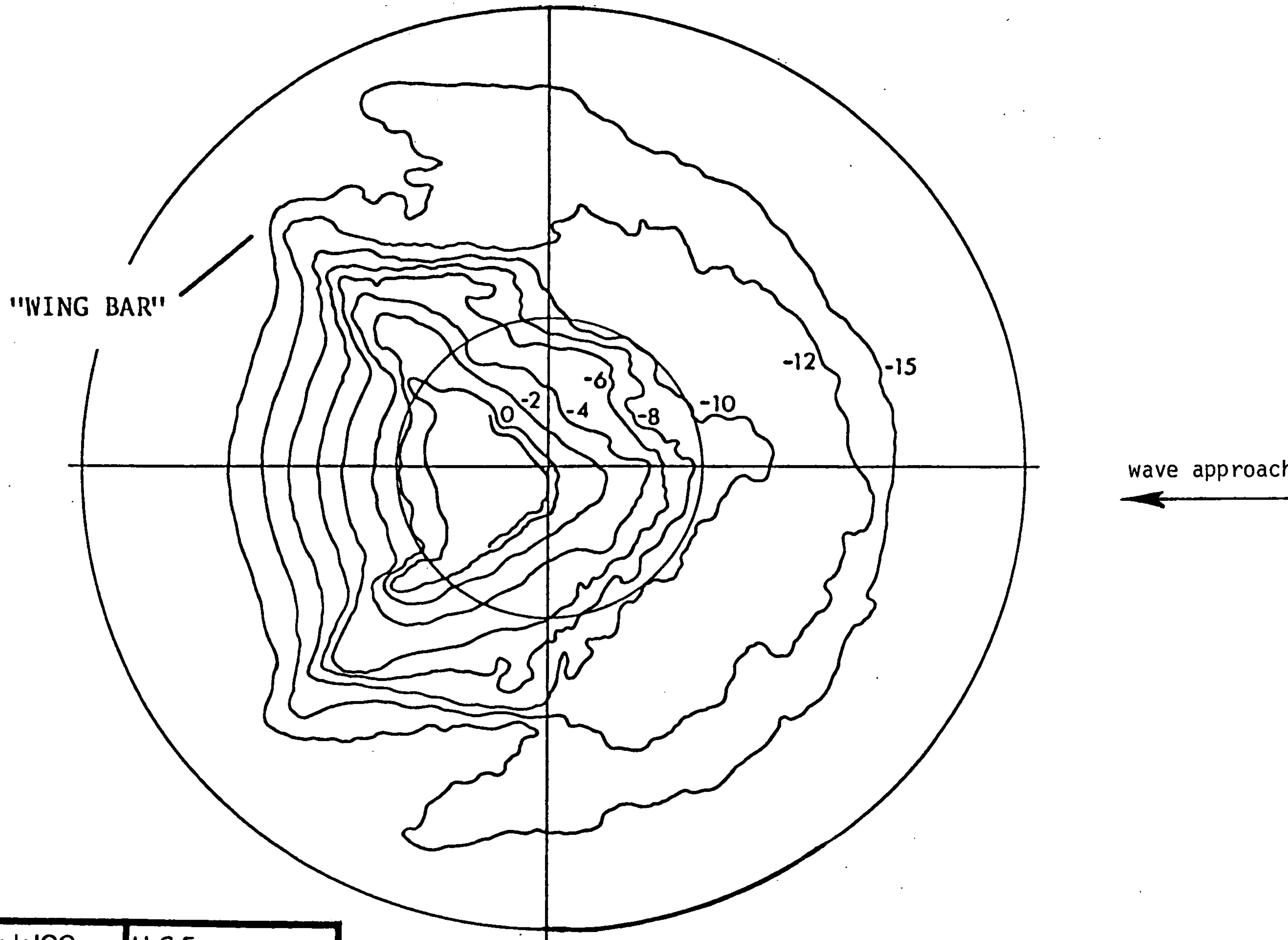
often of plunging form because of the steep slopes which are utilized in their construction to minimize the required volume of dredging to construct them. Strong longshore currents develop alongside these islands forming longshore trenches. The current carries suspended sediment from the zone of plunging breakers and deposits it to form symmetrical "wing bars" extending out from either side of the back (the sheltered lee side) of the island. (See Figure 3.30). Scale effects due to the inability to scale down sand grain size were found to have little effect on the resulting shape of the eroding island.

In the same model series, Kamphuis and Nairn found that when irregular waves were used, the morphological features of the eroded island though similar in character were far less pronounced and that erosion or scour rates were reduced by a factor of 3 to 4. However, the limited depth of erosion below which the initial profile remains unchanged, remained virtually the same with irregular waves. This depth was seen as analogous to the ultimate scour depth for structures placed directly on the sea floor. From this finding, it was therefore postulated that scour around structures predicted from regular wave model studies is reasonably accurate when only ultimate depth is considered. However, scour patterns are smoothed out and the rate of scour is much less with irregular waves than with swells.

While ultimate scour depths are often of greatest concern for structures on the sea floor, the rate of scour or erosion is most important for sacrificial islands because it defines the useful life span of the island. Since erosion is a manifestation of littoral transport for the islands, the rate of erosion is strongly dependent on wave height, wave period, and grain diameter.

3.1.7 Caisson Retained Islands

Caisson retained islands are a special case of the sacrificial sand island. The concept was developed from a need to construct islands in



SCALE: 1:100	H: 6.5 M.
TIME: 40 HRS.	T: 8 SECS.
D_{50} : 0.105 MM	

FIGURE 3.30

Artificial Island Contours (in m)
(after Kamphuis and Nairn, 1984)

deeper water, where volumes of fill material became large enough to be inhibitive on the grounds of cost (usually for depths greater than 20-25 m). The caisson retained island usually consists of a caisson of octagonal planform placed on top of an underwater mound. The toe of the caisson is protected either by sacrificial gravel berms or by rock (Myers et al, 1983) to prevent undermining by scour.

The caisson retained island could also be classified as a shallow water surface penetrating gravity structure somewhat comparable to those discussed in Section 3.1.5. The structure is characterized as an obstruction which is large compared to the wave length of incident waves. In shallow water, the primary mechanism of erosion is due to the increased orbital velocities resulting from wave reflection, wave diffraction and even wave breaking. Scour holes are most pronounced at the corners of the structure where turbulence in the form of eddy shedding combines with accelerated currents. Fleming et al. (1983) developed a numerical model and calibrated it with physical model results to predict erosion around caisson retained islands. Figure 3.31 shows an eroded island predicted by the numerical model. Figure 3.32 is a conceptual sketch of the scour pattern around a caisson retained island.

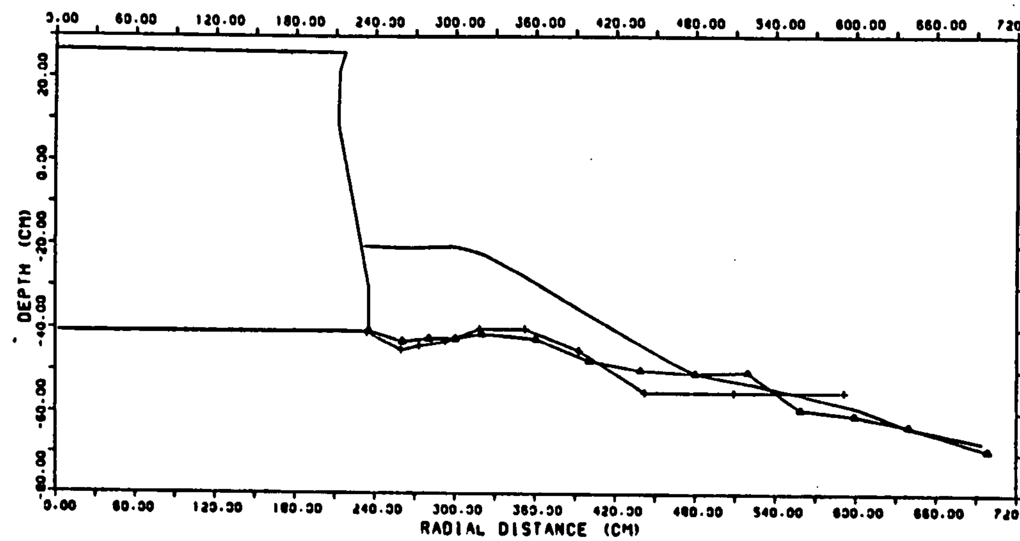
3.2 PROTOTYPE EXPERIENCE BY TYPE OF STRUCTURE

This section of the report presents information on prototype experience obtained from the literature search, interviews and the questionnaire survey. The discussion is subdivided according to type of structure. Although some structures are more appropriate than others for application to the Scotian Shelf, it was considered that this section should be kept sufficiently general to include a full range of possible alternatives. Reference is made to fuller descriptions given in case histories prepared as part of this study, listed in the Appendix, and archived by COGLA.

COMPARISON OF PHYSICAL AND MATH MODEL RESULTS

20 YEAR STORM

ANGLE 0.0



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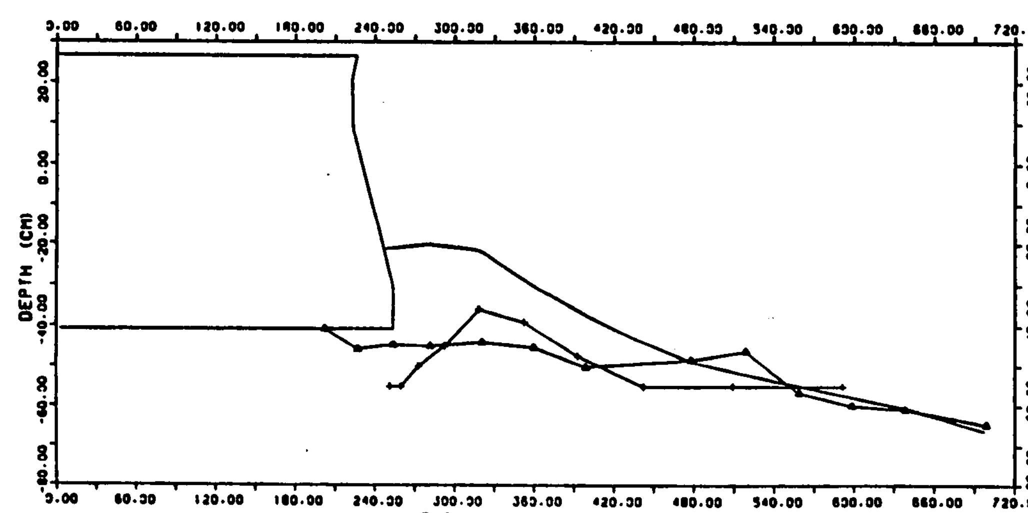
- INITIAL BED
▲ PHYSICAL FINAL
+ MATH FINAL

COMPARISON OF PHYSICAL AND MATH MODEL RESULTS

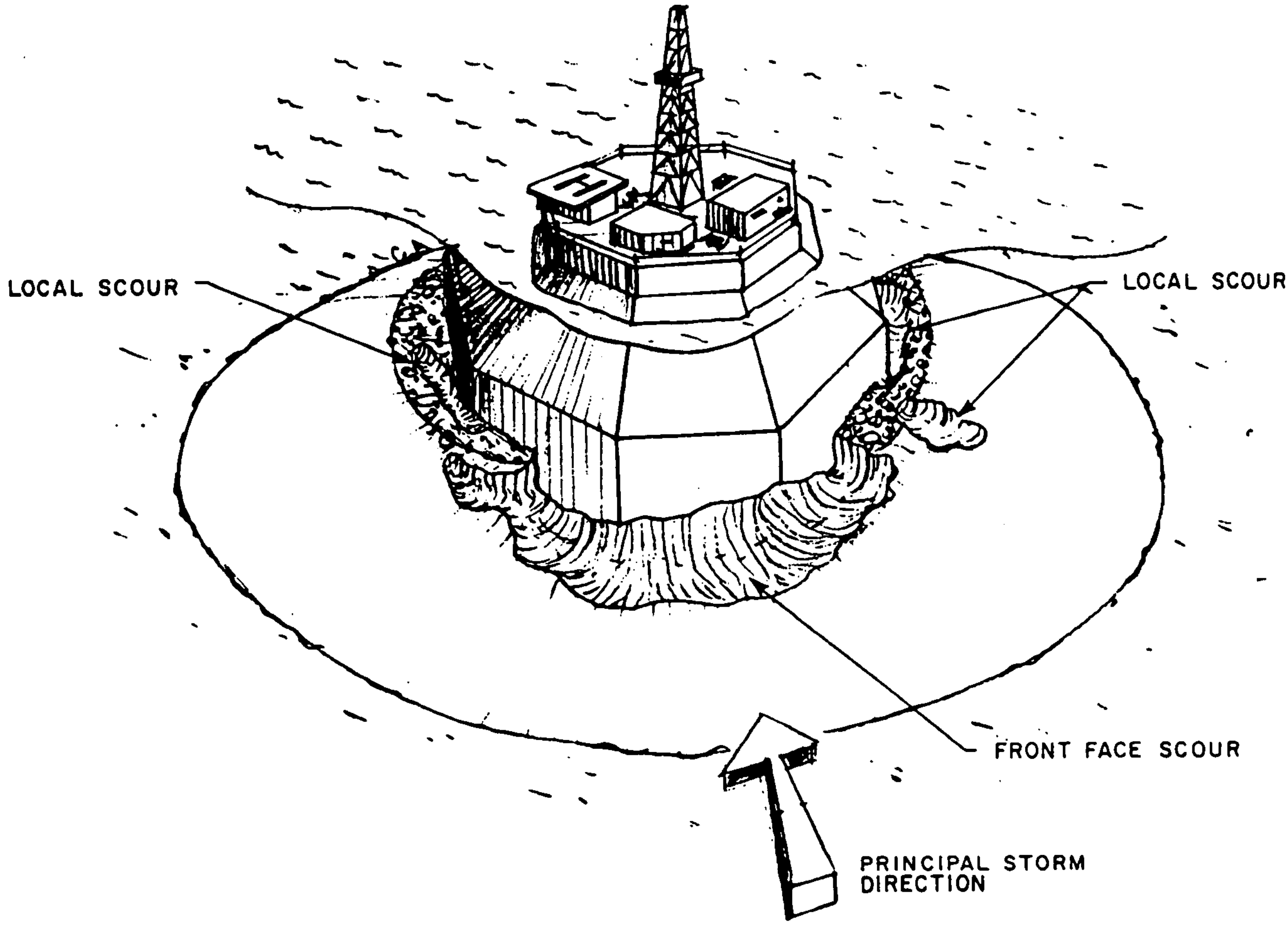
20 YEAR STORM

ANGLE 22.5

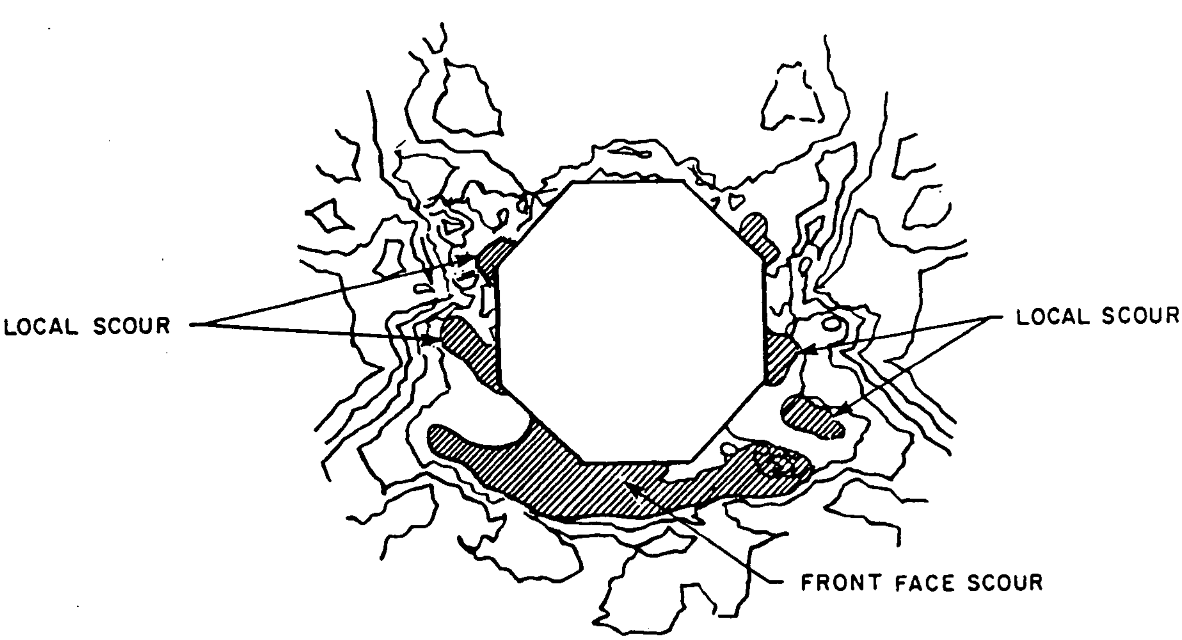


LEGEND: - CAISSON
- INITIAL BED
▲ PHYSICAL FINAL
+ MATH FINAL

FIGURE 3.31 Comparison of Erosion Predicted with Physical and Math Models (Fleming et al., 1983)



Idealized Scour - Sacrificial Berm
Caisson Island



Numerical Model
Scour Predictions

FIGURE 3.32

3.2.1 Multi-Member Structures

3.2.1.1 Pile Supported Structures

A considerable portion of the literature reviewed pertained to scour around pile supported structures. Some of this is associated with research activities, particularly laboratory studies, while a number of papers describe the use of various techniques for scour prevention. Often these techniques have application to other types of structures as well. Experience with such techniques are described for Gulf of Mexico and North Sea production platforms.

A good description of scour prevention and repair techniques tested by Amoco in the North Sea is presented by Angus and Moore (1982) which is an update of information on similar techniques by Watson (1973). The platforms described in these papers are piled structures with 1 m diameter legs driven into the seabed to depths ranging from 15 to 90 m depending on the soil conditions. Water depths range between 20 and 40 m. Extreme design wave heights are estimated to be in the region of 16 m with maximum bottom current velocities of 1.0 m/s. It is reported by the authors that Amoco's structures are designed to withstand between 1.5 and 3.0 m of scour.

Angus and Moore also discuss a number of different types of scour repair and/or prevention techniques that have been used. It must be deduced from this that scour depths did become unacceptable and therefore came close to or exceeded the above limits. Seven different techniques, covering items such as sandbags, nylon nets, plastic collars, tires, artificial seaweed and gravel pads, are described. It appears that the only technique to provide long-term benefit has been the gravel pad although the automobile tires were also effective up to the time of writing. All other methods required periodic maintenance and repair and were generally considered to be inadequate for long-term protection. This experience has been summarized in the archived case histories. (A4.1.1).

The use of a gravel pad was also recommended by Van Dijk (1981) who describes Shell's experience with scour problems in the southern North Sea Leman and Indefatigable fields where scour of up to 3.5 m was experienced in 1978 and 1979. Shell's present practice is to place a protective layer around its platforms before scour starts.

Other experiences with scour around Gulf of Mexico platforms are described in papers by Posey (1970) and Sybert (undated). The latter author describes the scour problems and remedial measures associated with five platforms in water depths ranging from 11.6 to 13.7 m. Current measurements were not available but diver estimates of current velocities were 1.5 to 2.5 m/s. The upper soil layers were unconsolidated and consisted of fine sand with a few shells and clay stringers. Dish-shaped depressions were formed under each platform, some of these being 3.6 m deep and extending as far as 18 m from the platform. On the basis of laboratory tests, an inverted filter was installed at each platform in three layers, each approximately 1.2 m thick with progressively coarser material from bottom to top. Approximately 5,000 m3 of material were used for each platform and at the time of writing, which was four years after the installation, no significant changes had occurred in the mats. This experience has been summarized as a case history. (A4.1.5)

The following information has been provided by respondents to the questionnaire survey.

Norpipe AS recorded 3 m of scour around the platform jacket legs and inside the jacket of the B11 compression platform in the southeast North Sea. Water depth at the location was 33 m. The 100 year return period wave height is assessed to be 22 m with a period of 14.4 seconds. The seabed current is given as 1 m/s. The bottom material has a median grain size of .2 mm. No information on remedial measures was provided and it is understood that the problem has yet to be resolved. This experience is also summarized in the case histories. (4.1.2)

Elf Aquitaine A/S recorded minor scour within design limits for their QP/DP2 piled jacket drilling platforms in the Frigg Field North Sea. These platforms lie in 100 m of water. The 100 year return period wave height is assessed to be 29 m with a period of 16 seconds. The seabed current is given as 0.3 m/s. The bottom material is a fine to medium sand with a median grain size of 0.1 mm. General scour of 2 m was allowed for in the original designs and regular diving and sounding monitoring have been undertaken since the installation of the platforms in 1975/76. See case history for further details. (4.1.1)

3.2.1.2 Jack-Ups

Three papers dealing with scour problems around jack-up platforms were reviewed of which two described experiences on the Scotian Shelf with the Orion Gulftide, a Mobil Oil operated drilling rig in 1977 (Song et al. 1979; Murphy and Yan, 1983). Scour data on these and other jack-up installations on the Scotian Shelf was compiled for Mobil by Geomarine Ltd. (1983). Questionnaires were also reviewed relating to the deployment of the Orion Gulftide and from a combination of the above sources, a detailed case history was prepared including summaries of diving inspection reports from Geomarine (1983). Water depths at the planned drill sites ranged from 13 to 35 m. Significant wave heights with a 25 year return period exceeding 13 m were expected and near bottom tidal currents of 0.8 m/s were known to exist. Three storms during the summer and fall of 1977 resulted in scour around the spud cans. After each occurrence, remedial measures were taken including the placement of sand bags, fibre mats, chain link fence and a grout pad. However, none of these measures proved successful and stability of the rig was eventually maintained by jacking down the legs periodically. By late January 1978, maximum scour depth had exceeded 3 m. This experience has been summarized as a case history. (A4.1.6)

Following these events, an airlift system was developed which caused the legs of the jack-up to penetrate to a depth sufficient to provide stability even with the maximum expected scour during the drilling period. Leg penetration depths of greater than 6 m have been achieved with this system. Since the initial installation in the mid-seventies, several jack-ups have been equipped with the air-lift system. Model tests of other techniques for scour prevention around the spud cans of a jack-up platform have been carried out by the Danish Hydraulic Institute for North Sea applications (DHI 1980). The use of 2 m wide hinged plates under the action of wave and currents was investigated. It was concluded that, to provide protection under strong wave and current conditions, no area of the spud can could be exposed. Even then, inspection after storms was recommended.

3.2.1.3 Other Types

An interesting paper by Wilson and Abel (1973) describes the measures taken to prevent scour around the base of a drilling platform on the Scotian Shelf in 1972. The platform was the semi-submersible Sedco H which, in this case, was set on the bottom in 27 m of water. It was anticipated prior to the drilling program that scour could develop around the three platform pontoons which measured 24 m in diameter and physical model tests were therefore conducted by the Delft Hydraulic Laboratory to predict the extent of the scour. From these tests, it was found that scour depth could reach 6 m after a period of 8 months. Based on these tests, scour protection in the form of a nylon mesh was developed and these mats were installed on the pontoons on arrival of the semi-submersible at the site. The paper describes the installation procedure and the performance of the mats during the drilling period. In general, they were very effective in reducing scour although they were susceptible to movement during storms due to wave effects on the soil and movements of the rig itself from wave loading. Because of these movements, with subsequent repair costs, the long-term performance of the mats would probably be questionable. See case history. (A4.5.1)

3.2.2 Pipelines

3.2.2.1 Pipes

As with the various bottom founded structures described in the previous sections, numerous scour problems and scour protection methods have been described in the literature for application to pipelines where removal of the soil underneath portions of the pipe leads to spanning, high stressing and failure of the pipe. In most cases, protective methods are similar to those already discussed; gravel mattresses, fabrics and artificial seaweed.

Shell reported scour problems where pipelines crossed the shoreline with erosion depths of 2 to 3 m occurring during a single storm (Van Dijk 1981). In the case of pipelines, figures of 41 to 81% exposure during one year have been found together with seabed level changes of up to 5.5 m. Specific scour experiences are quoted as follows:

- 1975 Scour of 5.5 m depth, 43 m wide by 49 m long (in direction of current.

- 1977 Pipeline exposure to a depth of 2 m under the line for a considerable length.

A method of counteracting the effects of scour on pipelines is described for a Shell pipeline from a SBM (Single Buoy Mooring) off northern Wales in which the scoured areas were first infilled. The pipe was then covered with a mattress consisting of a latticework of reeds and brushwood bonded to a sheet of polypropylene. (Offshore Services, November 1976). The method, developed by ACZ of Holland, entailed using 70 such mattresses 70 m wide, which were then covered with 90,000 tonnes of armour stone dumped from a side-dumping barge. The installation rate averaged one mattress per day and the total project required 4 months for completion. According to Van Dijk (1981), Shell's present practice is to place a stone mattress on its pipelines before scour starts.

The following information has been provided by respondents to the questionnaire survey.

Elf Aquitaine Norge A/S recorded scouring of a 16" gas pipeline in the NEF Field, North Sea. The pipe was installed directly on the seabed in water depths averaging 100 m between the northeast Frigg Field and the Frigg Field, North Sea. The 100 year return period wave height is assessed to be 20 m with a 16 second period. The seabed current is given as 0.3 m/s. The bottom material is a fine sand with a D50 grain size of 0.1 mm. A year after installation, some free spans were discovered up to 30 m in length. The longer free spans were stablized by covering with riprap using the Seaway Sandpiper. Elf Aquitaine also recorded scour problems with 2 x 26" gas pipelines connecting the TP1 and CDP1 platforms in the Frigg Field. The environmental conditions were similar to those given above. The pipeline was initially stabilized and anchored by a number of large concrete blocks of up to 25 tonnes in weight. Two scour processes were noted. The concrete blocks initiated scour and free spans of up to 35 m in length were found with 50% of the total length being affected. The remedial measures adopted consisted of providing additional support by means of grout bags and covering with riprap with material having a D50 of 50 mm.

Norpipe AS recorded experience with the Ekofisk-Emden pipeline near the German coast in water depths between 8 to 10 m. The maximum wave height is assessed to be 8 m with a period of 10 seconds. Seabed currents are a maximum of 4 m/s and bottom material has a D50 grain size of 1 mm. The initital scour protection consisted of trenching and natural backfill. Inspections showed that the pipeline suffered exposure as a result of scour and remedial action by backfilling with crushed rock was undertaken. This experience is archived. (A4.2.2.)

Wescan Maritime Consultants recorded experience with a 0.9 m pipeline off Mispec Point, Nova Scotia. Water depths ranged from 0 to 35 m. The significant wave height was assessed as 3 m with a period of 8 seconds.

Seabed currents vary between 1 and 3 m/s and the sea bottom material was silty mud and sand overlying rock which outcroped in places. The pipe was initially buried and covered with shot rock but over a period of 13 years scour caused exposure and serious spanning and some movement of the pipeline. Remedial action consisted of supporting the pipe by means of grout bags at approximately 15 m intervals to reduce the spans. No direct action was taken against the scour process. The spans chosen were determined to be small enough not to cause over-stressing of the pipe. This experience has been summarized in the case histories. (A4.2.3)

ICI Linear Composites Limited have provided details of full scale trials on a 762 mm diameter pipeline. Late in 1968, an array of seeded fronds was anchored close to a 46 m length of pipeline in 9 m of water. The array consisted of tufts of polypropylene fibrillated tapes spliced into a support system and the support system anchored to form a 64 x 46 m mesh as shown in the diagram. (See Figure 3.33). At the beginning of the operations in October, the pipeline was exposed to varying degrees of scour along all 46 m and offshore from the area where the fronds were to be placed. After installation, a build up of sand was first noticed after the first 5 lines of fronds were laid. In a couple of days, 200 mm of sand had accumulated. A further build up occurred whilst the laying continued. A preliminary survey shortly after installation of the erosion control system showed that a 1 m build up of sand had occurred over the whole of the fiber area and yet the pipe was completely exposed again 6 m offshore from the fiber.

The first full survey of the effect of the fiber array was carried out on January 26, 1968 and the overall picture is shown in the diagram. A 1.5 m bank of sand was produced over the pipe. Moreover this 1.5 m sand bank extended over the whole of the fiber array, tailing off over a centre distance of 3 m from the edge of the fiber rectangle. The pipe itself became exposed again 9 m from the offshore edge of the frond seeded area. A further survey carried out in April 1969 confirmed the results, hence the frond array had built up a 1.5 m sand bank over two months and maintained it for a further 3 months during winter weather.

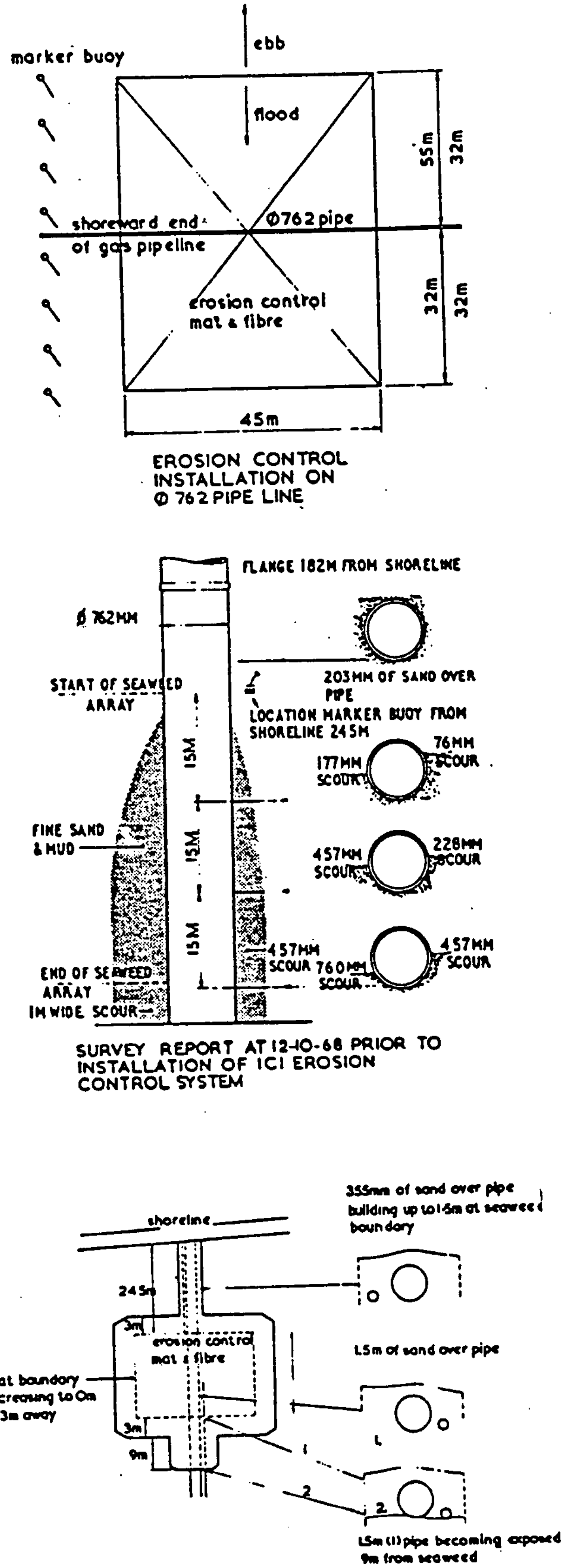


Figure 3.33 Example of an Erosion Control Fibre Mat by ICI Linear Composites Limited (Linear Composites Erosion Control Systems)

Details of scour experience around early North Sea pipelines are abstracted from a paper by Elk - 1975 included in the case history. Information is given on scour depths spanning and monitoring techniques but no relevant environmental data is provided. (A4.2.4)

A list of more than thirty contracts for pipeline scour protection, carried out mainly in the North Sea in the past seven years, was obtained from Westminster Seaway, an offshore contractor. Many of them were remedial work to counter actual scour problems. In each case, a special cover layer placement vessel, the Sandpiper, had been used.

3.2.2.2 Valve Chambers

The only article in the literature referring to scour protection of pipeline valve assemblies is based on model studies by the Danish Hydraulic Institute for the Danish North Sea gas pipeline (DHI, 1983). The model scale was 1:20 and two flow conditions were modelled; these were currents alone and currents and waves combined in the same direction. Tests were performed with and without scour protection. The scour protection consisted of textile mattresses with sewn-in and sand-filled channels for ballast (See Figure 3.34). Widths of 1 and 2 m were tested. It was found that the 1 m wide mattress allowed some scour to occur but the wider mattresses provided the required protection corresponding to about 5 years of wave and current loading in the North Sea. It was concluded that this technique would produce the necessary results.

3.2.2.3 Shoreline Interface

Considerations of scour prevention around pipelines approaching a shoreline are addressed in an experimental study by Herbich (1977). Shoreline change in the field has been examined by DeWall and Christenson (1979). Both these topics were examined in detail in Section 3.1.3.2. Van Dijk (1981) shows that 45% to 81% of the first

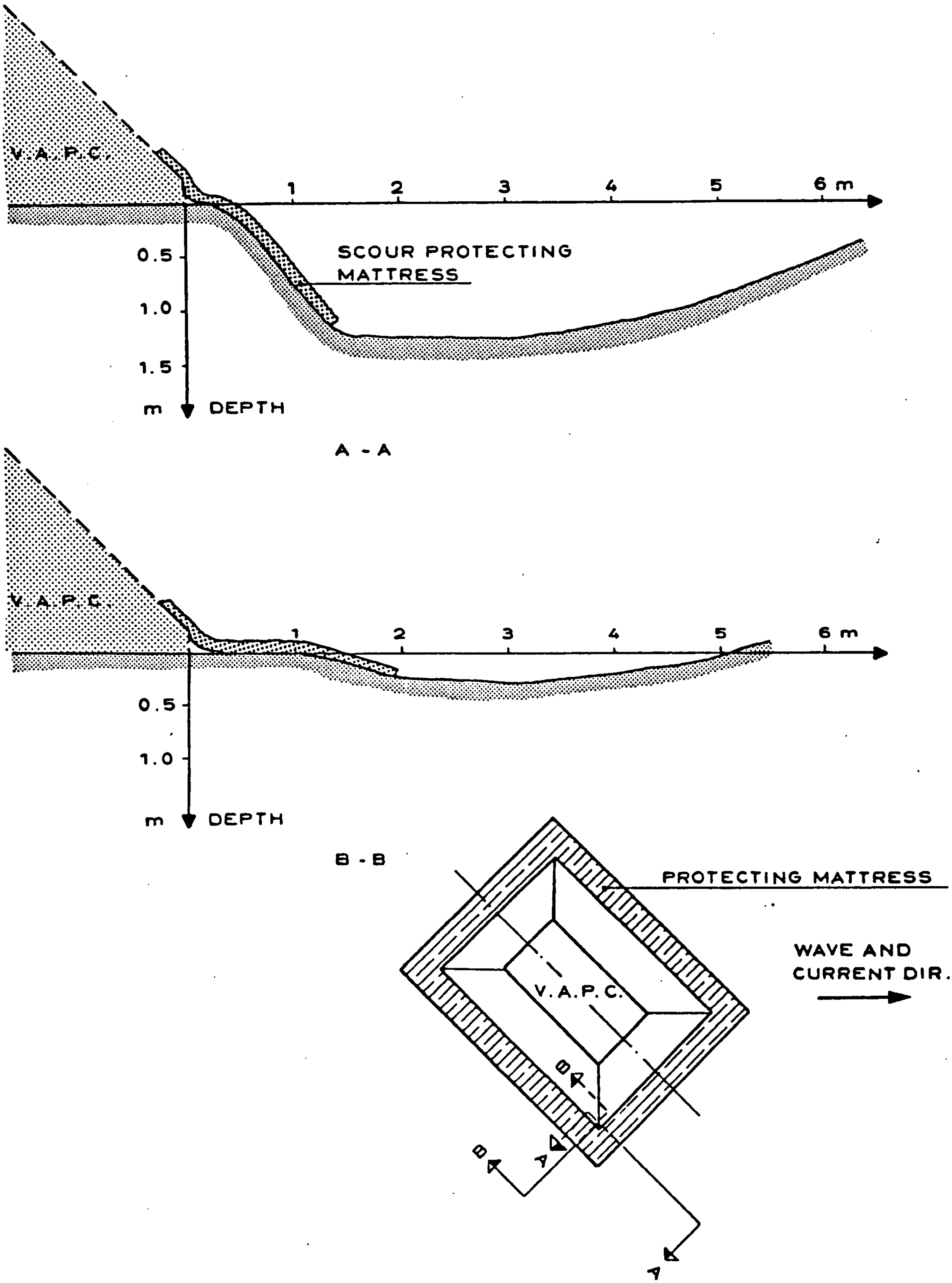


Figure 3.34 Scour Protecting Mattress for a Valve Chamber (DHI, 1983)

2000 metres of pipeline originally buried to a nominal depth of at least one metre near the shore at Bacton in the U.K. was uncovered at various times over a period of five years. The locations of pipe exposure varied so that less than 10% of the length in question remained covered for the entire period. The section concerned extends to a depth of about 30 m. Conditions at the same site at Bacton also clearly point to the hazards of pipes crossing dunes or cliffs. These areas may become weakened, where they are disturbed to install the pipe, initiating locally increased shore erosion.

3.2.3 Gravity Structures

3.2.3.1 Surface Penetrating Structures

In this discussion, "gravity structures" is used to cover a range of bottom grounded structures, the most familiar being the typical concrete gravity base oil production platform which is used in the North Sea. Few scour problems have been experienced with this type of gravity structure. Hoeg (1983) suggests that of all the gravity platforms installed in the North Sea only one, TP1 in the Frigg Field, has developed significant scour, the platform in that case having a square base. Foss and Warming (1979) report on experience with three concrete gravity structures of the Sea Tank design in the North Sea including TP1. The structure design included a skirt, the performance of which was also carefully monitored. Observations to date do not indicate any serious scour problems around two of these structures. Scour to a depth of 2 m developed at the corners of the TP1 base during the first few months after installation in 1976. Further scour was prevented by placing gravel bags in the depressions. In general, it appears that scour protection methods have not been employed around gravity platforms in the North Sea although in the case of the Ekofisk Storage tank, which has a diameter of 90 m, scour protection in the form of a graded stone mattress was provided around the structure to a distance of 10 m (Carstens & Sharma, 1975).

Scour protection methods for offshore berthing piers and neashore storm barriers are described by a number of writers. An article in Civil Engineering, July 1983 describes the methods used to prevent scour along the Eastern Scheldt storm surge barrier in Holland. It was estimated that over 5 km2 of seabed required scour protection and mats of woven polypropylene were laid on either side of the barrier. Concrete blocks were attached to the mats to provide ballast. In some cases, mats consisting of layers of woven polypropylene and asphalt were used but in generally shallower water. No data were given on storm current magnitudes.

The use of scour prevention techniques on an offshore berthing structure are described by Apelt and MacKnight (1976). This structure sited in 17 m of water off North Queensland, Australia consisted of concrete caissons measuring up to 47 m by 41 m with four columns about 12 m square projecting through the water surface. A series of model studies was carried out to determine possible scour effects and to test projection measures. The method eventually employed consisted of flexible concrete mats weighing 20 tonnes each with concrete blocks 0.5 m square cast on wire ropes. The model test showed that this method of protection should be completely satisfactory in waves 3.5 m high, although waves 7 m high would cause varying amounts of movement. During the first year of operation, the area was subject to several storms with wave heights to at least 3.5 m. An examination of the structure showed no sign of damage.

Bishop (1980) describes experience with scour at the Christchurch Bay Tower, U.K. This tower is a small offshore structure with gravity foundation placed in 9 m of water. It was an experimental tower for measuring wave forces in near-breaking or breaking waves. The original tower suffered foundation problems during the first major storm that it was subject to. The foundation design specifically excluded a skirt and after the storm it was found that the tower was rocking quite severely due to the scour of material underneath the base which left a ridge of material. At the same time, the tower had sunk some Ø.5 m into the seabed. Remedial works were not successful. A second tower was designed with larger base and foundation skirt Ø.7 m deep. Saucer-shaped scour occurred, the skirt became exposed and the foundation was grouted due to loss of some of the material. Nevertheless, the second design was considerably more stable than the initial design. Further details are archived. (A4.5.2)

The following information is summarized from questionnaire survey returns.

AGIP SPA recorded information on a mobile mat supported drilling rig installed at Beniboye, Nigeria in 1982. Water depth at the site was 7.7 m. The mean and maximum annual wave height were 2.44 m and 4.57 m, respectively. The mean bottom current velocity was Ø.3 m/s and the seabed material had a D5Ø grain size of Ø.1 mm. The initial scour protection consisted of plastic interwoven artificial seaweed mats with fronds 1 m long. Diver inspections indicated scour of up to 2 m deep around and underneath the structure. The remedial action taken involved placing sandbags in the scoured areas. See case study. (A4.3.2)

Elf Aquitaine Norge AS recorded information on their platforms TP1, TCP1 and CDP1 in the Frigg Field, North Sea. Environmental conditions were similar to those detailed in previous sections. As mentioned above, TP1 suffered some erosion at one corner of the square base which was resolved by sandbagging. CDP1 was initially installed with an antiscouring carpet of filter cloth which was fixed to the platform bases and rolled out after installation and weighted down with sandbags. Because of problems with stability of the filter mats, gravel dumping was required around the perimeter of the slab. This experience is summarized in the archived case studies. (A4.3.3)

Halcrow - U.K recorded information on the Kish Bank Lighthouse installed in the Irish Sea in 1965. The structure stands in 19 m of water. The 100-year return period wave is estimated to be 19 m with a period of 10 seconds. The maximum seabed tidal current is given as 0.6 m/sec and the bottom material is a fine to medium sand with a D50 grain size of 0.2 mm. Soundings taken every two years indicated that by 1981 erosion of up to 5.5 m had taken place centred 80 m from the lighthouse. Remedial action consisted of filling the scour holes with dredged gravel with a D50 grain size of 35 mm. Further information on this structure is given in the case studies. (A4.3.1)

Bitumarin BV provided extensive information on scour protection methods used on the Oosterschelde storm surge barrier, Holland. This project is located in 25 to 35 m of water. Significant wave heights of 4 m with a period of 8 seconds can be expected. Design bottom currents were assessed to be 2 m/s and the seabed material had a D50 of 0.2 mm. Three different methods were used:

- direct placing of asphalt on the seabed to form an impermeable layer,

- laying of a permeable, prefabricated mattress of bitumen coated stone sandwiched between filter cloth with a steel mesh reinforcing layer,

- laying of a stone filter blanket with armour layer between joints in the above system.

3.2.3.2. Submerged Gravity Structures

Scour around submerged structures have been described for a research platform installed off West Germany (Maidl and Stein, 1982). The platform measure 75 m in diameter at the base and is 4.5 m high with 45o sloping walls. The water is 3Ø m deep. A number of techniques were tested including the following:

- precast concrete slabs attached with hinges to the edges of the platform base,

- sand bag clusters in nylon netting,

- artificial seaweed anchored by synthetic and steel fabric mats.

The concrete mats which were considered to provide long-term protection based on a 2-year observation period experienced failures over longer periods as a number broke away from their connections and others experienced local deterioration due to spalling. It was considered that a more robust design was required. The concrete filled nylon mats appeared to provide protection although some damage was experienced. The sand bag clusters were considered not to provide long-term protection as they tended to be displaced from the structure. The artifical seaweed did lead to rapid sediment build-up but the synthetic anchoring mats failed in a number of cases and were not considered to be a feasible solution. Two or three years of observation of steel mats for anchoring the seaweed indicated that this combination might provide longer term protection. This experience is summarized as a case study. (A4.5.2)

3.2.3.3 Footings/Isolated Seafloor Structures

Some information on scour around isolated footings in the nearshore zone is provided in an unpublished report by Dewall (1981). Fifteen footings with six different shapes and constructed from reinforced concrete or aluminum ballasted with concrete were tested in water depths of 9 m and 18 m off the coast of North Carolina. The site is fully exposed to ocean waves with a mean annual significant wave height of about 1 m and currents of less tha Ø.3 m/s. Nearshore bottom sediment is moderately well sorted, medium to fine sand. During the tests, the largest wave height was about 4 m. No correlations are given in the report between scour extent and environmental conditions. A number of the footings had scour protection in the form of sand bags, artificial seaweed, gravel and filter matting and combinations of these. All footings were small being only Ø.6 m on the side and Ø.1 m thick except for two; one of which had dimensions twice these and another in the form of a cube which was Ø.6 on the side. One footing was circular with diameter Ø.6 m. Another had a "cutting edge" extending below the bottom of the footing.

Observations after 42 to 43 days indicated that scour hole diameters ranged from 2.25 to 4.5 times the footing size (diameter or side length) and scour depths ranged from Ø.12 to Ø.65 times the footing size. Scour diameter at the largest footing was about the same as at the smaller ones but at the thicker cube footing scour depth was twice as great. Of those footings not protected, the square plate with the "cutting edge" was most stable; observations after 4 months indicated that it had become buried.

Gravel weighted filter cloth provided some protection although it was concluded that its size should be 5 or 6 times the footing dimension. Observations of the effectiveness of the artificial seaweed were inconclusive, and DeWall suggests that the seaweed itself may act as an obstruction leading to larger area scour, although this observation has not been borne out in other applications referred to in this report.

Limited observations at the 18 m water depth indicated that scour of a similar magnitude as observed at the 9 m depth would occur there as well.

DeWall concludes that footing shape is not a significant factor in determining ultimate scour geometry. Experiences in the North Sea with large gravity platforms indicate that this is not the case as related in this report. The shape may determine whether scour occurs at all while the size of the structure in relation to wave characteristics may be important in determining when shape becomes important. From his experiments, DeWall recommended further testing of filter cloth mats with the size of the cloth being in the order of 6 times the structure diameter.

3.2.4 Islands

3.2.4.1 Sacrificial Islands

Sacrificial beach islands have been in use in the Beaufort Sea since 1970 where they were introduced largely because of their resistence to ice forces. Experience has shown the islands are however vulnerable to erosion during the open water season when they are exposed to waves and currents. Due to the nature of the dredged sediment used to construct the islands, (usually a fine to medium sand with a D50 of 0.2 mm) erosion of the islands can progress very rapidly. The erosion of Alerk, a prototype island in the Beaufort Sea, was monitored by Esso Resources Canada with aerial photographs, over a period of three storms in October 1980. During these storms which each lasted less than a day producing significant wave heights of about 3 m and wave periods of around 7 (as determined from a simple windwave hindcast), the island was eroded from the edge of the horizontal drilling platform to the centre of the island, loss of some 50 meters within a month. Many more examples of the erosion of artificial islands are reported by Harper and Penland (1982), one of these islands provides an even more striking example of the vulnerability of hydraulically filled sacrificial sand islands. In the summer of 1978 Isserk was constructed for Imperial Oil with 2 million cubic metres of fill. The island was located in 12 m of water, had a diameter of 225 m and 5 m of freeboard. On the 17th of July 1978 the island was occupied and drilling was in progress and by the 29th of August, only six weeks later, the island had been reduced to a fully submerged shoal.

Evidently, the application of artificial islands is limited to sites where open water seasons are very short or where wave climates are not severe. In the Beaufort Sea artificial islands have become an acceptable method of construction, however they are vulverable to rapid erosion under severe storm conditions.

3.2.4.2 Caisson Retained Islands

Caisson retained islands were developed from a need to construct islands in deeper water in the Beaufort Sea where volumes of fill material required for simple islands becomes large enough to be inhibitive on the grounds of cost. The Tarsuit Caisson retained artificial drilling island was installed for Canmar/Dome Canada in the Canadian Beaufort Sea in 1980-1981. The structure is an octogonal concrete caisson consisting of four independant concrete boxes with bevelled edges, with a main dimension of 110 m which rests on a submerged island berm of dredged fill. The water depth is 22 m with the top of the berm being 6.5 m below sea level. The fine sand of the berm was protected with a 1 m thick layer of dredged gravel. In July of 1982 Tarsuit was exposed to a storm with a peak significant wave height of 3.8 m and an estimated return period of six years, the gravel which had a D50 of 40 mm, was locally scoured to depths of 1 to 1.5 metres extending 5 m laterally. Following this storm and on the basis of results from model tests, a one metre apron of quarried rock was installed. Since its installation the rock apron has been exposed to various storms with peak significant wave heights in excess of 2 m resulting in no significant erosion. A more detailed account of the monitoring of Tarsuit is available. (A4.4.1)

4. SCOUR PROTECTION METHODS

4.1 General Description

4.1.1 Design Allowances

A fairly common method of dealing with the problem of sour is to assess the maximum depth of scour likely to occur at the structure location and make conservative allowances for the effect of this in the design. Several examples of this approach are given in brief detail below.

4.1.1.1. Gravity Structure Skirts

A rigid steel or concrete curtain wall may be provided at the toe of gravity structure. The length of skirt is calulated to be such that undermining of the structure will not occur should the maximum scour expected be achieved.

4.1.1.2. Flexible Aprons

A hinged or articulated apron of steel or concrete is attached to the toe of gravity structure which is able to conform to the scour profile. The nature and width of the apron are chosen to reduce scour and eventually provide a natural angle of support which will prevent undermining of the structure.

4.1.1.3 Pipeline Burial

The conventional method for scour protection of pipelines where erosion is likely to be a problem is pipe burial. The pipe is laid in a trench excavated in the sea floor and backfilled to original seabed level either with excavated or some more stable material.

In some cases, a gravel mat is placed over the buried pipe. In other cases there is significant long shore movement of materials in the surf zone, the trench is left open and allowed to fill by natural processes. However, this procedure may be hazardous since it could lead to jacking from infilling underneath the pipeline as described in Section 3.1.

Pipeline burial is the conventional means of scour protection for pipelines in the vicinity of the shoreline interface. Pipelines are buried to a sufficient depth that they will not be exposed by profile change. The pipelines are laid in a trench which has been backfilled with the dredged material or with selected material if the excavated material is unsuitable.

The theoretical knowledge on shoreline change does not yet provide a reliable answer for the required depth of burial. Therefore, the profile change for a specific site must be determined by field surveys or laboratory testing.

At some locations, pipes have been laid between sheet pile cofferdams through the surf zone. A proposal suggested for the Ekofisk/Statfjord to Kallstø gas lines was to lay 5 m diameter precast concrete tunnel sections for a distance of 700 metres throught the surf zone to carry the pipes.

Table 4.1 details some typical pipeline burial depths related to work in the North Sea.

It should be borne in mind that pipe burial is sometimes undertaken for other reasons than purely as protection from erosion. Trenching also reduces the risk of damage from trawling, fishing and dragging anchors.

4.1.1.4 Additional Pile Length

For piled jacked and jack-up structures, the length and strength of pile, caisson or spud can penetration are calculated to provide fixity of the piles and overall stability of the structure assuming that the seabed is lowered by the amount of predicted scour. Many jack-up platforms are

Table 4.1 Examples of Typical Pipeline Burial Depths

Pipeline	Burial: Surf Zone	Burial: Offshore
Shell Leman Bank Pipeline (U.K.) Joint Leman	3-m cover	3-m cover
Trunk Line (U.K.)	3-m cover	1-m cover
B.P-Forties	2-m cover	1-m cover
Ekofisk-Teesside (U.K.)	3-m - 2.5-m cover	1-m cover
Ekofisk-Emdem (Germany)	See remarks	Not determined
Mobil-Wilhemshaven (Germany)	Minimum 3-m cover	Not applicable
Placid (Netherlands)	varying 2 to 7-m cover	2-m cover

now equipped with an airlift system to allow for greater depths of leg penetration.

4.1.1.5. Sacrificial Protection

Sacrificial Islands

As the name implies, the design of the artificial islands includes an allowance for erosion of the sacrificial beach. This design method is practical in the Beaufort Sea because of the lack of suitable armour material. The islands are constructed from dredged sand which in the Canadian Beaufort typically has a median grain size of D5Ø = Ø.2 mm, consequently erosion is very rapid. The sacrificial beach which acts to dissipate incoming wave energy is typically sloped at 1:12, a 1:3 slope starts at the limit of uprush and rises to the horizontal drilling platform. Obviously, the design of the islands should include for the provision of a dredge to carry out remedial infilling of scour caused by waves. Erosion of artificial islands has recently been investigated with physical model studies by Kamphuis and Nairn (1984). A series of model islands at different scales were tested to determine the scale effects inherent in mobile bed models. While insight was gained into scale effects and some agreement was found with prototype results, accurate quantitative prediction techniques for erosion rates require further model testing at scales approaching the prototype size and most importantly prototype results. A comprehensive numerical model for the erosion of artificial islands which includes deformation of the mound is not available. However, a model is presently being developed at Queen's University in Kingston and will aid in design of these islands. Also, some proprietary numerical models exist for the erosion of artificial islands (Moir, 1985).

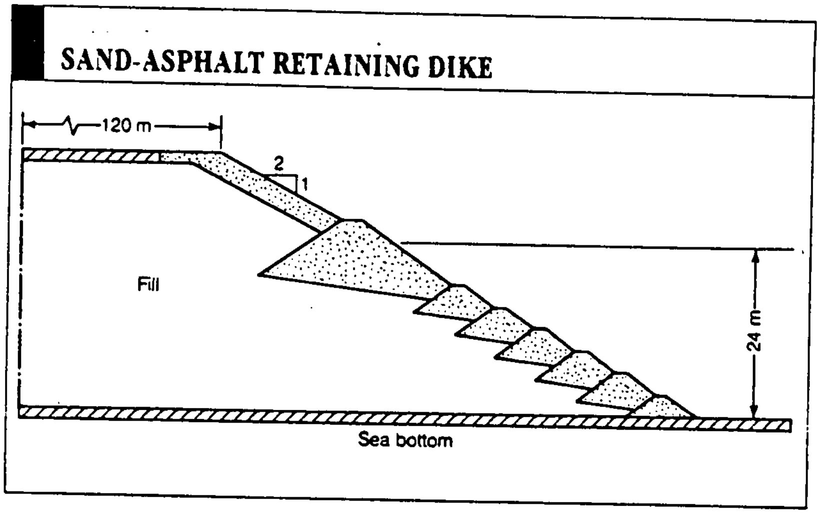
Artificial islands have been protected by a variety of materials as discussed by Tilmans et al (1982) and Robertson (1983). A common technique is to place sand bags on a filter cloth layer in the surf zone region of the beach. However, as a slope protection method, sand bags have only demonstrated limited effectiveness in the Beaufort Sea. Also, the environmental cleanup is very costly where sand bag removal is required.

Most design technology for the Beaufort to date concerns exploration platforms with short design lives of one or two open-water seasons. The design of production islands may require a more permanent slope protection to extend the design life. Robertson (1983) suggests the use of thick sand asphalt mats on their own or in conjunction with concrete blocks for extended-life slope protection (See Figure 4.1). However, it may prove more economical to retain dredging capacity to reconstruct islands following major storms.

Caisson Retained Islands

The caisson retained island usually consists of a caisson placed on top of an underwater mound. The toe of the caisson is protected either by sacrificial sand or sacrificial gravel berms or by rock, Myers et al (1983). Design considerations include the lateral extent and the depth of the protective gravel and rock layers which are contingent on the grading of the protective material.

Numerous numerical and physical models have been used to determine design dimension. Physical model studies have been performed by Brebner and Kamphuis (1977) and W. F. Baird Associates (1981). Recently Fleming et al (1983) (see also Fleming, 1983), have developed a comprehensive numerical model for investigating scour processes around a caisson retained island. The model consists of three major stages, these being a hydrodynamic stage, a wave transformation stage and a sediment scour stage. Bed changes due to scour observed in a physical model performed by Esso Resources Canada were reproduced well by the numerical model so that the latter could be used to investigate scour at full prototype scale for different bed materials. This model was used by Acres Consulting (1983)



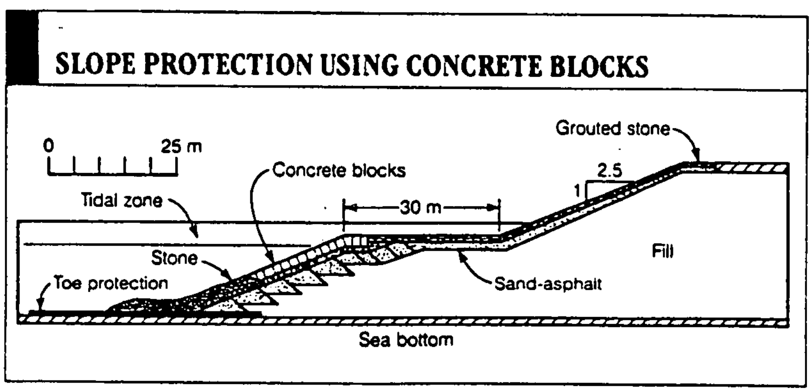


Figure 4.1 Extended-Life Slope Protection Methods for Artificial Islands (Robertson, 1983)

to determine design parameters for Esso Resources Canada. Apart from the detailed design some general conclusions from this report are worth mentioning.

The gravel blanket, at the minimum should extend beyond the point of which significant erosion of gravel is expected to a depth where scour of the sand at the toe of the gravel layer is not unacceptable. Also, the results from the numerical model indicate that erosion would be reduced if a well-graded material is used since the shear force due to effective roughness and the entraining force would be reduced. This suggests a gravel-sand mixture might be advantageous as a protective layer to the underwater mound.

4.1.1.6 Underpinning

Underpinning is often used as a remedial measure for stabilization of spanned sections of pipeline. For areas where sediment deposits are intermittant over a rock seabed, spanning should be anticipated and the underpinning technique may be used as a preventive measure. One type of underpinning developed by Dunlop Limited (reported in Ocean Industry, Feb. 198Ø) consists of attaching nylon bags coated with polychloropene to the underside of the pipeline, and filling them with grout to hold the pipe down as well as supporting it. The disadvantage of this method is its reliance on divers for installation which can prove costly.

4.1.2. Scour Reducing Measures

These methods act to reduce scouring current velocities and either cause a reduction in scour or in fact attract accretion of material. Consequently, they may be referred to as active protection measures.

4.1.2.1 Artificial Seaweed

The concept of artificial seaweed was developed on the basis of the effectiveness of using vegetation to establish fore dunes adjacent to beaches. In an application of this method, artificial seaweed is used to reduce local velocity of currents in the boundary layer as well as the turbulent flow near the seabed (Maidl and Stein, 1981) and cause a reduction in scour or in fact attract accretion of material. Hindmarsh (1980) states that from model flume work and studies at Trondheim, flow velocities around structures can be reduced by up to 80%. Prototype experiments by Maidl and Stein also confirm that artificial seaweed effectively reduces velocities, in their tests velocity was reduced by 20 to 30% at 0.30 m above the seabed.

Artificial seaweed has been used in both the conventional upright manner and in a hanging position. Various adaptations of upright seaweed have been developed ranging from the basic method of a mat of polypropylene fibres (with specific gravity less than 1.0) to synthetic fronds (See Figure 4.2.) The upright seaweed is secured by sand bags, Paraweb mats (an interwoven polyester fibre) or steel fibre mats. The seaweed not only prevents erosion but promotes deposition of sand onto the mat ultimately leading to the build up of a fibre reinforced mat. However, the deposited sand is usually not sufficient to anchor the mat of fibres and the mats must be anchored to the seabed. In prototyhpe tests, Maidl and Stein found that Paraweb mats anchored with earth nails were lifted from the seabed and therefore judged insufficient for North Sea conditions. In further tests with upright seaweed attached to steel fibre mats anchored to the seabed, depositon of sand was observed and the mats were being held in position. Success with Paraweb mats has been reported by Hindmarsh in the northern North Sea in the Piper Field for Occidental and by Shell in Brunei. They may also be deployed over a pipeline (with the option of artificial infilling) as shown in Figures 4.3, 4.4 and 4.5.

Hanging seaweed which employs polyester fibres is usually used around piled or gravity structures (see Figures 4.6 and 4.7). It is possible to attach the system prior to installation of the structure (see Figure 4.8). The packages which contain the fibre are mounted on a supporting member, surrounding the structure to be protected. The system is secured to

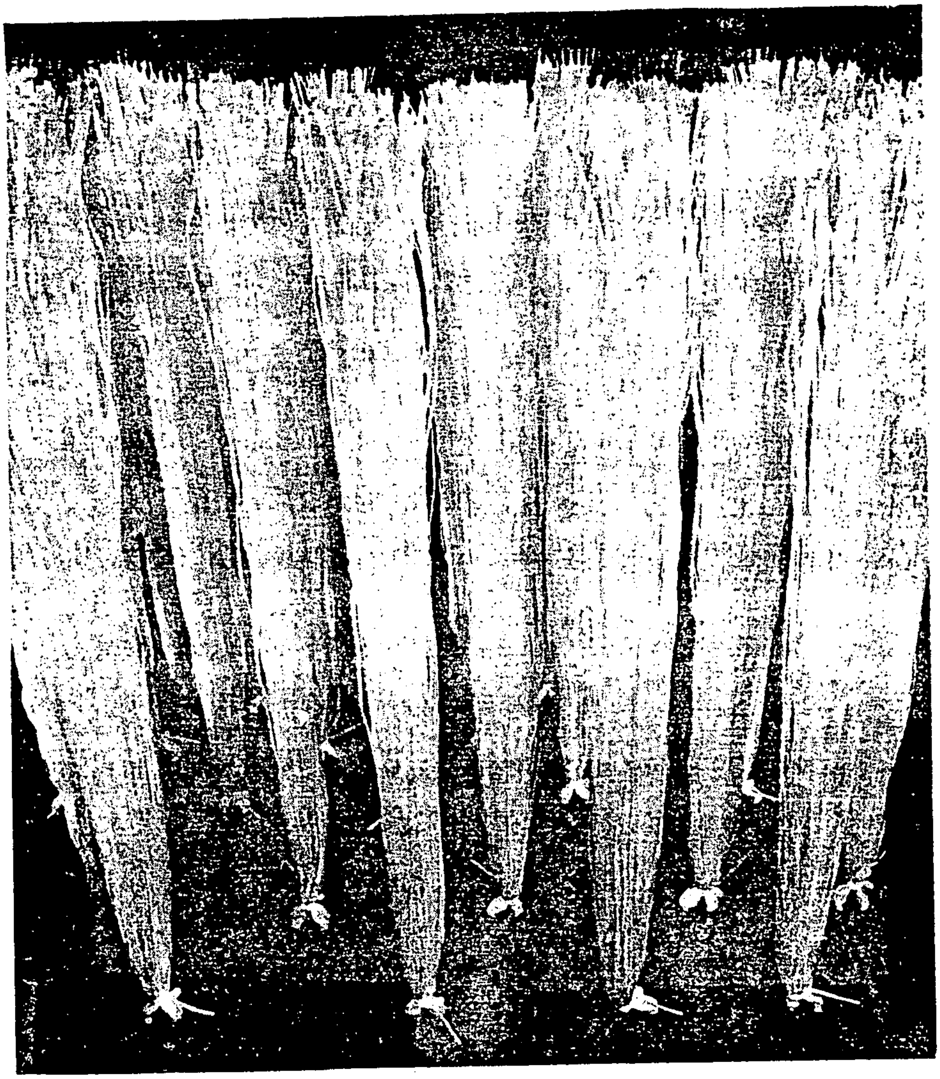


Figure 4.2 Upright Artificial Seaweed in the Form of Synthetic Fronds (Linear Composites Erosion Control Systems, ICI Linear Composites Limited)

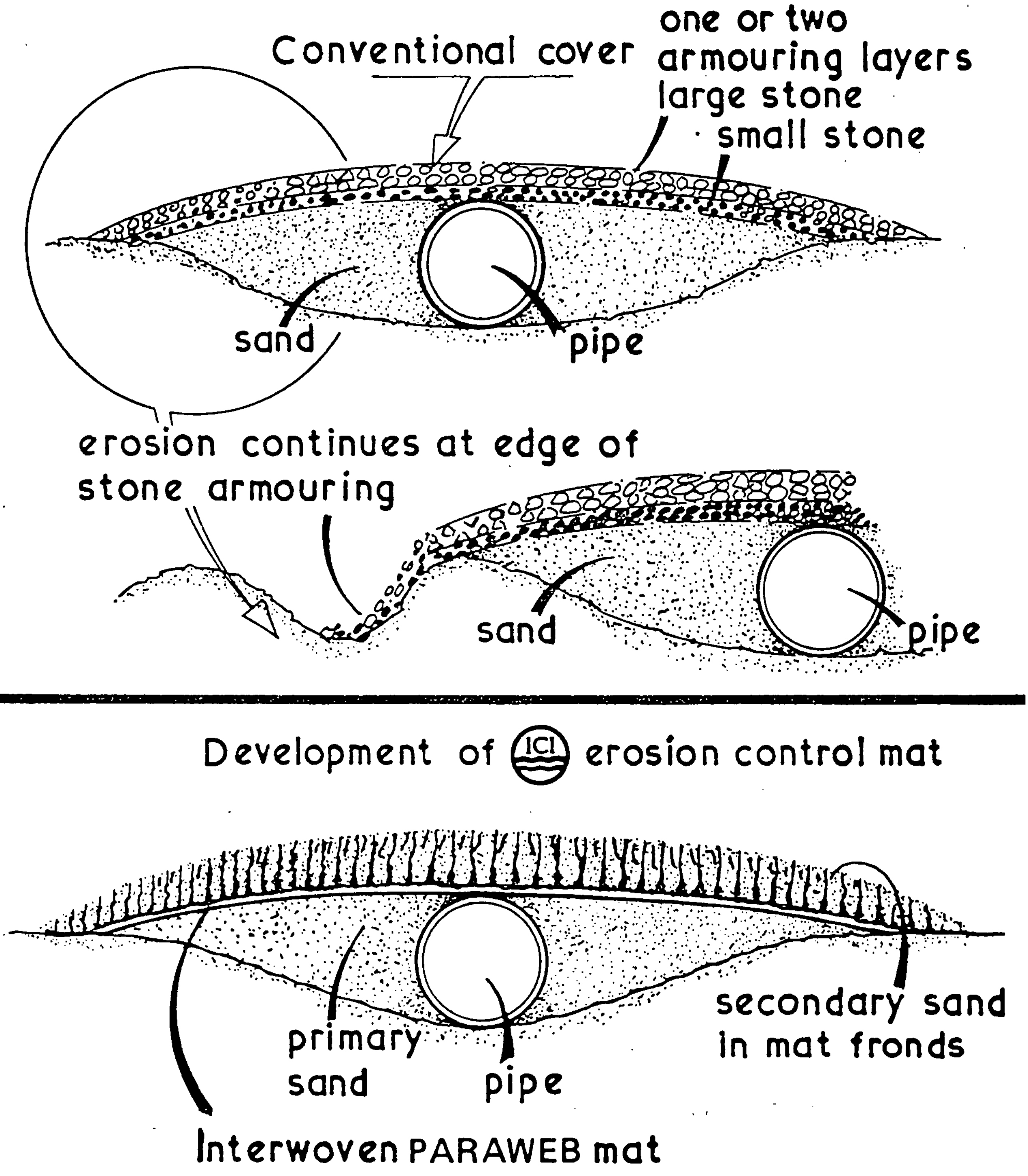


Figure 4.3 Cross-Section of a Pipeline Cover with Artificial Seaweed (Linear Composites Erosion Control Systems, ICI Linear Composites Limited)

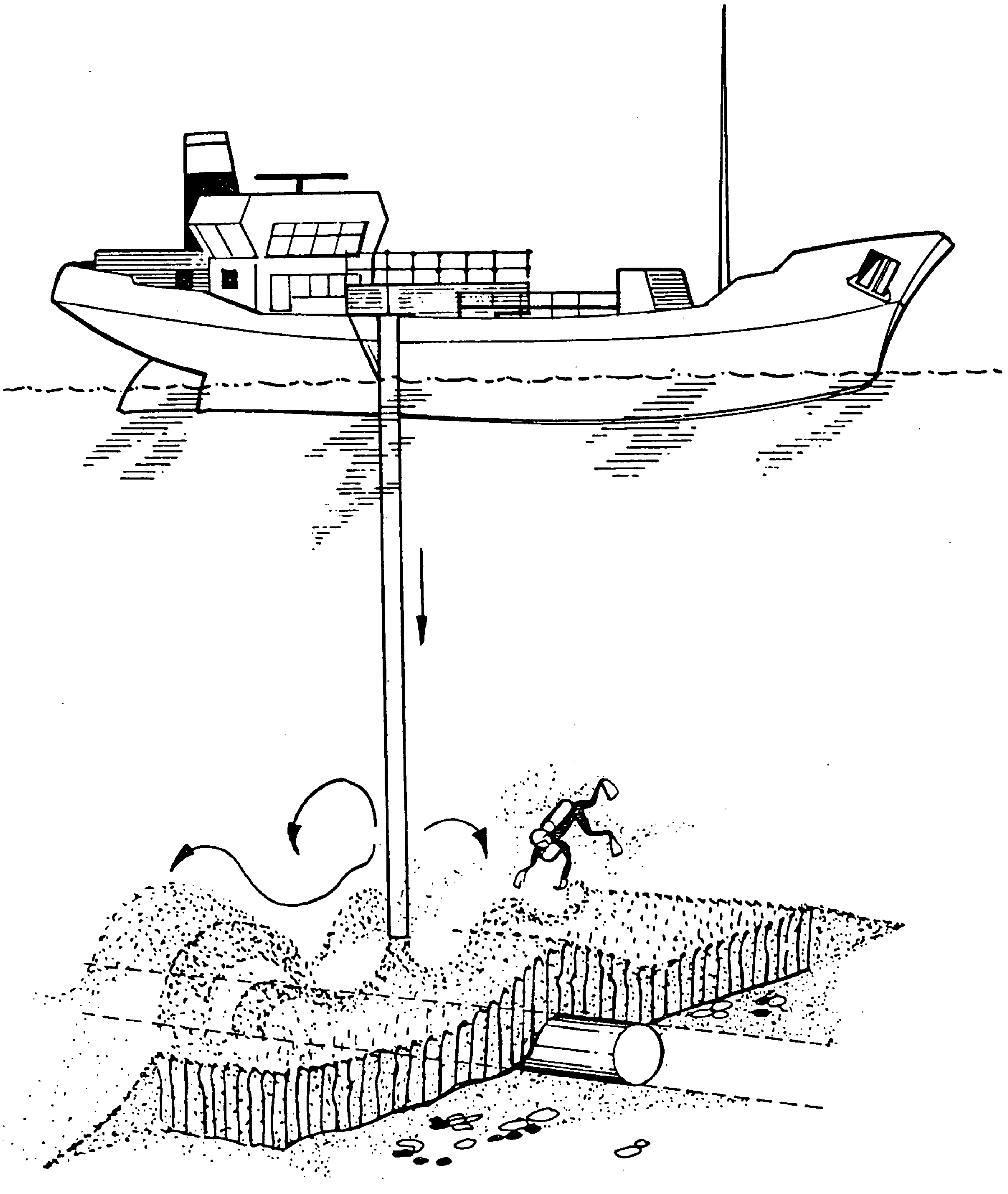


Figure 4.4 Filling an Artificial Seaweed Mat with Sand
(Linear Composites Erosion Control Systems, ICI Linear Composites Limited)

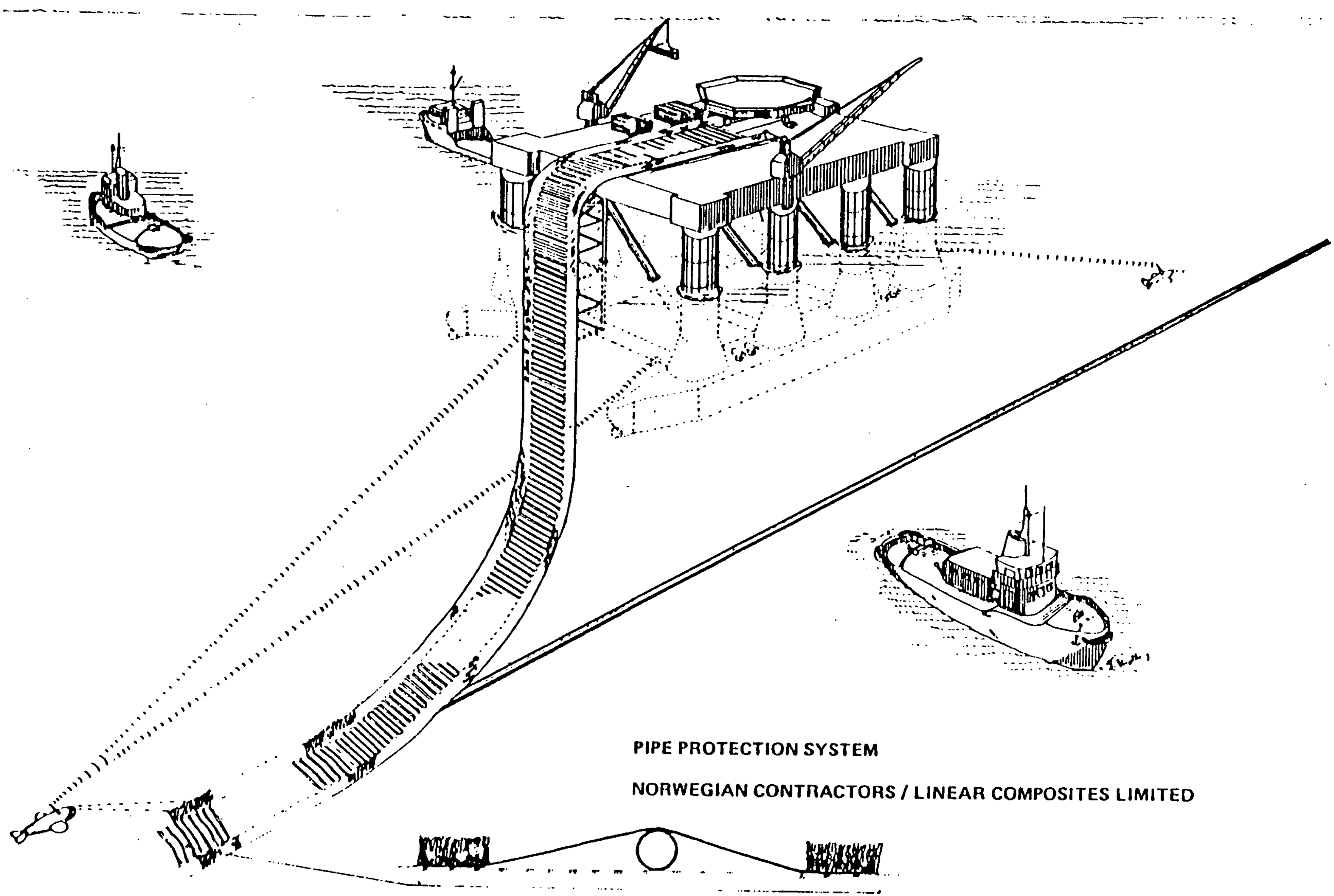
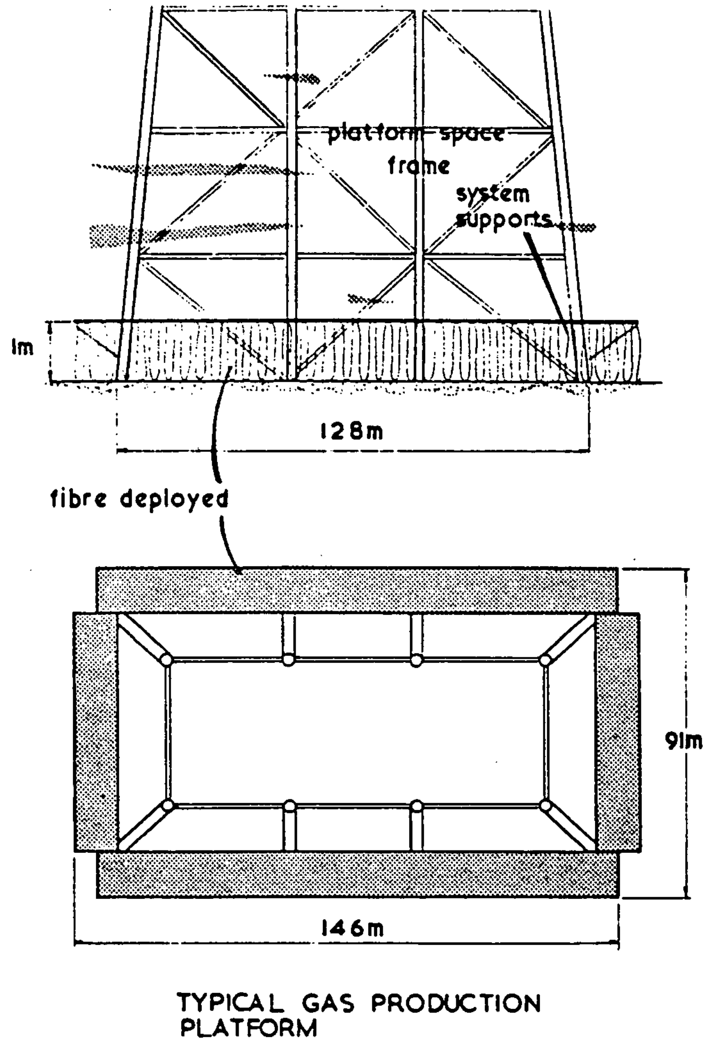


Figure 4.5 Deployment of an Artificial Seaweed System
(Linear Composites Erosion Control Systems, ICI Linear Composites Limited)



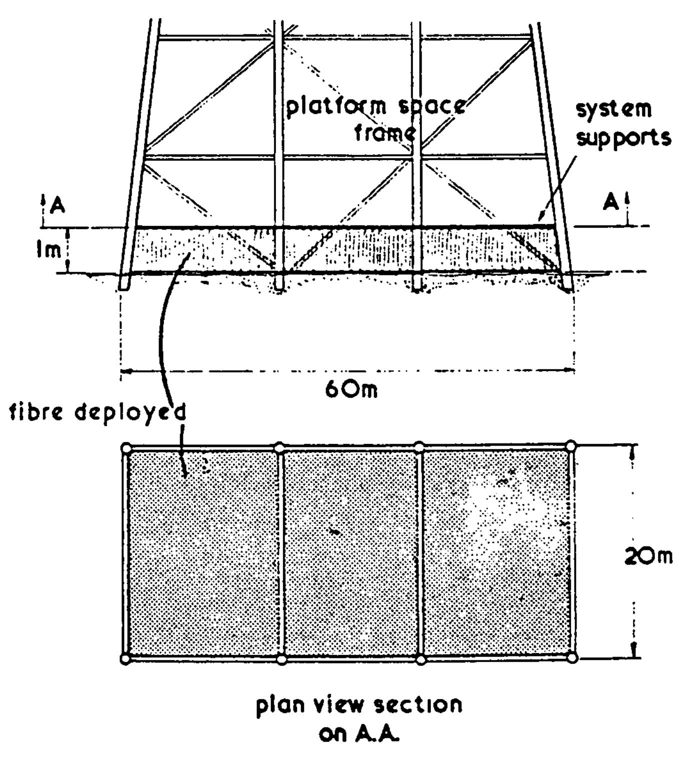


Figure 4.6 Upright Seaweed Protection Under a Piled Platform (Linear Composites Erosion Control Systems, ICI Linear Composites Limited)

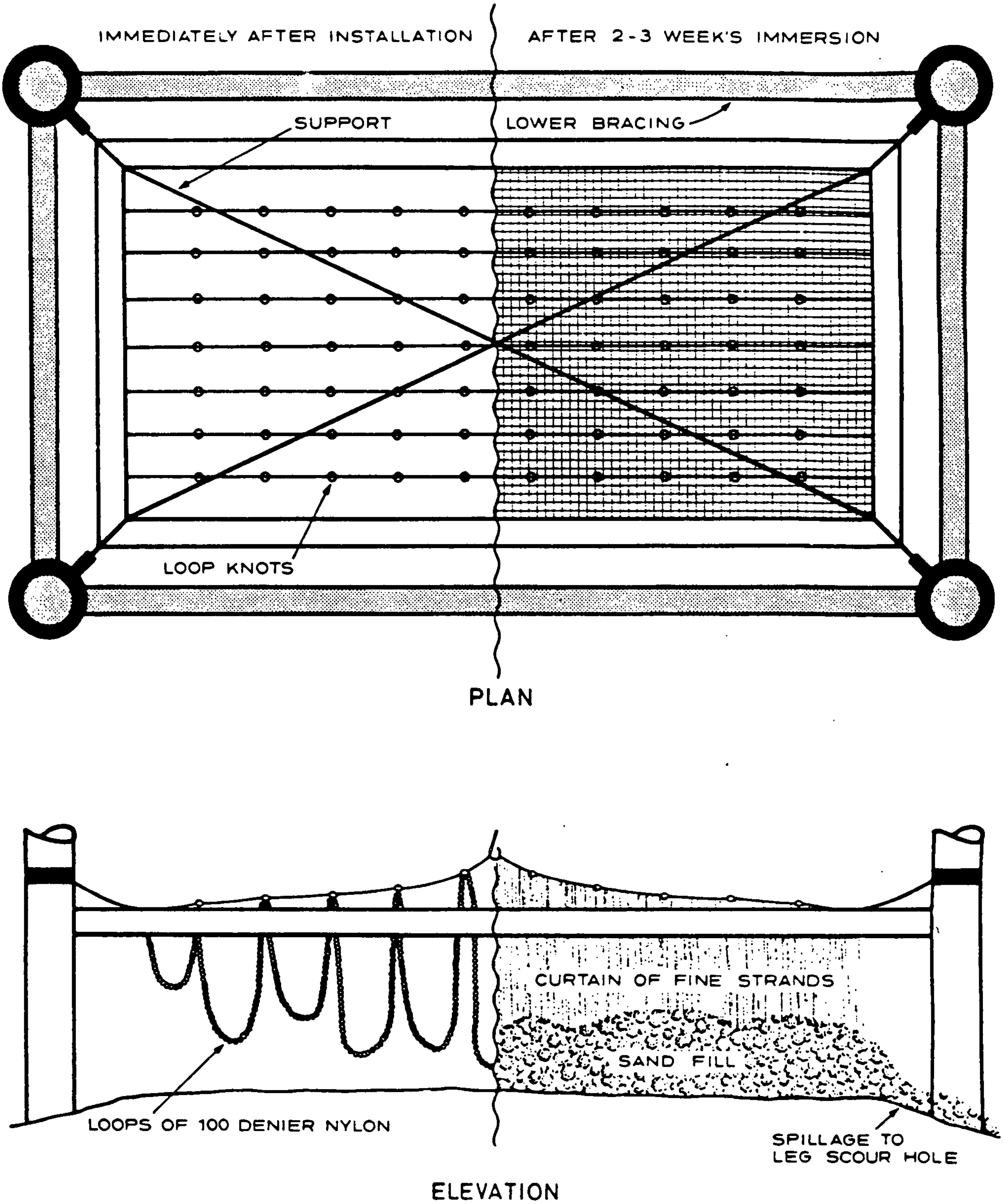


Figure 4.7 Hanging Seaweed Protection for a Jacket Platform (Watson, 1973)

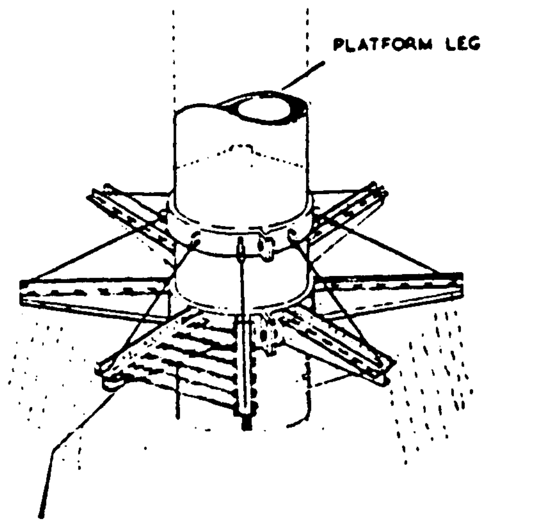


Figure 4.8 Pre-Installed Protection for a Platform Leg (Linear Composites Erosion Control Systems, ICI Linear Composites Limited)

subframes which in turn are mounted to steel or ferro cement outrigger base frames which are an integral part of the base structure. The system is kept in this form until the structure has been placed on site at the seabed, when the system is activated and the fibre array deployed.

4.1.2.2. Others

A scour protection technique recently developed by Gulf Applied Research is described in a paper by Loer (1983) and consists of device which is fitted to a pile before or after scour has occurred. The device shown in Figure 4.9 inhibits the formation of the primary vortex which is the cause of local scour, (See also Figure 4.10). Loer states its success has been proven in laboratory studies as well as in prototye tests in the Gulf of Mexico. In laboratory tests the device, called a `Scour Brake', has been shown to promote infilling when used as a remedial measure to a scour hole around a pile. The device will only avert local scour, global scour will remain unchecked.

4.1.3 Armour Cover Layers

The principal method of direct scour prevention is some form of armour layer. This can either be permeable or impermeable and rigid or flexible. The favored systems are permeable, allowing release of uplift water pressures and flexible, allowing conformation to scour settlements without losing structural integrity. Permeable systems generally consist of a filter layer to retain the underlying seabed material and an armour cover layer to withstand the erosive forces.

IMPERMEABLE

4.1.3.1. Soil Stabilization

Over the past thirty years or so, methods of soil stabilization by mixing cement or lime with weak insitu soils have been developed. These

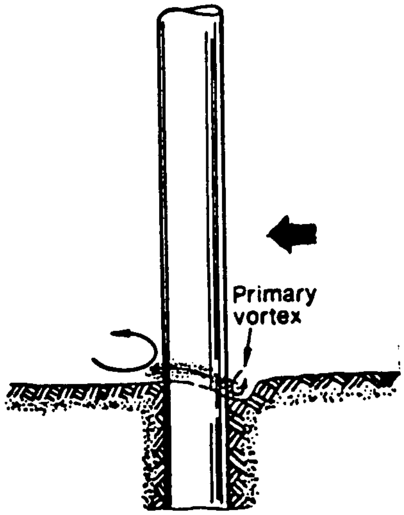


Figure 4.9 WHAT CAUSES SCOUR HOLES
As the water streams toward the structure, a swift downwash occurs on the upside of the leg that begins the formation of the scour hole. This mechanism continues, resulting in a horseshoe-shaped hole around the platform leg. (Loer, 1983)

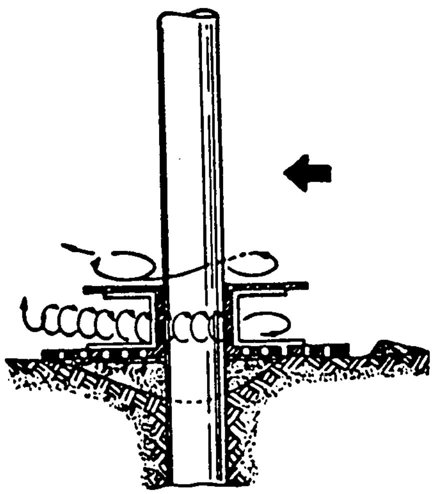


Figure 4.10 SCOUR BRAKE INSTALLATION
As indicated here, the scour brake prevents the primary vortex from interfacing with the seafloor sediments. Also, the hydraulic jumps created around the circumference of the device are caused by vertical spines that alter the vortex-shedding mechanisms in such a way that it results in reduced wake and prevents sediment lifting. (Loer, 1983)

methods have, in the past, been limited in scour protection applications to locations above low water.

The mixture of cement/lime, water and insitu soils are compacted and on setting can form an impermeable layer of material with the properties of concrete. Typically, the mixture would consist of 10% cement and 10% water and be up to 300 mm thick. With this depth and being rigid and impermeable, some problems have been experienced with stablilized soils when settlement and cracking have occured.

The Takenaka group in Japan has developed a technique for the deep chemical mixing (DCM) of insitu soils that could prove effective in protecting underwater structures from scour damage (See Figure 4.11). The DCM method is a soil-improvement process that it is claimed can solidify soft seabed soils into stiff soils which have uniaxial compressive strengths of 20 to 50 kgf/cm2. The method involves mixing the seabed soil with cement slurry and additives in insitu using a purpose made rig. The DCM method has been used to improve the soft alluvial clayey soils prevalent in coastal Japan, but should also be capable of solidifying and improving sandy soil layers from 3 to 5 m in depth. This improvement soil layer would provide a stiff cover layer capable of protecting the seafloor from erosive forces.

Both the above processes require rigid quality control to ensure satisfactory results.

4.1.3.2 Impermeable Mastic-Asphalt Layers

This scour protection consists of a layer of mastic-asphalt, typically 200 to 300 mm thick. Mastic-asphalt is a mixture of bitumen, sand and filler. The sand-filler is minimal. By adding excess bitumen above the necessary fill the voids in the sand-filler mixture an impermeable material is created. At high temperatures, the mastic-asphalt behaves like a viscous liquid with a viscosity of 40 - 100 Pas. at

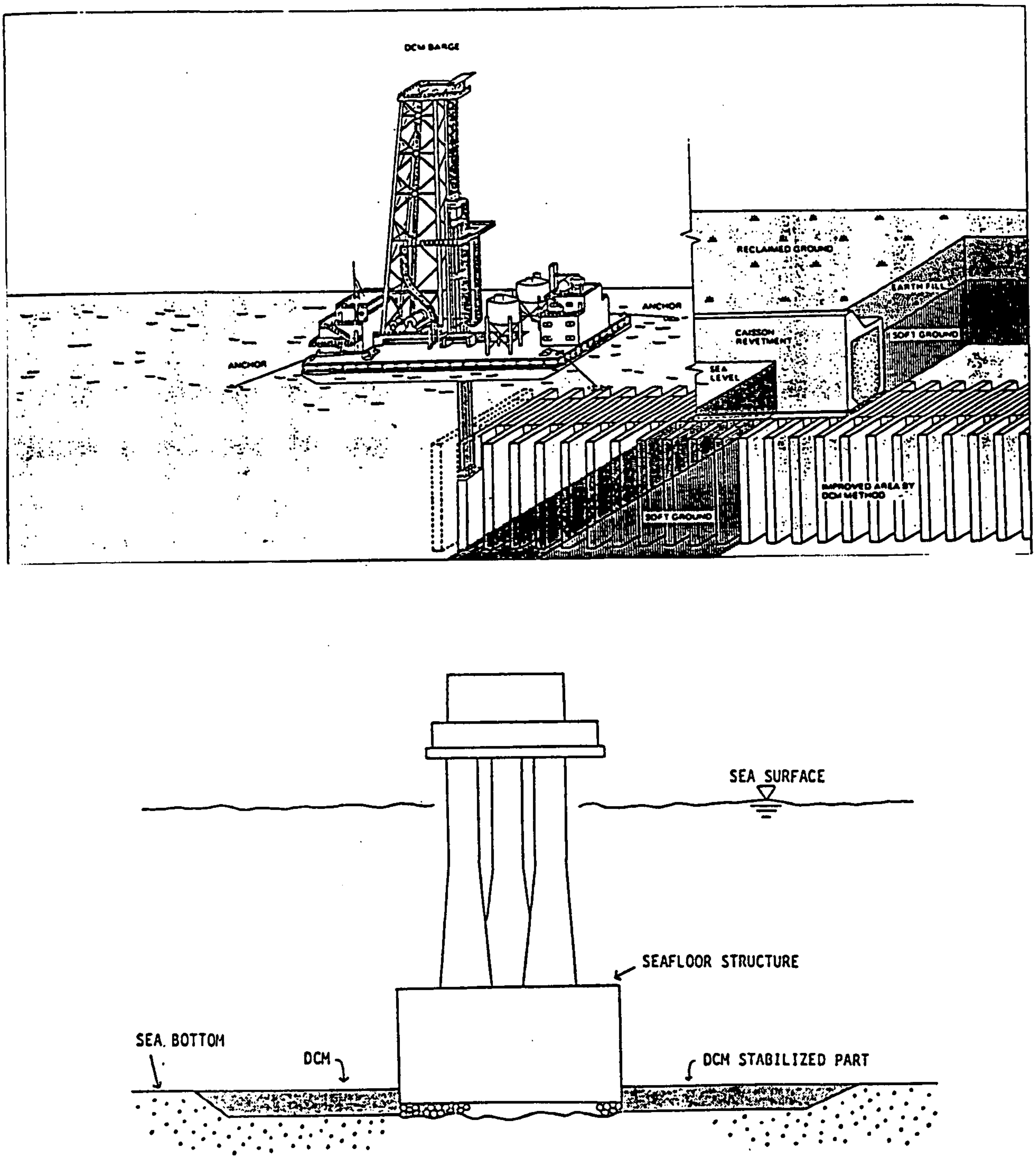


Figure 4.11 A Method of Deep Chemical Grouting (Takenaka Group)

temperatures of 120 - 160o C. Because of this viscous behaviour it is possible to pour hot mastic-asphalt under water on the sea bottom where it will flow out forming a uniform layer.

A recent application of this method was carried out by Bitumarin on bottom protection for the Oosterschelde storm surge barrier, Holland, using the vessel "Jan Heijmans" which is equipped with a supply pipe and distributor. (See Figure 4.12). With this distribution, the mastic was brought and spread on the sea bottom in 5 m wide sheets 80 to 100 m thick overlapping each other to a total thickness of 200 to 300 mm. The second hot sheet placed over the first cold sheet ensure that both sheets melt together to give a uniform, sandtight impermeable construction. (See Figure 4.13).

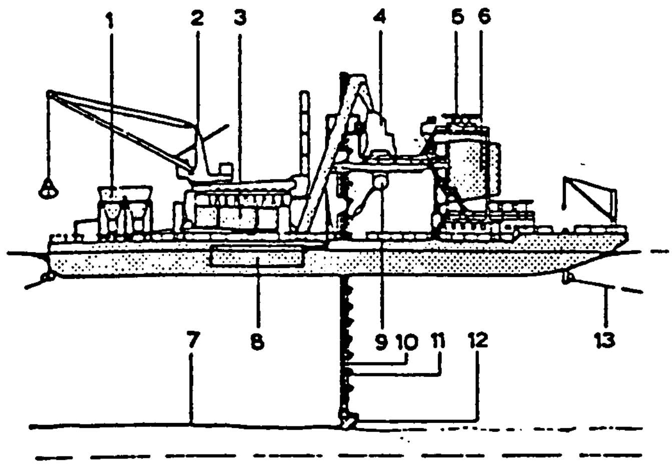
After cooling, the mastic-asphalt behaves like a visco-elastic material which means that it is capable of conforming to slow deformations of the subsoil without cracking or losing its sandtightness.

In places where the sea bottom is sloped, the same procedure can be applied. After placing a stone layer on the seabed, this is grouted with mastic-asphalt. The stone layer prevents extensive flow of the mastic-asphalt on the slopes during the hot executing phase.

PERMEABLE - FILTERS

4.1.3.3. Stone Filters

Graded stone filter mats are probably the most commonly used method to prevent or remedy the effects of scour. They have been used extensively as under blankets and aprons at the toes of rubble mound breakwaters, islands and gravity structures, around individual piles and as global protection around piled structures and as a cover to pipelines.



1. sand
2. transhipping crane
3. drying drum
4. mixer
5. control room
6. filler silo
7. sand mastic carpet
8. bitumen tanks
9. stirring kettle
10. supply pipe
11. filling gate
12. distributor with feeler
13. anchoring cable

Figure 4.12 Deployment of an Impermeable Mastic-Asphalt Layer by the "Jan Heijams"
(Linear Composites Erosion Control Systems, ICI Linear Composites Limited)

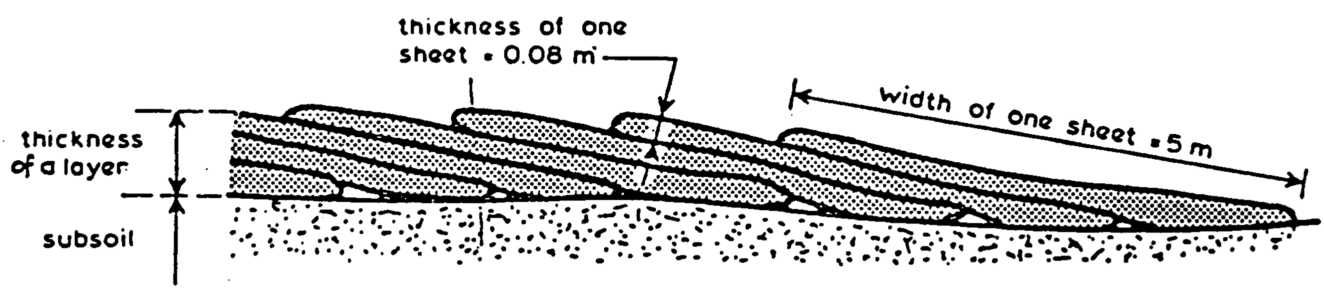


Figure 4.13 Sheets of Mastic-Asphalt Lying Like Roof Tiles
(Linear Composites Erosion Control Systems, ICI Linear Composites Limited)

The principle factors to be considered in designing a stone filter are:

- it must relieve water pressure and yet retain the finer bed soil particles and

- it should completely stable under the erosive wave and current forces at the location.

To achieve the above requirements, the filter apron will generally consist of two or more layers of stone ranging from the smaller grading required to retain the seabed to the larger surface layers required to resist the environmental forces. A commonly accepted criteria for determining the grading of the various layers is given in Section 5.3.1.

Succesive layers of filter stone would be built up until the grading of the armour stone required to combat erosion was reached.

Over the last fifteen years, there has been a growing tendency to utilize fabric filter cloths to replace the stone filter beneath the armour layer.

In some instances, a layer of stone, gravel or sandbags have been used directly on the seabed sized to counter erosive forces but without the benefit of a filter layer. Generally, this has led to unacceptable levels of scour as the underlying material is removed through the voids and the armour settles to a level at which the bed material is no longer affected by the erosive forces. The method generally requires a continuing remedial placing of additional armour until with time a stable and acceptable profile is reached.

The armour material used can itself cause scour resulting from increased bed roughness and the modification to flows resulting from its size. This is particularly the case at the edges of the protective apron. It is normal practice to extend the filter layers several metres past the point at which the cover stone is required to reduce this effect. There are therefore advantages to limiting the size of the surface layer to the minimum required. In this respect, a number of projects have resorted to the use of high density materials such as granites, mineralized rock or steel mill slag which have small size relative to weight.

The main problems with filter layer systems is in the availability of suitable materials within the grading limits required, and in the accuracy with which the layers can be placed. The apron will only be as effective as the adequacy of coverage will allow.

A method which counters difficulties in placing segregated filter layers involves spreading a single evenly graded mixture which contains all the elements required for filters and cover armour. With the passage of time, a natural filter is created as finer material is removed from the mixture and the remaining material is graded from fine at the bottom to coarse at the top through the depth of the layer.

The success of this method hinges on finding a material with the proper gradation that will provide an armour layer to resist the design shear stress. Figure 4.14 provides an example of the initial and armour layer gradation and Figure 4.15 demonstrates the method as a cover layer for a pipelines trench.

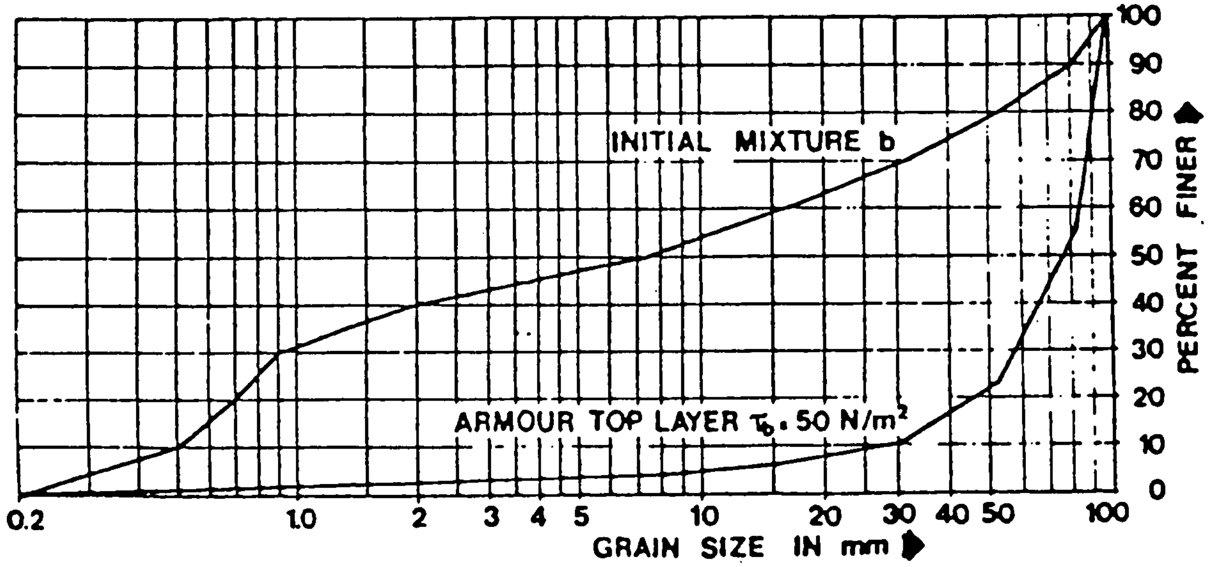


Figure 4.14 Grading Curve for Gravel Dredged from the Seabed (Roelofsen, 1980)

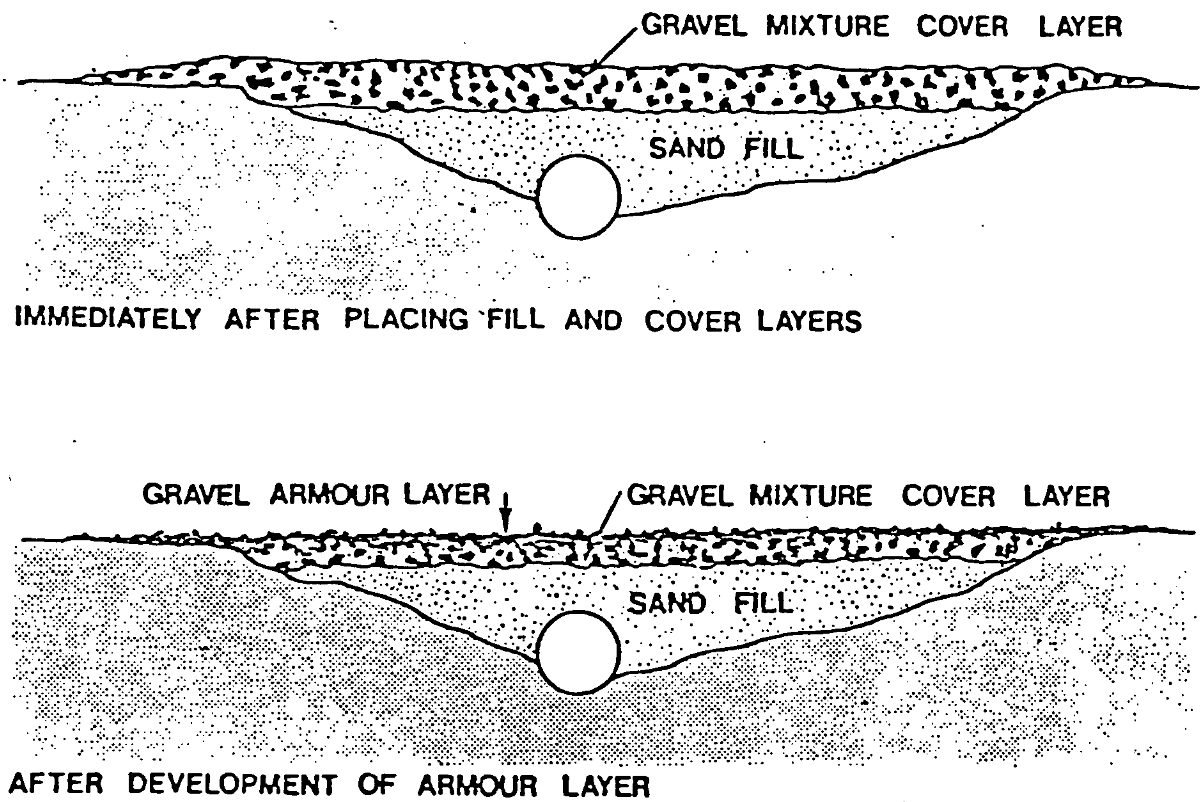


Figure 4.15 Sketch of Cover Layer on a Pipeline Trench (Roelofsen, 1980)

Problems associated with this method are that considerably more material than finally required must be placed. If an extreme storm is experienced before the natural armour layer is created by the normal environmental movements, then gross quantities of material may be lost.

Armour cover layers associated with either a graded stone filter or filter fabric are generally regarded as the most effective scour protection method in general application.

4.1.3.4 Filter Fabrics

Over the last fifteen years, a large selection of filter facbrics have been marketed. These are manufactured from a variety of synthetic polymers such as polypropolene, polyethylene, polyester, polyamide, etc. The fabrics are either woven or bonded into fibrous felt mats. The fabrics can have a density of fibre or weave which retain fine silts or consist of a network of ropes suitable for retaining gravels and rocks. (See Figure 4.16)

Filter fabrics have been used directly on the seabed adjacent to structures to provide erosion protection. The fabric is attached to the structure and weighted, anchored or staked in place.

In practice, adequate anchoring has proven difficult to achieve and its breakdown in extreme conditions could lead to rapid erosion-related failure.

The principal factors to be considered in choosing one of the many fabrics available are:

- the ability to relieve water pressure and yet retain the fine seabed soil particles,

- the ability to act as a filter without ultimately clogging with the finer materials,

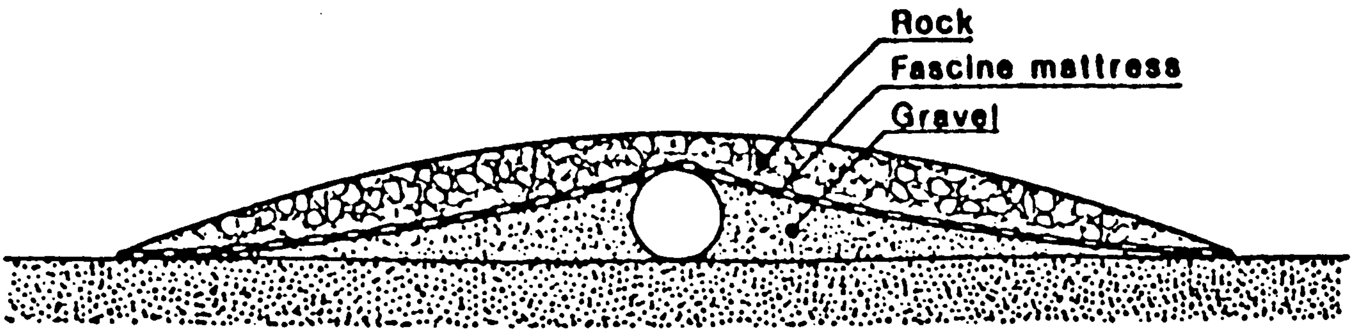


Figure 4.16 Rock-Covered Fascine Mattress (Herbich, 1981)

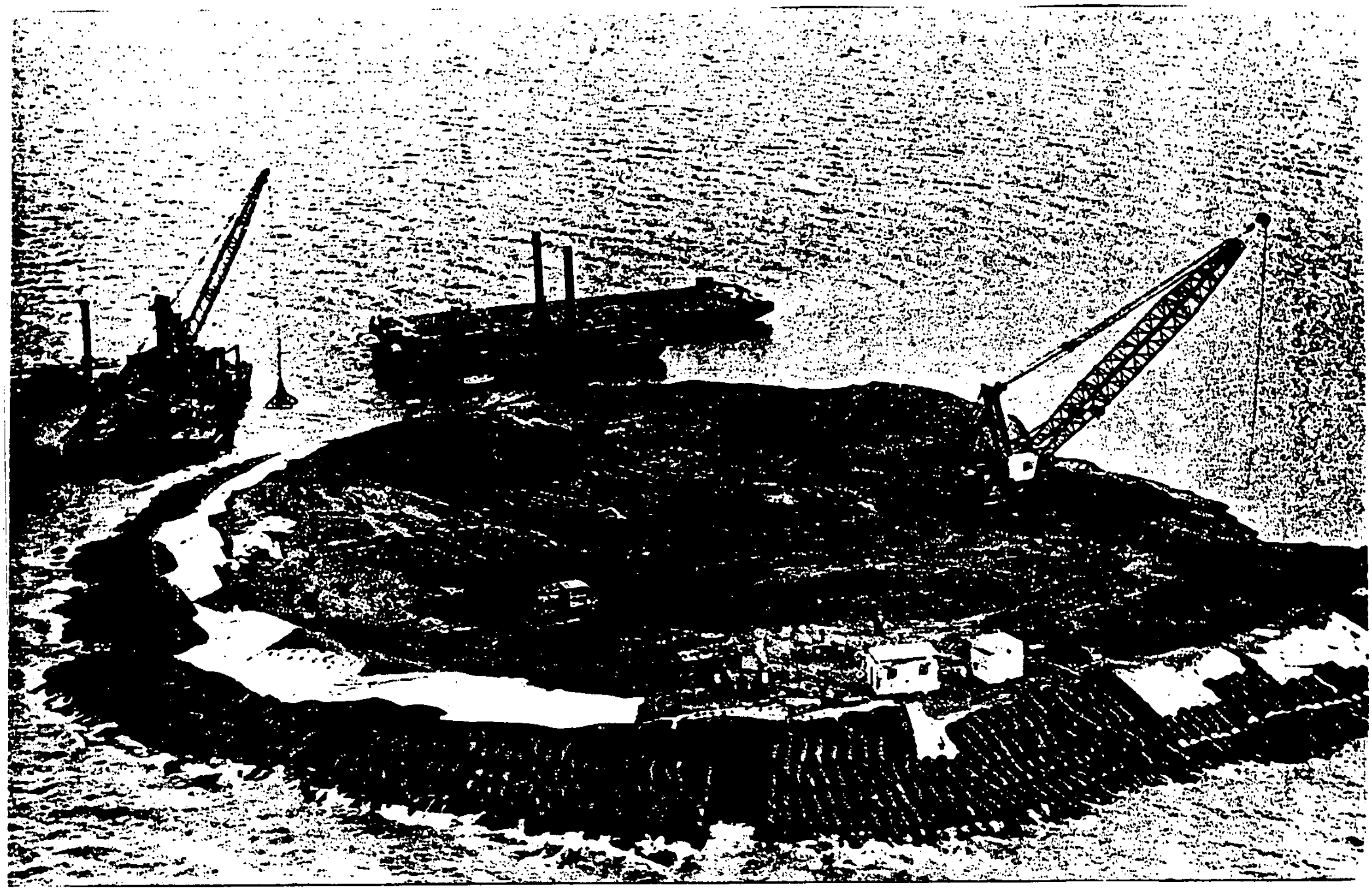


Figure 4.17 Sandbag Protection of an Artificial Island (Robertson, 1983)

- the ability to withstand tearing and puncturing during installation and operation and to withstand the environmental forces related to wave and current action and

- the ability to resist deterioration caused by chemical and marine life.

It has been found that with a correctly sized fabric, a natural filter established in soil immediatley behind the fabric with time. This reduces the permeability of the fabric until an equilibrium condition is reached.

Careful analysis should be conducted before using filter fabrics with soils having:

- liquid limit values greater than 40%

- Plasticity index values greater than 15%

Filter fabrics form the basis of fabricated concrete or sand filled bags and mats which are covered elsewhere.

The conventional method of satisfying all the above factors is by the use of a graded stone filter whose surface layer is made up of a stone size which will withstand environmental forces and whose underlying layers retain the finer particles of the preceding layers down to the seabed.

Disadvantages of armoured stone filters is the difficulty and cost in obtaining the appropriate stone sizes for the various layers at most locations and in placing the filters satisfactorily in deeper water. Filter fabrics are generally more available, cheaper and easier to place. However, the difficulties in anchoring and the potential for rapid failure are serious disadvantages with filter fabrics. A fairly common solution is the use of a filter fabric beneath a layer of gravel or rock which serves both to anchor the fabric and to provide protection against the environmental forces.

It has been suggested that the selection of "Equivalent Opening Size" (EOS) and "Percent Open Area" (POS) for the fabric be based on the following criteria:

a) Filter fabrics adjacent to granular materials containing 50% or less by weight fines, minus 200 micron or 0.2 mm material

$$\frac{\text{d15 size of soil (mm)}}{\text{EOS of filter fabric (mm)}} > 1$$

POS not to exceed 35%

b) Filter fabric adjacent to all other soils

-EOS no larger than the openings in the U.S. Standard sieve No. 70 (0.2 mm)

-POS not to exceed 25%

To reduce the chances of clogging, no fabric should be selected with a POS less than 4% or an EOS with openings smaller than a US Standards Sieve Size No. 100 (0.149 mm). When possible, it is preferable to select a fabric with openings as large as allowable by the criteria.

In soils where there is a higher proportion of fines, (D15 less than 0.149 mm) or a substantially uniform small particle size, some form of supplementary filter may be necessary. This may either be a blend of medium and coarse sand to be placed under the selected filter fabric in order to promote the development of natural graded filter, or a non-woven fabric to be applied under the selected filter fabric.

PERMEABLE - ARMOUR

4.1.3.5 Riprap

Riprap has been successful in protecting pipelines, pile supported structures and all types of gravity structures.

4.1.3.6 Sandbags

The use of sandbags both as a method of providing permanent protection and as a remedial measure following the occurence of scour has in the past been fairly widespread. Its use has not only been limited to underwater scour protection but also as slope protection in the breaking wave zone, for example, in artificial exploratory islands in the Beaufort Sea. (See Figure 4.17) Sandbags would generally be used in association with an underlying filter medium either of graded stone or fabric. Jute, filter fabric or woven plastic bags are filled with grout, sand, gravel or material available at the site and placed as accurately as possible in the area to be protected. In deeper waters where accurate placing is difficult, divers would then arrange the bags as required to give adequate protection. Sandbags of up to 2 cubic metres each have been deployed although conventionally normal 50 kg sacks would be used. The general concensus from literature appears to suggest that sandbags may be appropriate for temporary structures or urgent rememdial measures but not as a permanent solution.

Problems noted in the literature included:

- large numbers of sandbags are required for even minor scour holes,

- many bags are lost in the dumping process,

- bundles of bags laid around platform bases without a filter do little to change flow characteristics and scour continues beneath the sandbags,

- installaion and maintenance of the system is expensive in diver time and,

- once the integrity of a bag is lost its usefulness immediately disappears leading to possible rapid failure.

4.1.3.7 Flexible Concrete Block Mats

A variety of flexible precast concrete block mat systems have been marketed in recent years.

These generally consist of precast concrete blocks with cast-in voids which are threaded onto steel or polypropylene ropes in a grid to form a mat sized to be easily transported and placed by conventional trailers and cranes. They are generally laid on a fabric filter system and can be additionally secured with helical scour anchors. This alternative is of particular relevance on slopes in the breaking wave zone where uplift pressures can be significant. Blocks are generally of a complicated geometry which can provide an element of interlocking between units. The voids in the units assist in relieving water pressure uplift forces and also dissipating flow velocities. Units have been produced by the laying sequence along without ropes or cables but these require hand placing. Other precast block units have been bonded directly to a filter cloth base which facilitates placing and provides a filter medium in one operation.

Some manufacturers stress that the cables forming the mat grid should not be considered when calculating the stability of the protective layer which should be dependent on the weight and interlock of the units. The cables or ropes in this case are provided purely for transportation and placing.

An advantage of this method is the speed and accuracy with which the units can be placed. This could be of particular value where the periods available for construction are limited by extreme weather conditions or ice cover. Because of the relatively low volume of the units and the ease of transportation, the units could have advazntages in areas where supplies of quarried rock are unavailable. Where the above constraints do not apply, the system is likely to present an expensive alternative.

4.1.3.8 Gabion Units

Gabion type units consist of compartmented rectangular containers made of either steel rod, hexagonal wire or polypropylene strand mesh which are filled with stone. The steel is generally galvanized and can be plastic coated. The compartments can either be filled after placing or prefabricated prior to installation. The compartments and interior walls provide strength and help retain the shape both during filling, installation and in use. Stones used are generally cobble sized or larger and can be placed either by hand or mechanically. When made up into mats, the units are easily placed using purpose made spreader lifting units. In use, the gabions provide a flexible and permeable surface which asists in dissipating uplift pressures and erosive forces and also in allowing the units to conform to any scour-related settlements. When used in an erosion protection context, the units require either a graded stone or fabric under filter.

Problems have been encountered where the environment has a particularly abrasive quality, for example, when used as slope protection or groins in the surf zone where the beach consists of coarse sand or cobble.

4.1.3.9 Filter Fabric Mattresses

Articulated mortar filled mats are made up of a matrix of filter fabric compartments woven or stitched into a mattress configuration which are laid in position and filled with mortar by pumping. When hardened, the compartments in the mattress form individual mortar blocks which are joined by the fabric cell divisions. In some systems, steel or polypropylene ropes run through the mortor blocks in two directions to further bond the material together. The compartments can be created of a size to match the weight of block required to resist the erosive forces at the location to be protected. It is usual for the mattresses to be laid over a suitable graded stone or fabric filter.

The system would provide an economical, relatively easy to install and flexible solution in remote areas where suitable mortar sand is available but conventional quarried stone is absent. The basic mattresses, filter cloth and accessories have relatively low bulk and are easily transportable; the only other material required being cement.

4.1.3.10 Permeable Mastic-Asphalt Layer

The use of conventional bitumen bound materials for coastal scour and wave protection has been limited in the past by its impermeable nature. Where uplift pressures are significant, use of these materials can become uneconomical. Bitumarin have developed a method of producing a permeable bitumen bound material (FIXTONE) which resolves this problem. It is claimed that FIXTONE is a durable, flexible, permeable material which can reduce material quantities compared with conventional riprap protection.

FIXTONE is a mixture of single graded angled stone typically 20 to 40 mm and a premixed bituminous mortar. To achieve the optimum binding characteristics with maximum permeability, a coating of 1 to 2 mm is suggested. The mixture consists of the following:

80% angular 20 to 40 mm aggregate
20% mortar

The mortar is composed of the following:

60% sand
20% bitumen
20% filler

The percentage of bitumen in the mix overall is about 4% by weight.

The material is mixed in a conventional asphalt plant by a two-phase mixing system. Firstly, the mortar is prepared after which the stones are added at a temperature of 130 to 150oC.

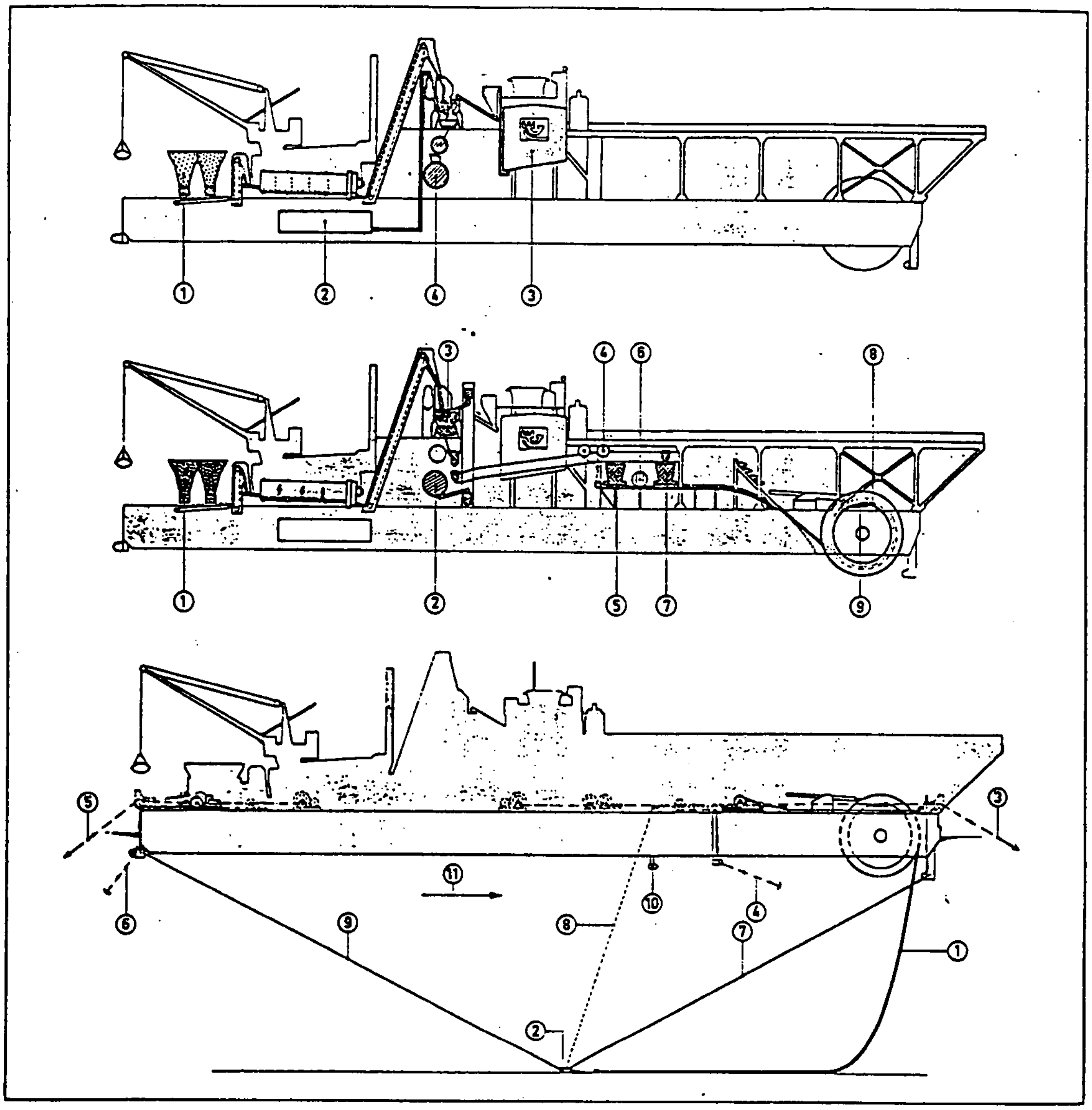
To prevent leaching of underlying fine material, a filter layer is required; typically a filter fabric.

The matrix can be laid in-situ in intertidal situations, by prefabricating mattresses on hand, incorporating wire mesh and lifting cables or, in deep water situations, by prefabricating aboard a purpose made vessel.

Such a vessel, the Bitumarin "Jan Heijmans", has the filter cloth, wire mesh and cables on drums which are drawn out and coated with the FIXTONE mixture produced by an on-board asphalt plant. The finished mattress is drawn onto a laying roller for subsequent placing. The prefabricated mattresses are typically 150 to 220 m long and 17 m wide (See Figure 4.18).

4.1.4 Remedial vs Preventative Measures

Many of the scour protection methods mentioned in the preceding sections may also be used for remedial work. The most common procedure



From top to bottom: the production of asphalt mastic, the manufacture of fixtone mattress and laying a fixtone mattress

1 Sand
2 Bitumen
3 Filler
4 Asphalt mastic

1 Crushed stone
2 Asphalt mastic
3 Fixtone
4 Filtercloth
5 1st layer of fixtone
6 Wire-mesh reinforcement
7 2nd layer of fixtone
8 Edge ballast
9 Tail beam

1 Fixtone mattress
2 Anchor beam
3 Bow cable
4 Forward side-cables
5 Stern cable
6 Rear side-cables
7 Anchor-beam lifting cables
8 Anchor-beam guide cables

Figure 4.18 Deployment of a Permeable Mastic Asphalt Layer by the "Jan Heijams" (Bitumarin) for repairing local scour around piled structures, foundations or pipelines is to dump dredged gravel into the scour hole. This method is relatively inexpensive and generally successful. Sand bags may also be used to fill scour holes but they are more difficult to deploy since they must be placed carefully to avoid damaging the bags. The Scour Brake technique (described in Section 4.1.2.2) has been used as a remedy for local scour around piles. Scour under jacket platforms and spanned sections of pipeline can also be remedied by artificial seaweed. However, this method does not provide an immediate solution and its success is not guaranteed. Furthermore, the seaweed in most cases must be deployed by divers which makes the method costly. Spanned sections of pipeline can also be repaired by the installation of grout filled bags to anchor and underpin the pipeline. Where undermining of a foundation occurs the only remedial measure is grouting.

While may be other types of remedial measures however, the most common techniques have been described above. The use of dumped gravel is the only technique which does not utilize divers, this combined with a modestly encouraging rate of success render it the most cost-effective method.

4.2 Preferred and/or Proven Techniques According to Type of Structure

Scour protection methods are briefly summarized according to type of structure in this section. (For greater detail see Section 4.1). Also, Table 4.2 gives a summary of scour protection methods by structure type in matrix form.

4.2.1 Multimember Structures

4.2.1.1 Pile Supported Structures

Pile supported structures such as jacket platforms have been protected

Table 4.2 Summary of Scour Protection Methods

SCOUR PROTECTION METHODS

METHOD \ TYPE	MULTI-MEMBER STRUCTURES				PIPELINES			GRAVITY STRUCTURES			ISLANDS	
	INDIVIDUAL PILES	PILE SUPPORTED	JACK-UPS	OTHER TYPES	PIPES	VALVE CHAMBERS	SHORELINE INTERFACE	SURFACE PENETRATING STRUCTURES	SUBMERGED GRAVITY STRUCTURES	FOOTINGS/ ISOLATED SEAFLOOR	SACRIFICIAL ISLANDS	CAISSON RETAINED ISLANDS
DESIGN ALLOWANCES												
1. Gravity Structure Skirts								x/o				x
2. Flexible Aprons								x/o				x
3. Pipeline Burial					x/o		x/o					
4. Additional Pile Length	x/o	x/o	x/o									
5. Sacrificial Protection											x/o	x/o
ACTIVE SCOUR REDUCING METHODS												
1. Artificial Seaweed	x/o	x/o			x/o		o	x/o	x	x		
2. Others	X	X/O										
PASSIVE SCOUR REDUCING METHODS (Impermeable)												
1. Soil Stabilization								x	x		x	x
2. Mastic Asphalt								x/o	x		x	x
3. Concrete Aprons												
(Permeable)												
1. Stone Filters	x/o	x/o			x/o	x	x/o	x/o	x	x	x/o	x/o
2. Fabric Filters	x	x			x	x	x	x/o	x	x	x/o	x/o
3. Riprap	x/o	x/o			x/o	x	x/o	x/o	x	x	x/o	x/o
4. Sandbags	x/o	x/o	x/o		x/o	x		x/o	x	x/o	x/o	x/o
5. Concrete Block Mats						x		x	x	x	x/o	x/o
6. Gabion Units						x		x	x	x	x	x
7. Fabric Mattresses						x		x	x	x	x	x
8. Mastic Asphalt								x/o	x		x	x
9. Scrap Tires	X/O	X/O										

x Potential Application
o Application Noted in Literature

from scour by several methods, primarily by dumped gravel or by seaweed in either the upright or hanging position. The most common method, owing to its cost-effectiveness, is the use of dumped gravel. As more experience is gained, the use of the `Scour Brake' developed by Loer (1983) and described in Section 4.1.2.2 could also prove to be a useful technique for averting local scour around piles.

4.2.1.2 Jack-Ups

The most common method for jack-ups is to increase the penetration of the legs enough so that stability of the structure will be maintained even under maximum design scour. Many jack-ups are now equipped with airlift systems to allow for greater leg penetration. Dumped gravel may also be used to inhibit scour around jack-up legs.

4.2.1.3 Other

In some cases semi-submersible rigs are set on the seabed. The only reported method of scour protection is the installation of nylon mesh mats over the pontoons of the rig, while they provided effective protection in the short term, excessive movement during storms cast doubt on the success of long term performance. Dumped gravel mattresses placed before installation of a rig would probably be better as a protection technique.

4.2.2 Pipelines

4.2.2.1 Pipes

Pipes are usually protected by armoured cover layers. A common and proven method is the inverse filter system with a rock armour layer overlying a gravel layer which surrounds the pipe. Often a synthetic mattress is placed between the gravel or sand layer and the rock layer.

4.2.2.2 Valve Chambers

The only example in the literature referring to scour around pipeline valve assemblies concerns a laboratory test at 1.20 scale. A textile mattress or apron with sewn-in bags of sand for ballast was found satisfactory in this case.

4.2.2.3 Shoreline Interface

Pipes crossing the surf zone must be buried in a trench below the deepest profile limit expected for the design life.

4.2.3 Gravity Structures

4.2.3.1 Surface Penetrating Structures

The most widely used method of protection is toe protection with rock or gravel aprons. Flexible concrete or synthetic weighted mats attached to the toe of the structure have also been used with success. A foundation skirt is also included in most designs to prevent undermining.

4.2.3.2 Submerged Gravity Structures

The protection of these structures is similar to that described in Section 4.2.3.1.

4.2.3.3 Footings and Isolated Seafloor Structures

Footings may be protected from scour by gravel weighted filter cloth or by dumped gravel.

4.2.4 Islands

4.2.4.1 Sacrificial Islands

In most cases no protection is provided and scour must be periodically infilled by a dredge.

4.2.4.2 Caisson Retained Islands

Gravel and rock aprons are used at the toe of the structure.

5. MEASUREMENT, ESTIMATION AND DESIGN PRACTICES

5.1. Pre-Construction Environmental Data Collection

There are no universally accepted standards for the quality and extent of environmental data that should be collected prior to the design of a marine structure. Clearly the proposed route or location of a structure with respect to exposure to waves, intensity of tidal currents, occurence of storm surge and mobility of bottom sediments should have been a large influence on the amount of data required.

More often than not, the volume of environmental data collected for a particular development will be dictated by the time available rather than by the needs of the project. In areas where there has already been extensive activity, such as in the North Sea and the Beaufort Sea, the data bank grows with time to the extent that less general data collection need be carried out than would be required in an area devoid of any previous investigations.

Waves

For waves it is common-place to find that the collection of one year of data is specified as a workable and practical minimum. However, past experience and analysis has shown that the wave climate for a particular location may vary considerably from year to year and that ten or more years of data would be more appropriate for design conditions to be established. Even then the confidence limits for estimates of extreme events such as the 1:50 or 1:100 year event is not particulary high. (Baird and Hall 1980).

In a very few areas a lengthy record may be available from long established recording stations, for example, in Logy Bay, near St. John's Newfoundland, but in most circumstances this is not the case and initiation of a decade of recording is not a practical proposition. However, wind data is usually far more readily available over long term periods and this may be used to hindcast wave conditions over the equivalent time frame.

Modern wave hindcasting methods vary quite considerably in their complexity and data input requirements. For parametric methods it will usually be necessary to pre-process the wind data to a) infill periods when data is missing due to instrument failure and b) adjust wind speeds to represent clear overwater velocities. The latter is to allow for the position of the instrument which is usually affected by sheltering and roughness of the surrounding terrain. Parametric methods of wave hindcasting can be quite accurate for situations where fetch lengths are smaller than the dimensions of typical weather systems and well defined such as in enclosed or semi-enclosed areas of water. It is also implicit that the wind field is sensibly constant spatially and there are a number of different ways in which variations in time, leading to growth and decay, may be handled (Fleming, Philpott, Pinchin, 1984). Parametric methods are highly cost effective and are suitable for reproducing long time-series of wave data from corresponding wind records. They are therefore suitable for synthesizing long-term wave climatic statistics.

The more sophisticated methods of wave hindcasting are represented by the two-dimensional spectral hindcasting models of which there are a number of different versions, but only a few which incorporate the effects of shallow water; refraction and shoaling, for example those of DHI and Resio. These models are suitable for consideration of individual storm events or relatively short sequences of such events. They are usually too costly for modelling long time series and are usually confined to studies of specific extreme events. For the application of these models well defined boundary conditions are also required. In addition the pressure field over the entire area to be modelled must be known in reasonable detail both spatially and in time.

The pressure field may be used to derive the wind field using a suitable atmospheric model. Alternatively the wind field may be applied directly but it must be defined in considerable detail in order to drive the model. Such detailed data is not usually available and some sort of intermediate processing is inevitably required.

In the case of parametric methods it is possible to convert a long term wind data set into an equivalent wave data set and then to perform a statistical analysis on the result in order to obtain extreme values. In the case of two dimensional spectral hindcasting techniques such processing would be extremely costly because the volume of data and processing required would be so great. The alternative approach is to statistically analyze the pressure or wind records to provide the extreme storm conditions from which the wave conditions may then be derived. Clearly, if the simulation models are correctly formulated and the statistics are consistent, these amount to the same thing. Since the processes involved are non-linear this is not necessarily so.

Special mention should be made of the problem of defining extreme wave conditions in shallow water or sites that are partially protected by shallow banks or islands. Standard extreme value analyses do not ordinarily incorporate consideration for shallow water conditions. In these circumstances a much more detailed study of wave propagation patterns in the area of interest is required either by using a two-dimentional shallow water model as described above or by other shallow water wave modelling techniques.

Currents

Current measurements are usually collected as a prerequisite of design of marine works, but the period of time over which they are collected is usually limited to only a few weeks. If currents are predominantly tidal, a two week measurement period provides basic data with respect to spring and neap tide cycles.

In areas where storm surges enhance the current significantly or create the infrequent, but dominant design condition it is a matter of chance as to whether such conditions are actually monitored.

Currents are often only measured at one or two levels in the water column and some theoretical velocity distribution is used to estimate velocities at other depths. However, the need to take measurements at a number of positions in the water column has become accepted and efforts to collect detailed profile data are made. With regard to sediment transport problems it is clearly the near bottom current velocity that is the most relevant to the problem. Graff (1984) has suggested a framework of analysis suitable for practical engineering. It involves examination of the components at times bearing a fixed relationship to times of high water at the nearest port. This information can then be used to provide a principal flow direction which can be fitted to a simple sine wave to account for spring/neap variations. A linear relationship between reference tidal amplitude and the current is assumed and the analysis of a relatively short length of current record, then provides a means for extrapolating tidal stream vectors according to the tide levels predicted at the reference port. The method may also be used to investigate storm surges and simple prediction of extreme flows. This method appears to provide a good framework for analysis of current data in areas where the tide curves and their spatial variations can be well defined at reference ports. As such it has been developed in the North Sea which is well surrounded by reference ports. This is not the case in the Scotian Shelf so that application of the method to that area may be difficult.

The use of numerical models to simulate current flows has become fairly widespread practice. These models are generally restricted to vertically integrated two-dimensional flow which must exclude considerations of stratified flow, wind induced surface currents etc. Three dimensional models are now being developed, but it is likely to be some time before they will be generally applicable. There are still a number of fundamental problems with regard to turbulent exchange that need to be resolved.

Given well defined boundary conditions the two dimensional flow models are capable of providing some useful information with respect to tides, surge and wind induced currents. As for waves the extreme events are examined by first devising the extreme storm events and using them to drive the model.

Bottom Sediment

No pre-design studies would be complete without at least some rudimentary bottom sediment sampling investigations. However, beyond that the level of studies have varied considerably and may include both geophysical surveying and side scan sonar investigations. Bottom tracers have been used to estimate rates of sediment transport but the value of this type of exercise is highly dependent on achieving a high rate of recovery. Also the results are limited to the environmental conditions over the detection period which may be limited.

In areas where active sand waves have been thought to exist, detection of their movement may be attempted by repeated precision surveying. However, the movement of those features is usually only of the order of a few metres a year, depending on the water depth. It is also possible that sand may be moribund only to be mobilised under rare extreme events. Installation of self recording instruments can resolve that dilemma. Larger scale features such as the banks that are found in the North Sea can only be detected over several decades and periodic sounding charts over similar orders of time are thus required. Consequently it is a matter of chance as to whether suitable data does exist.

Pre-design studies related to shore crossings have to consider the seasonal variation of the beach profile and the long term erosion pattern. As such the monitoring that is required is similar to that for any other coastal dynamics study. The seasonal variation of wave climate, particles size grading and repeated shore surveys are required as basic data.

To conclude, pre-design investigations can often be quite meagre in relation to the volume of capital works that are envisaged and it is only in relatively recent times that attempts have been made to deal with the potential environmental problems more comprehensively and practice still lags well behind. For example, exploratory drilling in Hudson Bay had been in progress for a decade, yet prior to commencing the work, no wave measurements were made.

5.2 Experimental Estimation and Design

5.2.1 Physical Scale Models

Physical scale models represent the area in which there has been most activity with attempting to predict the extent of scour for particular structures. This has come about largely due to the considerable problems associated with theoretical treatment of the problem (See Section 5.3).

In common with other sediment transport problems scour phenomena in unidirectional flow have received rather more attention than in oscillatory flow. Also much of the work that has been carried out has been directed at the design of bridge piers in rivers. This has not caused undue concern as it is widely believed that scour depths in oscillatory or combined wave and current regimes should be less than or equal to the equivalent unidirectional flow. However, this view may need careful qualification.

There are no accepted standards for carrying out physical model studies of scour phemonena. Pragmatically, experimental design is governed by the model facilities that happen to be available. It follows that a fairly wide range of scales and sediment sizes have been used and there is an impression of a fairly ad-hoc approach to such modelling procedures. Rather than go through the numerous publications that have been uncovered it is more useful to report the major works which incorporate the results of others.

The greatest volume of results apply to local scour around cylindrical piers. Unfortunately relatively few experimental series are of a totally general nature with a range of variance of all of the variables involved. Breusers et al (1977) provide a good summary of the subject to that date. The vortex mechanism that gives rise to different scour patterns around piles has been described at length in Section 3.1. The eddy structure may be composed of any combination of the three basic systems of the horeshore-vertex system, the wake-vortex system, or the trailing-vortex system. The occurrence depends on the geometry of the pier and the free stream conditions.

Breusers et al (1977) summarise many of the most interesting references. On the basis of all of the experimental data collected, dimensional analysis suggested a relation of the form

$$S_u = f\left(\frac{\bar{v}}{\bar{v}_c}, \frac{d_{50}}{D}, \frac{h}{D}\right)$$

where S_u is the equilibrium scour depth, D is the width of obstruction, $\bar{v}_c$ is the critical mean freestream velocity for sediment movement, d_{50} is the bed material mean size and h is the water depth. This expression neglects the influence of slope of obstruction, Froude number, bed material and gradation. It was further concluded that the scour depth may, on the basis of experimental results be described by a function of the form

$$S_u = f\left(\frac{\bar{v}}{\bar{v}_c}, \frac{h}{D}, \text{angle of attack}, \text{shape}\right)$$

and specific form was suggested as

$$S_u = f_1\left(\frac{\bar{v}}{\bar{v}_c}\right) \cdot \left(2.0\tanh\left(\frac{h}{D}\right)\right) \cdot f_2(\text{shape}) \cdot f_3\left(\alpha, \frac{L}{D}\right)$$

in which

$$f_1\left(\frac{\bar{v}}{\bar{v}_c}\right) = 0 \quad \text{for} \quad \frac{\bar{v}}{\bar{v}_c} \le 0.5$$

$$= \left(2\left(\frac{\bar{v}}{\bar{v}_c}\right)-1\right) \quad \text{for} \quad 0.5 \le \frac{\bar{v}}{\bar{v}_c} \le 1.0$$

$$= 1.0 \quad \text{for} \quad \frac{\bar{v}}{\bar{v}_c} \ge 1.0$$

f_2 (shape) = 1.0 for circular rounded piers
= 0.75 for stream-lined shapes
= 1.3 for rectangular piers

$f_3\left(\alpha, \frac{L}{D}\right)$ as shown in Figure 5.1

It may be noted that the influence of particle size does not appear in the final expressions. The lack of influence of particle size is a fairly common feature among reports of results of physical model tests. This appears to be at least in part due to the limited range of sizes used in such tests. Logically there must become a point, both for small and large sediment sizes at which some influence begins to emerge more strongly.

Clark et al (1982) consider some of the problems of modelling scour processes around slender piles in waves and currents. They provide a comparison of research on relationships between pier diameter and maximum scour depth alone as reproduced in Figure 5.2. Results for unidirectional flow give ultimate scour depths varying between 1.0 and 2.3 times the pile diameter. One experiment from Wells and Sorenson (1970) was for waves only and in which the maximum scour depth was only 0.3 times the pile diameter, a factor of between 3 and 8 less than the unidirectional equivalent.

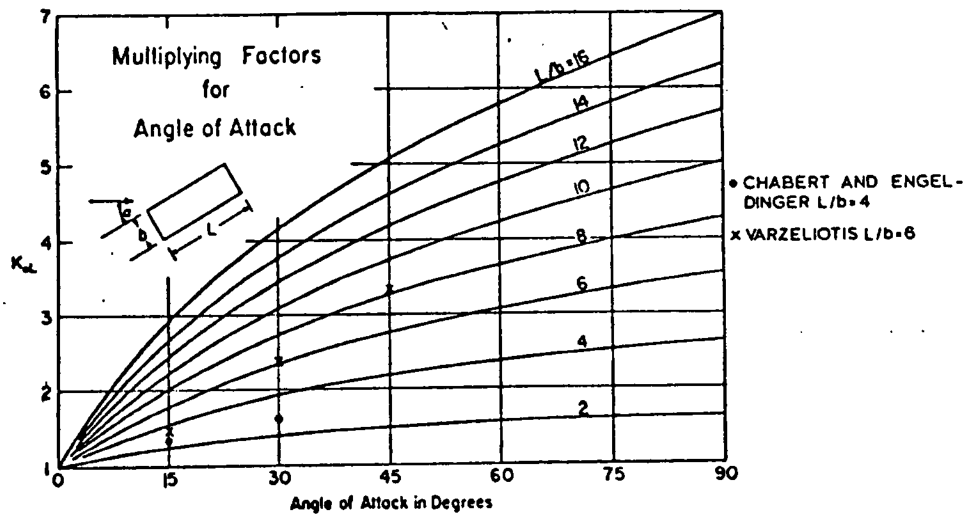


Figure 5.1 Correction Factor for Angle of Attack on a Rectangular Pier (Note L/b is equivalent to L/D) (Breusers, 1977).

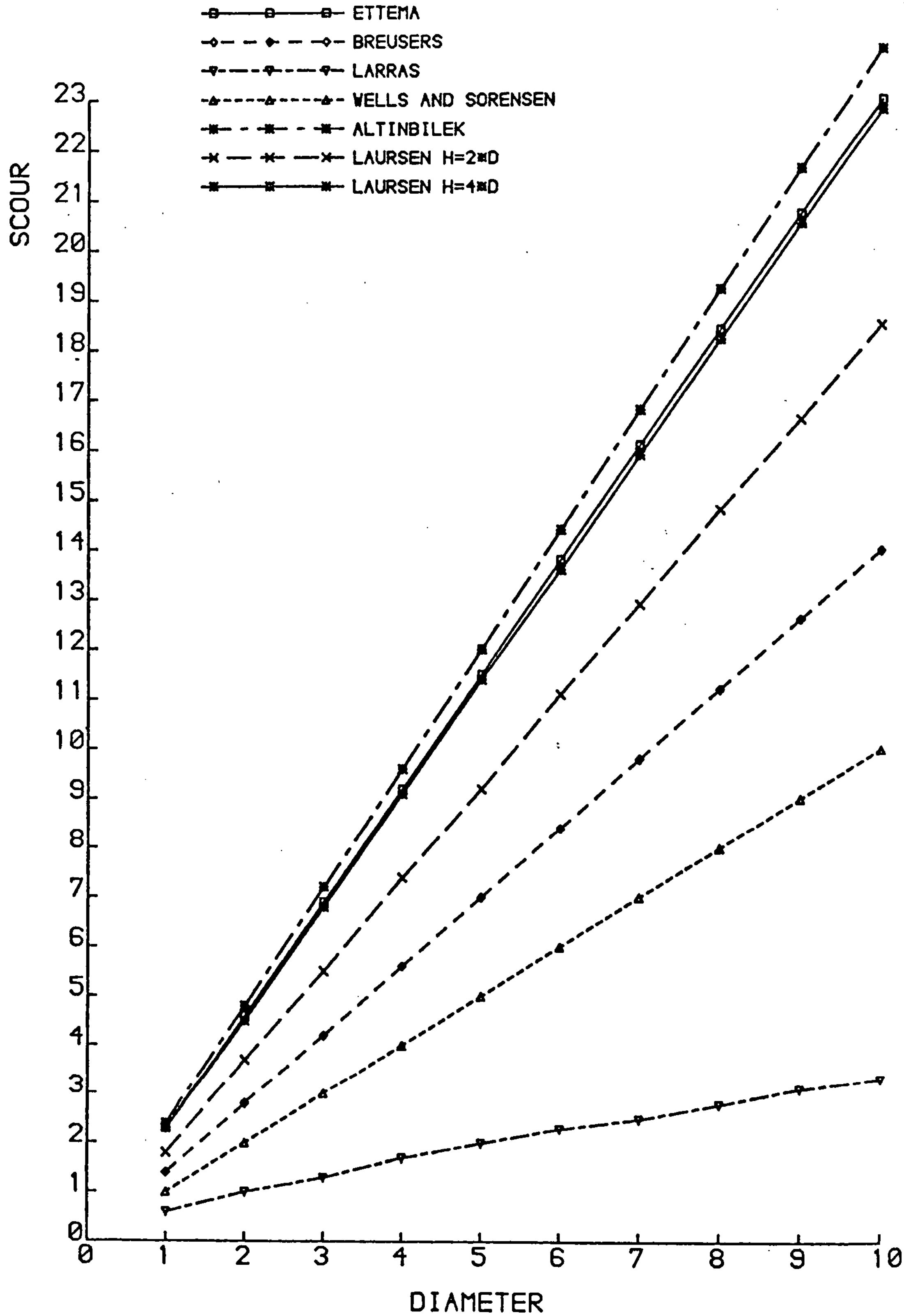


Figure 5.2 Comparison of Some Previous Research on the Relationship Between Pier Diameter and Maximum Scour Depth (Clark et al, 1982)

They also came to the following conclusions concerning scaling laws.

" (a) Undistorted models must be used (geometric similarity).

(b) The model bed material must have the same shape of the grain distribution curve, and the same compactness and natural slope as the prototype material.

(c) Under conditions of fully developed turbulent flow in the model the Froude Law ensures the similarity of the water levels, structure of the velocity field and its dynamic effect on the bed; in this case the flow is independent of the the viscosity ie.

$$Re_m > Re_{sq}$$

where Re_m is the model Reynolds number and Resq

is given by

$$Re_{sq} = \frac{63R}{k\sqrt{\lambda_r}}$$

is the friction head loss coefficient for

the bed based on the equation

(d) The equation relating the scales of the sediment size, specific gravity, sediment discharge, and time of scour formation with the model geometric scale are the same as those applicable to similarity of undistorted river models.

(e) Clark et al suggest that lower density sediment should be used to avoid particle sizes less than 0.05 mm.

(f) A distortion of the relative roughness of the bed for a short section of the channel (downstream of the basin) has practically no influence on the development and size of the scour."

Clarke et al. (1982) also observe that for design of experimental work the following points should be considered:

(1) The minimum limiting condition for the depth/pile diameter ratio should be 2.0; otherwise the characteristic horse-shoe vortex and flow structure cannot develop sufficiently and the scour pattern will not develop properly

(ii) The minimum limiting condition for the ratio of flume width to pile diameter is about 5 and if it is in the range between 5 and 7 the results should be treated with some caution as far as the shape and extent of scour hole are concerned".

Imberger et al (1982) carried out flume experiments on scour around circular piles and concluded that the parameter u_*/u_{*c} was probably the most important for the determination of the depth of the scour. In order to test this hypothesis they gathered all available data from the literature for circular cylinder scour tests in models and reduced the results to a single plot. Figure 5.3 shows the resulting variation with the data plotted being obtained directly from the publications of Breusers et al. (1977), Charbert and Engeldinger (1956), Jain and Fischer (1980) Qadar (1981), Shen et al (1966) as well as Imberger et al (1982). The sediment types varied from calcareous sediment with a D_{50} = 0.06 mm through sands varying in diameter from 0.2 mm up to a maximum diameter of 3.0 mm/sec. The corresponding range of u_{*c} was consequently between 12 mm/sec and 35 mm/sec. The range of cylindrical pile diameters covered by the data was 25 mm to 200 mm.

Imberger et al (1982) only presented data for which the ratio of flume depth to cylinder diameter was greater than 3. They observed that a very definite trend about the line of best fit and the degree of scatter is not unusually severe for sediment transport or scour experiments. They therefore suggested that the most important correlation is demonstrated in Figure 5.3 and the influence of different void ratios, soil structure and scour on the resistance is all provided through the value of the critical shear velocity.

Niedoroda et al (1981) provide a good descriptive account of the physics of scour in the ocean environment and in particular the differences between scour in unidirectional flow and combinations of waves and currents.

Their conclusions were as follows:

" 1. The mechanisms of scour around discrete ocean structures included the generation of the horsehoe vortex due to the stagnation pressure gradient associated with the relatively thick boundary layer of steady or slowly varying ocean current, convective accelerations of the main flow in both steady and oscillating flow, wake turbulence, and bottom boundary layer separation associated with waves.

2. The mechanisms for scour hole development due solely to waves is substantially different than the mechanisms for scour hole development due to steady or slowly varying currents.

3. The mechanisms for scour hole development due to the combined effects of both waves and currents are complex.

   In the case of a strong current and weak wave orbital flow, the development of the scour hole will be dominated by a horseshoe vortex with a pulsing intensity. The scour hole will develop faster than in a corresponding steady current case due to the excess bottom fluid shear stress developed by the wave orbital boundary layer. In the case of a weak

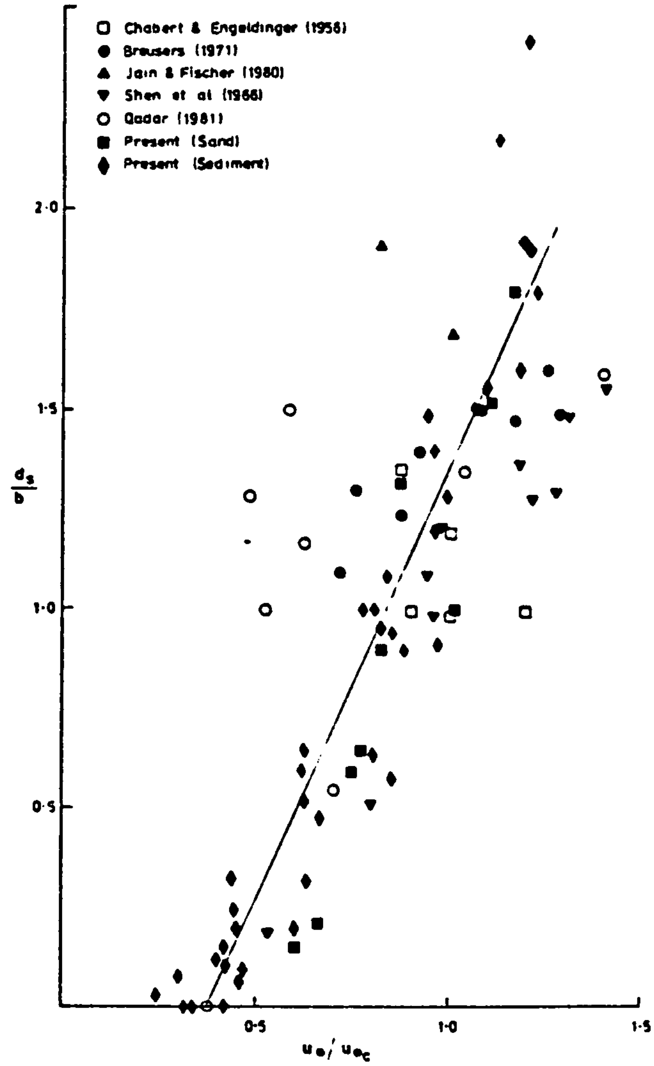


Figure 5.3 Dimensionless Plot of Scour Depth Against the Shear Velocity for Cylinders Showing the Line of Best fit from Present Results (b is the Pile Diameter) (Imberger et al., 1982).

current and a strong wave orbital flow, the scour hole will have a shape similar to that developed by a steady current. This feature will develop more rapidly than it would in the present of a steady current alone due to the additional bottom fluid stresses developed by the orbital boundary layer. However, the equilibrium depth of the scour hole can be less than that associated with a steady current whose magnitude is equal to the sum of the weak current and the maximum orbital velocity.
When the magnitude of the wave orbital flow and the steady current is nearly the same, the situation is difficult to evaluate. The scour hole shape and size will generally be controlled by the horeshoe vertex related to the steady current component. Erosion will progress rapidly due to the waves. However, it is not possible to generalize whether the scale and intensity of the pulsing horseshoe vortex will be larger or smaller than the corresponding steady current case.

4. The size and depth of the scour hole produced by the combined effect of both waves and currents acting on a flow obstruction which is large with respect to the oscillation amlitute (Keulegan-Carpenter number is small) will be considerably smaller than that predicted from a small cylinder, except when the steady current is larger than the maximum wave orbital velocity. This is caused by the fact that the horseshoe vortex will not wrap completely around the obstruction before the flow reverses due to the wave orbital velocity."

The above demonstrates that whilst the processes in waves and currents can be markedly different from pure unidirectional flow the potential for scour or the equilibrium scour depth is generally less when the bottom shear stress for the two cases is equivalent. It also points out the fact that once the obstruction becomes large compared to the amplitude of oscillation results for so called "slender piles" cannot apply.

An interesting series of model tests was carried out by the Hydraulics Research Station, Wallingford (Rance, 1980). A number of idealised shapes were investigated with varying magnitudes of background currents. The wave conditions remained constant and lightweight material (bakelite) was used so the results present little of general applicability. The physical model experiments were carried out in a wave tank nominally 16 m long and 3 m wide with a still water depth of 0.5 m. The nominal Froude scale was 1:100 with model wave periods ranging between 1.0 and 2.0 seconds. The bakelite material had a median diameter in the range 0.39 to 0.83 mm with a specific gravity of 1.4. A series of tests was done for all of the shapes with a single wave period of 1.25 seconds both with and without a superimposed background current. The ratio of peak oscillatory flow velocity to unidirectional flow velocity was about 2.5 to 1 so that wave action was predominant. The results of these experiments are shown in Figures 5.4 through to Figure 5.8. These show areas of different depths of scour in terms of object diameter.

Physical model tests relating to scour processes around large objects have been carried out for pre-design studies of artificial islands to be constructed in the Beaufort Sea. Moir et al (1984) have carried out extensive tests at 1:25 scale on a caisson retain island which consisted of an octogonal caisson sitting on top of a submerged mount. The bed material used in the physical model was of similar size as to be used in the prototype (0.3 mm D50) and tests for gravel protection were also carried out.

The other type of artificial island is the pure sand or gravel type. Extensive tests at different model scales have been carried out at Queen's University, Kingston. These have used sand with grain diameters of D50, 0.105, 0.18 and 0.56 mm. The results of these tests are summarized by Kamphuis and Nairn (1984) who also suggest the basis for suitable scaling relationships.

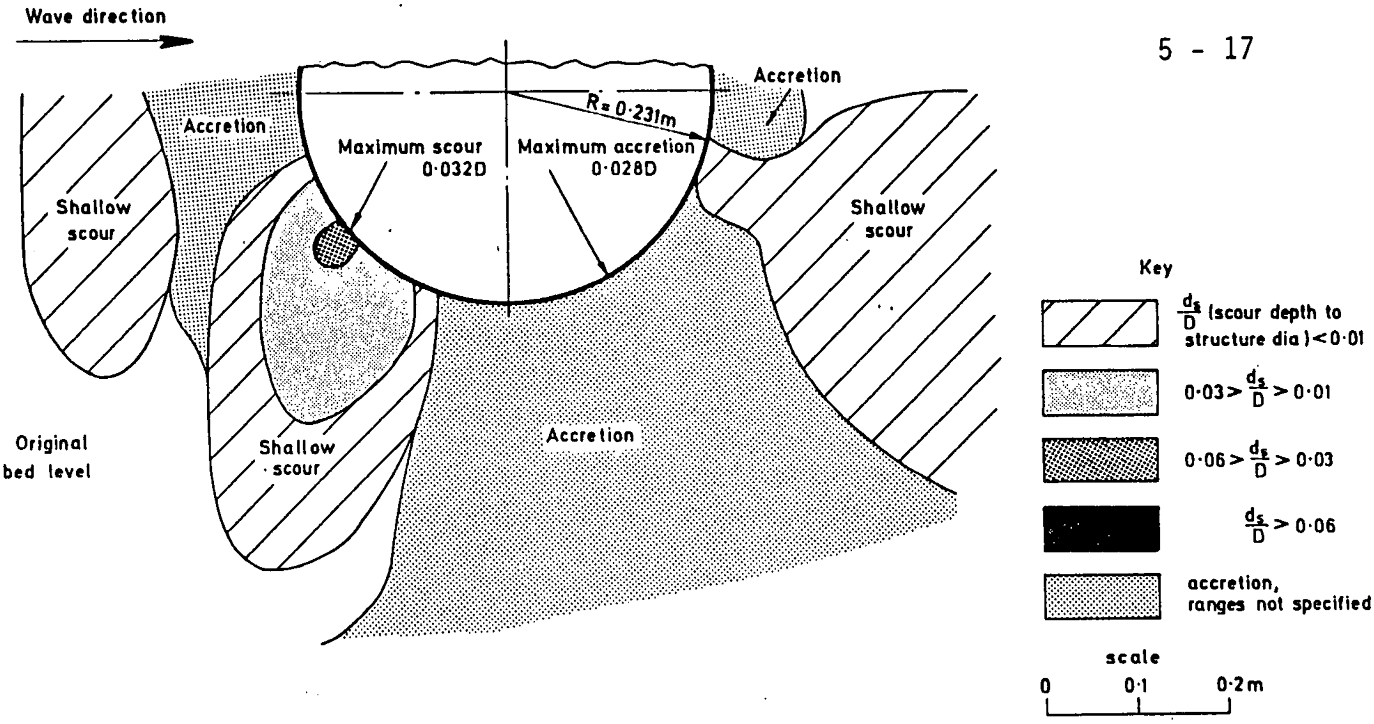


Figure 5.4a The Physical Model of a Circular Structure Showing Bed Topography Under Wave Action (Rance, 1980)

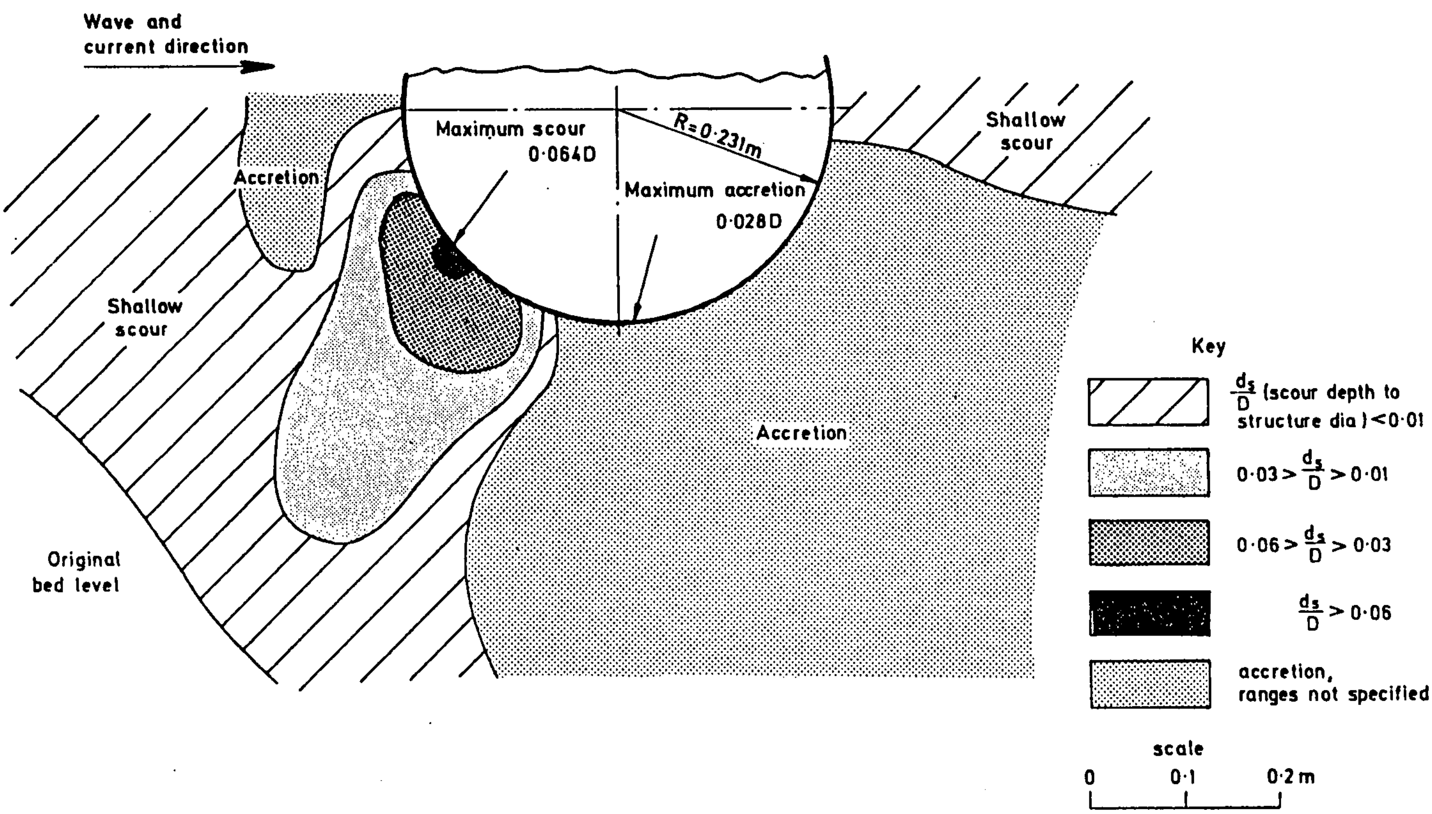


FIGURE 5.4b The Physical Model of a Circular Structure Showing Bed Topography Under Wave and Current Action (Rance, 1980)

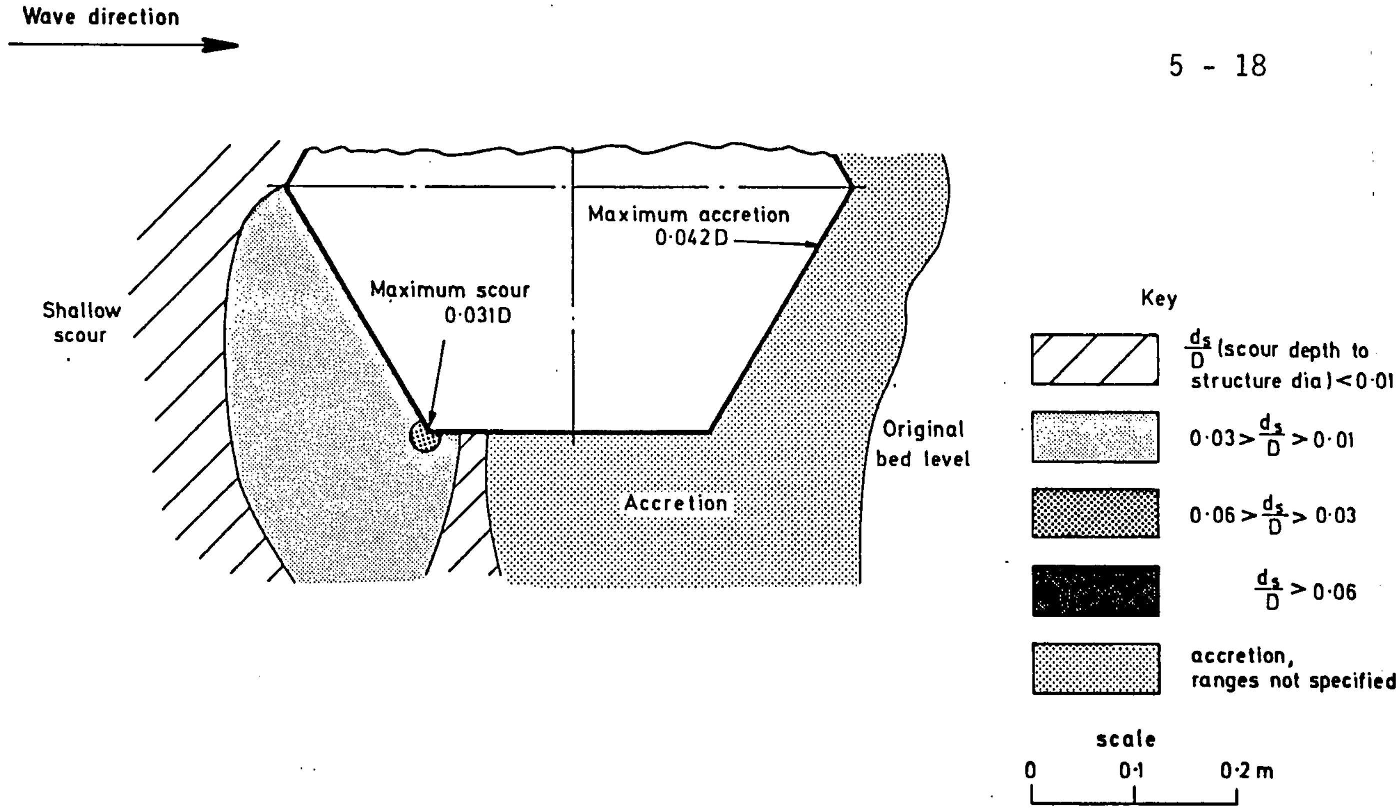


Figure 5.5a The Physical Model of a Hexagonal Structure with Leading Corner Showing Bed Topography Under Wave Action (Rance, 1980)

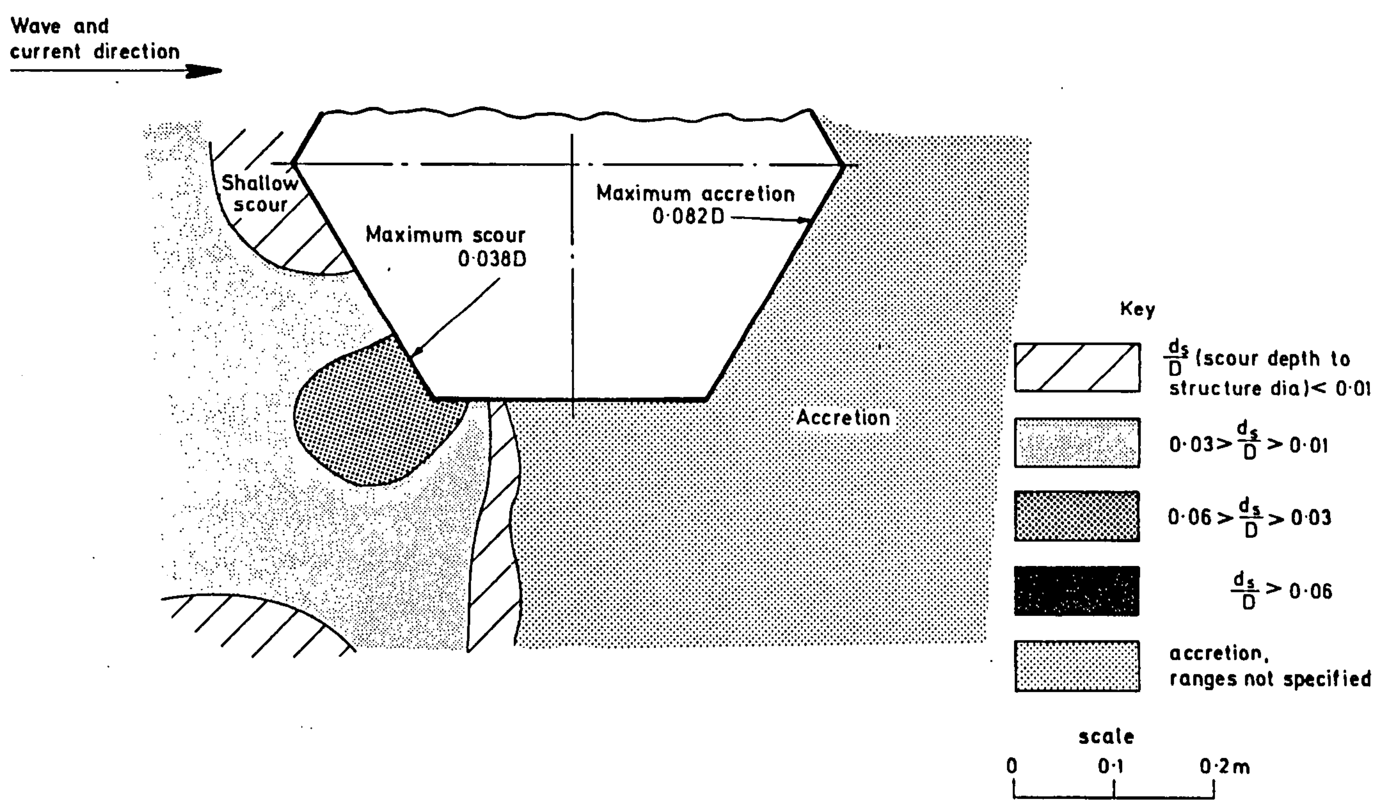


Figure 5.5b The Physical Model of a Hexagonal Structure with Leading Corner Showing Bed Topography Under Wave and Current Action (Rance, 1980)

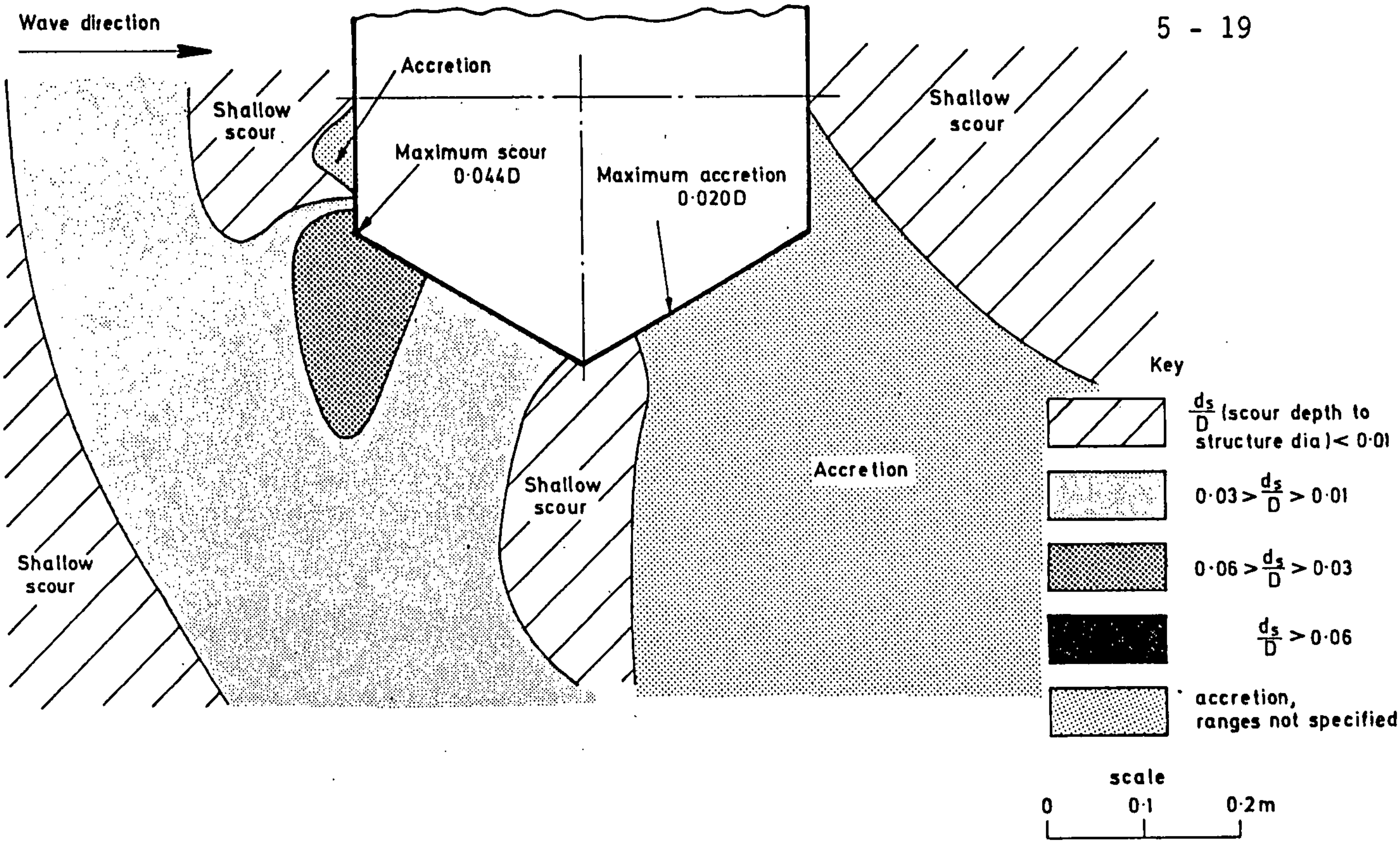


Figure 5.6a The Physical Model of a Hexagonal Structure with Leading Face Showing Bed Topography Under Wave Action (Rance, 1980)

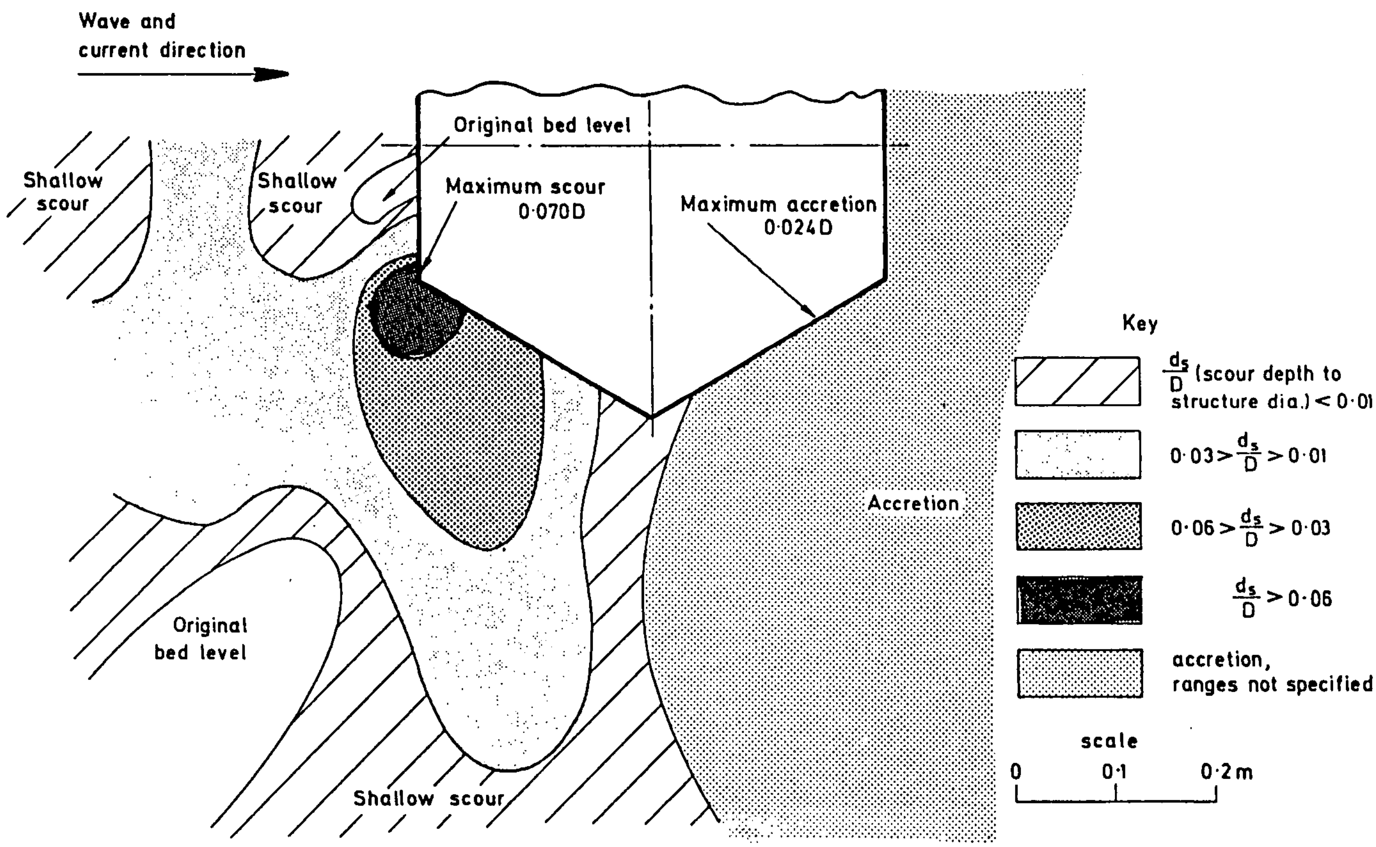


Figure 5.6b The Physical Model of a Hexagonal Structure with Leading Face Showing Bed Topography Under Wave and Current Action (Rance, 1980)

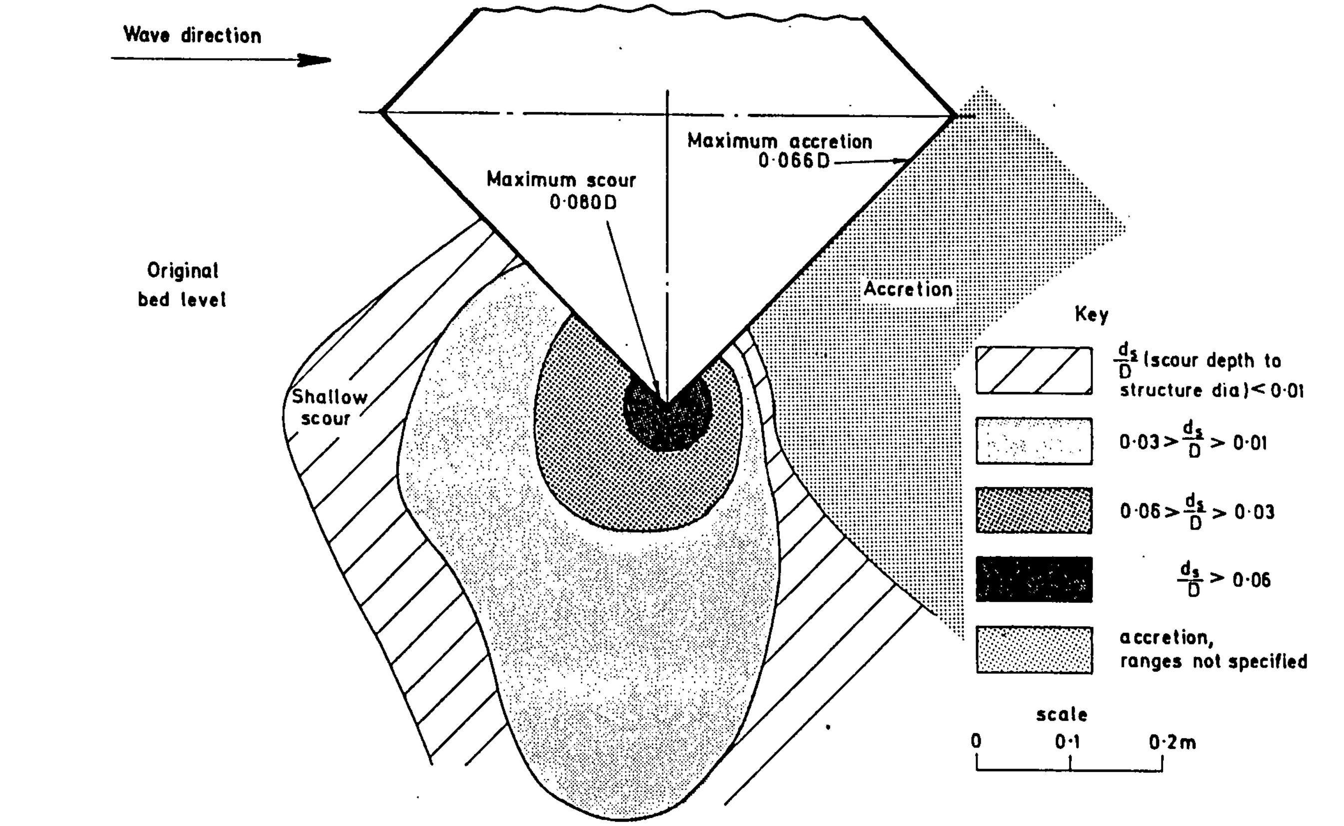


Figure 5.7a The Physical Model of a Square Structure with Leading Corner Showing Bed Topography Under Wave Action (Rance, 1980)

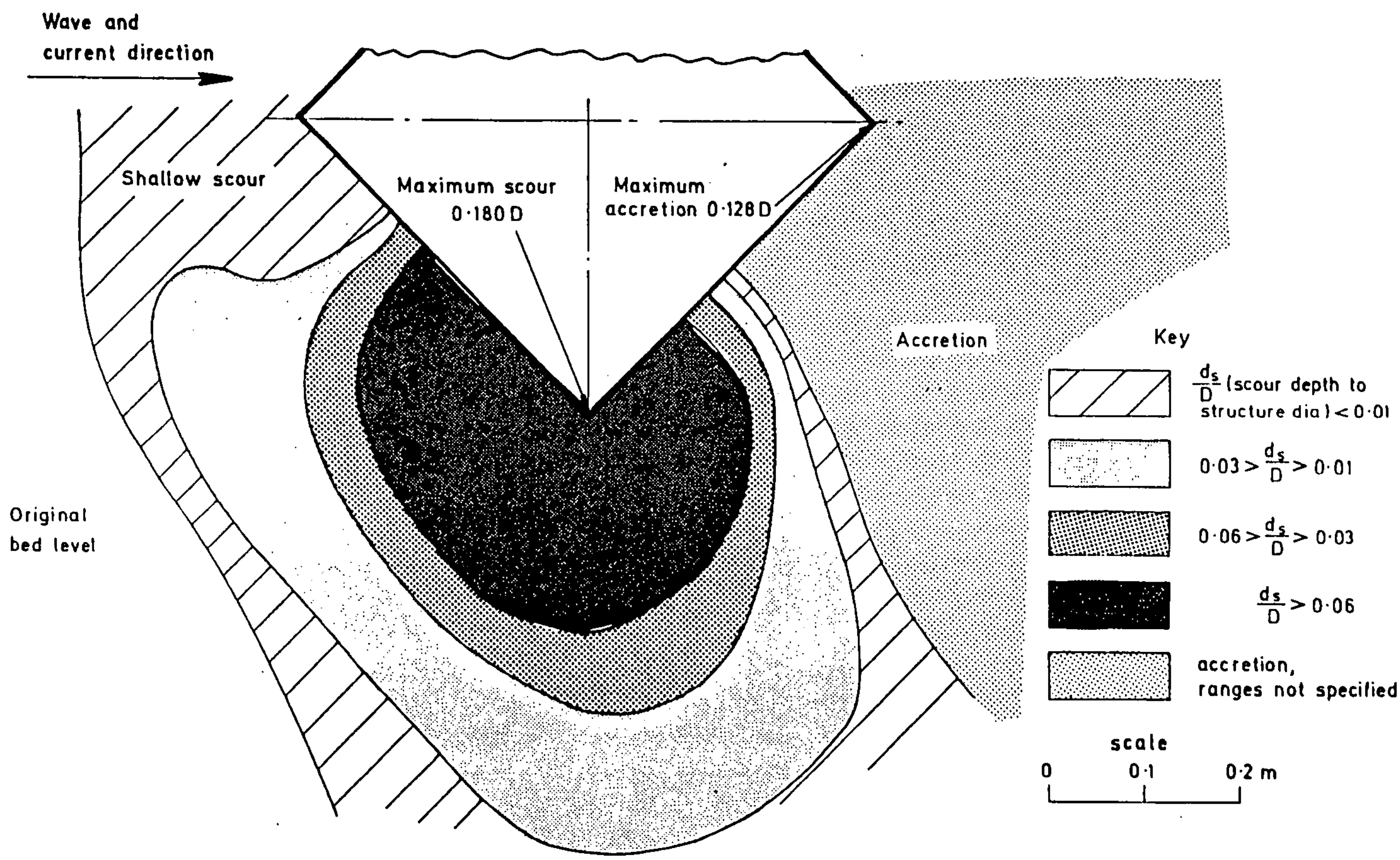


Figure 5.7b The Physical Model of a Square Structure with Leading Corner Showing Bed Topography Under Wave and Current Action (Rance, 1980)

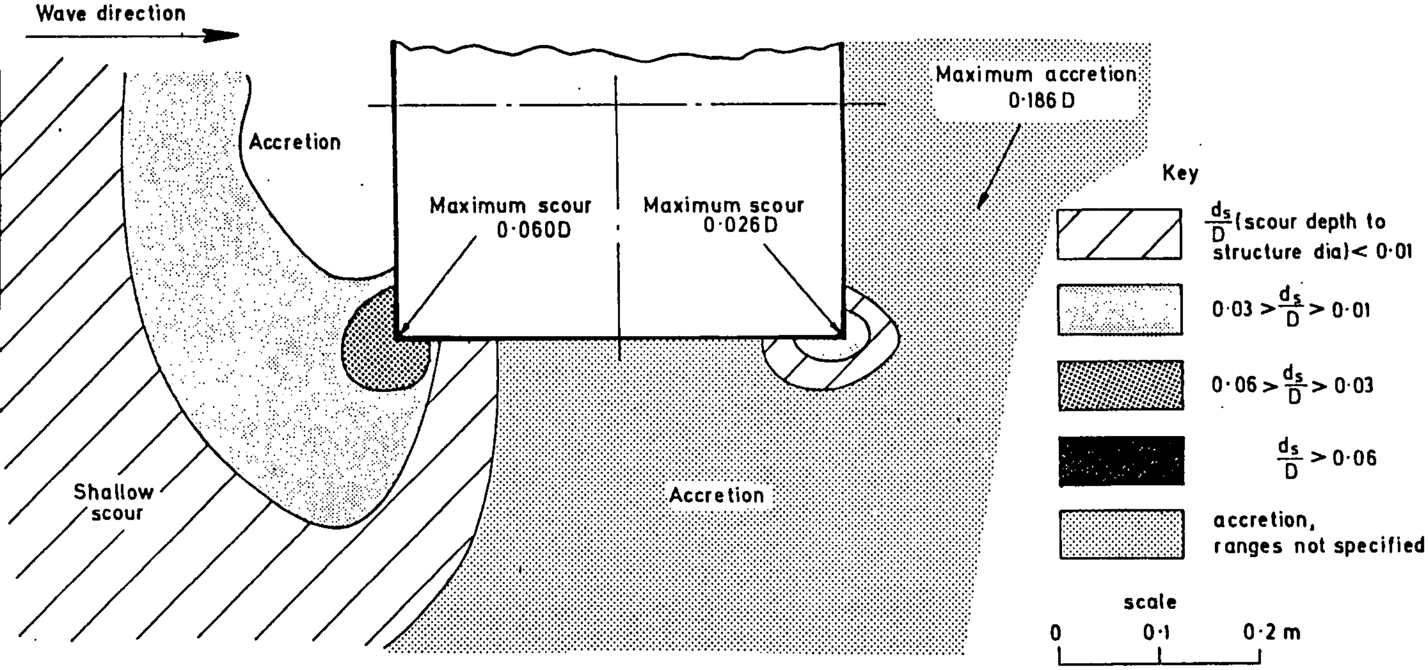


Figure 5.8a The Physical Model of a Square Structure with Leading Face Showing Bed Topography Under Wave Action (Rance, 1980)

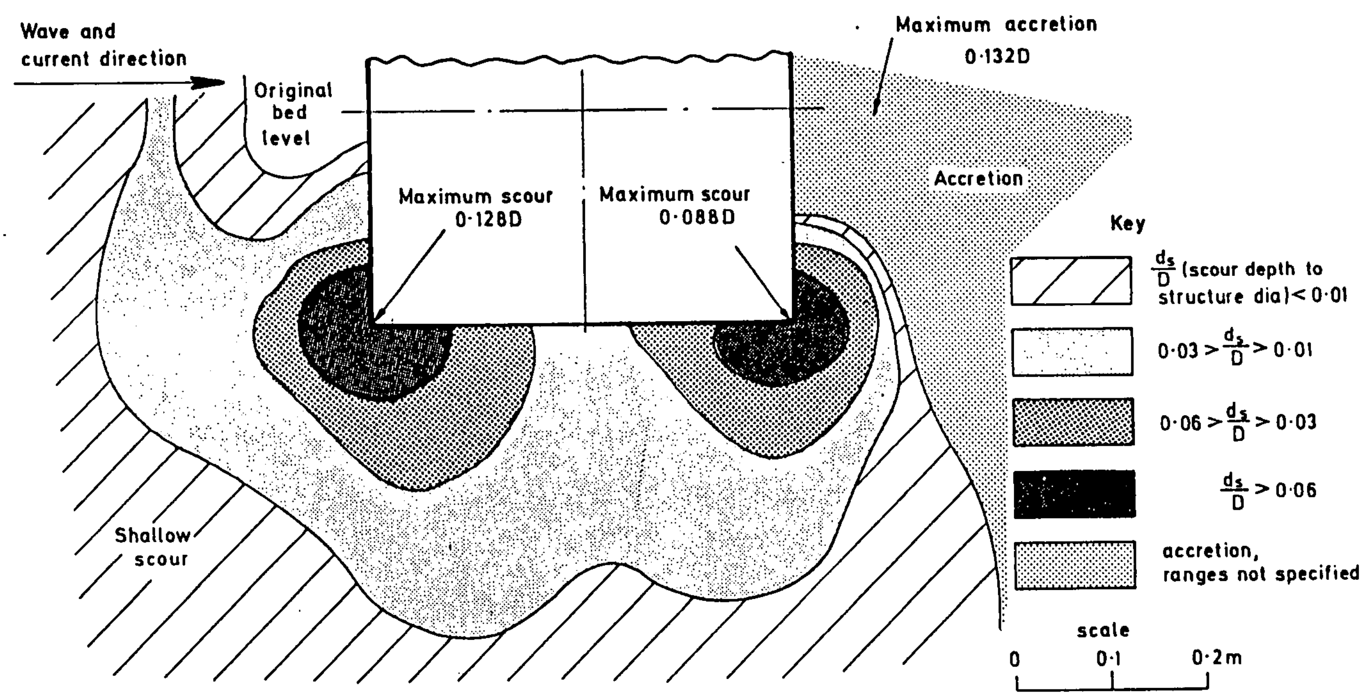


Figure 5.8b The Physical Model of a Square Structure with Leading Face Showing Bed Topography Under Wave and Current Action (Rance, 1980)

Physical model studies related to scour around pipelines are much less common than studies concerning cylindrical piers. Some studies have been done with unidirectional flow over pipelines while no notable studies have been completed and reported for oscillatory flow. Kjeldsen et al (1973) carried out physical model tests to investigate scour due to uniform flow across a pipeline resting on the bottom and pipelines at various stages of burial. Flow velocities betwen 0.25 and 0.45 m/sec were used, the mean diameter of the bed material was 75 microns and four different pipe diameters were investigated. In common with results for vertical piles it was found that the development of scour and the equilibrium scour depth is dependent primarily on geometry and weakly dependent on the particular grain size. Also the water depth should be unimportant as long as it exceeds three diameters. For pipelines initially resting on the bottom the equilibrium scour depth was given by the expression

$$S_u = \left(\frac{v^2}{2g}\right)^{0.2} . D^{0.8}$$

where v is the mean flow velocity, D is the pipeline diameter and S_u is the scour depth below the invert of the pipe. This expression was given as valid in the range:

Froude Number: $0.1 < Fr_D < 0.7$

Reynolds Number: $9.8 \times 10^3 < Re_D < 2.1 \times 10^5$

Bijker (1983) presents a formula based on the model test of Kjeldsen et al (1973), van Ast and deBoer (1973), Jansen (1981), van Meerendonk and

Roermund (1981) and Proot (1983). The results give and empirical function similar to Kjeldesn's.

$$S_u = 0.93 \left(\frac{v^2}{2g}\right)^{0.26} D^{0.78} d_{50}^{-0.04}$$

Bijker's function demonstrates that there is a slight influence due to median grain diameter. D50. Again, as suggested in the previous section on scour around cylindrical piers the limited influence of grain diameter may be a result of the restricted range of boundary conditions under which the test were performed. Table 5.1 contains a list of the boundary conditions for the tests used to derive Bijker's equation.

Zravkovich and Kirkham (1982) carried out model tests to investigate the development of scour underneath pipelines for oscillatory flow. This investigation only dealt with pipelines which had an initial gap between the seabed and the pipeline. Quantitative results are presented in Figure 5.9. For low values of the Keulegan-Carpenter number K_C a small sand ripple formed reducing the initial gap, at larger values of K_C substantial scour developed.

Research on the scour of pipelines under the action of oscillatory flow is presently underway at Delft University. It has been postulated that the ultimate scour depth will be a function of orbital excursion, wave period and grain diameter. Preliminary qualitative results showed scarcely any influence of pipe diameter.

No results are presented for partially or fully buried pipelines. In the case of the latter no scour occurred in the physical model whereas in oscillatory motion buried pipelines can be uncovered due to build up of pore water pressure within the sea bed. This may greatly increase the relative mobility of the bottom sediment thus setting up the required differential for scour to occur. Herbich (1981) describes a series of physical model studies designed to investigate the effect of storm waves

Table 5.1 Boundary Conditions for Physical Model Tests on Scour of Exposed Pipelines (Bijker, 1983)

Name	water depths (m)	velocities (m/s)	grain diameter (μm)	pipe diameter (mm)	pipe above (+) on (0) below (-) bottom level
Kjeldsen (1974)	0.43, 1.43	0.20-0.52	74	0,06-0,5	0 , -
van Ast and de Boer (1973)	0.21, 0.26	0.29-0.65	220	0,049, 0,088	+ , 0 , -
Jansen (1981)	0.36, 0.38	0.10-0.25	150	0,04, 0,05	+ , 0 , -
van Meerendonk and van Roermund	0.31	0.17-0.25	150	0,05	+ , 0 , -
Delft Hydraulics Laboratory (1982)	0.38, 0.40	0.28-0.58	90, 170	0,019, 0,075	+ , 0 , -

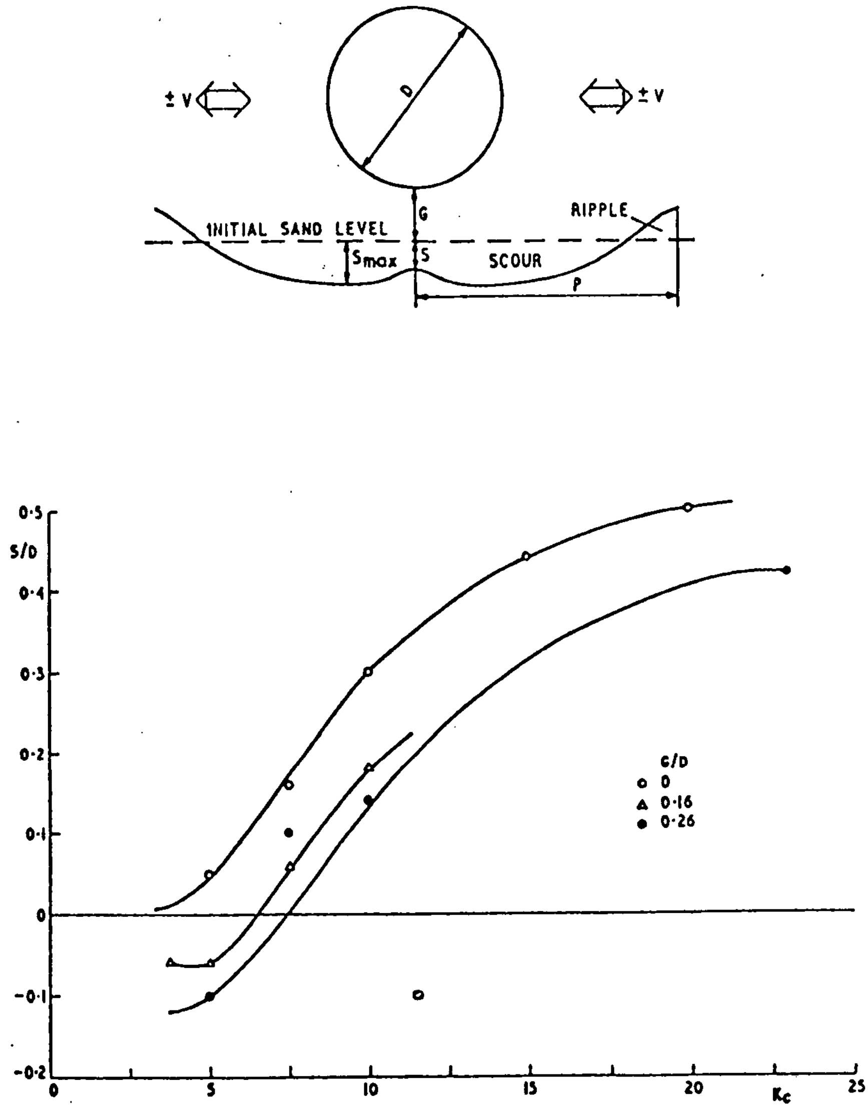


Figure 5.9 Variation of Scour Depth Under the Cylinder Axis with Keulegan Carpenter Number (Zdravkovich and Kirkham, 1982)

on buried pipelines approaching a shoreline. Other investigations are also described but the results are only qualitative.

Since profile evolution is primarily a two-dimentional phenomenon with on-offshore sediment transport, it can be modelled in a two-dimensional wave flume. Due to the inability to model grain size correctly, the laboratory profile will be distorted. Several authors (Vellinga, 1978, 1982; Hughes,1982; Sayao and Guimaraes, 1984) have presented reliable methods to account for model distortion in the interpretation of results based on the distortion of the fall velocity scale for sand grains. Herbich (1977) has also presented a method of calibrating distorted physical model results with prototype profiles. Of interest, in the model investigation is the response of the profile to a design storm at various water levels. From this information, the lower limit of shoreline change at any depth can be determined and used in design for depth of burial.

This completes the present dicussion on physical model testing relating to scour problems. The material presented is that which is considered to provide some generality of results. Numerous examples of problem specific model tests can be found in the literature the results of which are not considered in general terms.

One of the most striking features of mobile bed physical model tests that can be observed when photographs are provided is the size of the bed forms in relation to the structures being tested. They are usually very large due to the fact that the bed sediment used is not scaled. This is also much more pronounced in the case of combined wave and current action. The influence of roughness of the bed is discussed in section 5.3.1. It is a fundamental parameter in boundary layer and sediment transport dynamics. It seems questionable that this type of distortion should be ignored and simple geometric similarity is assumed to apply to the prototype. Since such similarity does not apply to other types of mobile bed models this seems unlikely.

5.2.2 Field Experiments

Full scale field experiments are considerably more expensive and difficult to carry out compared to their laboratory counterparts. At the same time the fundamental difficulty associated with interpretation of two-phase mobile bed scale models has led to frequent calls for more prototype data.

One of the most comprehensive series of field experiments concerning scour around natural and artificial objects was carried out by Palmer (1970). Field studies were carried out by divers which allowed direct observations, measurement and photography of scour development around different seabed structures. Supporting studies were conducted from a small submersible and field investigations at offshore platforms and in a tidal inlet provided additional data. Palmer provides an extremely detailed description of scour processes based on careful observations. However, the extent of quantitative data available is limited.

Figure 5.10 shows the geometry of objects used during Palmer's studies. Figure 5.11 shows the measured data for the relationship between cylinder diameter to scour pit lip distance and Figure 5.12 shows the ratio of pit diameter to object diameters as a function of object diameter. Some of the conclusions that he made are as follows:

(i) The height and shape of the obstacle has a significant effect on sediment transport rates early in the history of scour. Eventually the object diameter becomes the dominant factor.

(ii) The extent of scour was found to be independent of wave oribtal velocity in the range 0.25 to 0.6 m/sec.

(iii) The extent of scour was found to be independent of grain diameter in the range 0.12 to 0.63 mm.

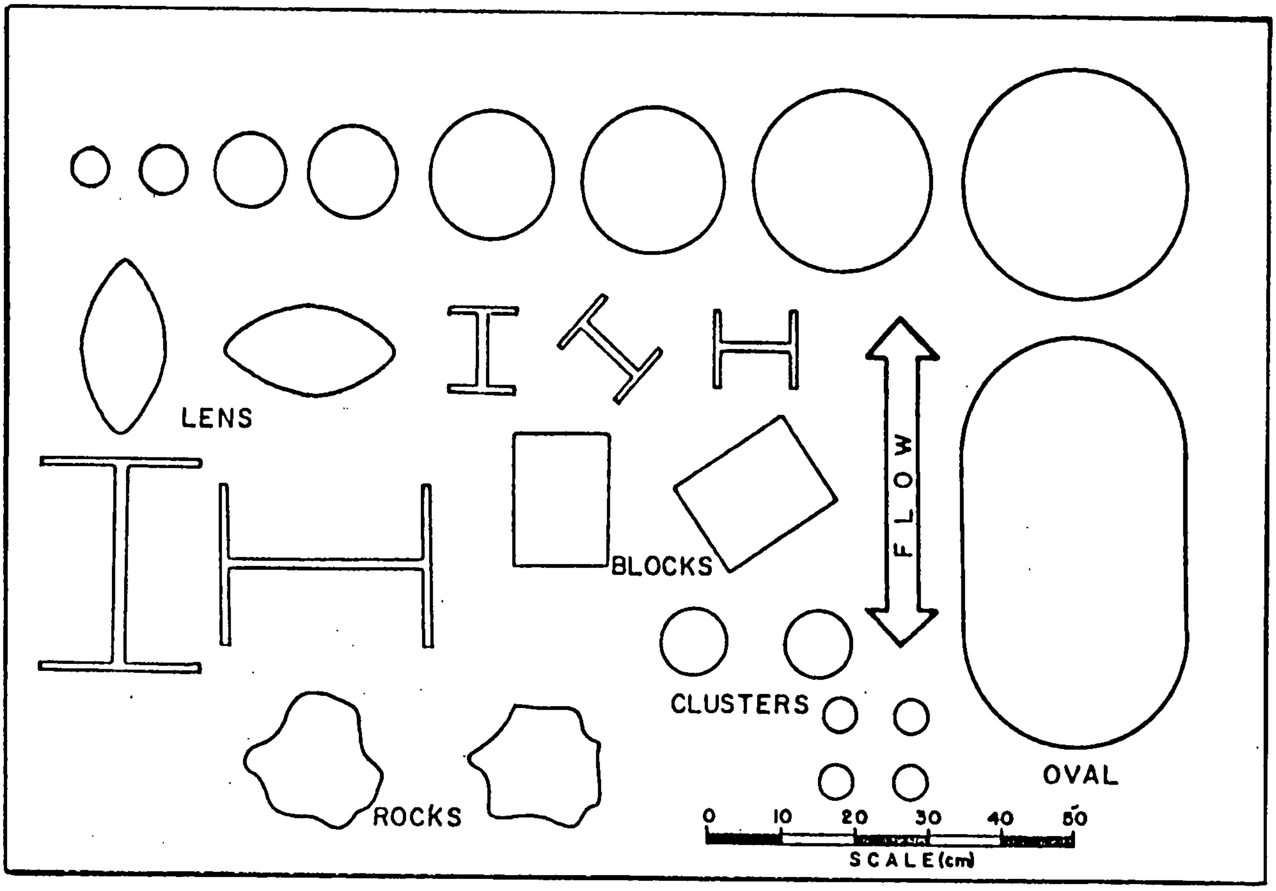


Figure 5.10 Geometry of Objects Used in Palmer's Field Studies (Palmer, 1970)

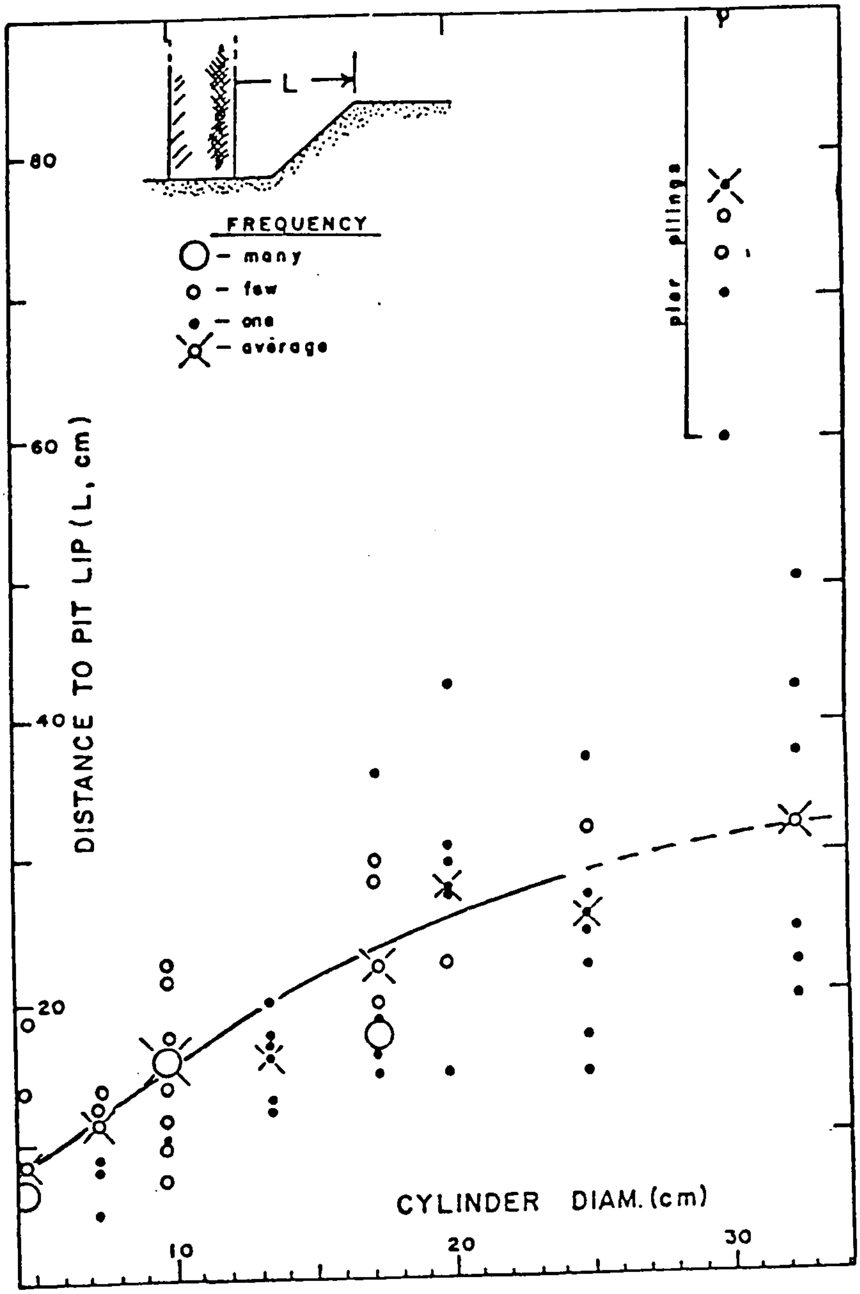


Figure 5.11 Distance to the Pit Lip versus Cylinder Diameter-Measured Data (Palmer, 1970)

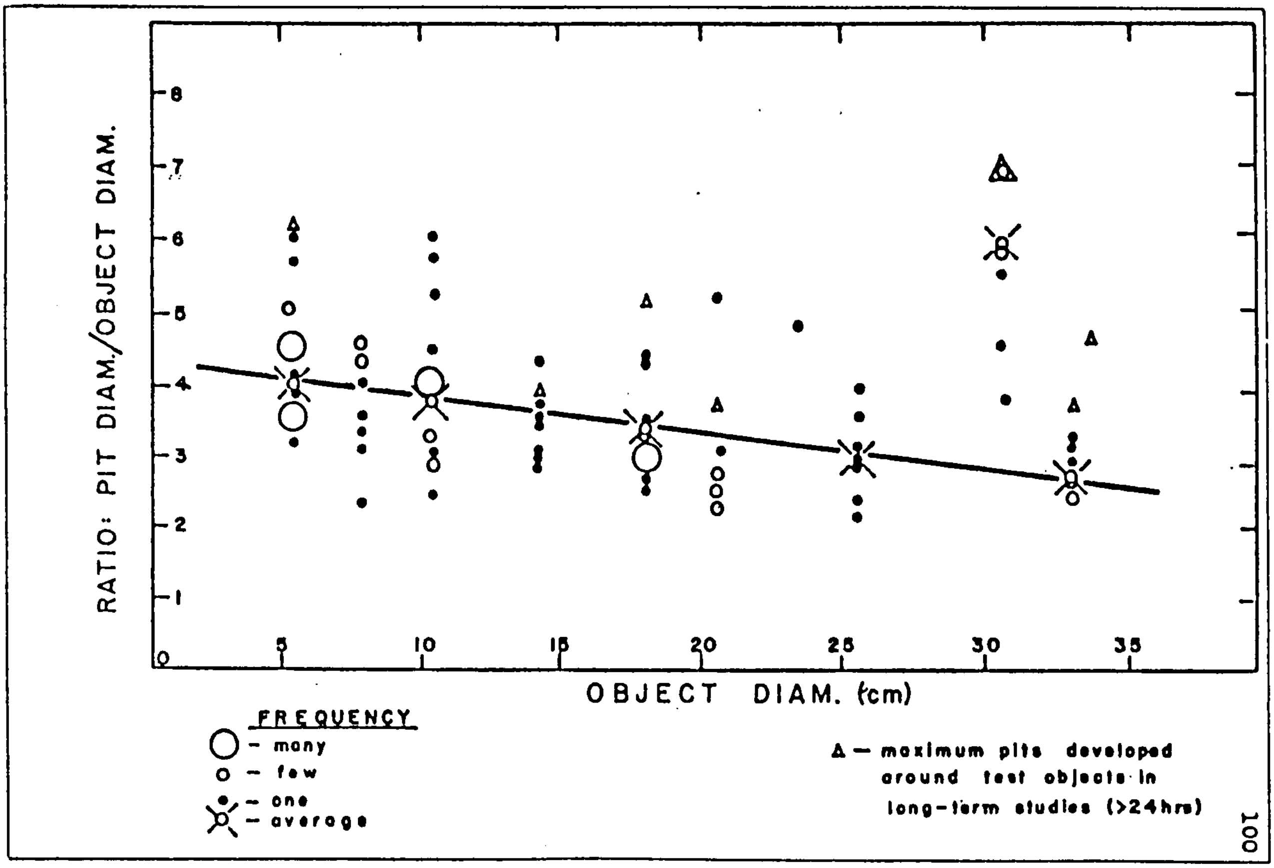


Figure 5.12 Ratio of Pit Diameter to Object Diameter as a Function of Object Diameter (Palmer, 1970)

(iv) An increase in object size causes an increase in scour development, but the percentage growth rate deceases. Thus small objects reach the terminal scour condition earlier than larger ones.

Palmer also looked at individual elements in clusters and found that following scour at each object the pit colalesced. This pit scoured according to that for a single object having a diameter equal to the combined diameter and separation distance of the elements. He also found that inclination of a cylinder object up to 45 degrees had no effect on the size of scour hole.

Palmer's experiments were all on relatively slender objects. In contrast Maidl and Stein (1981) report on prototype experiments with a number of different types of scour protection mats around a gravity structure. The scour protection systems tested included artificial seaweed, paraweb mats and concrete filled mats. Comments are made on the performance of each material but no conclusions are drawn with respect to the most satisfactory method.

Bishop (1980) describes experience with scour at the Christchurch Bay Tower, U.K. This tower was a small offshore structure with a gravity foundations placed in 9 m of water. It was an experimental tower for measuring wave forces in near breaking or breaking waves. The original foundation design specifically excluded a skirt and after the first major storm it was found that the tower was rocking quite severely due to scour of material underneath the base. Remedial works were not successful.

A second tower was designed with a larger base and a foundation skirt 0.7 m deep. Saucer shaped scour occurred exposing some of the skirt and some grouting was carried out to replace lost material. Nevertheless, the second design was considerably more stable than the initial design.

As previously pointed out, the movement of sand waves, which is not itself scour, can cause similar problems. Tesaker (1980) has reported a tracer experiment carried out on the Ekofisk field (North Sea). He concluded that fine sediment in 50 to 70 m depth of water could be mobilised fairly easily and since currents could be of the order of 0.25 m/sec for nearly 40% of the time significant movements could quite easily take place. The use of wrecks to evaluate the potential for long-term scour was also discussed. However, the conclusion was that "contrary to much experimental work, no relationship was apparent between either the width and shape of wrecks and the depth of scour, or between the height of the wreck and the depth of scour".

Dyer (1980) has reported measurements carried out in Start Bay (U.K.) where divers have measured the crest elevation of a sand wave relative to reference stakes. It was found that under mean tidal conditions, there was a maximum bed elevation of about 0.5 m with a gradual movement of the wave of about 2.0 m. The biggest changes accompanied storms when a whole crest of sand wave could be flattened out with an associated decrease in elevation of 1.5m.

The above sand wave dimensions are fairly modest compared to those that can be found in the Southern North Sea. Here, sand waves may be of the order of 15 m in height and move at a rate of the order of 10 m per annum.

The estimation of scour for the design of pipelines at the shoreline interface involves the prediction of nearshore sand level changes on unobstructed beaches. DeWall and Christenson (1979) present an analysis of shoreline change for locations along the U.S. East Coast, Gulf Coast, West Coast, and Great Lakes Coast. Through the synthesis of 33 data sets (1049 profiles) representing eight coastal locations, DeWall and Christenson used a regression analysis to relate extreme wave height (that height which is exceeded 12 hours per year) to depth of maximum scour. The relationship is shown in Figure 5.13. While this relationship would not be adequate as a universal design criterion for burial depth of pipelines it may be applied to sites with similar beach characteristics.

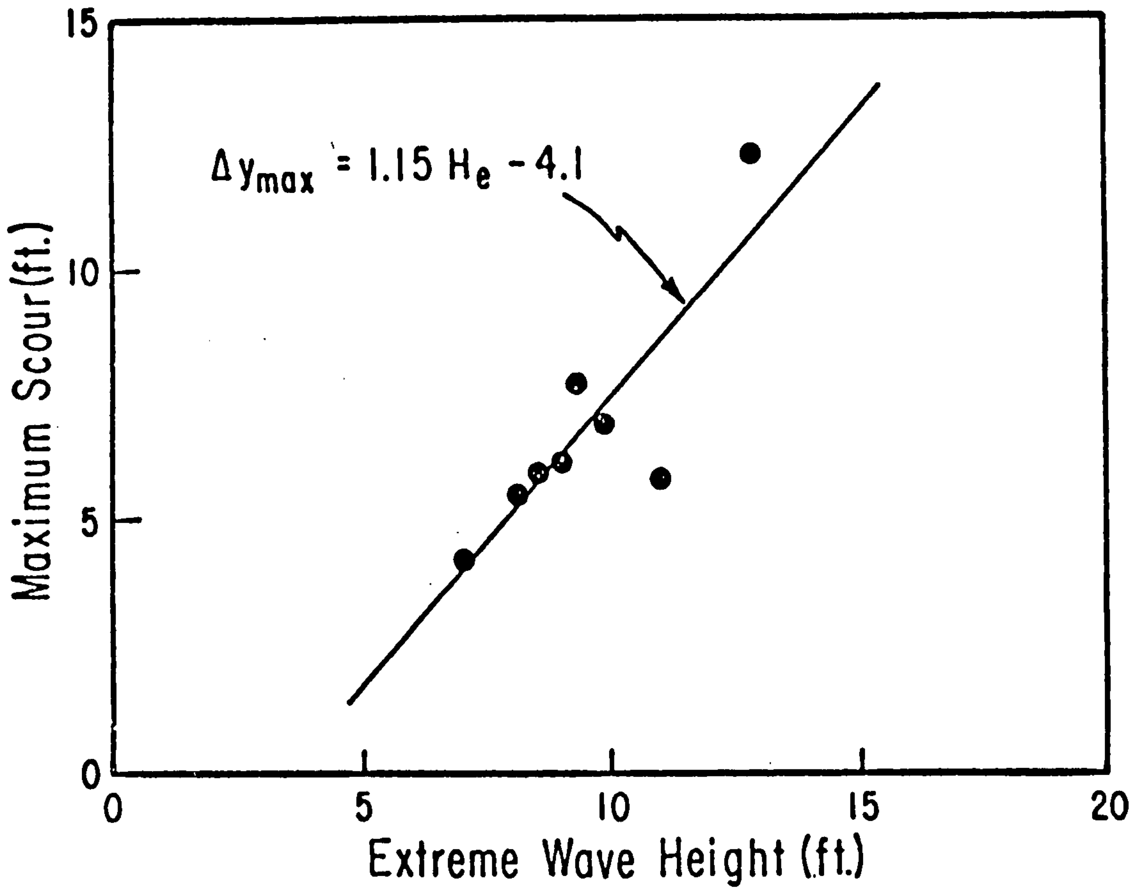


Figure 5.13 Maximum Scour Depth for Pipelines at the Shoreline Interface Related to Extreme Wave Height (DeWall and Christenson, 1979).

DeWall and Cristenson point out that their relationship will underestimate maximum scour because of the frequency of shoreline surveys in the data set. The more frequent surveys have a higher probability of documenting extreme profile changes. A field survey program should at least be able to pick up seasonal changes which are generally much greater than year to year changes.

5.3 Theoretical Approaches

The prediction of movement of cohesionless sediments finds its roots in investigation of, and application to, unidirectional flows. In this context the division between bed load and suspended load movement has been developed although exact definition is lacking other than by a conceptual description. Whilst some success has been achieved in prediction of mean sediment transport rates in steady unidirectional flows it is recognized that a complete deterministic description of the complexities of two phase motion including the generation of turbulence and turbulent exchange it is not likely to be forthcoming for some time to come. Predictive models are thus, at best, gross simplications of detailed motions and are heavily reliant on empirical coefficients. It follows that the range of applicability of such models should be strictly limited to the range of particle sizes and flow conditions for which empirical coefficients have been evaluated.

The development of analytical frameworks for estimating the distribution of sediment in the water column or its rate of transport under the influence of both waves and currents has relied heavily on theories developed for unidirectional flow. This has been by means of modification of the shear stress acting on the bed,the power expended at the bed or the efficiency of the transporting process. When defined, the distribution of suspended sediment in the water column has, in most cases, rested on the one dimensional diffusion equation.

5.3.1 Theory of Sediment Transport

There exists a number of formulations for sediment movement due to waves and currents. It is difficult to assess how far they can usefully be employed in effectively predicting scour processes, but in some circumstances results have been satisfactory (Fleming, 1983).

In most sediment transport theories it is necessary at some stage to estimate the shear stress acting on the bed.

In the case of turbulent flows which predominate in natural conditions, the solution of the boundary layer equation is complex and to date only approximate solutions have been proposed. Jonsson (1975) describes a solution in which he neglected the phase difference between shear stress and velocity and assumed fully developed turbulence and a logarithmic velocity distribution in the boundary layer. He measured complete velocity distributions throughout the wave cycle and hence derived empirical coefficients for the derivation of the wave friction factor.

Kajuira (1968) presented a theoretical solution to the problem based on a three layer model each with different structures of eddy viscosity. The model agrees quite well with experimental work described above. However, the fundamental problem with any of these approaches is in defining the bed roughness. Estimates vary between measures of grain diameter, functions of ripple height and functions of ripple height and length - none of which have been properly verified in common conditions found in prototype, particularly when ripples are present. In this context it is interesting to note that in Jonsson's laboratory work ripple roughness was represented by triangular elements. Recent research work has shown that sharp peaks so formed lead to vortex shedding that can be quite different in character to that when properly formed ripples are used. As a result the characteristics of the boundary layer are different and it follows that estimates of effective or equivalent roughness length should be different.

Estimates of the shear force acting on the bed can be made by inference from the velocity gradient, measurement of turbulent velocity fluctuations or direct measurement of shear force. The first of these is common practice, and the last method has been used successfully in the laboratory when using a shear plate with different sizes of roughness elements. A shear plate has not been used for ripple type roughness and it is difficult to see how it could be effectively used in prototype situations.

Measurement of turbulent velocity fluctuations and hence shear stress has been successfully carried out in the laboratory using a laser doppler anemometer. The technique requires very careful setting of the laser beams. Results for a flat bed and oscillatory flow have shown that the shear forces associated with turbulence are highly irregular in nature. When ripples are introduced it has been found (Savell, 1984) that the vortices thrown off by the ripples have the effect of masking the turbulent velocity fluctuations, thus making interpretation of data very difficult in terms of calculating shear forces involved. This can occur for several ripple heights above the bed.

Classification of the flow regime in the boundary layer itself deserves some attention as there is a degree of ambiguity in the technical literature with respect to consistent limits. The turbulent regime, which may be assumed to apply to prototype conditions is divided into rough wall and smooth wall regimes. At the same time it may be noted that laboratory experiments are most often confined to laminar or transitional conditions.

For a rough bed, the flow regime depends on the bed geometry as well as the flow parameters. All flow regime criteria may be reduced to a relationship between relative roughness of the bed and a Reynold's number. Comparison of limits proposed by different investigators show considerable variability. As long as conditions are unambiguously rough turbulent this should not raise a problem. Otherwise interpretation of boundary layer measurements will be influenced by whichever regime is assumed to apply. It may also be noted that in any event the proper estimation of roughness length is also important to this exercise.

Again the roughness length plays a central role in the perception of near bed hydrodynamics when placed in the conventional analytical framework. Because of this it is important that good measurements or predictions of bed form geometry are made. This is essentially a function of wave orbital motion at the bed, ambient current velocities and grain size characteristics.

It is generally agreed that ripple length is dependent on the oscillation amplitude at the bed until a maximum ripple length is reached. This appears to be a function of sediment grain diameter, density and oscillation period. The ripple height increases with increasing oscillation amplitude to a maximum and thereafter decreases to an ultimately flat bed. The most comprehensive work in this area is represented by Mogridge and Kamphuis (1972), Swart (1976) and Nielsen (1979), of these the last is the only one that appears to include both laboratory and field data. Nowhere is there any comprehensive data concerning the combination of both oscillatory and unidirectional motion.

Finally the definition of sediment transport relationships usually requires the definition of both bed load and suspnded load movement. Suspended load models usually take the form of the one dimensional turbulent diffusion equation with some specified distribution of turbulent diffusivity. Bed load models usually relate the sediment transport to the dimensionless ratio of bed shear to grain weight. The concentration at the top of the bed load is taken as a reference concentration to the suspended load distribution.

Nielsen (1979) developed an interesting solution which makes use of the time variation of the bottom boundary condition and is modelled in terms of a pick-up function representing a non-zero net transport at the bottom boundary. This provides a boundary condition for the suspended load. However, theoretical models of suspended load are hampered by a lack of experimental data on the mean and distribution of turbulent diffusivity in an oscillatory flow.

There have been a number of approaches proposed to deal with the combination of waves and currents essentially in terms of shear forces. The original work of Bijker (1971), implicitly assumes a current dominated environment and ignores the interaction between wave and current components. Madsen and Grant (1976) approach the problem from a different viewpoint by assuming that the current is weak relative to the wave orbital velocities. This is justified by the observation that even when the mean current velocity is larger than the peak orbital velocity due to waves, the smaller scale of the wave boundary layer makes the wave velocity dominant close to the bed. The work of Madsen and Grant includes consideration of instantaneous and orthogonal shear components as well as motion both outside and within the bottom boundary layer. The equations of motion of the combined flow are solved by assuming that the convective acceleration terms are negligible and using an eddy viscosity concept to model the shear divergence. This is assumed to be linearly varying and constant through the wave period.

This brief review of the theoretical aspects of seabed hydrodynamics and sediment movement serves to provide a perspective of the parameters that require good definition in any experiment whether it be in the laboratory or in the field. However, knowledge or definition of sediment transport rate alone does not provide any information with respect to the degree of scour other than the implication that a more active zone of sediment transport will be one in which scour potential is large. However, scour potential can also be large when the hydrodynamics of an area are changed sufficiently to induce sediment movement where othewise there would be none. This is known as `clear water scour'.

There are a number of situations in which conventional theories of sediment transport may be applied to scour problems. A gradient in sediment transport rate arises from a spatial or temporal in equality in the driving forces. As long as flows remain reasonably horizontal and free of significant eddies some applications may be considered.

(i) Large Gravity Structures and Artificial Islands. This class of problem is one for which the obstruction to the flow is large compared to the wavelength of incident waves. Consequently both wave and current conditions vary significantly around the structure. (See Section 5.3.2.).

(ii) Dishpan Scour. The scour that can occur in the vicinity of a jacket structure must be caused by the changes in flow through and around the clusters of piles. If the flows can be predicted even on the basis of reduction in cross-sectional area, there is some potential to calculate changes in sediment transport rate.

(iii) Sand Waves. One means of theoretically dealing with the movement of very large sand waves is to consider the difference in sediment transport rate at the crest and the trough of the wave. Simple consideration of continuity provides the means of calculating migration rates, but this pre-supposes that the height of the sand wave is known.

Conventional sediment transport theories are also relevant to predictions of shoreline movement both alongshore and on/offshore. Estimates of seasonal and long term changes are necessary for the design of shore crossings for pipelines.

There are no "theoretical" models for the development of local scour at individual pile legs or around clusters of pipes. Herbich et al (1984) provide a semi-empirical framework that is heavily based on physical model experiments. It may be summarised:

1. Generally for scour by waves and currents a parameter α is defined.

$$\alpha = \frac{H^2 \, LV^2 \, (V + (1/T - V/L) \, HL/2h)^2}{((\rho_s - \rho)/\rho) \, g^2 \, h^4 \, d_{50}}$$

where H is wave height, L is wave length, T is wave period, V is current velocity, h is water depth, ρ_s and ρ are the density of sediment and water respectively and d50 is the median diameter of the sediment.

2. Subject to experimental limitation scour will not occur when α is less than 0.02.

3. The relative general scour is given by

$$\log_{10} \left(\frac{S_u}{h} + 0.05 \right) = -0.6331 + 0.3649 \log_{10}\alpha$$

4. For scour around a pile under the combined action of waves and currents a parameter is defined

$$\beta = \frac{H^2 \, LV^3 \, D \, (V + (1/T - V/L) \, HL/2h)^2}{((\rho_s - \rho)/\rho) \, \nu g^2 h^4 \, d_{50}}$$

where ν is the kinematic viscosity.

5. When α is less than 0.02 only local scour around the pile will take place. The local relative scour depth based on experimental data is given by

$$\log_{10} \frac{S_u}{h} = -1.2935 + 0.1917 \log_{10} \beta$$

6. When α is greater than 0.02 the scour will be both genral and local and in which case the experimental data gives the relation

$$\log_{10} \frac{S_u}{h} = - 1.4071 + 0.2667 \log_{10} \beta$$

These relationships are strongly empirical and are only based on one data set (Wang and Herbich, 1983). Nevertheless the correlation with the experimental data is extremely good. This leaves the question of scaling to prototype to be considered.

Jain (1981) has presented a comparative study of formulas and data related to maximum clear water scour around circular piers in steady flow. As a result he proposed the relationship

$$\frac{S_u}{D} = 1.84 \left(\frac{h}{D}\right)^{0.3} F_c^{0.25}$$

where F_c is the threshold Froude number given as $F_c = V_c/\sqrt{gh}$

One expression for scour depth under a pipeline initially resting on the bottom is given is section 5.2.1. That was based on experimental data for uniform flow. Herbich (1984) gives an analytical method for estimating the maximum scour depth under a pipeline also for currents as developed by Chao et al (1972). The average jet velocity underneath the pipe is

$$\bar{V}_1 = V_0 \left(\frac{2(Sp/D)^2 - (Sp/D) - 1}{2(Sp/D)^2 - 3(Sp/D) + 1} \right., \quad S \geq D$$

The limit of scour depth may then be found by calculating the point at which the average flow velocity under the pipeline exerts a bottom shear stress that is less than the initial shear for the bottom sediment.

A more rigorous theoretical treatment of flows around pipelines and hence scour depth is given by Bijker (1976) together with some experimental results. Comparison to the unidirectional flow condition is also discussed.

The results for pipelines discussed above imply that the pipeline is initially resting on the bed. It does not address the conditions of pipejacking, uncovering of buried pipelines or the self-burial of pipelines. The scour depth below a pipeline is required for some aspects of design but the all important question of free spans cannot be predicted in this way.

Given that there exist some somewhat shakey theoretical means of predicting scour depths around seabed structures there is a requirement to provide protective measures if considered necessary. The design of a structure will always tolerate some scour so that the extra loading that might result is accommodated. In the event that some scour protection is deemed necessary the theoretical means of design are broadly limited to predicting the reduced mobility of coarser material. In fact the indications are that the use of gravel size material for scour protection is one of the most effective means.

Some sediment transport models can be stretched to deal with the movement of gravels but attention must be paid to the data on which such models were originated. Unless it is intended that the scour protection material should be sacrificed as in the case of artificial islands, gravel protection will usually be designed to be relatively immobile so that it is the threshold of motion condition that is more important.

Posey (1970) has suggested that a reverse filter should be designed for protection according to the following inequalities:

d_{15} filter < 5 d85 base

4 d_{15} base < d15 filter < 20 d15 base

d_{50} filter < 25 d50 base

where d_{15} is 15 percent finer than etc.

5.3.2 Numerical Models

Given the present state of knowledge only a few of the pure hydrodynamic phenomena can be reliably predicted by theoretical or numerical models. These include tides, surges, wave refraction, shoaling, diffraction and direct reflections. Investigation of most other phenomena require a combination of prototype measurements, physical model tests or mumerical models.

Numerical modelling is a mathematical process by which theoretical relationships are discretised to be represented by numerical approximations. This invariably involves the use of digital computers which are becoming more elaborate with time. These have not, as yet, been developed to deal with problems in three dimensions with two phase flows.

Those numerical models of scour that have been applied to particular problems have generally used conventional sediment transport models coupled with two-dimensional wave and current models. This means that quasi-steady state principles are used in a situation.

Rance (1980) used numerical models of wave diffraction around a large cylinder together with a sediment transport model and predicted sediment distribution and wave action.

Kobayashi et al (1981) used numerical models of wave refraction and wave generated currents together with a beach sediment transport model to predict the deformation of a gravel artificial island. They also used a wave diffraction model to predict sediment movement patterns around a caisson island on a flat sea bed. In neither case did they attempt to examine the evolution of the sea bed in response to the changing conditions. The results were not verified.

Fleming et al (1983) combined numerical models of current flows, wave refraction, wave reflection, wave diffraction and sediment transport around a caisson island placed on an underwater mound. The model was run to investigate the deformation of the mound as a function of time. The changes measured in a physical model were reproduced well and the model was used to investigate the use of different protective materials. It was also used as means of interpreting the physical model results at prototype scale and predicting the effects of random seas with differing direction properties.

Perlin and Dean (1983, 1985) have developed a fully implicit finite-difference, N-line numerical model to predict bathymetric changes in the vicinity of coastal structures. The wave field transformation includes refraction, shoaling and diffraction.

5.4 Post-Construction Monitoring

The same principles apply to post-construction monitoring that have already been discussed in Section 5.1 - Pre-construction Environmental Data Collection. As field data is difficult and expensive to collect there is a natural reluctance to discontinue some of the data collection once a project is completed.

The basic ingredients of a post-construction monitoring programme related to scour problems should be to maintain good coverage of winds, waves and currents. In addition inspections of potential scour areas are usually carried out by divers as a matter of routine. The basic difficulty from the design point of view is that it is rarely possible to observe the depth and extent of scour that might occur at the height of a storm. This clearly requires some sort of insitu measuring system such as pressure transducers buried around a structure.

In the case of pipelines, surveys need to be carried out on a regular basis as part of a routine maintenance excercise. It follows that in areas where sand wave movement has been detected careful monitoring should continue in the post construction period.

6. GUIDELINES FOR SCOUR DESIGN PROCEDURES

6.1 Existing Standards and Codes

Several regulatory bodies have published guidelines for the planning, design and construction of offshore platforms. In all cases only limited mention is made of scour and no detailed guidance for scour design is provided. The collection of pre-construction environmental data is covered in somewhat more detail, but not with specific reference to seabed scour. This section presents a summary of existing standards and codes from several of the better known regulatory authorities.

6.1.1 American Petrolium Institute (API)

The API circulates two publications which make reference to scour design; the Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms (API RP 24, 15 ed., October 1984) and the Recommended Practice for Design, Construction, Operation and Maintenance of Offshore Hydrocarbon Pipelines (API RPIIII). The latter publication is presently under revision (October 1984) and since significant changes are anticipated, no information is presented. Relevant extracts from API RP 24 are listed in Appendix A5.1 (these include Environmental Data Collection, Site Investigations and Foundation Design) and direct references to scour are presented below;

> 1.3.9 Scour. Scour is removal of sea-floor soils caused by currents and waves. Such erosion can be a natural geologic process or can be caused by structural elements interrupting the natural flow regime near the sea-floor. Sand or silt on the sea-floor are particularly susceptible to scour. Scour can result in removing vertical and lateral support for foundations causing undesirable settlements of mat foundations and overstressing of foundation elements. Where scour is a possibility, it should be accounted for in design or its mitigation should be considered.

2.6.1 Hydraulic Instability of Shallow Foundations

> 2.6.15a. Scour. Positive measures should be taken to prevent erosion and undercutting of the soil beneath or near the structure base due to scour. Examples of such measures are (1) scour skirts penetrating through erodible layers into scour resistant materials or to such depths as to eliminate the scour hazard, or (2) riprap emplaced around the edges of the foundation.

6.1.2 British Standards Institute (BSI)

The BSI published a Code of Practice for Fixed Offshore Structures (1982). Two short sections are devoted to the influence of scour, causes of erosion and methods of protection as quoted below;

> 5.2.3.8 Influence of scour. When considering the depth of penetration of piles, allowance should be made for scour if this is likely to occur. Alternately, precautions should be taken to prevent scour occurring adjacent to the piles.
>
> 5.3.10 Erosion. The possibility of erosion of soils from around and beneath the structure should be assessed.
>
> 5.3.10.1 Causes of Erosion. Erosion of the sea bed soils may be caused by:
>
> (a) the influence of waves and currents passing over the sea bed at velocities sufficient to dislodge particles of the sea bed.
>
> NOTE: In this connection, the effect on the sea-bed velocities of the obstruction formed by the structure should be taken into account, preferably as a result of model

tests. It should be noted that the velocities likely to cause scour may be markedly directional either due to maximum current direction or to the configuration of the structure.

(b) the relief of pore water pressure from beneath the structure, which may cause removal of soil from beneath the foundations (sub-surface erosion);

(c) The combined influence of (a) and (b).

5.3.10.2 Protection against erosion. Adequate methods of protection against erosion should be provided where unsuitable conditions are likely to arise. One method of protection is the use of fabric filters surrounding the foundation overlain by riprap, sand bags or granular material of sufficient size not to be removed by the influence of scour. Studies should be made to determine the width of apron necessary for a particular location. In view of the uncertainty regarding the procedures used to design such systems, it is important that regular surveys of the erosion protection system are carried out throughout the life of the structure. See references 5.14 and 5.20.

Reference 5.14 corresponds to HRS (1978) in the references of this report and 5.20 is a series of papers entitled Scour Prevention Techniques Around Offshore Structures which were presented at a seminar of the Society for Underwater Technology and which are also included among the references of this report.

6.1.3 Det Norske Veritas (DNV)

The DNV present Rules for the Design, Construction and Inspection of Offshore Structures (1977). Some considerations of the influence of scour on structural stability are given as follows:

> F3.7 Modification due to scour
>
> F3.7.1 Scour will lead to complete loss of lateral resistance down to the depth of scour and should be considered so in the construction of the p-y curves for soil layer susceptible to scour, see Figure F3.3.
>
> F3.7.2 Scour will also reduce the effective stress, p_o, further down which should be considered by using the scoured base as mudline in the construction of p-y curves. This has been demonstrated in Figure F3.3. In sand this will reduce the value both of the k_l parameter and the design lateral resistance, p_d, defining the p-y curve for a certain pile element.

6.1.4 Canadian Standards Association (CSA)

A standard entitled "Off shore Pipeline Systems #Z187" is presently in the process of being published and should be available in 1985.

6.1.5 Canadian Oil and Gas Drilling Regulations

As part of the process in obtaining a drilling program approval certain information must be furnished and forwarded to the Canadian Oil and Gas Lands Administration (COGLA). Where the program is to be carried out offshore, Section 8(f) requires the following information to be provided:

> (i) particulars of the nature of the seafloor in the proposed drill site

(ii) the prevailing environmental conditions in the area of the program.

Section 89 on Well Prognosis presents further detail on the form of information described in (i) and (ii) of Section 8. Section 176 prescribes the scope of observations including wind speeds, wave height and period and current velocities that must be taken once drilling is in progress.

6.2 Pre-Construction Environmental Data Collection

6.2.Ø General

The collected environmental data must be sufficient to adequately define the wave climate, current field and bottom sediment characteristics. The hydrodynamic conditions at the seafloor are of particualr concern to scour design.

6.2.1 Wave Climate

The wave climate must be defined to determine the peak orbital velocity of oscillatory motion near the bed which plays an active role in scour. It has become apparent from the previous sections of this report that there is a lack of clarity concerning the influence of waves on the ultimate scour depth. Therefore, an accurate determination of wave conditions by a sophisticated method for the sole purpose of predicting scour might not be entirely justified until its importance has been better established. On the other hand, the wave climate must be accurately defined to determine structural loading and operational conditions for the design of marine structures. Therefore, wave climate data, more than sufficient as input to scour design, will normally be available.

Wave climates are generally derived from three principal sources; wind-wave hindcast, direct wave measurements and visual sea state observations from mariners. Wave hindcasts which have been calibrated by at least one year of measured wave data will provide adequate information to define normal wave conditions and to perform extreme value analyses. Very often no measured data, or only two or three months of measurements are available.

Two types of wave hindcasting should be considered for design purposes. The use of parametric methods provides an accurate and cost effective approach to hindcasting for areas where fetches are well defined (such as enclosed or semi-enclosed bodies of water) and where the wind does not vary spatially to an unreasonable extent. Parametric methods are suitable for synthesizing long-term wave statistics.

The other method of hindcasting is by two-dimensional spectral models. As input, this type of model requires a pressure field (over the entire area to be modelled) defined to reasonable detail both spatially and in time and accurately delineated boundary conditions. These models are far too complex for modelling long time series and their application is restricted to the consideration of individual storm events. The two-dimensional spectral methods have proven to be very reliable especially where rapid changes of wind speed and direction occur, however they are very expensive to run.

Whichever method of hindcasting is chosen, special attention should be given to shallow water sites that are partially protected by shallow banks or islands. Most standard hindcasting models are not capable of dealing with shallow water effects. However, parametric methods may be combined with refraction analysis preferably using directional spectral transfer methods while the most advanced two dimensional models can allow for simultaneous wave generation and refraction.

6.2.2 Currents

Currents are the major cause of scour around seafloor structures particularly in deeper water. While surface currents are often studied in great detail with respect to the dispersion of oil spills, much less attention is customarily given to the vertical distribution of currents and to currents close to the bottom. Current measurements are usually taken for a period of a few weeks, which may be adequate to provide basic data for the principal diurnal or semi-diurnal tidal components. However, in areas where wind-driven storm currents are important it is a matter of chance whether such conditions are adequately monitored in short series of measurements.

In Section 5.1 a framework of analysis for currents based on water levels was introduced (Graff, 1984). However, it was suggested that difficulty would arise in applying the method to locations where numerous tide reference points do not exist (for example the Scotian Shelf). The current field may also be determined by numerical flow models which are generally restricted to vertically integrated two-dimensional flow. Vertical distributions of current velocities are necessary to interpret and apply the results of two-dimensional models to seafloor scour. It is apparent that if seafloor scour design is to be approached in a systematic manner more extensive field measurements of current must be obtained, both spatially (including a vertical distribution) and temporally. The current measurements should be taken over a series of storm events to evaluate the importance of surge and wind induced currents.

6.2.3 Bottom Sediments

Bottom sediment studies should be carried out to determine the nature and thickness of the surficial deposits. If the deposit is sand or gravel an approximate grain size distribution is useful. Model studies have indicated that grain size has little effect on scour, however, this is in part attributable to the narrow range of boundary conditions examined in most tests. An evaluation of approximate sediment transport threshold conditions, as well as rates and the extent of sand wave activity should be considered.

6.3 Selection of Scour Prediction Procedure

Most prediction techniques for ultimate scour depth have been derived from physcial model studies, some have originated from field experiments and very few can be classified as analytical or theoretical methods. Numerical models cannot be applied until analytical methods are established. A summary of the information that does exist is presented in Table 6.1, contributions from various authors in the form of equations, plotted relationships and qualitative comments are classified according to prediction technique, applicable hydrodynamic conditions and structure type. In sections where a plethora of studies exist, (for instance, physical model studies of scour around cylindrical piles) only comprehensive summaries (for example Breusers et al, 1977) or the most recent studies are presented.

For some structure types more than one predictive technique is available, yet for others no reliable techniques have been established. A brief summary with comments on the most common prediction methods for the various structure type is presented. (Section 5 provides a more detailed discussion).

For piled structures or cylindrical piers under the influence of currents alone, Breusers et al (1977) present an empirical relationship based on a vast quantity of model tests (many of which were bridge pier investigations). Herbich et al (1984) produced semi-empirical equations for general and local scour around piles under the combined effect of waves and currents. The hydrodynamic condition of waves alone is seldom considered for seafloor scour of piled structures in deepwater. There are no direct references to relationships on global scour around groups of piles. Herbich et al (1984) provide an empirical formula for "general" (as distinct from "local" scour) due to waves and currents (See 5.3.1.). However, it does not appear to be of general application and it is not clear from the text under what conditions it may be applied.

There are presently no quantitative means for predicting scour around buried or partially buried pipelines under any hydrodynamic condition.

Table 6.1 : Scour Prediction Design Guidelines
Literature Sources by Type of Structure and Design Method

		PILED STRUCTURES (& JACK-UPS)	PIPELINES				LARGE GRAVITY STRUCTURES			SMALL FOOTINGS
			Buried or Partially Buried	Sitting on the Bed	Spanned	Shoreline Interface	General	Sacrificial Beach Islands	Caisson Retained Islands	
PHYSICAL MODEL	Waves	Armbrust(1982)(b)		Delft Univ. (1983)	Zdravkovich & Kirkham (1982) (b)	Herbich(1977, 1981)(a)	Rance(1980)	Kamphuis & Nairn (1984)	Brebner & Kamphuis(1977) Baird & Assoc.(1981) Moir et al (1984)	
	Currents	Breusers et al (1977)(a) Imberger et al (1982)(b)	Kjeldsen et al (1974)(c)	Kjeldsen et al (1974)(a) Bijker(1983)(a)	Zdravkovich & Kirkham(1982) (c)					
	Waves & Currents	Clark et al(1982) (c) Armbrust (1982) Wang & Herbich (1983)(a)					Rance(1980)			
FIELD EXPERIMENTS	Waves					DeWall & Christenson (1979)(a)(b)	Maidl & Stein (1981)(c) Bishop(1980) (c)broken waves		Myers et al (1983)	
	Currents	Palmer(1970)(b)		Littlejohns (1977)						Palmer(1970) (c)
	Waves & Currents	Palmer(1970)(b)					Tesaker(1980)			Palmer(1970) (c) DeWall(1981)
ANALYTICAL (SEMI-EMPIRICAL)	Waves									
	Currents	Jain(1981)(a)		Chao et al(1972) (a)						
	Waves & Currents	Herbich et al(1984) (a) general & local scour								
NUMERICAL	Waves						Rance(1980)			
	Currents									
	Waves & Currents							Kobayashi et al (1981)	Fleming et al (1983)	

(a) - equations
(b) - plotted relationships
(c) - qualitative

Kjeldsen et al (1974) and Bijker (1983) present similar relationships (based on physical model data) for determining scour underneath a pipeline initially sitting on a bed which is exposed to currents. The only guidance provided for the case of waves alone acting on pipelines was for the special case of pipelines with an initial gap above the seabed (see Zdravkovich and Kirkham, 1982). Research was reported to be underway at Delft University examining the influence of oscillatory flow on the scour around pipelines initially sitting on the bed (Bijker, 1983). The problem of scour of pipeline installations at the shoreline can be examined through physical model tests as demonstrated by Herbich (1977, 1981). DeWall and Christenson (1979) present a relationship for maximum scour depth at the shoreline based on a collection of field data. This relationship should only be used for shorelines comparable to those in the original data set, and even then there are reasons to suppose that the shoreline change will probably be underestimated. The scour around large gravity structures is usually investigated by physical model studies. Rance (1980) presents data on a series of model tests for the scour of piers of various shapes under the influence of waves alone and waves and currents combined. In any model test of a mobile bed consisting of sand the question of scale effects should be addressed, however this is seldom done though often mentioned. Kamphuis and Nairn (1984) provide a discussion of scale effects encountered in a series of model tests on sacrificial beach islands. Field experiments of large structures are only used in a comparative capacity, specific experiments for design application would generally be too expensive. Numerical models of scour around large gravity structures have been attempted.

Fleming et al (1983) have modelled wave and current erosion around a caisson retained island numerically and were able to calibrate and verify the results with results from physical model tests.

It is apparent that several methods and techniques are available for scour prediction. However, as is the case with most sediment transport phenomena the estimates are seldom very accurate. Table 6.1 reveals the present paucity of knowledge on the subject of scour. This is especially apparent from the number of blank cells near the bottom of the table. As the theory of scour is developed a greater selection of analytical and numerical methods will be available. The development of seafloor scour theory can only come about through

i) the developement of hydrodynamic theory especially concerning turbulence and three-dimensional flow

ii) further model studies which investigate the question of scale effects most of which result from the inability to correctly model sediment grain size and boundary layer flows

iii) more systematic information on the scour around prototype structures.

6.4 Selection of Scour Protection Procedures

The types of scour protection methods which are available are presented in detail in Section 4.1 and in summary in Section 4.2 Protection procedures should be considered at the design stage of the project if significant scour has been predicted under prevailing bottom conditions by application of estimation techniques such as those described in Section 5.

The use of rock, stone, or gravel layers is the most common technique of protection. In deeper water and in other situations where currents predominate, a framework for design exists and is relatively straight forward. In cases where waves predominate the situation is more complex, empirical coefficients required to determine layer resistance are often lacking, and experimental methods required.

Artificial seaweed is less common and little can be presented in the way of design procedures. Many different types of artificial seaweed schemes have been presented in the literature, yet their effectiveness for particular situations can only be estimated through experimentation, preferably in the field.

There are two separate approaches to the use of rock or gravel protective layers. The armour layer may be designed to be stable for the environmental design conditions (i.e. the n - year storm) in which case the selected material will be rock or stone. The design of the armour layer could be determined using unidirectional flow formula for the inception of sediment transport and riprap design (see the Shields curve). A conservative estimate of required stone or rock size will be achieved when when waves and currents exist if the two component velocities are added to determine the equivalent shear stress. The use of a larger stone armour layer will require the use of filter layers, the design of which are summarized in Section 4.1.3.4. The armour layer can also be placed on a synthetic mattress, in place of filter layers, as shown in Figure 4.16.

Often suitable rock or riprap material is not locally available and an alternative approach is to use gravel as a protective layer. This method has the advantage of not requiring a filter layer without which placement is easier. However, the gravel will not be stable under certain storm conditions and some idea of the rate of transport and removal of the material is necessary to assess the useful life of the gravel layer. Van Hijum and Pilarczyk (1982) report on the profile development and longshore transport along coasts. However, little information is available in the literature concerning transport of gravel on the seabed. A remaining option is to determine the required thickness of the gravel layer by physical model testing under design storm conditions. (See Brebner and Kamphuis, 1977, and Baird et al, 1981). However, this involves a combination of physical and numerical modelling approaches. In this method a numerical model is first calibrated against the results of physical model tests and then scaled up according to the governing equations to run at prototype scale. Fleming et al (1983) used this for scour of the berm of a caisson retained island. Lack of experimental data on roughnesss coefficients for coarse gravels and stones under wave action somewhat limits the scope of this method at present.

To circumvent the difficulties of placement of filter layers under water, a hybrid approach has been used in which a single layer of widely graded material is placed (Roelofsen 1980). The advantage of this method is that it will naturally form filter and armour layers as fines are washed out of the surface layers. The difficulty is locating a suitably graded material. There is also a danger of severe erosion in the event that a design storm occurs before the protective armour layer has evolved.

6.5 Post-Construction Monitoring

Post-construction monitoring is essential because it provides two types of information of both local and general significance.

1. Data by which estimates of extreme conditions may be improved, enabling designs of subsequent structures to be placed in the area to be refined and giving warning of the need for improvement of scour protection measures for existing structures.

2. It also provides prototype correlations which can be used to refine predictive models and design techniques generally.

(Effective post-construction monitoring appears until now to have been the exception rather than the rule as demonstrated by the findings of this study).

A significant amount of environmental and seafloor data will have been collected in advance of implementation as required under Canadian regulations. Often, however, this will not include more than a few months of measurements and once only bottom surveys from which long-term extreme conditions were originally deduced for design purposes. However, since seafloor structures must often be depended upon for many years and frequently additional structures will be added in an active area, it is necessary to conduct post construction monitoring of scour related parameters on a continuing basis.

Canadian Oil and Gas Drilling Regulations already make provision for three-hourly observations of nine environmental parameters including sea state and currents. However, the requirements are very general and do not appear to include provisions for instrumented measurements or for monitoring the seafloor for actual occurrences of scour. Therefore, in cases where a scour hazard has been identified, post construction monitoring specifically designed to measure scour and scour producing phenomena should be made. In most cases measurements of bottom current will be of the greatest value and ideally these should be made some distance from the structure and at two heights above the bed to define the undisturbed boundary layer. The use of modern instruments with high sampling rates will provide data on any residual wave effects also.

Measurements made close to a structure, for example using meters cantilivered from the leg of a rig, though quite common, are of less value, except for research purposes, because the flow there is disturbed by the proximity of the structure itself.

Where waves are known to have an influence on scour, in particular with shallow water structures, direct measurement of waves should also be considered.

Post-construction monitoring for scour should be undertaken periodically and following major storms in places where there is a potential scour hazard. This should include diver inspections for local scour following storms and routine resurveys of the bed around the structure to check for slower global effects that might pose a longer term hazard.

It is widely believed that the most severe destablilizing effects of scour occur only during the course of severe storms, but unfortunately there does not appear to be an effective way of monitoring such ocurrences and no specific references to attempts at field measurement were found in the literature. A device capable of measuring the intergranular component of soil pressure on a long term basis in a marine environment would be required for this.

6.6 Remedial Measures

There are two scenarios under which remedial measures may be required. If scour protection was initially included in the design but subsequently failed and gravel, stone, or rock protection is desired, the size of the material should be increased and/or the thickness of the protective layer may be increased. Other remedial measures include underpinning pipelines with grout filled bags for which specific guidelines are available (Ocean Industry Feb. 1980) or the use of artificial seaweed for which guidelines are usually arrived at by prototype experimentation.

7. APPLICATION TO THE SCOTIAN SHELF

7.1 Outline of Physical Environmental Conditions

The information presented in this section has been abstracted largely from the Venture Development Project Environmental Impact Statement prepared for Mobil Oil Canada Ltd. The sections of primary interest were Volume III a & b, The Biophysical Assessment.

7.1.1 Hydrodynamic Conditions

7.1.1.1 Currents

Information has been gathered on currents on the Scotian Shelf from moored current meters, bottom drifter experiments and hydrodynamic models. The moored current meter data consists of time series of current speed and direction averaged over twenty minute intervals taken at five locations in the vicinity of Sable Island in 1972, 1973, 1976 and 1977 (Evans-Hamilton, 1977a). Current measurements were also taken with bottom drifters near the Venture site at Sable Island (Evans-Hamitlon, 1977b). A numerical flow model which included the effects of tides, winds, bottom friction and bathymetry was implemented by Ocean Science and Engineering Inc. (1971 a, b)) to synthesize the available current measurements and to simulate the natural circulation on the Scotian Shelf.

The current measurements were resolved into three primary components: tidal currents, wind driven currents and mean or coastal currents. A summary of the values of these three components is listed for six different hindcast sites around Sable Island In Table 7.1. While the tidal currents represent the major component of regular currents the wind-driven current becomes equally important during extreme events. Unlike tidal currents which are relatively constant with depth, wind-driven currents decrease with depth as shown in Table 7.2. It is evident from Tables 7.1 and 7.2 that sediment transport and scour are associated with storms

Table 7.1 Current Parameters at Sable Island (Site Locations Listed in Figure 7.1) (Evans-Hamilton Inc. 1977a)

	Site A	Site B	Site C	Site D	Site E	Site F (Venture)
Tidal Currents[1]						
Major Axis (mag.)	73.2	73.2	79.2	79.2	73.2	73.2
(dir.)	160/340	160/340	160/340	160/340	160/340	160/340
Minor Axis (mag.)	36.3	36.6	42.7	42.7	36.3	36.6
(dir.)	070/250	070/250	070/250	070/250	070/250	070/250
Mean Currents[1]						
(mag.)	6.1	6.1	6.1	6.1	6.1	6.1
(dir.)	260	260	260	260	260	260
Wind-Driven Currents[1]						
10-year (mag.)	70.1	73.2	76.2	76.2	64.0	61.0
(dir.)	080	055	070	070	120	060
25-year (mag.)	82.3	85.3	88.4	88.4	70.1	73.2
(dir.)	080	055	070	070	120	060
50-year (mag.)	94.5	97.5	100.6	100.6	79.2	82.3
(dir.)	080	055	070	070	120	060
100-year (mag.)	103.6	106.7	109.7	109.7	79.2	82.3
(dir.)	080	055	070	070	120	060

[1] Magnitude in $cm \cdot s^{-1}$ and directions in degrees True towards

Note: Accuracy (Ocean Science and Engineering Inc., 1971a,b):

Tidal Currents ± 25%

Wind-driven Currents ± 30%

Table 7.2 Design Current Velocities for the Venture Site (Evans-Hamilton Inc. 1977a)

d/D[1]	Direction (degree toward)	Current in the Direction of Maximum Wave Height ($cm \cdot s^{-1}$) — Recurrence Interval (years) 10	25	50	100
.10	025	134	140	149	155
.25	025	119	125	131	134
.50	025	101	107	110	113
.75	025	088	094	098	101
.90	025	070	073	073	076
d/D[1]	**Direction (degree toward)**	**Maximum Current ($cm \cdot s^{-1}$)**			
.10	060	143	158	177	186
.25	060	132	131	143	152
.50	060	101	107	110	116
.75	060	088	094	098	101
.90	060	070	073	073	076

[1] d/D is ratio of depth to total depth at site.

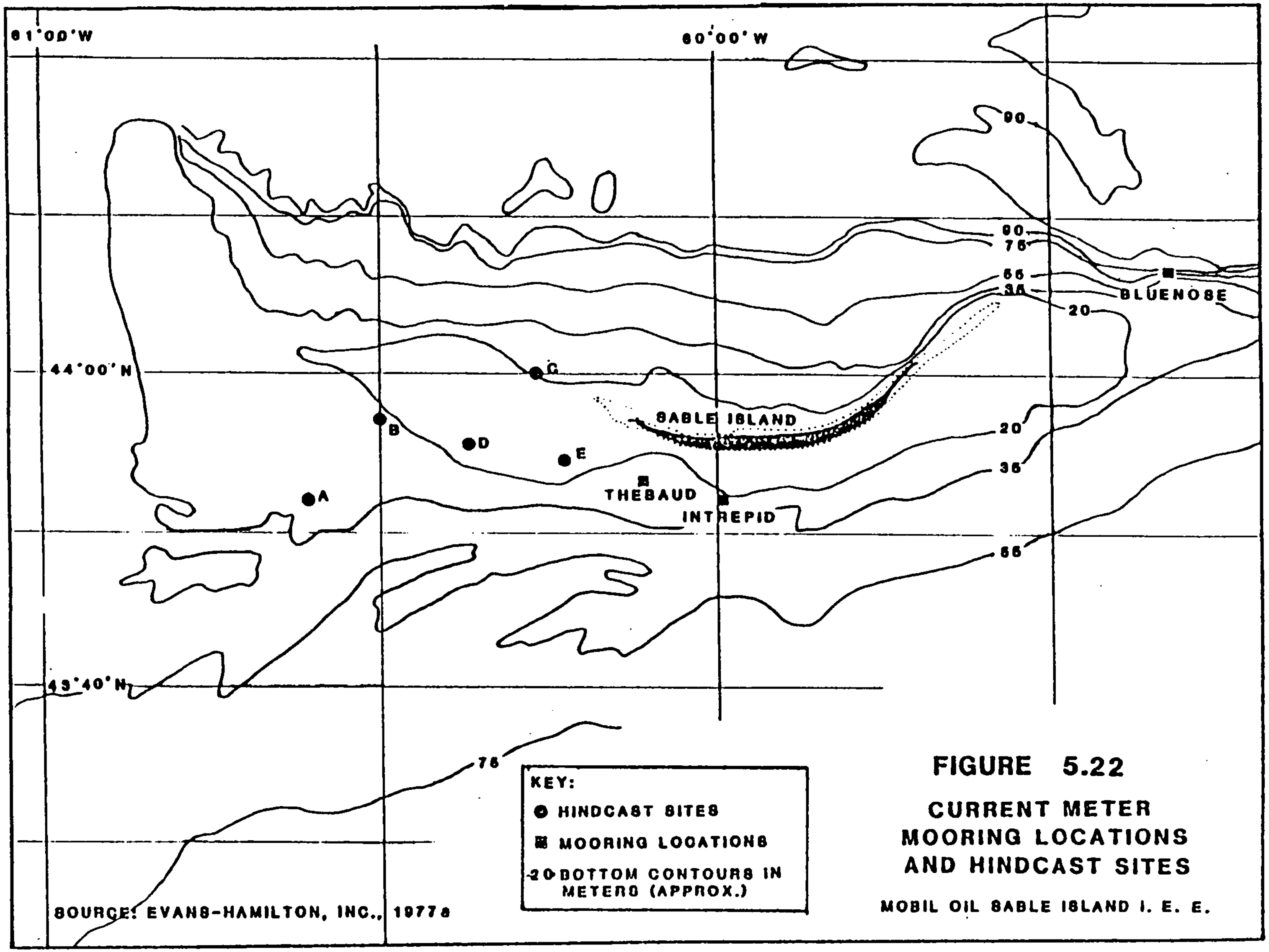


Figure 7.1 Mobil's Current Meter Mooring Locations and Hindcast Sites (Evans-Hamilton, 1977a)

because winds may in some instances double the currents due to tide alone, and hence nearly quadruple the shear stresses acting on the sea floor.

Table 7.2 also shows the strength of wind-driven currents when their direction coincides with the direction of maximum wave height, while this is important in the design for structural stability it may be less important for scour since the oscillatory motion of waves acts more as a stirring mechanism than a transporting mechanism. Of greater concern for scour would be the coincidence of tidal and wind-driven current in which case combined currents may exceed 2.0 m/s in extreme storm events. However, at most sites the extreme wind-driven currents are aligned with the minor axis of the regular tidal currents as indicated in Table 7.1, and the combined effects are less severe.

7.1.1.2 Waves

Owing to the lack of long-term direct measurements and the problems of interpreting visual observations, the detailed determination of the sea state for the Scotian Shelf will depend mainly on hindcasts computed using wind and surface pressure field data. The hindcasts would have been verified to the extent possible using direct measurements and visual observations.

The United States Army Corp of Engineers at the Waterways Experiment Station (WES) have developed a deep-water two-dimensional wind-wave hindcast model for various points on an orthogonal grid in the North Atlantic. One of these deepwater grid points lies near Sable Island. Results from the hindcast were modified to account for wave refraction, wave shoaling and island sheltering. A summary of a Gumbel extreme value analysis carried out for two sites within the proximity of Sable Island is presented in Table 7.3. The 100 year significant wave height varies from 13 m to 15 m with a significant period of about 13 s. The variability of the 100 year wave at the different sites indicates the sensitivity to bathymetry, water depth and sheltering. It should be noted here that in a recent study for MEDS, Resio (1982) suggests the extreme values determined from WES data may be overestimated. Because of problems with the coarseness of the pressure grid, the definition of the shoreline in Canadian waters and the accuracy of the wind field description (outlined by Baird and Readshaw, 1981) the WES data extreme value analysis is not considered adequate for design purposes.

The normal wave conditions were assessed by two wave hindcast techniques and by a compilation of visual sea-state observations reported by ships available from the Atmospheric Environment Service (AES). The ship observations provide an adequate estimation of average conditions and are presented as monthly wave height exceedance plots in Figure 7.3. Data on wave periods from ship observations is usually unreliable. The normal wave conditions have also been determined by hindcasts from the WES data and also by a wind-wave hindcast model with modifications for shallow water developed by SUNY. Exceedance and persistance data generated from the SUNY model for two stations are summarised in Figure 7.4 and Table 7.4 respectively. A distribution of sea and swell height by period from the WES data are given in Table 7.5 and 7.6 respectively. As expected, the winter season from October to April features the most severe wave climate much of which originates from the northwest. In the summer, waves primarily come from the southwest (these conditions are apparent from a series of seasonal wind roses for Sable Island shown in Figure 7.5).

7.1.2 Storms

Three principal types of storms effect the study area: the frontal system, the extratropical storm and the tropical storm (or hurricane). Wind-wave forecasts should treat frontal systems and hurricanes separately since they are different populations of random values (LeBlond, 1981). Extratropical storms are particulary deep low-pressure systems which form in the frontal zone over eastern North America and can be treated as part of the frontal population. Tropical storms (with mean winds 34 to 63 knots) on average, affect the Scotian Shelf less than once a year (Evans-Hamilton inc., 1977). Hurricanes (winds greater than 64 knots) rarely reach the Scotian Shelf.

Table 7.3 Comparison of Design and Wave Elevation Parameters from the Rowan Juneau Drilling Site and Site F near Sable Island (See Figure 7.2) (Evans-Hamilton Inc. 1981b)

	Recurrence Internal (yr)[1]			
	Site F		Rowan Juneau Site	
Design Parameter	(44°02'N,59°33'W)		(43°59'04"N,59°38'10"W)	
	50	100	50	100
Significant wave height (m)	13.9	15.1	11.9	12.7
Maximum wave height (m)	17.1	17.3	19.8	20.3
Maximum crest elevation[2] (m)	13.7	13.8	15.6	16.0
Wave direction from (deg. True)	180.0	180.0	205/60	205/60
Significant period (s)	12.9	13.1	12.3	12.6
Storm tide[3] (m)	0.5	0.6	0.4	.05
Astronomical tide[4] (m)	1.6	1.6	1.6	1.6
Storm still water level above bottom (m)	21.9	22.0	26.0	26.1

[1]Chart depth at site F is 20 m and at Rowan Juneau, 24 m.
[2]Measured relative to storm still-water depth.
[3]Measured at time of maximum wave heights.
[4]Measured relative to Canadian chart datum (place of lowest normal tides).

Table 7.4 Summary of Wave Height Persistance (Ocean Science and Engineering Inc. 1972b)

Season	Site	Wave Height ≥ 1.5 m			Wave Height ≥ 2.4 m			Wave Height ≥ 3.7 m			Wave Height ≥ 4.4 m		
		Maximum Observed (h)	Mean Duration (h)	Number of Durations	Maximum Observed (h)	Mean Duration (h)	Number of Durations	Maximum Observed (h)	Mean Duration (h)	Number of Durations	Maximum Observed (h)	Mean Duration (h)	Number of Durations
Winter	1	249	59	24	86	23	24	49	24	7	6	4	2
	2	330	69	22	156	36	25	58	20	16	41	22	6
	Deep	370	94	18	228	47	24	82	28	18	52	27	7
Spring	1	258	64	22	103	39	15	51	32	5	36	20	2
	2	389	56	27	99	34	17	53	25	9	44	22	4
	Deep	609	126	14	107	50	17	65	35	10	49	25	6
Summer	1	102	45	8	40	33	3	7	7	1	0	0	0
	2	82	35	9	49	21	3	25	25	1	0	0	0
	Deep	109	44	11	65	49	3	35	35	1	14	14	1
Fall	1	162	59	18	72	46	6	39	28	4	22	12	2
	2	165	47	19	63	36	7	35	18	5	25	16	2
	Deep	189	96	12	77	45	8	50	26	6	31	28	2

See Figure 7.2 for Site Locations

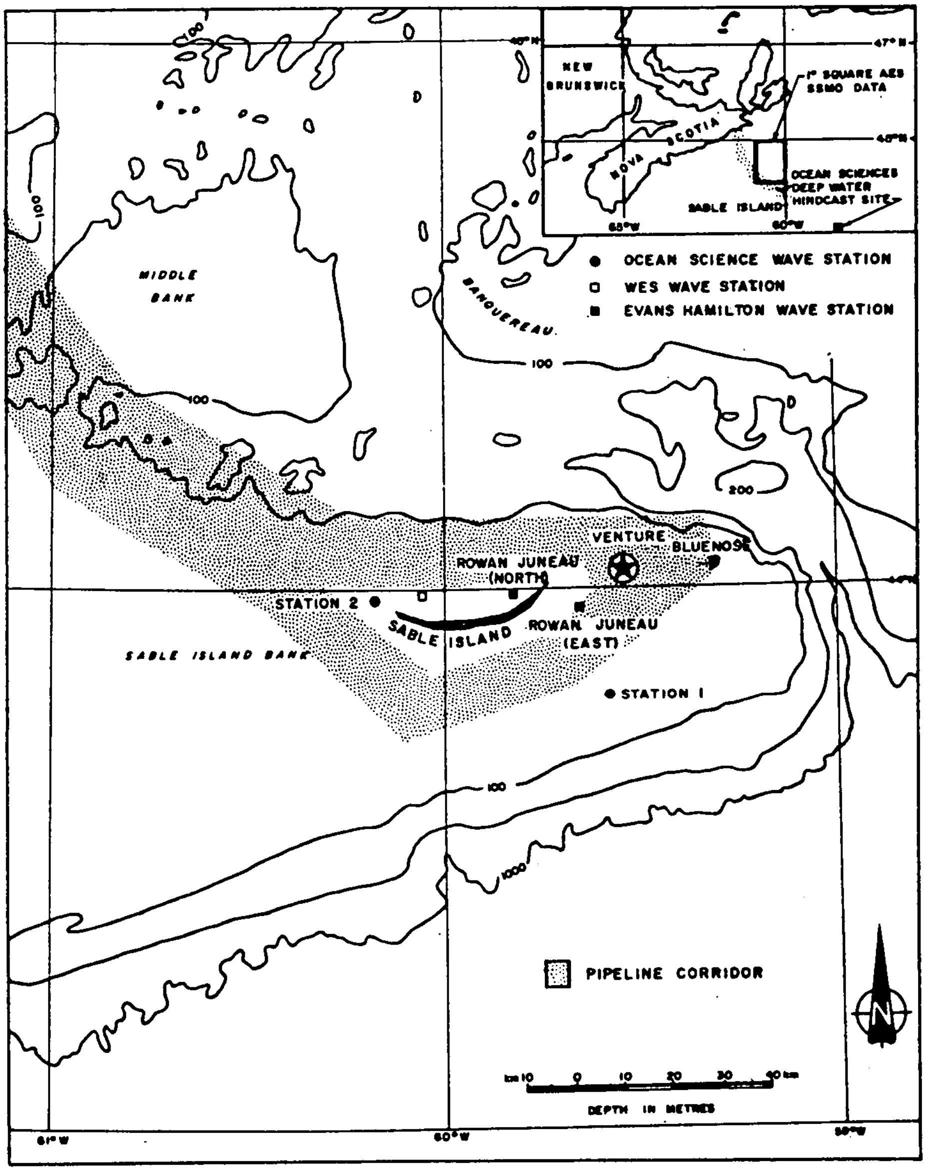


Figure 7.2 Wave Data Location Map (Evans-Hamilton Inc., 1981 a, b, 1982 a, b, c; Ocean Science and Engineering Inc. 1972 a)

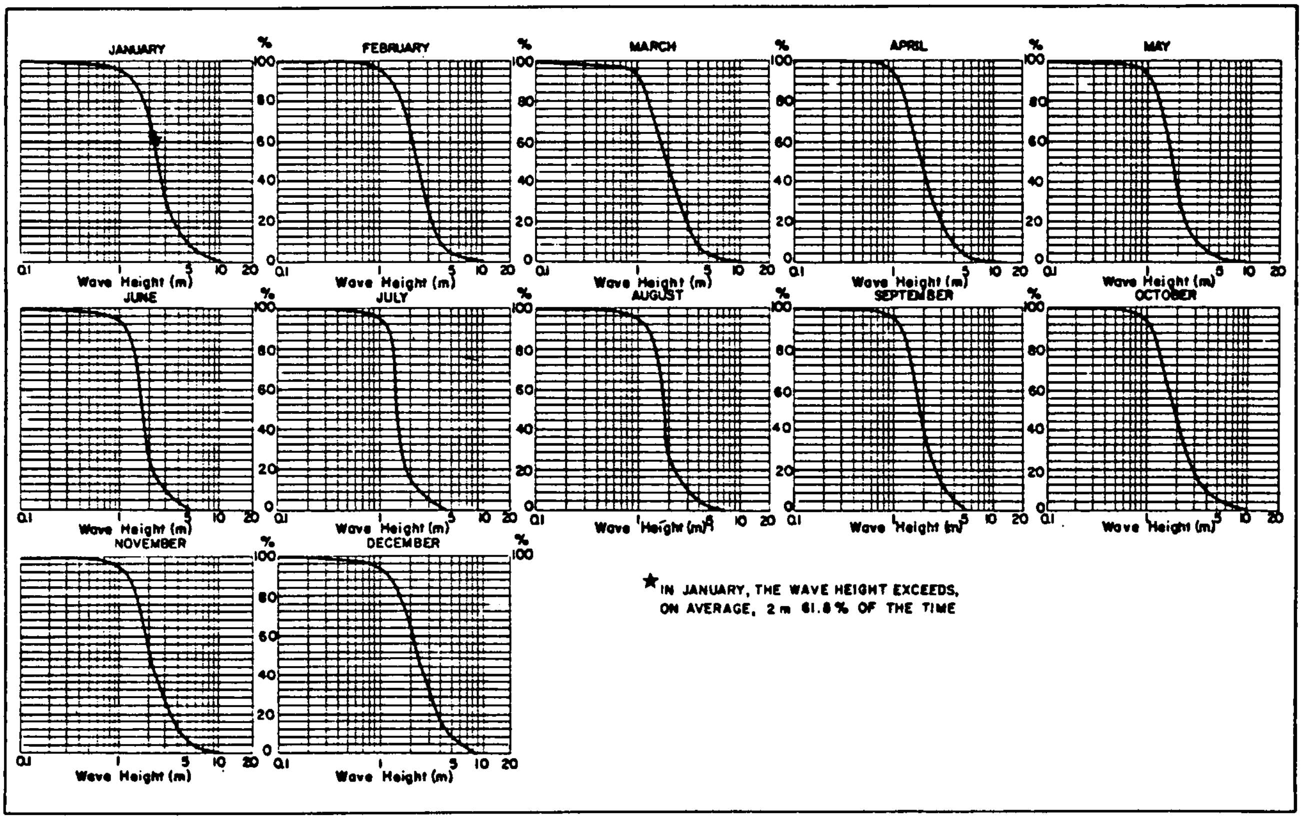


Figure 7.3 Monthly Wave Height Exceedance Plots from Ship Observation Data (May 1972-1977, AES Wave Climatology Project of the W.W. Atlantic 1978)

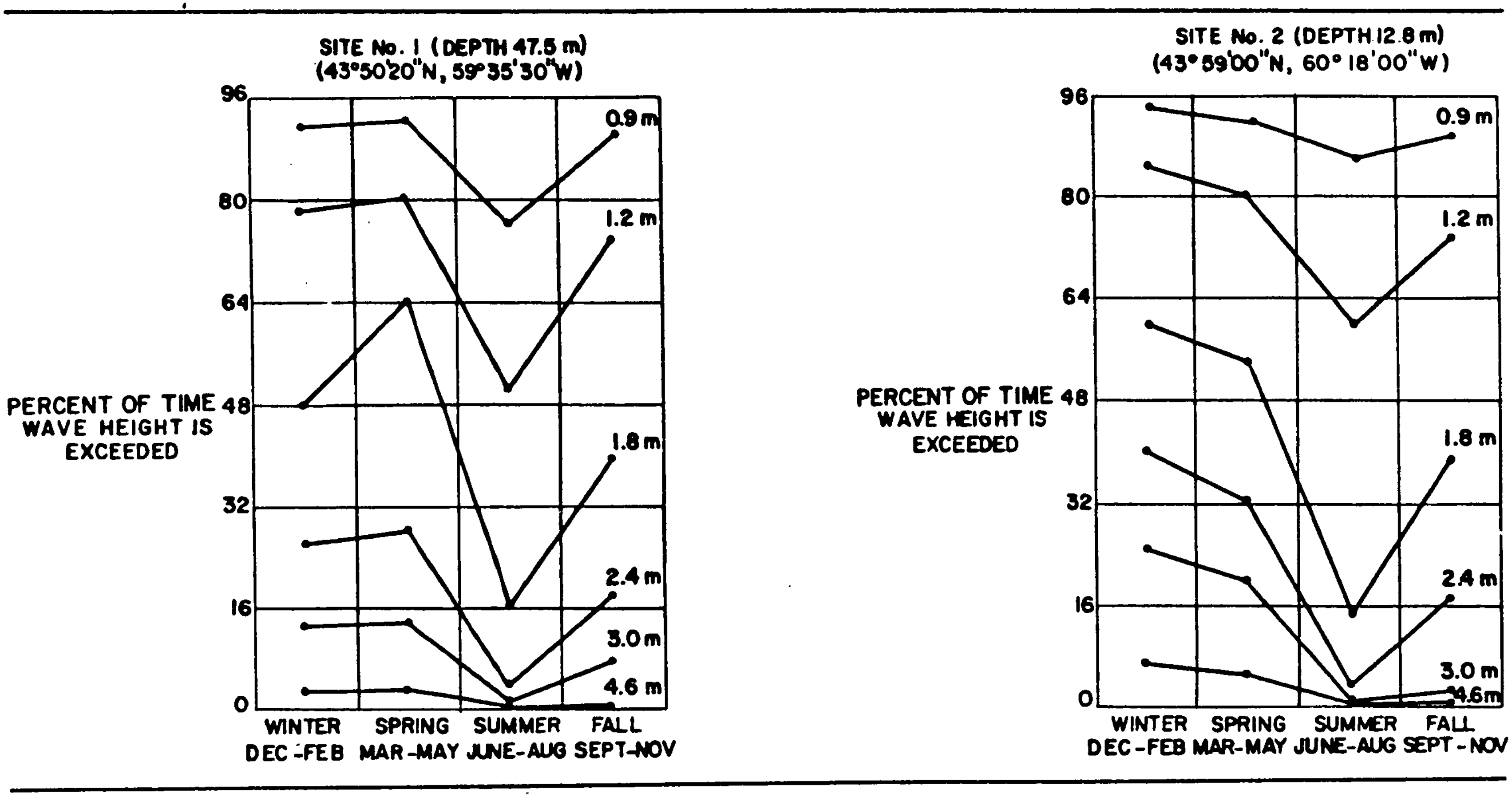


Figure 7.4 Significant Wave Height Exceedance from the New York University Model (Ocean Science and Engineering Inc. 1972 b)

Table 7.5 Percent Occurrence of Sea Height by Period for the WES Wave Station (see Figure 7.2) (Martec Limited, 1982)

Sea Period (s)	Sea Height (m) 0-1	1-2	2-3	3-4	4-5	5-6	6-7	7-8	8-9	9-10	10+	Total	Mean Height
0-1	0.219	.002	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.221	0.108
1-2	5.664	0.231	.029	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	5.924	0.270
2-3	17.527	1.044	0.110	.021	.007	0.000	0.000	0.000	0.000	0.000	0.000	18.708	0.453
3-4	16.588	2.636	0.194	.048	.003	0.000	0.000	0.000	0.000	0.000	0.000	19.468	0.764
4-5	7.310	10.630	0.336	.036	.002	0.000	0.000	0.000	0.000	0.000	0.000	18.313	1.125
5-6	.007	12.122	0.673	.067	.009	.002	0.000	0.000	0.000	0.000	0.000	12.880	1.590
6-7	0.000	2.092	3.496	3.451	1.323	.082	0.000	0.000	0.000	0.000	0.000	10.445	2.963
7-8	0.000	0.000	0.151	4.473	2.279	0.538	.089	.002	0.000	0.000	0.000	7.531	3.957
8-9	0.000	0.000	0.000	0.171	2.387	0.984	0.219	.045	.010	0.000	0.000	3.816	4.868
9-10	0.000	0.000	0.000	0.000	.057	1.160	0.641	.098	.031	.005	.002	1.993	5.941
10-11	0.000	0.000	0.000	0.000	0.000	.005	0.312	0.228	.027	.010	0.000	0.583	7.019
11-12	0.000	0.000	0.000	0.000	0.000	0.000	0.000	.039	.045	.005	.003	.093	8.226
12-13	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	.002	.005	.007	.014	10.073
13-14	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	.003	.003	12.329
14-15	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	.003	.003	12.770
15-16	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
16 or more	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
TOTAL	47.315	28.756	4.938	8.267	6.066	2.771	1.261	.411	.115	.026	.019		

Table 7.6 Percent Occurrence of Swell Height by Period for the WES Wave Station (see Figure 7.2) (Martec Limited, 1982)

Swell Period (s)	Swell Height (m) 0-1	1-2	2-3	3-4	4-5	5-6	6-7	7-8	8-9	9-10	10+	Total	Mean Height
0-1	.036	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	.036	0.137
1-2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
2-3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
3-4	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
4-5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
5-6	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
6-7	16.422	8.429	1.164	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	26.014	0.904
7-8	7.177	7.285	1.808	.067	0.000	0.000	0.000	0.000	0.000	0.000	0.000	16.336	1.190
8-9	3.868	4.755	1.138	0.473	0.000	0.000	0.000	0.000	0.000	0.000	0.000	10.234	1.342
9-10	3.256	6.073	3.827	1.122	0.221	.010	0.000	0.000	0.000	0.000	0.000	14.509	1.753
10-11	0.233	1.006	1.227	0.276	.096	.022	0.000	0.000	0.000	0.000	0.000	2.860	2.175
11-12	0.185	1.083	2.204	0.631	0.113	.048	.010	0.000	0.000	0.000	0.000	4.274	2.415
12-13	.072	0.396	0.641	0.456	.045	.031	.007	.002	0.000	0.000	0.000	1.649	2.587
13-14	.034	0.228	0.308	0.319	.084	.009	0.000	0.000	0.000	0.000	0.000	0.982	2.736
14-15	0.360	0.111	.0190	0.177	0.113	.014	0.000	0.000	0.000	0.000	0.000	0.641	2.883
15-16	.009	.043	.093	.091	.060	.027	.003	0.000	0.000	0.000	0.000	0.326	3.287
16 or more	0.226	.074	.007	.026	.027	.043	.017	.009	.003	0.000	0.000	0.432	2.031
TOTAL	31.553	29.483	12.605	3.636	.759	.204	.038	.010	.003	0.000	0.000		

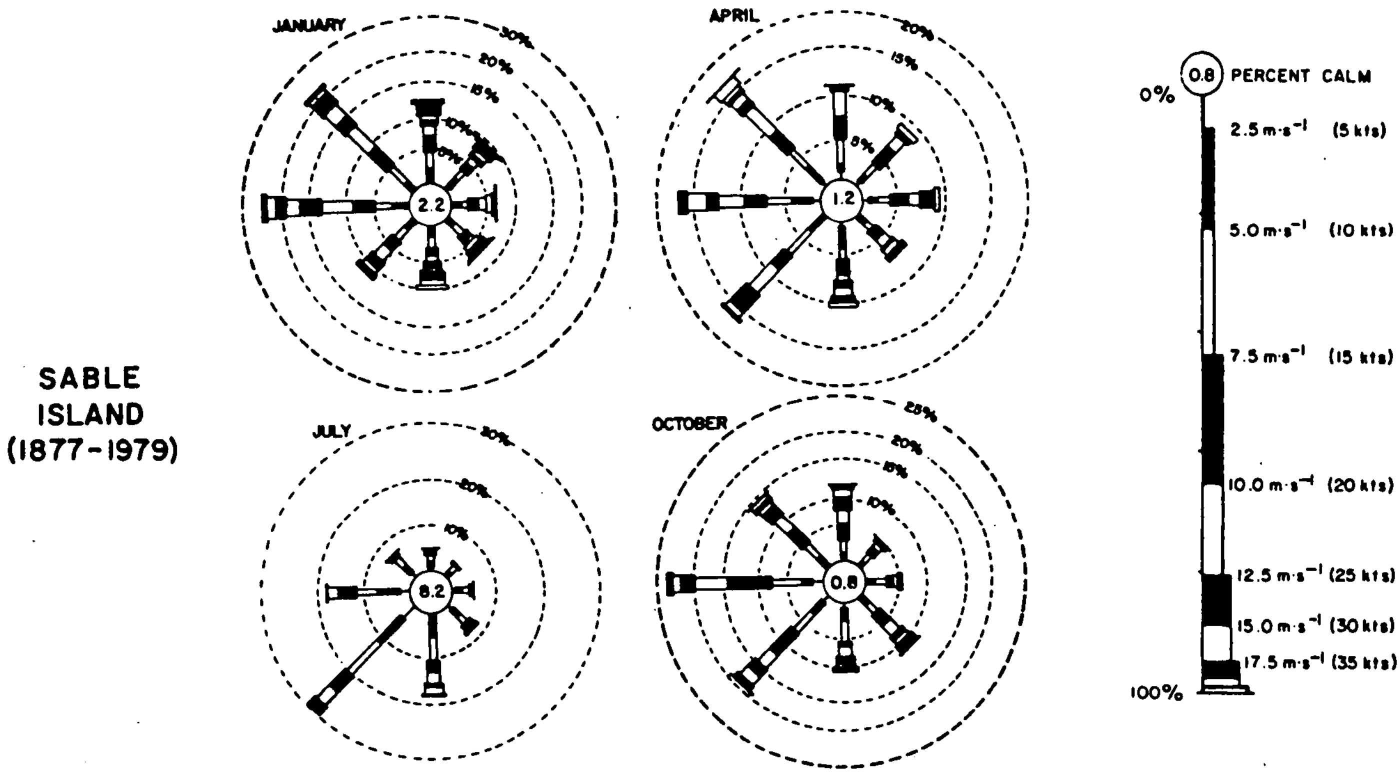


Figure 7.5 Seasonal Wind Roses for Sable Island (Mobil Oil Canada Ltd., 1983)

7.1.3 Bottom Sediment Conditions

The surficial geology of the Scotian Shelf has been mapped by King (1970), MacLean and King (1971) and MacLean et al (1977) and is shown in Figure 7.6. A large portion of the Scotian Shelf, almost exclusively where depths are less than 100 m, is covered by Sable Island Sand and Gravel with less than 50% gravel. This formation is a fine to coarse grained well-sorted sand with the inclusion of coarse gravel and rounded boulders at the upper grain size limit. A second subdivision of Sable Island Sand and Gravel is composed of less than 50% sand. Isolated deposits of this material are found in deeper water north of Sable Island. Both of these units are thin with deposits no thicker than 15 m.

7.1.4 Bedform and Sediment Mobility

The movement and size of sand waves is an important phenomenon when considering scour. Large sand waves exist to the north and to the south of the western part of Sable Island. The sand waves are more abundant on the south side where the sand is finer. In studies by Evans-Hamilton Inc. (1976) and Geomarine Associates (1983), sand waves were found to range from 2 m to 6 m height with lengths from 80 m to 3500 m around the island. The water depth varies from 10 m to 75 m. The existence of sand waves usually indicates that there is active sediment transport at these depths. Sand waves and the surficial geology in the Venture Field pipeline corridor are shown in Figure 7.7. The general sediment transport mechanisms are show in Figure 7.8.

The depths at which sea floor structures might be installed on the Scotian Shelf could range from less than 10 m to more than 300 m. The depth within the Venture Field pipeline corridor varies from 20 m near Sable Island to a maximum of 100 m on the banks and to 320 m in the channels, (See Figure 7.9) Petrie (1975) states that at a depth of 100 m, tidal currents have velocities of 15 cm/s. According to the Shields curve this

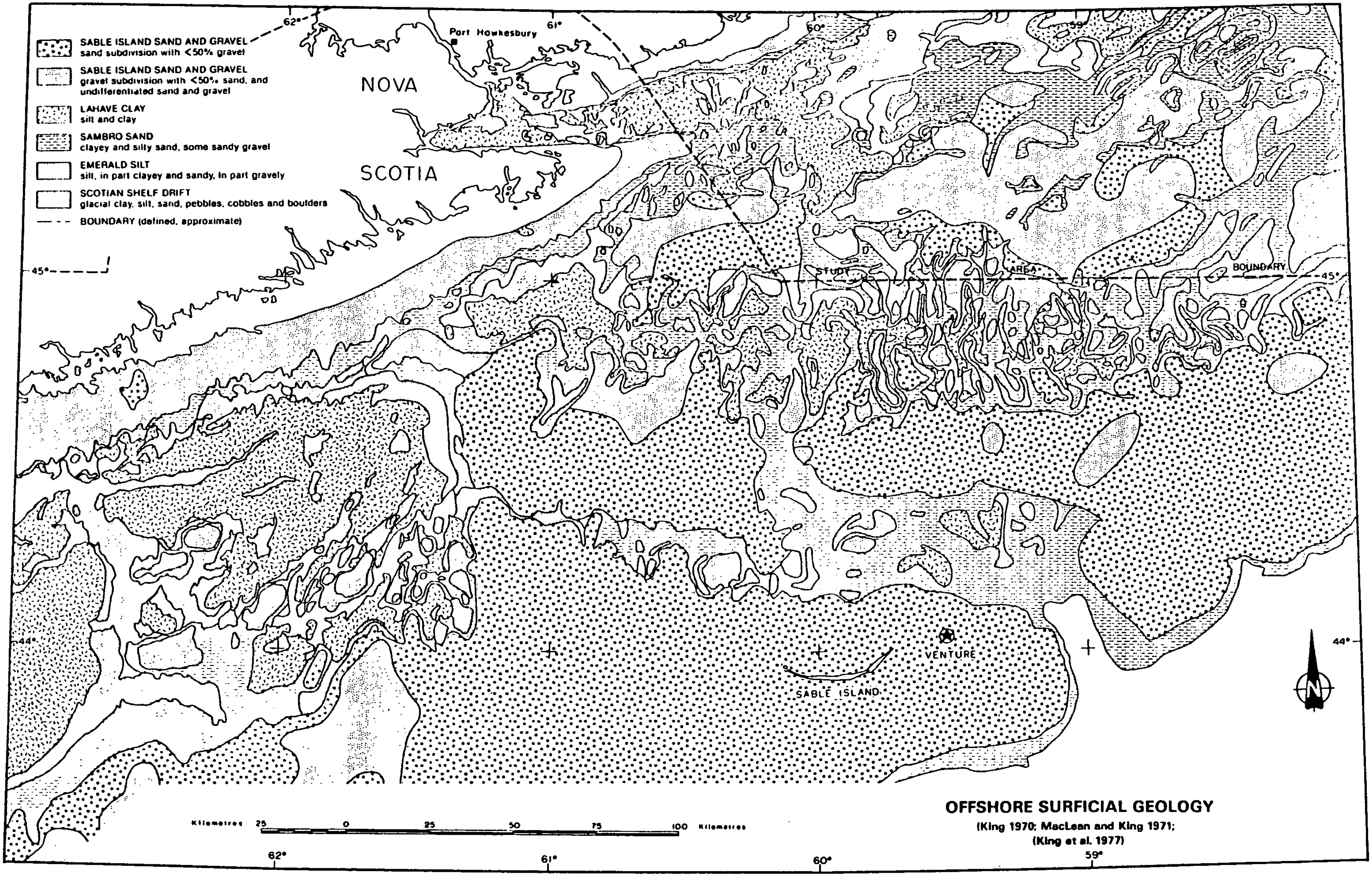


Figure 7.6 Offshore Surficial Geology (King 1970; MacLean and King 1971; MacLean et al 1977)

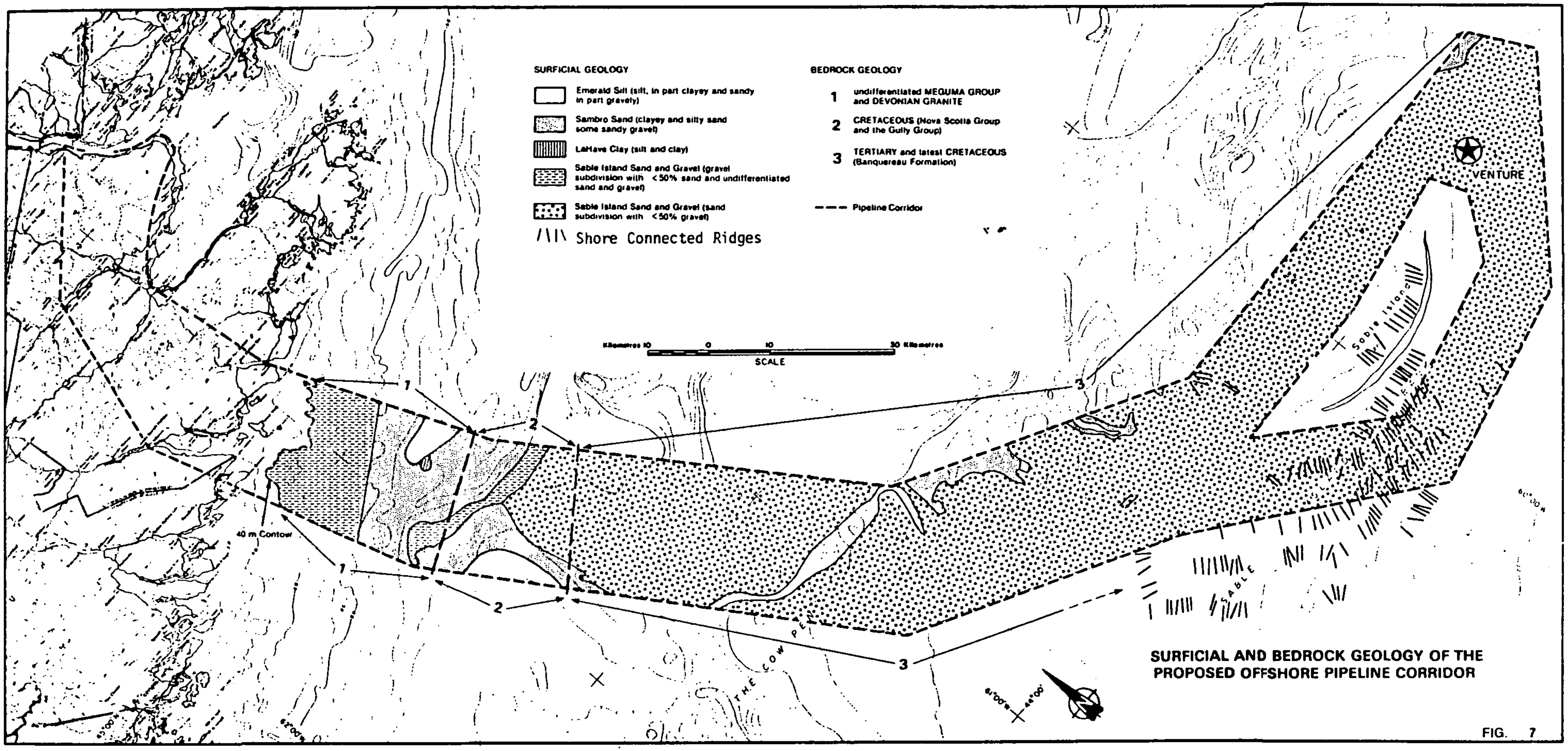


Figure 7.7 Surficial Geology and Sand Waves in the Venture Field Corridor (Mobil Oil, 1983)

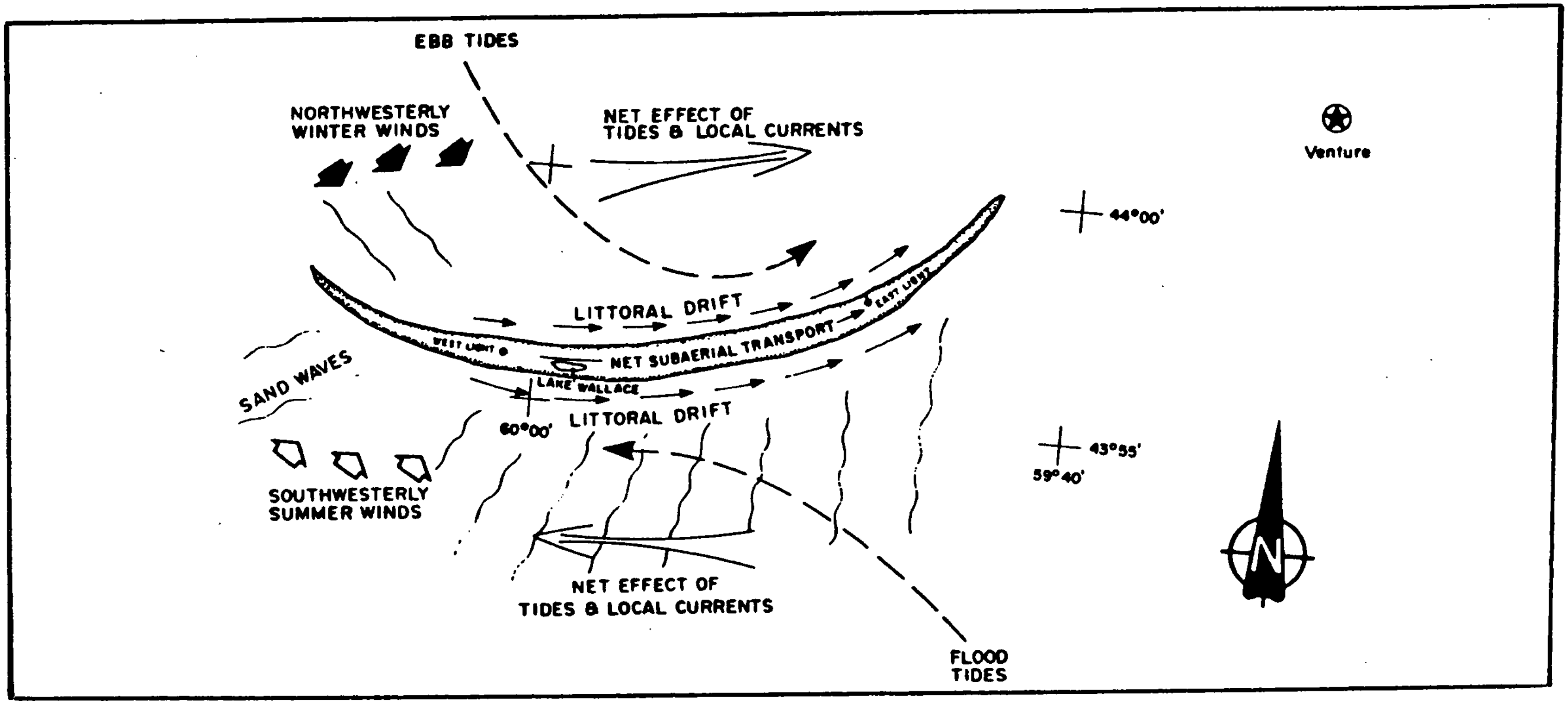


Figure 7.8 Sediment Transport Mechanisms Around Sable Island (James and Stanley 1968; Evans-Hamilton Inc. 1976, 1978; Martec Limited 1980)

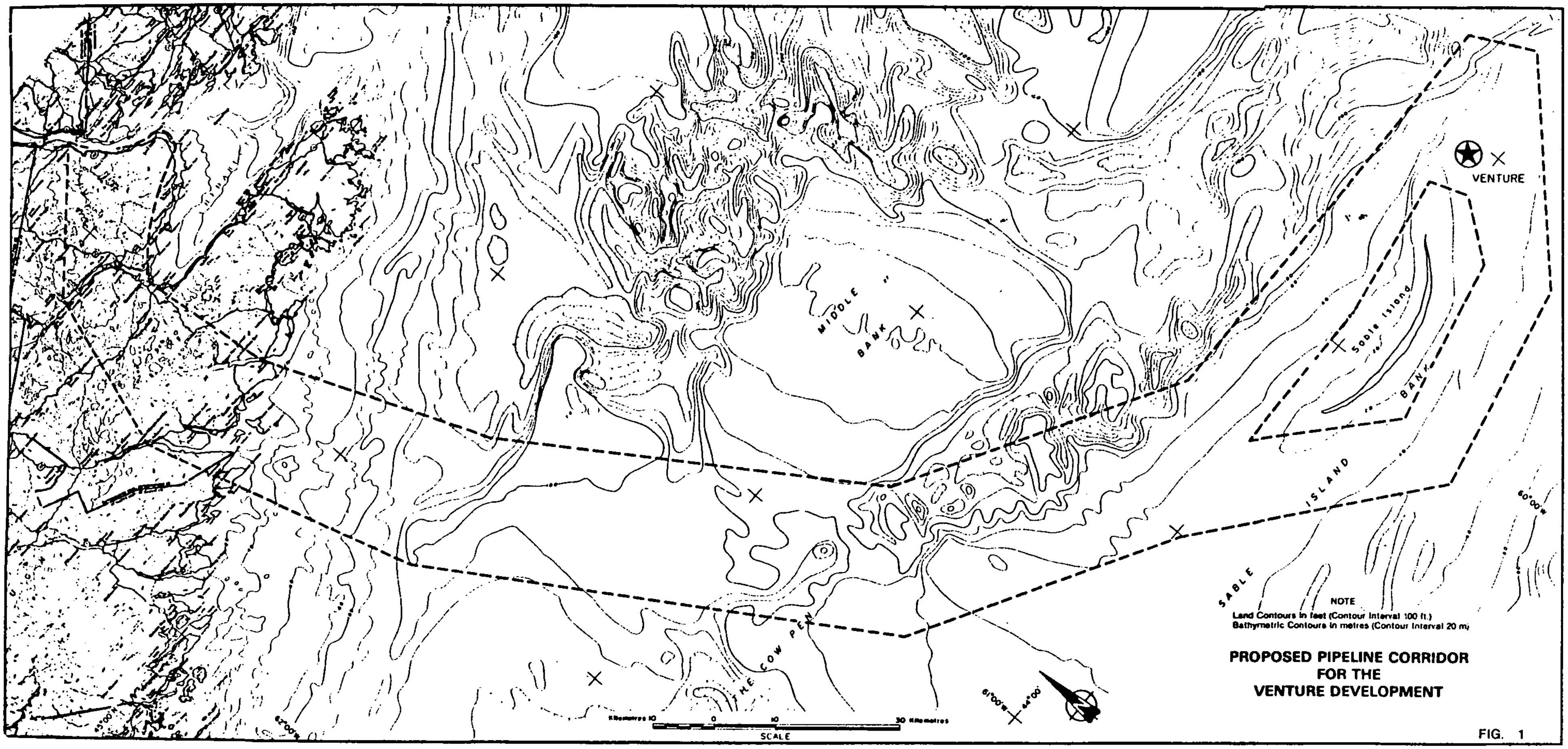


Figure 7.9 Proposed Pipeline Corridor for the Venture Field Development (Mobil Oil Canada Ltd., 1983)

current is just sufficient to initiate sediment motion of fine sands. This also falls within a range of threshold velocities postulated from 0.1 m/s (Allen, 1970) to 0.4 m/s (Evans-Hamilton Inc. 1975). Furthermore, ambient velocities due to flow patterns induced by seafloor structures will be locally increase 2 or 3 times by convective acceleration. Storm conditions would not extend this depth for incipient sediment motion because wind-driven currents and oscillatory fluid motion due to waves are insignificant at these depths.

Unfortunately, little is known about turbidity and other density currents within the deep channels and the potential for scour cannot be ruled out in these areas regardless of depth. Hollister et al (1984) have measured strong oceanic currents at great depths south of the Scotian Shelf.

7.2 Probable Types of Structure and Scour Hazards

Information from both Mobil Oil Canada Ltd., and Shell Canada, the two major interest holders with strong prospects for production, indicates that pile supported jacket type structures are favoured for production platforms on the Scotian Shelf. These would have at least four legs penetrating the sea bed and possibly six or even eight legs depending on the number of wells and other facilities required. Each production platform might support anything from two to about a dozen conductor pipes. Thus there could be anywhere from six to, say, twenty members penetrating the sea bed and generating eddies that could cause scour. Such assemblages of members can cause global or "dishpan" scour as well as local scour around each member. In addition to permanent platforms there may well be a continuing role for jack-ups that could result in increased scour in the vicinity of the production platforms. This could occur where jack-ups are installed close to production platforms to perform "workovers", that is to say repair and redrilling of existing production wells that are damaged or have become partially blocked during production operations. The addition of three lattice framed spuds in the vicinity of say a six legged platform might well increase the total scour at adjacent legs of the production platform.

The water depths at which piled jacket structures might be installed could vary from less than 10 m for the Ventue Field to depths of 80-100 metres for the Shell holdings. Within this range of depth there is a definite risk of scour as illustrated by experience with jack-ups in the area. (Case Study A4.1.6)

Although there are interest holding areas along the outer slopes of the Scotian Shelf which extend to depths exceeding 2000 m these lie well beyond the range of conventional scour. They are therefore not within the scope of this study although some reference to current forces acting on bottom sediments at and below such depths has been made elsewhere in this report. See Hollister, 1984.

For the Venture Field, plans are being developed for a gas pipeline connection to mainland Nova Scotia. A pipeline corridor has been designated and is shown if Figure 7.9. It is understood from Mobil Oil Canada Ltd. and Sable Gas Systems, who are both interested in this project, that relatively detailed route surveys have been made and presented in proprietary reports. Although these documents have not been made available for this study it is understood that the probable pipeline route (or routes) encompass a more or less comprehensive range of bottom conditions, requiring a wide range of pipeline installation and anchoring techniques. Depths range from less than 20 metes to more than 300 metres. Bottom conditions range from sand ridges to silts and muds to areas of more or less bare rock. Active sand waves or sand ridges would of course present a major hazard to pipelines and for this reason they are the subject of a companion study and are not further addressed here. Soft silts and muds usually occurring beyond the depth of conventional scour present other types of bottom instability problems touched on in 3.2 but not pursued in detail. Likewise pipelines laid on rock or other unavoidable bed materials present yet another type of problem. However, vulnerability to scour is maximized if there are areas where hard impermeable substrata are covered with relatively thin layers of mobile sand.

7.3 Scour Design Procedures for the Scotian Shelf

Preceeding sub-sections 7.1 and 7.2 outline what is known of environmental conditions and probable types of structures for the Scotian Shelf area. This section abstracts from the foregoing sections of the report those aspects of scour design procedures which appear best suited to these conditions. It is understood that in many cases the indications given here are already (1984) being implemented by the interest holders, by means of companion ESRF studies or by others.

7.3.1 Preconstruction Data Collection

7.3.1.1 Currents

Currents are most important in causing scour.

There is a clear need for additional current measurements near the seafloor in the vicinity of likely structures, and along the proposed pipeline corridor.

Because of the complex bathymetry of the area and the importance of storms in causing scour, measurements should be supplemented by numerical methods. Additional current measurements may be needed to calibrate and verify such models.

7.3.1.2 Waves

Waves will generally be less important than currents in causing scour over the Scotian Shelf.

However, as with currents, the complexity of the bathymetry must produce significant variations in wave conditions from location to location, especially where waves must tranverse areas less than about 20 m in depth.

The best way of systematically assessing these variations combines measurement and numerical modelling techniques as for currents. The fact that the Scotian Shelf is open to the Atlantic Ocean poses a modelling problem that can only be truly solved by interfacing a local detailed model with an oceanic model to simulate wave energy reaching the Shelf from the ocean. On the other hand, the more severe storms most commonly proceed from west to northwesterly directions, minimizing the oceanic contribution. Hence localized numerical models may be used alone under appropriate types of conditions, for example to determine extreme conditions as required for scour predictions.

For detailed investigation of a limited number of extreme storms the use of advanced two-dimensional spectral models and shallow water wave models are required because of the variations in water depth which suggest simultaneous occurrence of generation, refraction and frictional dissipation. Such models are complex and must be calibrated and verified for selected storms for which simultaneous wave measurements are available at several places in the area of the model.

To develop comprehensive extreme wave statistics for the Scotian Shelf the much less costly parametric wave hindcasting can be applied. The latest parametric modelling systems can cope with relatively complex fetch conditions. Where complex bathymetry is encountered close to the area of interest, wave spectra and characteristic wave parameters should be adjusted by use of wave refraction, spectral saturation and spectral transfer computations. By this process directional wave spectra at a hindcasting station are transferred and modified to produce corresponding directional spectra at several other places, usually in shallower water, in the same area. Partial breaking and breaking should also be accounted for.

7.3.1.3 Seafloor Conditions

Detailed surveys of bottom conditions including observations over time are required on the Scotian Shelf especially in areas of sand waves to determine or confirm rates of sand wave migration. Also, since scour problems were encountered with jack-ups in the area, close attention must generally be paid to bottom sediment texture at sites of all proposed structures placed in less than say 50 metres of water.

Recently, sand transport measurements have been made around Sable Island using radioactive traces (see Amos et al, 1985). Also, direct measurements of bed level changes will be made in an experiment sponsored by ESRF early in 1986.

7.3.2 Scour Prediction Methods

7.3.2.1 Piled Structures

Two aspects must be considered:

1. Local scour occurring close to individual members both vertical and horizontal. The latter usually comprising horizontal bracing above the bed. Use of jack-ups for workovers has also to be considered.

2. Dishpan scour due to the combined effect of a cluster of legs, conductor pipes, and horizontal bracing.

The two phenomena are additive, and hence both must be considered in long-term predictions.

Because there are no theoretical approaches to local scour, reliance must be placed on empirical data. There are more data on local scour around isolated piles and cylinders than about other seafloor structures. However, most of it is based on models, and the ranges of parameters considered is in most cases restricted (See Section 5.2). It will therefore depend on both local environmental conditions and rig design whether data from the literature may be used or whether further experiments will be needed. Scour caused by horizontal bracing close to but just above the bottom may be considered in the light of the research of Zdravkovich and Kirkham (1982), but again additional experiments may be required.

The case of dishpan scour is more problematic, if as suggested in Section 5, it is looked upon as the result of general acceleration of the flow in the vicinity of a structure it may be computed by using available sediment transport models. On the other hand, this will underestimate the effect if it includes the more subtle phenomena related to a general increase in turbulence due to the presence of the whole structure but beyond the influence of the identifiable vortex effects generated by the individual members. Unfortunately, this question has not been addressed in the literature and so the only suggestion that can be offered for the Scotian Shelf is that more research be conducted if the issue seems important when a particular structure is being designed.

7.3.3 Scour Protection Methods

Because there are plentiful supplies of gravel, rock, and crushed stone in Nova Scotia, there is little doubt that these materials would offer a preferred scour protection method provided a suitable means of placement is available. In this regard contractors in Europe have developed specialized vessels for placing gravel and rock mattresses, so presumably the same will be possible in Canada.

7.3.4 Post-Construction Monitoring

As noted elsewhere, the record for systematic monitoring of scour and scour causing parameters has been poor until now. Monitoring of the erosion of artificial islands in the Beaufort Sea has been undertaken by the various oil companies although the data collected is usually proprietary.

8 DISCUSSION AND CONCLUSIONS

8.1 Scour Descriptions

Throughout the literature contradictions arise concerning the comparative amounts of scour (including ultimate scour depth and the lateral extent of scour) under the conditions of currents or waves alone, and with a combination of waves and currents. These differences may in part be attributable to the lack of a standardized approach to the descriptions of local scour. For example, when just "scour" or "extent of scour" are referred to, it is not clear if the parameter intended is ultimate maximum depth, ultimate average depth, volume, lateral extent, area affected by scour, etc.

Likewise, when the statement is made, as it often is, that waves alone, or waves and currents combined cause less scour than currents alone the basis on which the comparison is made is often not clear. It seems likely that when waves are added to a pre-existing current, the maximum depth of scour is unlikely to decrease. When two different conditions, one with waves alone, and the other with waves and currents, both give rise to the same peak velocity or shear stress, then it is apparent that the condition with waves and currents will cause more scour than the one with waves alone. Finally, when two conditions, one with currents alone and the other with waves and currents combined give rise to equivalent velocities or shear stress it is again possible to accept that the current alone will cause the greater volume of scour, although in these circumstances the basis on which velocity or shear stress are equated needs to be defined.

So far as can be determined, the difficulty discussed usually arises in the description and interpretation of model results. Scale effects in physical models of mobile bed phenomena (such as scour) almost always lead to an incorrectly modelled boundary layer. In light of this, results from physical model tests may not give a clear indication of scour in the prototype. Niedorada (1981) presents theroetical arguments that suggest prototype scale waves may be relatively more important in causing scour than they appear from model results.

However, while it is obviously important to understand the meanings intended in technical writing concerning local scour, the distinctions may not be so important in practice because in most cases design must be based on extreme conditions under which currents will be maximized and wave effects, if they are significant, will be superimposed on those of the currents.

There is very little information and no prototype data on general scour. The phenomenom is sometimes seen as no more than a localized increase in bottom sediment transport due to the increased velocity caused by the obstruction presented by members of a structure. However, this may be an oversimplification that underestimates its magnitude because it ignores the likely increase in background turbulence that is caused by a structure in addition to the characteristic vortices. The only formula found in the literature for general scour suggests that waves and currents reinforce each other (Herbich et al, 1984). Although there are some problems of interpretation with that formula, the combined wave-current effect is in general accord with the prevailing view that wave action increases the sediment transporting capacity of currents.

8.2 Prototype Experience

The intensive and long-lasting effort in the course of this study to obtain good data on prototype experience of scour met with modest success for the effort involved. There seems to be, in truth, a lack of good prototype data on scour; in particular scour measurements that can be related to environmental conditions in a manner that would permit verification or modification of the relatively more plentiful amount of data on physical models and laboratory experiments. The scarcity of good data sets on field experience appears to be due more to the lack of detailed monitoring than the absence of problems. Also, when data is available, the necessary correlations are not often attempted. Current and sea state observations are normally made several times per day under existing regulations for the offshore oil and gas industry. These do not necessarily involve quantitative measurements. Also, monitoring of scour is not specifically called for - because in may instances it would not be justified.

However, the lack of data seems to result more from the ad-hoc approach that operational divisions of oil companies take to scour problems than to the lack of regulations. If significant scour holes are detected during routine diving inspections, arrangements are made to fill them quickly by the most economical available method. It is not part of their mandate to explore cause and effect. Hence, as long as the problem and the cost remain "manageable", little effort is devoted to technical study of the problem. This would appear to be the fairest assessment of the facts uncovered in this study. A common oral response to the request for data which bears out this conclusion is that while scour has been experienced in some areas and was formerly of some concern, experience has shown it is easily remedied when it occurs and hence is not usually seen as a significant problem. For this reason, there may also be an understandable reluctance to focus attention on this issue.

8.3 Scour Protection Methods

The use of rock, stone, or gravel layers is the most common scour protection technique. This may be generally attributed to its proven success and the relatively low cost of implementation. The protective layers may be designed to be stable under design storm conditions or they can be sacrificial as is usually the case with gravel layers. The latter approach is more common because the material necessary to provide stability under design storm conditions is often locally unavailable and prohibitively expensive to import. Another method which seems promising yet remains relatively unproven is the use of widely graded material which will naturally form filter and armour layers.

The use of artificial seaweed, though widely publicized, seems to remain more or less unproven. There are somewhat contradictory assessments of the success of the method in the literature. At best, its success is not obviously better than that of gravel protection and furthermore it is usually more expensive to deploy because it is diver intensive. Nevertheless, the use of artificial seaweed may be necessary in areas where gravel or rock is not available.

8.4 Scour Prediction Techniques

For most scour phenomena there are no true theoretical prediction techniques. At present, most techniques are empirically based methods derived from physical model results. The accuracy of these methods is questionable considering the ad-hoc approach to most model tests in particular the lack of attention given to scale effects. Though it is possible that scale effects in physical models may not influence the ultimate scour depth prediction from model tests, however this has not been proved in the literature.

The majority of model tests that have been performed relate to scour around vertical piles caused by unidirectional flow. Consequently most empirical formulae are for unidirectional flow. It is not surprising that prediction techniques which consider wave action are scarce considering the confusion that exists even in the qualitative descriptions of scour under the different hydrodynamic conditions and combinations of waves and currents. The development of scour prediction techniques around piles and pipelines under the action of waves and currents can only come about through the improvement of hydrodynamic theory for flow around obstructions and through a proper investigation of the influence of scale effects on model results.

Some success has been attained in the use of numerical models for scour around large gravity structures. Theses models utilize sediment transport techniques which are also often empirical yet in general theory is more developed than it is for scour phenomena. Numerical models require physcial model results or experimental data for calibration purposes.

8.5 Guidlines for Scour Design Procedure

The existing guidelines provided by various regulatory bodies are of little help in scour design. The matter of scour is not treated as seriously as other elements in the design of offshore structures.

The design procedures should be initiated with a collection of environmental data. The wave climate is often clearly defined to determine structural loading and operational conditions for ocean structures and this information is sufficient for scour design. However, the same is not true for currents. which are the major cause of scour, especially in deepwater. The common procedure is to take current measurements for a few weeks. While this is adequate to define tidal currents it will not likely be adequate to evaluate wind-driven storm currents which occur infrequently. An effort should be made to measure currents during a storm and also to make the measurements at various depths, including at the bottom both near and away from the structure. Extensive bottom sediment studies should be carried out to determine the nature of bottom sediments around the proposed structure as well as to locate possible gravel deposits for use in scour protection. Areas of active bedform movement should also be determined.

Scour protection should be considered if scour prediction techniques indicate that appreciable scour will occur. A framework for the design of gravel and rip rap protection methods does exist for deep water structures where currents predominate. Design methods for artificial seaweed have not been reported in the literature and must rely exclusively on experimentation in the field and previous experience.

A stable rip rap layer may be designed using unidirectional flow methods (for instance, the Shields curve for inception of sediment transport). A conservative estimate will be achieved when waves and currents exist if the two component velocities are added to determine an equivalent effective shear stress. When the rip rap size is large, filter layers should also be included following standard filter layer design procedures.

The design of sacrificial gravel layers involves the estimation of the required thickness to provide protection under design storm conditions. Analytical methods do not exist and the thickness is usually estimated through either physical or numerical modelling procedures, or a combination of both.

Post-construction monitoring fulfills two roles. First, it may improve the statistical evaluation of extreme environmental conditions to be used in the design of subsequent structures. Also, it provides prototype correlations between scour depths and environmental conditions which can be used to refine predictive models and design techniques. Diver inspections of scour are not fully adequate and it would be extremely beneficial if a reliable method could be developed to monitor scour during storm events when destabilizing effects are most critical.

8.6 The Scotian Shelf

A large amount of information concerning the environmental conditions in the vicinity of Sable Island has been gathered as part of the Venture Development Project Environmental Impact Statement prepared for Mobil Oil Canada Ltd. The information is general in nature and cannot be used for design purposes. However, the study is helpful in developing design considerations specific to the Scotian Shelf. These considerations have been outlined in Section 7.3

Information from two major interest holders on the Scotian Shelf (Mobil Oil Canada and Shell Canada) indicates that pile-supported jacket type structures are favoured for production platforms. Most of these structures will be installed in water depths from less than 20 m to depths of 8Ø to 1ØØ m. The predominant bottom sediment condition in the area is a fine to medium sand, and environmental studies have indicated that maximum currents can be at least 2.Ø m/s. Considering these environmental conditions, there is a definite risk of scour as illustrated by experience with jack-ups in the area. Pipelines have been planned as a link to mainland Nova Scotia and the routes will inevitably traverse a wide range of bottom conditions with depths ranging from 2Ø to 3ØØ m, and consequently various protection methods wil be required.

Considering the range of depths that exist on the Scotian Shelf, currents will generally be more important than waves in causing scour. The results from the Environmental Impact Statement for the Venture Field indicate a wide range of variability in environmental design conditions from location to location becuase of the complex bathymetry and therefore local conditions will have to be determined for each installation. There is a clear need for additional current measurements near the sea floor in the vicinity of likely structures and along propsosed pipeline routes. The measurements should be supplemented by numerical models because of the complex bathymetry in the area. A statistical evaluation of extreme wave conditions may be determined with numerical models (which must be capable of addressing shallow water conditions) calibrated with measurements. Localised wind-wave parametric hindcasting models may be used since the most severe wave climate originates from the northwest, which has a limited fetch. Two-dimensional spectral models will be necessary to evaluate the oceanic contribution of wave energy originating from the south and the east.

8.7 Review of Scour as a Problem

A prevailing opinion in the oil industry is that scour is not a critical issue for most sea floor structures. This is apparent in almost all subjects of discussion within this report. Often scour protection methods are not included in the initial design and instead remedial measures are undertaken as required. The paucity of knowledge regarding scour prediction techniques, the ad-hoc approach to model tests, and the neglect of systematic monitoring of scour around prototype structures all reinforce the existing attitude. A possible exception to the general attitude concerns artificial islands which are more vulverable where model tests and prototype monitoring have been more widely implemented.

The question which remains to be answered is whether or not the problem of scour warrants greater attention than it has already received. This leads to the conclusion that further studies are required to collect and analyse both model and prototype data of scour more systematically than has been done hitherto.

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APPENDIX A

SCOUR AROUND SEAFLOOR STRUCTURES

THE QUESTIONNAIRE SURVEY

THE QUESTIONNAIRE SURVEY

Note: Copies of completed Questionnaires are available though not included in this report. They may be accessed at the offices of the Environmental Studies Revolving Fund in Ottawa.

A3.Ø The Questionnaire Survey

As a means of locating and assessing the suitability of unpublished protytype scour information a questionnaire survey was instituted in the early stages of the study. A circulation list was generated covering offshore oil companies, contractors, consultants and laboratories worldwide. A standard questionnaire package, which is attached, consisting of:

- An explanatory letter,
- Study Information Summary Sheet
- "ESRF at a Glance" on explanatory brochure published by the Government of Canada and
- Five copies of the questionnaire

was distributed to the selected list of contacts.

Responses to the survey fell into various categories.

1. No information available or outside respondents area of operation

2. Information supplied other than questionnaires, i.e. furtheer contacts, brochures, relevant papers, brief summaries, etc. Included in this category were 11 respondents invloved in offshore development who did not consider scour to be a significant problem.

3. Questionnaires returned. Some respondents returned as many as five questionnaires and the total number finally received was 31 of which on 11 noted scour as a problem. A number gave general information on scour studies and others gave details of commercial scour protection methods.

The table below gives statistics of returns under the above headings.

Questionnaires Issued	135
1. No Information	26
2. Information	26
3. Questionnaires	15
Total Replies	67
% of Total Responding	5Ø%
Total Positive Replies	41
% of Total Positive Responses	3Ø%

The initial resopnse to the survey was disappointing, however, by making personal , telephone and mail follow up contacts, a fairly respectable return was eventually received. Returned questionnaires generated further contact and possible sources of information. The final list of questionnaires issued and responses are given in Appendix A3.1.

Of the one hindred thirty-five companies circulated, 5Ø % eventually replied and of these, 3Ø % provided useful information of some description. The time scale for replies may prove of interest to others who may be considering a questionnaire survey in the future. The first questionnaires were mailed on February 23, 1984 and the final replies to the initial circulation were received on September 12, 1984.

Following an assessment of the questionnaires, a second stage circulation was instituted covering seven companies and involving nine projects. It was fest that these projects would provide suitable subjects for further investigation and inclusion in this Report as more detailed case histories (Appendix 4).

A summary of relevant data taken from the questionnaires received is given in Appendix A3.2 tabulated for multi-member structures, pipelines, gravity and other structures. All questionnaires returned are included in Appendix A3.3.

ESRF SCOUR STUDY
QUESTIONNAIRE REPLIES

OFFSHORE OIL COMPANIES AND CONTRACTORS

COMPANY	COMMENTS
AGIP, SPA Milano, ITALY	Questionnaire returned - Beniboye Gravity Structure - Nigeria
AMOCO London, U.K.	Paper on scour and field data - Southern North Sea
BP International London, U.K.	No recorded scour problems on steel jacket structure in North Sea at Magnus, Forties & West Sole
CANMAR (Dome) Calgary, Alberta CANADA	Questionnaire returned for Tarsiut & Uvilute Islands. Beaufort Sea paper attached.
Chevron Oilfield Research Lehabre, California U.S.A.	Brief information on design of platforms. Generally scour does not affect design. Where scour observed-bottom stabilization materials placed.
Chevron Petroleum UK London U.K.	No questionnaire. Basis for treating scour in principle.
Chevron Western Region Concorde, CA. U.S.A.	No observed scour around several West Coast USA platforms.
Elf Aquitaine Norge A/S Stavanger, NORWAY	5 Questionnaires returned on Frigg Field - N. Sea
Elf UK Plc London U.K.	No direct experience of scour.
EXXON Company USA Houston, TX. U.S.A.	Gulf of Mexico Design for Limited amount of scour in soft silty clays. Experienced few problems.
EXXON Production Research Co. Houston, TX. U.S.A.	No information Literature List
Gulf Canada Resources Ltd. Calgary, Alberta CANADA	Questionnaire returned for Beaufort Sea Predevelopment Study
Hamilton Bros. Oil & Gas Ltd. London, U.K.	Floating Production Systems. No scour problems.

COMPANY	COMMENTS
Loop Inc. New Orleans, LA. U.S.A.	Details of SALM System off Grand Isle, LA. in 1ØØ ft. of water-low seabed currents. Scour not a sugnificant problem.
Mobil Norway Stavanger NORWAY	Scouring not a problem on Statfjord Field. Two concrete platforms in 145 m water seated on clay.
Norpipe A.S. Stavanger NORWAY	2 Questionnaires returned B-11 Compression Platform North Sea & Pipeline, Ekofisk-Emden, near German coast.
Pennzoil Expl. & Petroleum Co. Houston, TX. U.S.A.	Questionnaire returned for Block K-1Ø North Sea.
Phillips Petroleum Co. Bartlesville, OK. U.S.A.	Scour generally not a problem. Noted only two problem locations-Puerto Oriday, Venezuela & Salavati Island, Indonesia. However no data available.
Shell UK Exploration & Production Strand, London, U.K.	General approach to design, monitoring and remedial works for Southern & Northern North Sea noted.
Union Oil of Great Britain Middlesex, U.K.	No recorded scour problems on Union Oil facilities.

ESRF SCOUR STUDY
QUESTIONNAIRE REPLIES

CONSULTANTS AND LABORATORIES

COMPANY	COMMENTS
Bitumarin B.N. HOLLAND	Brochure 5 Questionnaires
Commissioner of Irish Lights Dublin 2 IRELAND	Permission to use Kish Lighthouse information
Fenco Engineers Inc. Toronto, Ontario	4 Questionnaires. Paper - "An Anti-Scour System for Drilling Rigs Using Airlift". Murphy and Yan
Gore & Storrie Limited Toronto, Ont. CANADA	Inland Waters Scour
Martec Limited Halifax, Nove Scotia CANADA	Questionnaire returned. Scour around ODECO Gulftide at Migrant Wellsite, Scotian Shelf
NGI-Norges Geotekniske Institute Ø8Ø1 Oslo 8 NORWAY	Questionnaire returned. Report on Full Scale Observations of North Sea Gravity Platforms.
Nicolon B.V. THE NETHERLANDS	Brochure
Ontario Hydro Toronto, Ontario CANADA	Information on Lake Gris OH-GPV Cable. Suggested further contacts.
Taisei Corporation Eng. & Const. Tokyo, JAPAN	Questionnaire. Thermal Discharge Channel.
Takenaka Komuten Co. Ltd. Tokyo, JAPAN	Brochures on "DCM" & "Seabetter"
Texas A and M University College Station, TX. U.S.A.	Questionnaire returned covering lab studies.
U.S. Army Corps of Engineers Fort Belvoir, V.A. U.S.A.	Forwarded reports requested by K. Philpott
Vrijhof Ankers, b.v. THE NETHERLANDS	No information. Some work with Delft on "Erosion near Anchors"
Wescan Maritime Consultants Ltd. Calgary, Alberta CANADA	Questionnaire & drawing returned for Canapoint SPM terminal pipeline St. John, New Brunswick.
Western Canada Hydraulic Labs Ltd. Port Coquitlam, B.C. CANADA	Two questionnaires returned for Dome Beaufort Caisson Islands

APPENDIX B

SCOUR AROUND SEAFLOOR STRUCTURES

LIST OF PROTOTYPE CASE HISTORIES

PROTOTYPE CASE HISTORIES

INDEX

NOTE: These case histories are available but are not included in this report. They may be accessed at the offices of ESRF in Ottawa.

APPENDIX C

SCOUR AROUND SEAFLOOR STRUCTURES

LIST OF CONTACTS

LIST OF CONTACTS

A & M University, Texas / R. Reid, J. Herbich
ADI Limited / Fredericton, New Brunswick
AGIP Petroleum Co., New York / Henry Weicel
AGIP, SPA, Milano, Italy / J.C. Burton
Albery Pullerits Dickson & Assoc., Don Mills, Ontario
American Gas Association, Richard Schollhammer
AMOCO Production Co. (International), Chicago, Illinois
R.V. Anderson Associates Ltd., Willowdale, Ontario
ARCO Marine, Inc., Long Beach, California
Ardon International Ltd.
Association of Oil Pipelines, Washington, D.C. / Mr. G. Donald Rilev
W.S. Atkins & Partners, U.K. / M. Duckett
B.P. International Ltd., U.K. / Gerald Goulson
Bird and Hale Ltd., Toronto, Ontario
Biturmarin B.V., Holland
Bong-Doras
British Gas Corp., London / Mr. M.E. Ford
British National Oil Corp., U.K. / J.T.C.Hay / I Walker
British Petroleum Ltd., U.K.
R.J. Brown and Associates of America Inc., Houston, Texas
R.J. Brown and Associates, Netherlands / Andrew Palmer
Brown and Root, Houston
Brown & Root, U.K. / S. Williams
Canadian Eng. Surveys Co. Ltd., Calgary
CANMAR (Dome), Calgary, Alberta
Canterra Energy Resources, Calgary / Alan Laundry
Canterra Resources Marine Services / John Henderson
Cansult Limited, Don Mills, Ontario
Carr & Donald & Associates, Toronto, Ontario
CBLC Limited, Halifax, N.S.
C-Core Memorial University, Bill Winsor
CERC (Vicksberg) / R. Whelin / Fred Camfield / J.Grines

Chevron Canada Resources / E. Cudby
Commissioners of Irish Lights / Mr. Boyd
Conoco Inc., U.K.
Conoco (U.K.) Ltd., U.K. / R. Ganguly
Danish Hydraulic Institute / T. Sorensen, / K. Erling
Dalhousie University / D. Huntley
Davy McKee, U.K.
Delft Hydraulics Laboratory, Netherlands / van der Weide
Det Norske Veritas (Canada) Ltd., Calgary, Alberta
De Voorst Laboratory, Netherlands
Dome Petroleum, Calgary Alberta / Rick Myers
Drexel University PA. / Dr Richard Weggle
Earle & Wright Consulting Eng., U.K. / M. Hancock
Eastern Designers & Company Ltd, Fredericton, New Brunswick
EBA Engineering Consultants Ltd., Edmonton, Alberta
Elf Aquitaine, U.K.
Elf Norway, Norway
Engineering Topographic Laboratory, U.S. Army, Fort Belvoir VA. / Allan De Wall
Esso Australia Ltd., Sydney, Australia
Esso Resources Canada Ltd., Calgary, Alberta / J. Moir / G. Spalding
Exxon Co. USA Production Department, Houston, Texas / F. Chuck / S.J. Reso
Exxon Research & Engineering Company, Houston, Texas / R.B. Harley
Fenco Engineers Inc., Toronto, Ontario
Fenco-Shawinigan Engineering Ltd., Halifax, Nova Scotia
University of Florida, Gainsville, Florida / Dr. Robert Dean
Geomarine Associates, Halifax / Alan Ruffman
Global Marine Development Inc., Newport Beach, CA.
Golder Associates, Toronto, Ontario
Gordon Wilson Associates Inc., Vernon, B.C.
Gore & Storrie Limited, Toronto, Ontario
Gulf Applied Research, Houston, Texas
Gulf Applied Resources, Houston, Texas / Ken Loer

Gulf Canada Resources Ltd., Calgary, Alberta / Bill Livingstone / Dave Townend
Gulf Oil Exploration & Production Co., Houston, Texas
Hydraulic Research Station Ltd. U.K. / S. Huntingdon
Halcrow M.O.E. / P. Godfrey
Hamilton Bros. Oil and Gas Co., U.K. / C.J. Arinder
Hamilton Brothers Oil Co, Denver, Colorado / D Shannon
K. Hardtde Associates Limited, Willowdale, Ontario
Home Oil Limited, Perth, W. Australia / J. Anderson / B. Kiovean
Howe International Limited,Ottawa, Ontario
Husky Oil / J. Henderson
Imperial College of Science & Technology, U.K.
Institute of Hydrodynamics & Hydraulic Engg. / Denmark
Integ-International Engineering Ltd., Vancouver, B.C.
J.P. Kenney & Partners, U.K. / Dr. P.W.J. Raven
Kilborn Limited, Toronto, Ontario
Klohn Leonoff Ltd. Consulting Engineers, Richmond, B.C.
LaLonde, Girouard, Letendre & Assoc. Ltd., Montreal, P.Q.
Land and Marine Engineering Ltd., England
Lavalin Inc., Montreal, Quebec
Lawrence Allison Contractors Inc., / N. Harvey, L.A.
Loop Inc. / Robert C. Thompson
MacLaren Plansearch Limited, Dartmouth, Nova Scotia
Marine Gmaro (part of N.R.C.) / Jack Bolder
Marathon International Oil Co. / Findley, Ohio
Marathon International Petroleum GB Ltd., U.K.
Mar-Land Engineering Limited, Scarborough, Ontario
Martec Limited, Halifax, Nova Scotia
McAlpine Offshore Ltd., U.K.
McDermott Marine Construction, New Orleans, LA
Urban F. McCulloch, Ontario
McElhanney Surveyors & Eng. Ltd., Vancouver, B.C.
McLelland, Ventura, CA.
Mil. Cambridge Ass. / Dr. Madsen

Mobil Canada (Halifax) / S. Aitken / Michael Coolen / P.W. Fulton / Stephen Hatcher

Mobil England, London, England

Mobil Norway

Mobil Oil Canada Limited, Nove Scotia

Mobil Research & Development Corporation, Texas / D.M. Coleman / R. Stacey

N.F.C. Jack Sollet

National Seagrant College Program / David Duane

NORDCO, St. Johns / D. Ross

Pan-Arctic Oils / M. van Leterin

Petro-Canada / Paul Sabatini

Philips Petroleum Company / M. Staven

Posford, Pavry and Partners, U.K. / I. Stickland

Shell Canada / Roy Fodchuk

Shell International / J.N. Ringers/ H. Lensen

Stevens Institute of Technology / R. Stephens

Texaco / R. Schoddert

University of California, Berkely C.A. / R.L. Weigel

Woods Hole Oceanograpghic Institute / C. Hollister