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Summary

This report present a state-of-the-art review of how foundation stability and platform-soil interaction of jack-up platforms, is taken into account by the oil industry, classification societies, national and international authorities and others involved in the design, installation and operation of jack-up platforms.

Existing rules and regulations have been reviewed and analysed with respect to foundation requirements and a summary is presented.

Various design approaches are looked into in this report and will be further analyzed in Part Project 2: 'Analysis Methods'. Present installation and operation procedures have been examined with respect to foundation problems involved.

The report is based on a literature study combined with the input from the participating companies to a questionnaire as well as interviews with key personnel from several of the participants and also a number of designers and operators not participating in this project.

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JACK-UP PLATFORMS
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FOREWORD

This report presents the work carried out within Part Project 1 (PP1), 'Evaluation of Previous Experience' of the Joint Industry Project: 'Foundation Stability of Jack-Up Platforms'. The project is at the time of writing sponsored by the following fourteen companies and institutions:

Amoco Norway Oil Company
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Bethlehem Steel Corporation
Chevron Oil Field Research Company
Getty Oil Company
Gusto Engineering B.V.
Hitachi Zosen Corporation
Minerals Management Service
Mitsubishi Heavy Industries
Mitsui Engineering and Shipbuilding Co.
Nippon Kokan K.K.
A/S Norske Shell
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The work presented herein is the result of a study where the main work was carried out during 1982 and gives a state-of-the art review on how foundation considerations presently is treated within the oil industry and by the regulatory authorities.

The report finally gives recommendations for further investigations to be carried out in the next phases of the project.

EXECUTIVE SUMMARY AND CONCLUSIONS

GENERAL

The purpose of part project 1 (PP1) was to gather and review available information about todays practice in design, installation and operation of jack-up platforms regarding the platform foundation, independent leg footings of the spud-can type and mats.

The work was split into several subtasks:
1) Review of rules and regulations
2) Review of present design methods
3) Review of installation and operation procedures
4) Review of case histories and comparison between real performance and design assumptions.

The gathering of this information has been performed with a variable degree of success. A relative broad review of published information in the geotechnical literature and in offshore magazines and proceedings from offshore conferences was carried out when preparing the project specification. This review has continued during 1982 and a few additional sources of information has been found.

The questionnaires distributed to all participants were returned by most of them and represented the basis for further interviews and questions during a visiting round to many of the participants and also to other companies involved in design and operation of jack-up platforms.

The result of this data gathering was a considerable amount of input to points 2) and 3) on design methods and procedures, respectively. However, it turned out to be very difficult to find well documented case histories in addition to the papers published already.

Some of the participants have supported us with extra documents which have been reviewed and some are still under review on case histories. Parts of this documents could be of value for the case history part of the data gathering. This will be further evaluated within the project.

The problems connected to gathering of well documented case histories was recognized by the authors and the Steering Committee at an early stage. Still we had hoped to be more successful on this point.

In the following the conclusions of PP1 with recommendations for further work will be summarized.

RULES AND REGULATIONS

It has not been the intention of the authors to propose changes in todays rules and regulations neither to the classification societies nor to the national or international authorities. Merely the technical aspects and relevance of the existing rules and regulations regarding foundations of jack-up platforms have been treated here.

In Section 2 the existing rules and regulations have been presented and commented on, and in Appendix B an extract of foundation related clauses have been summarized.

The majority of jack-up platforms are normally operating as mobile units. This is reflected in the existing rules and regulations for jack-ups. Where no specific input data exist it seems that rulewriters have had problems with the identification of the problems. It is also the authors' impression that the rules for mobile structures have been written mainly by marine and structural engineers and that the interaction with the foundation soil has been looked upon more or less as a black hole. Soil-structure interaction requirements have thus been attempted to be eliminated through simplifying assumptions.

DESIGN METHODS

The problem of optimizing a design for world wide conditions has been described. It is found that generally a certain amount of conservatism is sought to be included in the footing design. However, the soil conditions assumed in the design calculations varies considerably from one designer to another. Generally it has also been found that the design soil conditions have not been varied in order to define eventual limits for operation. However we have also seen designers who have made considerable efforts in this area.

As mentioned above a certain degree of conservatism is sought to be included in the footing design. This relates mainly to the structural design of the footing and to the top part of the leg where the assumption of pinned conditions at the lower end of the leg generally leads to conservative design. However, how the various assumptions regarding the foundation influences the 'overall distribution of conservatism' is still an unanswered question. How the various possible combinations of soil conditions may influence the safety of the platforms is generally not documented in the design documents. To answer some of these questions the work to be performed under PP2 and PP3 on analysis methods and parameter studies may be a good step forward.

Design soil pressure

The magnitude of the soil reaction pressure is generally fairly controllable and fairly accurately estimated. Sea bed unevenness and the presence of obstacles like boulders are accounted for by many designers at least up to some magnitude.

Rotational stiffness of foundation

The present design assumptions of pinned conditions some 10 ft below mudline is considered to be nonconservative under special circumstances. The assumptions regarding the degree of fixity of the legs of independent leg type jack-ups require a thorough study as proposed to be carried out in PP2 and PP3.

Penetration and preload

Different bearing capacity theories are used today for prediction of penetration depth of jack-up footings. The degree of accuracy is merely a question of soil data assumptions. Often the penetration depth specified in design documents is a pure assumtion related to previous experience rather than a value related to specific soil conditions.

The preload considerations at design stage are generally that the most stressed leg shall be preloaded at least to the same vertical load as it will be subjected to during design storm conditions. The conservatism or may be the lack of conservatism involved in this preload design method needs to be investigated.

Pull-out resistance

Previous experience has shown that pull-out resistance generally is not a problem provided relevant jetting/suction releaser systems are present and operating.

INSTALLATION AND OPERATION PROCEDURES

Site investigations

The study has so far revealed that the policy of oil companies and drilling contractors varies considerably regarding the extent and type of site investigations prior to installation of a jack-up platform. It will be one of the main goals of this project to prepare the basis for the considerations involved in a specification of site investigations by improving the understanding of soil-structure interaction, and through a series of parameter studies pinpoint the parameters having main influence on the foundation stability.

Penetration prediction

Previous investigations by Gemeinhardt and Focht (1970) and Endley & al. (1981) have shown that penetration predictions can be carried out with relative good confidence. The relevance of the bearing capacity formula for this purpose is unquestionable. There are, however, a few things that may need to be looked at in some more detail. One of these things is the stability of the hole created by a spud-can penetrating into a soft clay. The caving in of the hole walls may represent a possible hazard similar to 'punch-through' of stiff top layers or crusts and it is recommended that this is further evaluated under PP2 and PP3.

The effects of previous footprints represents a danger during installation on locations where other jack-ups have been located previously. To avoid such hazards a survey of the sea bed with side-scan sonar is recommended to identify the presence of such footprints.

Elevation

An evaluation of the touchdown and elevation phase is the first possibility to check penetration predictions. It is believed that problems could have been forseen at this stage in several cases if a further examination of the soil conditions had taken place before application of preload.

Preloading

The most critical situation during preloading of independent leg platforms is probably punch-through. Punch-through of stiff layers on top of or embedded in softer strata has been the cause of damage to several platforms during the preloading phase.

From the industry we got the impression that most concern is now directed towards improvement of the capacity of the jacking system so that jacking with preload could be possible. This would allow a faster adjustment of the levelling in case one leg sinks faster than the the others and could help speeding up the preloading phase on locations with large penetration depths.

The preload capacity varies considerably from one design to another. The requirements for preload is one of the major points of interest in the further study. The present assumption that preload should at least be equal to the maximum vertical load under extreme conditions is not necessarily sufficient. The effect of load inclination, but also the phase difference between maximum horizontal load and maximum overturning moment (giving the maximum vertical load) needs to be investigated.

Common practice is, as we have been informed, to use full preload independent of the local environmental and soil conditions for a specific location. This is generally giving a high level of safety against instability during operation as the footing capacity as checked through the preload in this way for most conditions will be considerably above what is really needed. This is probably one of the more important reasons for the low rate of foundation accidents during normal operation combined with the fact that the platforms very seldom have been subjected to full design storm conditions. One should thus be very careful when proposing that the full preload need not necessarily be applied at all locations, but there will obviously exist combinations of environmental loads and soil conditions which would make a reduced preload a better solution in order to reduce extreme penetrations and avoid excessive pull-out resistance. It is felt that this should be addressed further in the parametric study.

STABILITY CONSIDERATIONS

Based on the information gathered during this study we have found that stability evaluations regarding sliding stability of shallow embedded footings like spud-cans on dense sand or stiff clays or mats on softer clay, deepseated and shallow bearing capacity failure of mats and the effect of inclined loading on independent leg footings are normally not carried out.

Evaluations of this kind require more detailed knowledge of the soil conditions and a correspondingly more detailed soil investigation. The philosophy/policy of the involved companies regarding soil investigations will to a great extent also be reflected in the treatment of stability evaluations on a case to case basis.

Seafloor instability is believed to be the cause of more than one platform failure. Wave pressure induced instability has been observed in delta areas, and underwater slides represent a major threat to jack-up platforms installed in sloping delta areas. The seafloor instability problem is complex and cannot be treated in too much detail within the project, but it need to be addressed in PP3 at least to a limited extent.

1. INTRODUCTION

1.1 GENERAL

Foundation failure has been the main cause of several jack-up accidents ranging from total loss of lives to a varying degree of damage to the structure and the drilling equipment. Since the jack-up concept was introduced in the middle of the 1950'ies a considerable amount of experience regarding installation and operation of this type of offshore structure has been accumulated.

Nevertheless, the damage records still increases and the experience of the 1980's shows clearly that there is a need for an improved understanding of the foundation behaviour of jack-up platforms.

The first part of this project 'Foundation Stability of Jack-Up Platforms' is a state-of-the-art investigation trying to illustrate the most important foundation problems related to the jack-up platforms and how they are treated by the industry and the regulatory authorities today.

Design methods and design criteria as well as the installation and operation procedures are reviewed to define how they may effect the safety against foundation failure.

One of the aims of this project was to carry out a review of case histories regarding foundation behaviour during installation, operation and severe storms. Such full scale data and case histories has not yet been made available. Such cases will in most cases be rather scarce, especially regarding the soil-data but also regarding actual load conditions.

In the absence of well documented failure cases a more theoretical approach will probably have to be followed in the continuation of the project.

1.2 PROCESS OF EVALUATION

This evaluation of previous experience is based on a literature study of relevant material published during the last decade. A reference list is presented in Chapter 5.

Further a questionnaire was produced and distributed to all project participants. The questionnaire addressed the problem areas of jack-up platforms which, based on the literature study, were assumed to be most critical and interesting regarding foundation behaviour and its influence on the overall safety.

Finally, after having evaluated the received answers to the questionnaires, discussion meetings were arranged with most of the participants and other relevant companies (See Appendix A). These discussions helped to clarify the answers to the questionnaires and also indicated new aspects regarding foundation behaviour that had not been covered by the questionnaire.

1.3 RESPONSIBILITIES W.R.T. FOUNDATION STABILITY CONSIDERATION

An effort was made within this study to try to establish a clear responsibility chart for treatment of the foundation aspects of jack-up platforms. The situation is, however, very unclear and due to the fact that jack-ups predominantly are being used as mobile platforms in different parts of the world and a for wide range of environmental and sea bed conditions, it turned out to be practically impossible to establish a responsibility chart, and it may be concluded that responsibilities tend to be clarified between the individual parties involved from case to case, and the way the parties involved handle this may also vary with the market demand for jack-up platforms.

1.3.1 National Authorities

Different procedures exist in different countries. To our knowledge only a few countries have regulated the procedures w.r.t. jack-up foundation investigation etc. in official forms.

1.3.2 Rig Owners and Oil Companies

As there is no formal overall agreement on the responsibility for site investigations and control of the suitability of a platform for a specific location, practice varies considerably from case to case dependent on the local experience and engineering judgement of both parties involved. Some oil companies require soil investigations for each new location and so do a few rig owners as well, while others base their foundation evaluation on the trial and error method.

1.3.3 Marine Surveyors and Insurance Companies

The marine surveying companies act as the underwriters' extended arm at location. The marine surveyors issue 'location approval' based on their judgement of the suitability of a platform for a specific location and the information made available by the rig owner. This information is often only a brief information about the soil conditions in order to evaluate the probable depth of penetration. The marine surveyors follow the move from one location to another. They are also to control that the installation procedures are followed regarding preloading and that the penetration of legs are within the design criteria considering environmental conditions, required air gap and available leg length.

1.3.4 Designers and Builders

The designer will normally define boundary conditions allowing the platform to operate over a wide range of soil conditions. As there are generally very little guidance in available rules and regulations regarding footing design these boundary conditions will to a large extent be based on the engineering judgement within the individual company and on the practical experience and available feedback from the operation of the platform. To some extent geotechnical consultants have been involved in this work, but there exist no unified approach. The boundary conditions regarding the foundation are specified in the design calculations and some times also in the operation manual.

1.3.5 Classification Societies

The classification societies are not involved in the evaluation of the suitability of a jack-up for specific location from a foundation point of view. Basically the classification societies will check the assumptions involved in the design of the footing and the assumptions regarding soil-structure interaction for an overall structural strength approval. From a classification point of view the platform is to be operated within the limits given by the design but no specification of how this is to be verified from location to location is usually given.

2. RULES AND REGULATIONS

2.1 INTRODUCTION

This chapter aims at giving an overall review of existing rules and regulations concerning the foundation stability of jack-up platforms. Firstly the regulatory parties which possibly may be involved in the operation of jack-up platforms is listed. Further the content of the rules and regulations are commented on from the geotechnical point of view. Finally the complete text of the discussed R & R are quoted and enclosed in Appendix B.

2.2 EXISTING RULES AND REGULATIONS [R & R]

The existing R & R which concern jack-up platforms either for design, construction or/and operation are as shown in Table 1 below.

TABLE 1. LIST OF RULES AND REGULATIONS

Classification Societies

American Bureau of Shipping:	Rules for Building and Classing Mobile Offshore Drilling Units, 1980.
Bureau Veritas:	Rules and Regulations for the Construction and Classification of Offshore Platforms, 1975.
Det norske Veritas:	Rules for Classification of Mobile Offshore Units, 1981.

Germanischer Lloyd	Rules for Construction and Inspection of Offshore Installations, Volume I, Offshore Units, 1976.
Lloyd's Register of Shipping	Rules for the construction and classification of mobile units, 1972.
Nippon Kaiki Kyokai	Mobile Offshore Units, 1978.

National Regulations

U.S. Codes of Federal Regulations:	46 CFR Shipping, Parts 107, 108, 109 Subchapter 1A, Mobile Offshore Drilling Units, October 1, 1980. 30 CFR Mineral Resources. Part 250. U.S Department of the Interior (Geological Survey), Mineral Management Services, OCS ORDERS.
U.K. Department of Energy:	Offshore Installation: Guidance on design and construction, July 1977.
Norwegian Maritime Directorate:	Mobile Drilling Units, Regulations laid down by Norwegian official control institutions, 1973 with amendments of 1975 and 1977, 1980 and 1981.

International Regulations

IMCO - Inter-Governmental Maritime Consultative Organization (now IMO)	Code for the Construction and Equipment of Mobile Offshore Units, 1980.

2.3 CLASSIFICATION SOCIETIES

2.3.1 Rules References

Generally the classification societies are not involved in the evaluation of the foundation stability for a specific location. But their rules include the assumptions to be made at the design stage for the structural verification of strength including also some assumptions regarding the footing and its interaction with the rest of the platform.

In Table 2 references are made to the different regulations (which are quoted in full text in Appendix B). In the following the geotechnical requirements are discussed in general, and finally the requirements of each classification society are shortly summarized in Table 3.

TABLE 2. RULES REFERENCES

ABS (American Bureau of Shipping)

Section 3 Classification Loading and Environmental Criteria

3.13 Units Resting on the Sea Bed

Section 4 Self-Elevating Drilling Units

4.5 Structural and Intact Stability Considerations

4.5.1.b Legs without Mat

4.5.5 Bottom Mat

4.5.6 Preload Capacity

4.5.7 Sea Bed conditions

BV (Bureau Veritas)

Chapter 2 Architecture - Scantlings - Foundations

2.3 Loads to be applied to the structure

2.34 Environmental loads

2.34-6 Reactions of the sea bottom on the structure

2.7 Stability - Water Tightness - Buoyancy

2.73 'In-Situ' stability for platforms lying on the sea bed.

2.73-1 General

2.73-2 Self-elevating platforms

2.8 Foundation

2.83 Foundation design

2.83-13 Self elevation platforms

DnV (Det norske Veritas)

Chapter III Special Design Principles

Sec.2 Self-Elevating Structures

Part C Hull, Legs and Bottom Mat

C300 Bottom Mat

Part D Overturning Stability on the Sea Bed

GL (Germanischer Lloyd)

Chapter 2 Basis for Dimensioning

Sec.7 Foundations, stability

E Overturning of mobile marine structures

LR (Lloyd's Register of Shipping)

Chapter B Structural Design

Sec. 3 Loading

310 Bottom Forces

Sec. 6 Self Elevating Units

602 Bottom Mat

NKK (Nippon Kaiji Kyokai)

Part P Mobile Offshore Units

Chapter 7 Self Elevating Units

7.2 Strength and Scantling

7.2.2 Legs

7.2.4 Bottom Mats

2.3.2 Geotechnical Requirements

The above list shows clearly how limited the evaluation of the foundation of jack-up platforms is dealt with by the classification societies. Some of the societies states clearly that the checking of bearing capacity and resistance to sliding of footing or mats on the sea bed is not included in the class

The main items to be examined as part of the class are:

(The specific requirements of each classification society are summarized in Table 3.)

Overturning Moment

Factor of safety against overturning:

$$F.O.S. = \frac{Stabilizing\ moment}{overturning\ moment}$$

F.O.S. are generally required to be > 1.5.
This may, however, vary with the design criteria which the overturning moment is based upon (operation, storm, extreme weather etc.).

Leg Penetration

The general theoretical consideration of the leg interaction with soil is to consider the leg pinned at the least 3m below sea bed, i.e. minimum penetration of 3m and no fixity effect of the soil.

To our knowledge also those societies which do not specify this value in particular are using the same principle with similar factors of safety.

Scouring

The most specific requirements concerning scouring is that the foundation area is to be reduced by 20% in the bearing capacity calculations. Otherwise, the rules just generally require that the scouring problem shall be studied without giving any further details.

Preload

Not all societies are requiring (in their Rules) preload facilities on the platform. For those societies mentioning preload, the requirement is only given in very general terms without any specification of preloading procedures.

Sea Bed Condition

As this is a location dependent parameter it is generally not included in the rig classification. However, in the design phase some assumptions have to be made. The classification societies will base their review on the assumptions given by the owner or from an 'all probable configuration' approach.

TABLE 3. Summary of Requirements by Classification Societies

Requirement	Overturning F.O.S	Leg Penetration	Scouring	Preload	Sea Bed Condition
ABS	'Sufficient'	Considered pinned at least 3 m below sea bed	- 20% reduction of area for mats	gravity + overturn. moment	Owner assumption
BV	Extreme 1.3 Normal 1.5	----	to be studied	individual preload	----
DnV	1.5	Not stated practice: pinned at 3 m below sea bed	to be studied	----	----
GL	1.5, 1.3 1.1	----	-----	----	----
LR	----	----	to be examined	----	all prob. configur- ations
NKK	----	Assume pinned 3 m below sea bed	----	----	----

2.4 NATIONAL REGULATIONS

2.4.1 General

Many countries do not have their own specific formalized regulations for the offshore activities along their costs. They might therefore only apply the International Regulations or adopt practice from other countries and procedures of legal aspects etc. are discussed from case to case.

We have studied the regulations for three countries: U.S.A., U.K. and Norway and to our knowledge no other country has any more specific regulation concerning foundation stability considerations.

As for the classification societies the rule reference are firstly listed (completely quoted in Appendix B) and further commented on the geotechnical concerned requirements.

2.4.2 U.S.A.

In the U.S.A. the regulations regarding jack-up operation are given in Code Federal Regulations (CFR) spesifically in 30 CFR *MineralResources* and in 46 CFR *Shipping*.

30 CFR: **Mineral Resources**
Chapter II - Geological Survey,
Dept. of the Interior

PART 250: Oil and Gas and Sulfur Operation in the
Outher Continental Shelf.
Reference is given to:
OCS ORDERS (Outer Continental Shelf)

i)	- Gulf of Mexico	Rev. A September 1980
ii)	- Atlantic	Rev. A ’ 1980
iii)	- Pacific	Rev. A ’ 1980
iv)	- Arctic	Rev. A February 1981
v)	- Gulf of Alaska	Rev. A September 1980

ii) iii) v): The first edition: Januar 1980

Order No. 2 Drilling operations

Clause 2 Drilling from Fixed platforms and mobile drilling units

Subclause 2.2 Mobile drilling units, Item C

The first issue of OCS orders effective from January 1980 did require full soil investigation for Mobile Drilling Units in all OCS Areas except the Gulf of Mexico.

OCS orders no. 2 clause 2.2

quote: *'Mobile Drilling Units.'*

Applications for drilling from mobile drilling units shall include the following:

a. (Environmental & Operation cond. for design)

b. (Environmental & Operation specifications for locations)

c. Sediment and sea bed data, including sea bed profiles, sediment consistency, allowable bearing and sliding loads and nearby potential bed hazards, i.e., sand waves, slimps and mud slides.

d. (Current classification)'

unquote

This item C on site investigation requirements was in Revision A deleted with effect from Sept. 15 1980, but transferred and included under the heading of *GeneralRequirements*, subheading 2.1.3 *WellSiteSurveys* by adding the following sentence:

'When requested, this data shall include sediment and seabed data, e.g. seabed profiles, sediment consistency, allowable bearing and sliding loads and nearbed potential seabed hazards, i.e. sandwaves, slumps, mud slides, permafrost, and deposits of frozen gas hydrates.'

It should be noted that the change of wording now allows the USGS District Supervisors to *selectively* require the submittal of the above data for all Regions, while the original wording demanded such documentation for all regions exept the Gulf of Mexico.

46 CFR Shipping

Chapter 1 - Coast Guard, Dept. of Transportation
Subchapter 1-A - Mobile Offshore Drilling Units
Part 108 - Design and Equipment
Subpart C - Stability
108.309 - Stability of Bottom

The requirements here are very much similar to those stated by the classification societies wrt to overturning moments. However, USCG does not require any F.O.S. on this parameter other than what might be included in the environmental criteria, etc.

2.4.3 United Kingdom

Department of Energy: Offshore Installations:

Guidance on design and construction, 1981

Part II, Section 3 Foundation and site investigations
3.1 Foundation
3.1.1 General

An extensive specification on foundation and site investigation requirements is outlined in detail.

A jack-up platform is by D.O.E. classified as a gravity foundation offshore installation and should therefore be considered similar to any fixed gravity platforms with reference to foundation requirements. However, we understand that the practical approach is somewhat different and that for a jack-up there is no requirements with reference to how the location approval is carried out by the marine surveyor/or anybody else involved.

2.4.4 Norway

Norwegian Maritime Directorate, 1975
Subchapter by: Norwegian Petroleum Directorate
Chapter III, Drilling
Section 16. Precautions when preparing the Drilling
Platform for Operations

Sea bed examination, is to be carried out, both before placing the platform and during operation (no requirements to this examination is specified).

2.5 INTERNATIONAL REGULATIONS

2.5.1 Rule Reference

IMCO: Chapter 2-Construction, strength and materials
Section 5, Special considerations for self-elevating units.

NOTE: The IMCO rules are only based on the CONSTRUCTION AND EQUIPMENT of the MODU, and should be considered as code of minimum requirement, available for any country.

This code does only evaluate the rig itself and not the specific location where it may operate.

2.5.2 Geotechnical Concerned Requirements

Clause 2.5.2.1 Bottom conditions

It is required to state the bottom conditions for operations to raise or lower the hull, in the Operating Manual. Any requirements to how these bottom conditions should be described is not given.

Clause 2.5.2.2 Overturning Moments

It is not required to consider the maximum overturning moment in elevating position. However, there is no specification w.r.t. F.O.S. on this point.

Clause 2.5.2.4 Scouring

The effect of possible scouring should be considered and the effect of any sour prevention arrangement where provided also should be taken into account.

Clause 2.5.5 Preloading

Independent leg rigs should be provided with preload capacity to pre-load each leg to the maximum applicable combined load. This preload is to be applied after initial positioning at a site.

3. PRESENT DESIGN METHODS

3.1 INTRODUCTION

3.1.1 Foundation Design Development

From the first jack-up installation in 1953 until today there has been a surprisingly small change in the approach to the foundation design, and the foundation design itself.

The first jack-up platform 'Delong Rig No. 1' and the first Mobile jack-up 'Delong McDermott', had pile-type foundations with cylindrical legs, had no footing structure and penetrated directly into the sea bottom. This design is today still applied to modern rigs and has proved to be a good and reliable design for certain sea bed conditions.

For pile-type foundations the operation areas are somewhat limited. In thicker strata of soft clays excessive penetration will normally be required to develop sufficient capacity. With increasing water depth the use of this type of foundation will be restricted to firmer clays and sandy/gravelly soils. The mat support was developed a few years later for soft sea beds but combined with piles to secure horizontal stability.

Spud-cans were first designed 1955 but also these were initially combined with piles through the cans.

The first foundation design without piles was built in 1956 with triangular truss type legs with large cylindrical spud cans. The ratio diameter vs. height were near one i.e. $(D/H = 1)$.

The philosophy behind the details of the different foundation designs is for us unconfirmed but we have got the impression that the different design developments in many cases have been more a question of different trials from various designers rather than a systematic development. Often the need to limit the draught for shallow water areas and ports may have been of greater importance to the designer than geotechnical considerations.

No specific design has yet turned out to be generally superior. Most designs have limitations w.r.t. sea bed conditions with corresponding advantages and disadvantages.

3.1.2 Purpose of Foundation

The purpose of the foundation is to transfer the load from the legs safely into the sea bed and maintain equilibrium for all loading directions without exceeding the capacity of the foundation soil which may lead to excessive and uncontrollable displacements of a single leg or the total platform .

The vertical capacity can be tested by help of gravity (preloading). The maximum vertical load due to combined gravity and environmental loading which may develop during a design storm can be tested in full scale.

Stability under combined vertical, horizontal and moment loading is considerably more complex and with todays available methods it is not possible to pretest the foundation for this general loading condition. However, an increased preload level will at least partly increase the capacity of the footing under inclined loads.

Due to the fact that the footing structure may in some cases, be utilized for buoyancy purposes in floating position, additional design parameters may also be involved with the background in hydrodynamics and hydrostatics.

3.1.3 Footing Design as Part of the Overall Design

Looking at an overall design concept of a jack-up platform we may divide the structure in subparts like barge, drilling/ living quarters, jack houses, legs and footing(s)

By the 'chain-strength philosophy' none of the above subparts should be of any different importance. However, in view of technological complexity and economy the footing structure is clearly the less favourable.

Apart from the spud/mat question there is very little to gain costwise by trying to optimize the structural design of the foundation. Also a small damage to the footing structure may involve complex and expensive repair arrangement. Therefore conservatism has been assumed to be the best solution to this unfamiliar field.

Consequently less investment ha been allocated to research and development of the footing structure where there is small margins to gain. Especially individual leg footings have been laid behind in development in favour of more 'critical' parts of the platform.

3.2 DESIGN ASSUMPTIONS FOR SOIL PRESSURE

3.2.1 General

At the design stage the magnitude and distribution of the soil reaction pressure will mainly be a function of structural and environmental parameters, the weight and variable loads to be carried and the selected shape and bearing area of the footing.

Jack-up platforms will generally have to be designed for general conditions and not a specific location. The soil conditions may thus vary from soft clays to rock as the extreme cases. For mats the unevenness of the sea bed will have to be considered at the design stage, and the presence of obstructions like boulders or debris may also need to be considered at least to some degree.

Any soil pressure assumption will for a specific location, lead to a penetration calculation which again will require a great deal of assumption for each soil mechanic parameter during the design stage.

Project participants and other companies working with jack-ups design, were asked the following questions:

A) What ranges of the soil pressure distribution do you apply in your foundation design?

B) What is the basic assumptions that lead to these answers?

The answers may be ranged in three categories:

I Legs without footing (pile type)
II Spud-can footings
III Mat footings

Answers A: I 100 - 200 t/m^2

A: II 20 - 50 t/m^2
III 1 - 3 t/m^2

Answer B: This question was very variably answered. Most

B: designers/builders had no independent answer and transferred the responsibility of the values they were using to their clients (the rig owner).

The most soil specific answers we received were from some oil companies. (Their engagement with fixed offshore units may explain this). Their assumptions were based on a generalization of different specified soil profiles.

3.2.2 Considerations of Soil Pressure Ranges.

From a structural point of view all pressures are within reasonable assumptions of net bearing area and total vertical load including preload. However, based on geotechnical considerations the number of parameters involved is numerous.

It is possible to find soil types which will correspond to all the soil pressure ranges given. On the other hand a more complex task is to map the continental shelfs to define relevant ranges of soil parameters and in this way arrive at design values for soil pressure ranges. Summaries of soil data for spesific areas may exist and were presented for major parts of the Gulf of Mexico by Young and Noggle (1978).

What may be done on paper, is to analyse the case of extreme conditions and study the effect of different parameters. This will be studied further in PP2 and PP3.

3.2.3 Pressure Distribution

Individual Footing

Traditionally all designers are assuming the gravity load to be evenly distributed between the legs. Environmental loads will however, cause different vertical loading on each leg. The directional variation of the environmental load may on the other hand give equal maximum vertical load on each leg (if we assume symmetrical shapes of the rig).

Any unsymmetric loading which may occur like effect of the centilever and unsymmetric storing of supply will be included in a worst possible case which is calculated for the rig. The design pressure is therefore the same for all legs and equal to the worst possible case for one leg. This will equal the preload level for all legs and no possible load case should produce a larger bearing pressure than this 'worst case'.

If we are looking at the pressure distribution on each individual footing, most designers assume this to be evenly distributed. (see clause 3.3. 'Assessment of rotational stiffness of foundation'. The effect of boulders and or debris has to some extent been taken into account by some designers. The structural strength of the footing will in most cases allow considerable variations in local pressure without special design considerations.

Some designers do include a study of the scour effects simply by reducing the bearing area with 20% according to the regulations of some of the classification societies while others consider a reduction of the area with up to 50% at the design stage.

Generally these assumptions are accepted in the industry and considered to be safe. This question will be studied further in PP2 but we will at this point only comment that many rigs have experienced different penetration on each leg and that this could indicate one or both of the two following situations:

1) Different soil properties at each foundation i.e.
   Variable bearing capacity

2) Uneven loading on the different legs at preload.

   Extreme loading may exceed the preload level and result in further penetration of the one leg.

After installation (incl. preloading) the variable load (supply etc.) should at any time be stored such that the centre of the rig remain at the centroid position. This is however, treated differently onboard in the different rigs. Some operators has stricter rules/guidelines of where to store the supply than others, and in some companies they are practicing that the load distribution is summarized and calculated every day.

Most reports from geotechnical consultants which has been available to this project are 'penetration studies'. In these reports the question of soil distribution has not been discussed. This may rather indicate that the question has not been of interest in this business rather than reflecting the complexity involved in it.

Mats

The basic approach is to assume linear distribution of of the soil reaction pressure against the mat. However, designers do also consider sea bed unevenness and spanning of the mat over sand waves to define critical conditions for the bending moments in the mat structure.

Scour is a major uncertainty in the assessment of soil pressure distribution and magnitude. Model tests and practical experience has shown that considerable scouring leading to undermining of large parts of a mat is possible provided the wave and tidal induced current velocities increases beyond a critical value. Scour is normally only a problem on cohesionless sandy to slightly silty soils. The bearing capacity of these soil types are considerably higher than for soft clay, and only minor contact areas are required.

This means that soil reaction pressures may nearly change to point loads compared with the dimensions of a typical mat. The most critical distribution of these *pointloads* against the mat depends on the structural strength of the mat parts and the mat geometry.

3.3 ASSESSMENT OF ROTATIONAL STIFFNESS OF FOUNDATION

3.3.1 Definition

This problem is limited to concern jack-ups with individual legs. For mat supported rigs this is considered as an overall stability consideration where other parameters are involved.

By 'rotational stiffness of foundation' we mean the soil ability to take rotational moments about axis in the sea bed plane. This is closely connected to the structural stiffness of the platform, i.e. the flexural rigidity of the legs and the distance between jackhouses and spud-cans, relative to the stiffness of the soil which depends on the shear modulus of the soil and the contact area.

3.3.2 General

The rotational stiffness of the foundation can be of major importance in the structural analysis of the whole rig and especially for the legs and the jack house structure. The rotational stiffness assumed in the design analysis will have to be considered by the classification society involved. It is however obvious that any stiffness assumption has to be based on assumptions regarding the soil conditions.

Most classification societies use the following assumption for the rotational stiffness of the foundation:

> *'Legs without mats, which may penetrate the sea bed. are to be considered pinned at least 3 m below the sea bed'.*

This assumption has generally been considered to be the most conservative approach which could be used. However, most parties involved in the design and classification do agree that this assumption is too rough and that a more specific and accurate assumptions could be made.

The presence of rotational stiffness will clearly tend to reduce the bending moment in the upper part of the legs and in the jackhouses. Under extreme loading the most stressed footing may tend to react softer, however the less stressed footings will probably have a considerable stiffness and moment capacity at least when the spud can is fully penetrated.

Consideration of rotational stiffness may have a significant influence for fatigue evaluations where more moderate loading conditions will be predominant.

Todays common practice is not to consider rotational stiffness. There exist, however, at least one point where this assumption will be non conservative. At the bottom of the leg in the connection between leg and spud-can the presence of rotational stiffness should be taken into account regarding local strength and fatigue.

As far as we are aware of there has only been a few attempts by designers to take rotational stiffness into account based on assumptions of soil conditions and relevant back-up calculations. Regarding the classification of platforms these platforms the classification society has approved but then with limitations w.r.t. soil characteristics at the locations where the unit will operate.

By methods which will be described in PP2 it has been shown that the rotational stiffness is not always zero. Studies of surface footings (not penetrated) have been carried out and the main parameters for the rotational stiffness have been evaluated.

The rotational stiffness is directly a function of the shear modulus of the soil. Consequently with reference to basic stress-strain relations of soil, the rotational stiffness will be large at low load levels and decrease with increasing stress levels.

This explains why the rotational stiffness should be considered in the fatigue analysis of the legs. The lower part of the leg will consequently be exposed to cycling bending stresses related to moderate environmental loading and a correspondingly increased moment capacity and stiffness of the soil.

Penetrated footings represent a more complex situation and generally a method based on an extrapolation of the surface solutions could be used. This will be considered further in PP2.

On several occasions cracks have been found and repaired in the lower sections of the leg and the connection to the footing structure. This may indicate that these parts of the legs represents areas with higher stresses than assumed, and could be explained by the presence of some rotational stiffness.

Apart from the shear modulus of the soil the contact area between the spud-can and the soil is a the critical parameter in the rotational stiffness calculation. Most spud-cans designed today have slight conical shaped bottoms and the bearing area will increase rapidly during the initial penetration. For stiff soil the maximum bearing area might not be obtained and this may cancel all relevant assumptions made w.r.t. rotational stiffness. Further scour of sandy sea bed may tend to undermine a footing and in this way reduce the contact area that developed during preloading.

3.4 PENETRATION AND PRELOAD CONSIDERATIONS

3.4.1 General

The penetration of a footing into the soil occurs when the applied bearing pressure exceeds the bearing capacity of the soil. Penetration continues until the bearing capacity corresponding to ultimate soil strength equals the applied footing pressure.

The applied footing pressure should at installation equal the preload level, which is generally defined as the vertical leg load equal to the maximum applicable combined load for to which the rig will be exposed. It includes weight of rig with maximum variable load onboard plus the vertical component of the overturning moment due to environmental loads (wind, current and waves).

As described earlier the vertical stability of independent leg type jack-ups can be controlled by the preload method. The sliding and overturning stability can, however, only considered through theoretical assumptions and calculations.

In Chapter 2 we looked upon the factor of safety against overturning. This is the only parameter considered by the classification societies in their evaluation of the overall stability of the rig in elevated position. This is a parameter independent of the local soil conditions and cannot garantee safety against bearing capacity failure under the most stresses part of the footing(s).

3.4.2 Allowable Penetration

At the design stage leg penetration is considered as an additional leg length which has to be estimated based on experience and to some degree on eventual information about the general soil conditions in the areas of operation Penetration calculation at the design stage before any specific location information is available, will have to be highly theoretical and pending more on assumptions of soil properties than the analyis method.

When describing the platform the maximum allowable penetration is basically a question of the leg length and is normally only critical for maximum water depth. The maximum allowable penetration can thus be defined as below.

(Max allowable penetration = leg length
-required length through the jack house
-airgap
-water depth)

This entails that with a smaller water depth the allowable penetration may be equally larger.The only practical limitation will therefore be the pull-out capacity as the pull-out force obviously will increase with penetration depth. This will be discussed in a following clause.

3.4.3 Design of Preload Capacity

Preload consideration at the design stage is not only a calculation of required load level, but even more important at this stage, is how it may be applied. Two systems are widely used today. Ballasting is typical for three legged jack-up platforms while the method using diagonal loading requires four (or more) legs.

The two critical design parameters for the preload ballasting arrangements are the total required preload amount, and distribution between the legs. For large jack-up rigs made for severe weather conditions (like North Sea), the required added preload may be up to 70% of the total rig weight before preloading. For rigs designed for less severe weather and smaller waterdepth this ratio will be around 25-30%. There exist however, also rig which have been designed with no preload possibilities.

The diagonal preloading system needs to be capable of adjusting the load on each individual leg and the preloading is therefore purely a question of the designing the jacking machinery/mechanism, and the rig may be able to increase the vertical leg load by 100%.

3.4.4 Overall Stability Considerations

The overall stability depends on the platforms ability to avoid all possible individual stability failure mechanisms: vertical penetration of individual legs or the most loaded parts of a mat, horizontal sliding and overturning stability. As described above each failure mechanism may be of different relevance to the design before a specific location is described. As far as we are aware of it is not common to carry out such global studies at the design stage. However, there exist exceptions where designers have carried out a broader evaluation of the foundation stability of a design.

What seems to concern most designers of independent leg type platforms is the punch-through problem. We are aware of designers who are studying this in great detail from a structural point of view. They admitted not to be able to solve the geotechnical problems, instead they studied the possible consequences of the punch-through and further they intend to develope their design to withstand these effects to a higher degree.

Other geotechnical considerations which could be included in the stability considerations might be the effect of cyclic loading and the sea floor instability itself.

3.5 PULL-OUT RESISTANCE AND MEASURES TO REDUCE IT

3.5.1 Definition

The pull-out resistance is defined as the downwards soil reaction force consisting of the overburden soil pressure on top of the footing, the side friction around the circumference and the bottom suction.

The bottom suction component of the force is normally reduced by different kinds of jetting systems also called suction releasers.

The pulling force is obtained by lowering the hull in a floating position and pulling the legs upwards with the jacking machinery while obtaining reaction forces from the buoyancy of the hull.

3.5.2 General

Few of the rig designers that we have been in touch with, have actually carried out calculations in order to determine the magnitude of possible pull-out resistance from a geotechnical point of view.

Studies of pull-out resistance of embedded objects has shown that a modified upwards bearing capacity theory will give reasonable values, compared with model tests.

One designer has however, carried out a more extensive study on suction of a sit-on-bottom type (mat supported) offshore structure (Ninomiya, et al. 1972). The study is based on experimental methods for obtaining the suction force, and empirical formulas were derived for a prototype structure. Effects of jetting systems were not included. These formulaes will be further studied in PP2.

The jacking capacity is generally designed by considering the maximum lifting force required. This is usually equal to the net weight of the hull including a practical amount of variable load. For some very few rigs the lifting capacity also includes the weight of the preload ballast.

Most jacking machinery may be reversed and hence the pulling capacity will equal the lifting capacity. The lifting capacity is by the designer assumed to be satisfactory also for pull-out.

Pull-out resistance is generally not looked at as an important problem, however, some cases of pull-out problems have been reported. The bottom suction effect represent a significant part of the total pull-out resistance and this underlines the need for an effective jetting (suction release) system.

3.5.3 Jetting Systems

It has neither been possible nor the intent of this project to study the different existing jetting systems in any detail. It is, however, our impression that all jetting system designs have been based on previous experience and earlier trials and errors. To our knowledge geotechnical considerations have not been involved in evaluation of the efficiency of jetting systems regarding the spacing of the nozzles, type of nozzles, pressure used, duration and pull-out velocity.

3.5.4 Footing Shapes

We have not included the footing shape in our pull-out resistance discussion so far. As mentioned in clause 3.1 we are not aware of design practice or separate studies which consider the shape of spud cans when evaluating the bearing capacity and the pull-out resistance of jack-up foundations.

For deeply embedded spud-cans the only practical shape related factor to pull-out resistance will be to make the top of the footing conical. This more streamlined shape may reduce the pull-out resistance to some degree and also allow the mud on top to be more easily washed off when retracting the legs.

4. PRESENT INSTALLATION AND OPERATION PROCEDURES

4.1 INTRODUCTION

Most investigations of losses or casualties to jack-up rigs which have occured in the past shows that the majority of accidents occur while the rigs were either moving between locations or in the operations just prior to or after a rig move.

In this project the procedures related to installation and operation of the rig and not the move operations will be evaluated. One of the aims is to evaluate how the present procedures might affect the foundation stability and safety of the rig after installation.

Installation and operation procedures varies considerably, not only between different operating companies, but also from rig to rig within the same company. Each operating company will usually have written operating guidelines. These have initially been worked out by the rig designers and later converted to include the operating company's experience.

The rig mover responsible will however, only use these guidelines as general information as he will consider each step during the move differently and judge from his own experience how to operate the rig. What might be an acceptable procedure at one location or part of the world might not be acceptable on other locations.

The Gulf of Mexico is considered as the best known jack-up operation area and hence most of the procedures have been developed with rigs in this area. For rigs outside this area methods acceptable in the Gulf of Mexico have been converted and modified to what the rig mover (or the company) feel could be an acceptable procedure for the new location.

The official 'Operation Manual' which is kept onboard and which is approved by the classification society contains essentials related to classification only, such as operation of jacking machinery, pumping systems etc. It does not contain practical guidelines requirements of how to operate the rig under different conditions e.g. punch- through, preload procedures, and pull-out problems. In the operation manual the relevant limiting operational condition are to be summarized. However, limitations describing the soil conditions are usually not included.

4.2 SITE INVESTIGATION

4.2.1 General

The broad variations of the soil conditions on which a jack-up platform may operate during its lifetime represent a potential risk. This hazard is attempted to minimize by various site investigations. Improved knowledge of the soil conditionds may allow the geotechnical engineer to predict foundation performance and allow the operator adjust the installation procedures at least to some extent in order to reduce eventual hazards.

The available geotechnical information, may vary a great deal from location to location. The required extent of soil investigation may also vary depending on the soil type and variation at the location.

The site investigation required for exploration drilling in new unexplored areas where no previous investigation has taken place may differ considerably from what is required for a location in a field already under development.

There exist today several methods for investigation of the sea bed. The cost involved may limit the extent of a site investigation. However, in each individual case there will be a certain amount of information required to reduce the risk of foundation failures. The cost of such investigations will vary but will generally be rather small compared with the consequences that the lack of information may cause.

A 'location approval' will be required to meet the minimum insurance requirements w.r.t. site investigation. The required investigation to satisfy a 'location approval' is fully depending on the marine surveying company. Each company may consider each location differently based on their experience and therefore require various parameters to be studied and recommend various site investigations to be carried out.

Some national authorities may also require specific site investigations before permission to drill is given to the oil company. It is then up to the oil company to evaluate the degree of investigation to be carried out.
It is reasonable to accept that no fixed procedure should be followed for a site investigation. The soil and sea bed investigation should however, reflect the already existing information as well as specific geotechnical problems related to the actual platform design and environment of the specific location.

What might be established in some detail is which parameters that should be investigated and to what degree of accuracy each parameter should be estimated. This is one of the goals of part project 3 (PP3) of this study.

Basically we may divide the site investigation methods in two kinds:

- Acoustic survey methods
- Soil sampling methods

Both methods have their limitations. The acoustic survey will give information about the soil stratification over the investigated area and indicate soil type and thickness of the different upper layers. But the absolute thickness of the layers and their soil properties can only be established by reference soil sampling and testing or in-situ testing, like the cone penetration test (CPT) and the remote vane test (RVT).

4.2.2 Acoustic Survey Systems

Acoustic methods are carried out from moving ships. The desired area around the specified location will be traced in a grid system.

The acoustic methods is based on profiling the materials below the ship for later to coordinate the recorded information to produce maps of the sea bed and the different layers below the sea bed. The grids of the area around the location may vary in overall size and intensity of runs. A close-grid survey is one where the sounding lines are spaced not more than 100m apart with another set of soundings run perpendicular to the initial ones. When investigating areas larger than about one square kilometer, the grid lines will normally be spaced at about a 1/10 th of the length of the side of the investigated area.

Side-scan sonar may be used to locate obstacles like large rocks, wrecks, etc., which may be hazardous for the jack-up footing. Sonar uses high frequency (40 - 250 kHz) soundings and will cover a certain area on both sides of the ship. Alternative methods may be the use of divers or by dragging a wire over the area between two ships like a fishing-seine.

The sea bed contours are maped by running a bathymetric survey by eccosounders with fairly high frequences (10 - 80 kHz). This survey will normally only give the water depth and not penetrate the sea bed.

The shallow seismic method allows penetration of the sea bottom by using a further reduced frequency. This survey method will not give detailed information about the soil itself but only the relative layering of the soil profile. Where a seismic survey is performed the depth of the survey is to be at least 50% in excess of the maximum allowable design penetration of the spud-cans.

The acoustic methods may not localize buried pipelines, submarine communications cables or other buried metallic hazards. To find such obstacles a Magnetometer survey may be implied.

4.2.3 Soil Sampling Methods

To investigate the soil properties itself a soil sample has to be taken and investigated in a laboratory, alternatively in situ tests may be performed with instruments inserted into the sea bed.

There are two categories of soil sampling operations at sea.

> Shallow penetration sampling, usually extending less than 10 feet below the sea bottom. This is a one time or single shot operation.

> Deep penetration sampling, can be extending to depth substantially below the sea bed.

The main technical/economical differences between the two methods is the requirement of fixety of position of the vessel from which the sampling are carried out. The shallow soil samples may be taken from an ordinary vessel. For a deep penetration sample however, it requires a vessel which maintain a fixed position, either by multiple mooring or by the use of dynamic positioning systems.

Shallow soil sampling

Different methods: from simple grabs or box dredgers, divers, gravity or vibrocores to sophisticated remote operated vehicles (ROV) may be used to pick up a sample of the sea bottom.

The gravity cores may penetrate the sea bed a few meters, large gravity cores has been able to get samples in soft clay, down to about 30m below sea bed, otherwise these methods will only get samples from the top layer of the sea bed.

The samples taken by these methods will normally be disturbed leading to reduced accuracy of the soil parameters.

Deep penetration soil sampling

Soil sampling at greater depths below mudline requires soil borings. These borings are carried similar to the oil-well drilling operation, but with lighter equipment.

The oil well drilling equipment on the drilling rig itself may also be employed for soil sampling by utilizing appropriate wirelines samplers under engineering supervision. However, this will be too late for predictions necessary for suitability evaluations and modification of installation procedures.

For the jack-up foundation purposes the soil boring may be carried out from anchored barges, or anchored or dynamic positioning boats. Barges are commonly used in inland waters but may also be used offshore for limited weather conditions.

Drilling operations from vessels with multiple mooring systems have been carried out in water depths much deeper than the capacity of the larger jack-up platforms.

Deep penetration sampling may also be carried out by ocean floor drilling rigs. These are small platforms which are lowered down to the sea bottom from a vessel and will operate from a position at the sea bottom. Such platforms may be divided into two cases; manually operated by divers and automatically or remotely controlled from a surface supported vessel.

4.2.4 In-Situ Soil Tests

All above discussed soil sampling methods are based on further examination of the soil a laboratories either on board or ashore. There exists also a number of in-situ tests which give some of the required soil properties directly without having to examine the soil in a laboratory. Many of these tests are considered just as good/or sometimes better than the soil sample investigation as the soil disturbance caused by sampling is avoided.

Different types of cone penetration tests may be carried out, directly on the sea bed. Some geotechnical consulting companies have developed their own testing facilities for sea bottom soil testing. These are heavy jacks that are lowered down to the sea bed from ships. The jack will then be able to push tests rods (CPT or vane) into the soil. The load/penetration relationship of the CPT-test and the torque/rotation relationship of the vane will be recorded onboard the ship.

In-situ tests of the sub soil may also be carried out by lowering suitable mechanical testing equipment into a soil well. CPT and RVT equipment for down-the-hole operation has been developed and is today in use.

4.2.5 Todays Soil Investigation Practice

As previously described there exists numerous methods for obtaining the desired soil information. Consequently, as the use of jack-up platforms has extended into areas with a more severe environment the attitude towards soil investigations has changed. Today most oil companies will require soil borings and others will also insist of having a full survey with shallow seizmic combined with soil borings and laboratory tests.

The operators (owners) will generally rely on the oil company's site investigation and assume it to be satisfactory. A few operators have developed their own site investigation programs which they insist on being followed every time.

Mat Supported Rigs

For mat supported rigs the sea bed topography is of major importance. Sea bed unevenness is more critical the harder the sea bed is. Mat supported rigs may have problems when operating on sandy or hard clay sea beds if the sea bed is uneven or inclined more than 1°. Although mat supported rigs have been designed particularly for soft clay conditions there will also be a lower limit of weakness of the sea bed soil which need to be avoided.

For mat supported rigs the undrained shear strength of the soil in the upper meter is a critical factor regarding sliding stability and if the increase in strength with depth is small deepseated overturning failure can become critical.

Today the sea bed topography seems to be the parameter of major concern to operators and oil companies when evaluating mat supported rigs while the soil properties seem to be of less concern.

Individual Leg Footings

Based on the traditional assumption that the preload testing of jack-ups, gives a reasonable assurance against foundation failure, and that the operation is carried out successfully without the benefit of prior soil investigation at the site, the desired sea bottom survey will vary from nothing to a full investigation with shallow seismics and deep soil borings.

The penetration depth can be a critical factor in the evaluation of the suitability of individual leg platforms prior to installation. The undrained shear shear strength of the soil and its distribution with depth below mudline will be a governing parameter in this evaluation and needs to be investigated in some detail, not only in top layer but down to a depth of at least one to two footing diameters below the expected penetration.

Today the investigation is often limited to top layer soil sampling and interpretation of shallow seismic surveys.

4.2.6 Economical Aspects

The soil investigation will under no circumstances be a major cost compared with total cost involved in drilling of an offshore oil or gas well or the value of the jack-up platforms. Even delays due to unexpected performance and eventual replacement of an unsuitable platform with a new one may cost considerably more than a well prepared investigation program. However, the site investigation is a part of the total budget and the benefit has to be related to the costs involved.

To be able to prepare an effective site investigation program and to evaluate its benefit an better understanding of the soil structure interaction and the consequences of misjudgement of the soil conditions will be required than what we feel is the normal situation today.

The cost involved in a site investigation program is heavily dependent on the extent of the investigation, the local conditions and also on the market situation. It is therefore not possible to indicate any fixed amount. The below numbers should just be taken as an indication based on the information gathered from different companies involved in offshore site investigations in autumn 1982.

Any soil investigation prior to the rig is moving on location will require the use of a barge or vessel. Shallow seizmic surveys will normally be carried out from small specialized vessels. Dayrates for a fairly equipped seizmic surveying ship all inclusive, may be around NOK 70.000 plus any mobilizing and demobilizing cost which may be added. If a remote operated vehicle is required, for additional inspection of sea floor, this may cost additional NOK 25.000 per day.

Specialized soil drilling vessels may cost approximately NOK 100.000 (US dollar 15.000) per day to hire in the North Sea. When including the staff and equipment required for the soil analysis, the cost will almost double (NOK 200.000 pr. day).

If deep boring from a drilling vessels is not required, the investigation cost may be considerably reduced. Most equipment used for soil investigation are mobile and may be brought onboard in any ship. Light supply vessels may as earlier described also be used. The marked daily rate for such ships vary a great deal and is generally cheeper than the surveying companies' own ships. The overall cost involved in such arrangement will depend more on the cost efficiency in equipping the vessel for soil investigation.

The time required for the different investigations will naturally vary a great deal with the weather conditions, the water depth and the soil conditions, and can affect the cost to a large extent.

An average, rather extensive investigation consisting of the seizmic survey and a deep soil sampling is outlined below.

A shallow seizmic survey eventually with gravity or vibrocoring, covering a grid of about 3 x 3 km is estimated to 3-4 days including mob- and demob. with a estimated dayrate of NOK 70.000	NOK 1/4 mill. (US dollar 350.000)
A soil boring with sampling, plus CPT and/or RVT from drilling vessel including some laboratory testing is estimated to require 3-4 days including mob- and demob. Dayrates estimated to NOK 100. to 200.000	NOK 1/4 - 3/4 mill (US dollar 35 - 100.000)

The above cost represent what could be necessary for an investigation required for an unexplored location where the rig may be operating close to its design conditions and where the soil parameters may be critical for the rig safety.

4.3 PENETRATION PREDICTION

4.3.1 General

Reference is made to the previous discussion under Clause 3.3, on penetration considerations at the design stage of a jack-up. The penetration prediction will obviously not be more reliable than each individual parameters used in the calculation. The quality of these parameters may (as discussed in Clause 4.1) vary a great deal with the extent and quality of the soil investigation.

4.3.2 Observed vs. Calculated Penetration

Considering the possibilities of unaccuracy involved in the penetration prediction procedures, it is surprising to see how accurate the penetration predictions sometimes are.

On the other hand the accuracy may be explained with adjustments based on practical experience which seems to be involved in every penetration prediction.

Only two studies on the penetration observations vs. penetration prediction for independent leg footings have as far as we know been published. Gemeinhardt and Focht (1970) and more recently Endley et al (1981) have presented the results of a large number of case studies of the measured and predicted values of the bearing capacity. Penetration for mats is generally not considered to be a problem. Even for the extreme soft clays in the Mississippi Delta areas only a few feet penetration of the mat has been reported. Hirst & al (1976) and Young & al. (1981) addresses the penetration of mats and give a limited comparison with observed values.

The published results of these studies of the ratio between predicted and observed penetration, confirms the large variation in this ratio. These reports also show the variation of the ratio versus the average shear strength of the soil. The predicted penetration tends to be more accurate for larger shear strength. The observed values varies about 70% for shear strength of about 2 t/m^2 but only 40% for 6.5 t/m^2. The average of all recorded ratios is increasing (not linearly with shear strength from about 0.75 to 1.25 for average undrained shear strength varying from 21 t/m^2 – 6.5 t/m^2 respectively. (Endley & al.,1981).

Discussions with the operating companies confirm the large variation in this critical ratio of actual penetration versus the predicted (calculated value). One operator answered: 'Generally the predictions are good, but sometimes we get BIG surprises!'.

This also indicate the general approach to this problem.

Penetrations larger than expected will not be considered a problem before the limitations w.r.t. leg length is reached. Shorter penetrations are usually only considered critical of the penetration depth is shorter than the height of the cone of the spud-can bottom and full contact is not achieved.

4.3.3 Penetration Prediction Calculation

The general soil bearing capacity formulae that different investigators have derived are generally based on a combination of theoretical solutions and experimental results.

Only smaller variations in the results will develop when using the different accepted methods.

Further, the load distribution through layered soil is a critical point for discussion. This will be of major importance in the case of 'punch-through' predictions.

The generally accepted methods which have been used in practice during the last 10 to 15 years are Skempton's (1951) method for uniform soils and Davis' and Booker's (1973) method in cases where soil strength increases considerably with depth below mudline.

PP2 will cover further discussion of the background for each of the different methods.

4.4 ELEVATION, PRELOAD AND PULL-OUT PROCEDURES

4.4.1 General

All procedures involved in the installation and moving off location, do vary a great deal from platform to platform. Firstly the variations will be based on the difference of the rig itself. Major variations are such as:

- three or four legs
- electric or hydraulic jacking machinery
- mat or spud-can footing
- jetting (suction releasers) systems mounted or not

The rig designer does initially make his recommendations of how the rig should be operated. Some of the designers we have been in contact with did however, express that this documentations often tends to be too theoretical. They are aware of of the fact that operators often look upon the problems differently and prefer to solve them more independently on a case to case basis.

However, many operating companies rewrite the designer's operation guidelines and include their practical experience and their company philosophy on safety considerations.

The case to case approach indicates, that these procedures will vary with the circumstances. Both weather and sea bed conditions will have to be considered w.r.t. the elevation, preload and pull-out procedures.

In the following a summary of geotechnical considerations involved in the installation and operation procedures is given without the intention of covering all possible situations that may develop.

4.4.2 Elevation

The weather condition during installation is a critical factor regarding impact forces on the spud cans and legs at touch-down. Generally this limits the installation possibilities to periods of calm sea and moderate wind.

After touch-down and elevation of the hull above the minimum airgap to avoid wave action the first confirmation of the penetration predictions will be possible. The bearing pressure corresponds now to the platform weight, and the first check on the predicted load versus penetration curve may be carried out and further adjustment of the penetration prediction could be made.

The air gap during preload may vary with the tide amplitude, soil properties and jacking machinery's capacity. Most jacking machineries are not capable of jacking during preload hence the jacks have to be locked in one fixed position for each preload level step during the penetration.

With such systems large tide amplitudes will involve a corresponding large variation in the air gap during preload. If the soil properties are well known and a uniform penetration is expected the rig would be allowed to have a larger air gap than ordinary recommended during this procedure.

When more than one leg is preloaded simultaneously uneven penetration of each individual leg may occur, only due to different soil properties beneath each leg. No operator has reported this to cause any major problems. However, the case of sudden penetration of one leg may be critical as it will happen faster than the jacks are able to compensate. This is the case for most 'punch through' situations when the spud-can suddenly penetrates a hard crust layer, or in the case of the spud slipping into an old footprint left from previous jack-ups operations at the same locations.

4.4.3 Preload

General

Mat supported rigs are not preloaded as far as we have been informed. Individual supported rigs with three legs will have ballast tanks and they may be filled in different orders to either preload one leg at the time or all three together.

The preload of one leg at the time is considered safer but is more timeconsuming.

A four legged rig may be preloaded by diagonal loading. This means that by releasing two diagonal legs more load is transferred to the other legs. Theoretically the load may be increased 100%.

Preloading in Soft Bottoms with High Tides

In soft bottom where high penetrations are expected and where there is a large tidal difference, the preload procedure may be complex and time consuming if the jacking capacity is less than required preload.

The assumed safest method will be to preload one leg at the time and start at maximum tide and by penetration and lowering, keep the air gap constant as the tide level decreases. At least one time interval of six hours will be required to preload each leg and for each time interval the penetration may maximum equal the height of the tide difference.

A four legged platform is more flexible in this respect as the preload is obtained faster on such a rig.

Preloading on Hard Bottom

In hard bottoms, sands or overconsolidated clay there might be a problem to penetrate the sea bottom at all. A certain penetration is desireable in order to improve the horizontal stability and reduce the consequnces of scouring. The jetting system which is developed for pull-out purposes has also been used for penetration purposes. This may give satisfactory penetrations but may involve a certain risk of further uncontrolled penetration.

By jetting water into the sand the bearing capacity is reduced by liquefaction and the required penetration may be obtained. The sand just below the footing is, however, after this procedure, disturbed with increased void ratio below the spud-can and the disturbed sand pad may be critical w.r.t. further liquefaction and hence uncontrolled penetration. The jetting method is therefore not generally recommended for penetration purposes, nevertheless the method is in use today.

Jetting may however, with great success be used to penetrate through a hard crust layer. The crust is usually overlaying soft material in which the above described pad of the disturbed sand, will not be present. In such a case, the risk of relying on the bearing capacity of the crust will be more critical than the risk involved with the jetting.

Preload Duration and Criteria for Dumping of Ballast

The final penetration depth should be reached after the required preload is applied. The footing pressure equals then the bearing capacity of the soil. Further settlement may however, occur due to consolidation effects of the soil. The platform is therefore left with preload onboard for some time to accelerate this process.

The duration of this preload consolidation is usually a fixed time which either the designer, the operating company or the rig mover will decide on. The time period may vary from zero to 24 hours, but is usually between 3 to 6 hours.

From a geotechnical point of view, the criteria for preload duration could be established as a function of the soil type. As far as we are aware of this has not been considered in practice.

In the case of 'punch through' or other sudden penetration problems, the dumping speed will be critical for having the preload released. The faster the unloading will take place the better. Today many rigs have quite complicated and individual systems which involves many operations before the load is released. There exists however, central preload control systems on the market, but to our knowledge not much used by the industry yet (Chiles 1982).

4.4.4 Pull Out

As far as we know there exist no written procedures neither from designer nor operator, of how a pull-out problem is to be solved. As described under the design methods a main unknown parameter for pull out calculations is the weight and the strenght properties of the backfill. Further the base adhesion or suction is a major factor and requires generally that suction release or jetting systems are intact and operating properly.

Different jetting systems are applied to eliminate the suction force below the spudcans.

- High pressure & low volume systems
- Low pressure & large volume systems
- Suction releasers which may only be suction compensating pipes from below spud or mat to free water volume.

Any preferences to the different systems has not been brought forward to our knowledge. The pull-out procedures has generally not been involved with much complications. However, cases where it has been impossible to withdraw the legs have been reported and in some cases the only solution to get the platform off location has been to cut the legs.

4.5 STABILITY CONSIDERATIONS

4.5.1 General

Depending on the extent and quality of the site investigation and local statistical data of the weather conditions, the overall stability of the jack-up platform can be evaluated.

Today such studies are usually carried out by geotechnical consultants. The content of the study may vary from one consultant to another, but is more dependent on the on the specification of the work. The oil companies that usually order stability evaluations do not have standard procedures for what this study should include. Usually this study is carried out by the same consultant involved in the site investigation.

The primary parameter to be examined is the vertical stability. For independent leg platforms this is essential. For mat supported jack-ups the penetration is generally very limited and therefore not of great importance. However, in very soft clays even mat supported platforms may penetrate until the buoyancy of the mat relative to the displaced soil volume increases sufficiently.

Most stability studies made available for evaluation within this project have only contained penetration predictions and simple overturning considerations. however, a few reports contain additional studies of sliding stability and warnings regarding scouring possibilities.

4.5.2 Overturning Stability

Overturning stability is the only compulsory stability requirement regarding classification. It does not include any geotechnical considerations and is purely a a function of the overturning moment, the weight of the platform and the geometry of the foundation. The platform is assumed to rotate about the footing edge or about the center of a spud can. The requirements vary slightly within the different classification societies.

From a geotechnical point of view the bearing capacity of the soil is an important factor when evaluating overturning stability. Overturning about the edge of a footing requires infinite bearing capacity and is thus impossible. The real safety against overturning is thus also a function of the bearing capacity of the soil.

For independent leg type platforms the overturning stability is covered by the preloading procedure. For mat supported jack-ups on soft clay sites the overturning considerations are more complex and probably more critical than for independent leg type platforms.

4.5.3 Sliding Stability

The studies of sliding stability regarding specific locations available to the project concern mat supported platforms. It seems to be a general assumption that sliding is not a problem for independent leg type platforms. This will be further studied in PP3.

Independent Leg Platforms

The distributions of the total horizontal load between the different legs should be investigated further. Common practice has been to distribute the horizontal force more or less evenly on all legs. From a structural point of view this may be a fair assumption as all the legs have been designed equal and the wind force is distributed equally provided the degree of fixity at sea bed is equal for all legs. Assuming pinned conditions at the bottom of the legs justifies the assumption of equal distribution of horizontal force.

From a geotechnical point of view the degree of fixity will most probably vary considerably dependent on the vertical load level and result in a nonuniform distribution of the horizontal force between the legs. The safety against sliding will thus vary for the different legs.

The penetration of a spud can into the soil will clearly improve the sliding resistance due to development of passive arth pressure and side friction on the embedded sides of a spud-can and the conical tip.

Differences in horizontal deflection between the different spud-cans may, combined with variabel rotational stiffness cause high bending stresses at the intersection of leg and spud can. Several accidents involving leg failure have remained unexplained and the above described mechanism could possibly have such consequences.

Mat Supported Platforms

For mat supported platforms the horizontal sliding resistance can be considerably less than for penetrated independent legs. Our experience is that although this is a critical failure mode (Young, 1981) it is rarely examined in detail by the operators or other parties involved in the installation and operation of mat supported jack-ups.

Short skirts are usually used around the periphery of a mat to increase sliding resistance, but also to prevent undermining by scour or pumping in and out of water under rocking motions. In several cases large sliding displacements have been reported in connection with hurricanes in the Gulf of Mexico. A detailed study of this problem will be carried out in PP3.

A major factor of concern for concrete gravity platforms in the North Sea area has been the effect of cyclic loading. This is not included in any evaluation of sliding stability of jack-up platforms known to the authors, and is a further item of interest for this study.

4.5.4 Bottom Stability under Wave Induced Loading

In addition to the soil stresses induced by the presence of the platform the wave induced pressures on the sea bed may contribute significantly. The wave pressure on the sea bed may produce relatively high shear stresses in the underlying soil and ultimately produce large displacements or total failure. The problem can be significant for soft underconsolidated to normally consolidated clays in water depths relevant for most jack-up designs.

The prediction of sea floor instabilities is very complex (Wright & al., 1972) and such studies have to our knowledge not been carried out for specific jack-up locations.

Different areas may, however, be characterized as possible sea floor sliding areas and particular precaution should be taken in these areas. Dependent on the severity such areas should eventually be abandoned by jack-up platforms.

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APPENDIX A:

LIST OF MEETINGS ARRANGED FOR PP1 REPORT

MEETING ARRANGED w.r.t PP1 REPORT ON

"Evaluation on previous experience"

E	COMPANY	LOCATION	MEETING PARTICIPANTS	
10.82	Shell International Petroleum Maatschappij BV	Hague	W.P.M. Van der Watering Dr. J. Cuckson J. Mr. Wilson Ph. M. Walter Tore J. Kvalstad Chr. Behrens	(head) Production Offshore Developm. & Design (Geophysics) VERITAS RD VERITAS RD
10.82	Forex Neptune	Paris	P. Morvan Tran Thanh Dang J. Edgren Chr. Behrens	Engineering Manager Head of Structural Eng. Group VERITAS, Paris VERITAS, RD
11.82	Arco Oil and Gas Co.	Dallas	W.M. Evans J.D. Allen R. Ruedrich J.J. Guerrero W.R. Worthington J. McEver Khoi M. Le W.L. Koederitz D. Jensen Chr. Behrens	Director Drilling Technology Design Civil Eng. Director Drilling Operations California Distr. S.Louisiana Distr. S.Texas Distr. Arco Alaska Inc. Dallas VERITAS, Houston VERITS, RD
10.82	*Sedco Inc.	Dallas	M. Oudshoorn D. Jensen Chr Behrens	Ass.Vice President Engineering VERITAS, Houston VERITAS, RD
11.82	*Baker Marine Eng.	Houston	H. Treu D. Jensen Chr. Behrens	President VERITAS, Houston VERITAS, RD
11.82	Getty Oil Company	Houston	J.D. Bozeman D. Jensen Chr. Behrens	Research Scientist VERITAS, Houston VERITAS, RD

DATE	COMPANY	LOCATION	MEETING PARTICIPANTS	
2.11.82	US Dept. of Interior Minerals Management Services	Washington		
			J.B. Gregory	Research Programme Manager
			S.E. Smith	Ass. Research Programme
			R. Insle	
			P. Bergrem	VERITAS, Houston
			Chr. Behrens	VERITAS, RD
3.11.82	Mineral Management Services Platform Verification Sect.	New Orleans	B. Martin	
			Chr. Behrens	VERITAS, RD
4.11.82	Bethlehem Steel	Beaumont	R. Scales	Chief Engineer
			D. Jensen	VERITAS, Houston
			Chr. Behrens	VERITAS, RD
4.11.82	*Nordrill Inc.	Beaumont	H.W	Vice-President Engineer
			D. Jensen	VERITAS, Houston
			Chr. Behrens	VERITAS, RD
5.11.82	*Global Marine Drill Co.	Houston	D.G. Schrieber	Operating Sys. Superv.
			K.E. Stansell	Manager Marine Operations
[illegible].11.82	*Zapata Off.Shore Comp.	Houston	A.A. Barrows	Sr. Structural Eng.
			D. Jensen	VERITAS, Houston
			Chr. Behrens	VERITAS; RD
5.11.82	*Sonat Offshore Drilling Inc.	Houston	A.M. Radway	Manager of Structural Systems
			D. Jensen	VERITAS, Houston
			Chr. Behrens	VERITAS, RD
8.11.82	*Friede & Goldman	N.Orleans	W.T. Bennet	Chief Offshore Design
			J. Alford	
			Sobhan Sengapta	
			Chr. Behrens	VERITAS, RD

* Company not partisipating in the Project.

APPENDIX B:

EXTRACT OF RULES AND REGULATIONS REGARDING FOUNDATION OF JACK-UPS

2.1.1.1 A B S (American Bureau of Shipping)
Section 3, Classification Loading and
Environmental Criteria.

3.13 Units Resting on the Sea Bed

Units which are to rest on the sea bed are to have sufficient positive downward gravity loadings on the support footings or mat to withstand the overturning moment of the combined environmental forces from any direction. Realistic variable loads are to be considered.

Section 4, Self-Elevating Drilling Units

4.5 Structural and Intact Stability
Considerations

4.5 Structural and Intact Stability Considerations

4.5.1 Legs

b ***Legs Without Mats*** Legs without mats, which may penetrate the sea bed, are to be considered pinned at least 3 m (10 ft) below the sea bed.

4.5.6 Preload Capacity
For units without bottom mats, all legs are to have the capability of being preloaded to the maximum applicable combined gravity plus overturning load. The factor of safety for combined loadings given in 3.11.2 may be used when considering structural aspects of the preload condition.

4.5.7 Sea Bed Conditions
The review of the unit is to be based upon the Owner's assumptions regarding the sea bed conditions.

2.1.1.2 B V (BUREAU VERITAS)

Chapter 2 Architecture - Scantlings - Foundations

Section 2.3 Load to be applied to the structure

2.34 Environmental Loads

6 - Reactions of the sea bottom on the structure

61 - The sea bottom reactions on structures of the platforms resting on the bottom counterbalance the gravity forces, the hydrostatic forces and the environmental forces. They thence have static as well as dynamic components. They are local forces.

Section 2.7 Stability - Water tightness - Buoyancy

2-73 "In situ" stability for platforms lying on the sea bed

1 - General

11 - Stability on the sea bed is a major condition for fixed and self-elevating platforms and it must influence platform design from the initial steps.

2 - Self-elevating platforms (jack-up)

21 - These platforms lie on the sea bed by means of retractable legs. These legs may have ends designed in the shape of piles in order to penetrate into the ground, or they may terminate on pads or mats resting on the bottom surface.

22 - Resistance to capsizing is to be reviewed all over again by using the following criterion: the stabilizing moment is to be at least equal to 1.3 times the capsizing moment under extreme environmental conditions, and to 1.5 times that moment under normal conditions (see 2-25 15).

Section 2.8 Foundation

2-83 Foundations design

13 - The self-elevating (jack-up) platforms are to be so designed that each pile may be tested at the extreme load provided by the general stability calculations, in the most unfavourable conditions (see 2-73.2). These tests are to be mandatorily considered as part of the safety investigation unless other means of checking are offered to the Society.

2.1.1.3 D n V: (DET NORSKE VERITAS)

CHAPTER III Special Design Principles

Ch. III Sec. 2 Self-Elevating Structures

Ch. III Sec. 2 Part C Hull, legs and Bottom Mat.

C 300 Bottom mat.

301 In the operating position account is to be taken of the loads transferred from the legs and the sea bed reaction, and the internal structure is to be designed to facilitate proper diffusion of the loads.

302 The effect of scouring on the support conditions is to be examined.

303 Any compartment which is not freely vented to the sea when the mat is resting on the sea bed is to be designed for a head of water to the design water level, taking into account the astronomical and storm tides.

304 Bottom impact forces due to rigid body motions of the unit while lowering the mat to bottom, are to be considered.

D. Overturning Stability on the Sea Bed.

D 100 General.

101 The safety against overturning is determined by the equation:

$$F = Ms/Mo$$

Ms = Stabilizing moment.
Mo = Overturning moment.

Both moments are calculated for rotation about the mostly stressed edge of the foundation.

102 Overturning stability is to be calculated for the most unfavourable direction and combination of environmental and functional loads. See also Ch. II, Sec. 4 A 102.

103 The safety against overturning is to be at least 1,5.

E. Overturning of Mobile Marine Structures

1. Proofs

1.1 In respect of mobile marine structures, e.g., jack-up platforms, that remain at a location for a brief period of time, that is, a few weeks or months, the proofs indicated in C and D for the foundation will, as a rule, be deleted, with the exception of those dealing with the safety against overturning and with the strength of the footage elements.

1.2 Safety against overturning: The overturning safety factors η_k, depending on the load case, shall not be less than the following values on the assumption of the most unfavourable weight distribution and tilting axis in the respective case (η_k = sum of the restoring moments divided by the sum of the overturning moments):

Load Case (GL)	2	3	4
η_k	1,5	1,3	1,1

It is presupposed in this connection that noticeable inclinations of the structure will not occur or will immediately be compensated by adequate measures.

1.3 As for the safety measures and checks necessary as compensation for the deleted sea bottom investigations and proofs, cf. F. 3.

1.4 The local strength of the footage elements at the touch-down point (feet) shall be calculated in conformity with the dimensions selected and with the soil pressures to be expected (cf. 2). The strength of the parts above ("legs") follows from Section 3. F.

2. Design of the footage elements

2.1 If ever possible - that is, if the solidity of the sea bottom so permits - the contact areas of jack-up platform legs shall be chosen, without any particular enlargement of their cross section, to equal the cross-sectional area of the leg. Depending on the sea bottom conditions to be expected, the leg pipe may remain open at its bottom end and solely be reinforced to absorb the local support forces (rocky ground), or else the leg shall be provided with a bottom that is to be adequately stiffened in order to absorb the bearing pressure (firm, sandy or cohesive soils).

2.2 Where soils of low bearing capacity are to be expected in connection with the utilization of a marine structure, specially enlarged footage elements (feet) may become necessary. An agreement with Germanischer Lloyd shall be reached in respect of their construction and linkage to the legs.

2.3 Permanently attached, large contact bodies (rafts) shall be designed in line with the principles laid down in Sections 2, 3, 4, and 5, as applicable.

2.1.1.5 L R (LLOYD's REGISTER OF SHIPPING)

Chapter B Structural Design

Section 3 Loading

Bottom forces

310 Reaction forces and moments resulting from all probable configurations of bottom contact and penetration are to be considered. The effects of differential settlement or loss of bearing area due to scouring, including the effect of relative horizontal movement of leg ends, is to be examined as relevant.

Section 6 Self Elevating Units

Bottom mat

602 Design of the bottom mat, if fitted, is to take account of leg loadings and sea bed reaction, and the internal structure is to be designed to facilitate proper diffusion of the loads. The effect of scouring on the support conditions is to be examined. In no circumstances are the scantlings to be less than those of a barge as derived from the *Rules for Steel Ships*. The scantlings of any compartments which are not freely vented to the sea when the mat is resting on the sea bed are to be based upon a head to the design water level, taking account of tide and storm surge.

2.1.1.6 N K K (NIPPON KAIJI KYOKAI)

PART P Mobile Offshore Units

CHAPTER 7 Self Elevating Units

7.2 Strength and Scantling

7.2.2 Legs

Legs are to be designed in accordance with the requirements in the following (1) to (7) in addition to the requirements in **7.2.1**. However, leg scantlings may be determined in accordance with a method of rational analysis or the results of the model test relating to the motion of the unit, to the satisfaction of the Society.

(1) Legs are to be either shell type or truss type, and as a rule, footings or bottom mats are to be fitted. Where footings or bottom mats are not fitted, proper consideration should be given to the leg penetration of the sea bed and the end fixity of the leg. In strength calulation of such leg, the leg is to be assumed as pin-supported at a position at least 3 *metres* below the sea bed.

(2) Legs in the field transit condition are to be designed in accordance with the following (a) and (b). The field transit move means the move which does not exceed a twelve-hour voyage between protected locations or locations where the unit may be safely elevated, however, during any portion of the move, the unit is not to be more than a six-hour voyage to a protected location or location where the unit may be safely elevated.

(a) The legs are to have sufficient strength for a bending moment obtained from the following formula.

$$M_1 + 1.2M_2 \quad (t-m)$$

where :

M_1 = Dynamic bending moment caused by a 6-*degree* single amplitude of roll or pitch at the natural period of the unit (*t-m*)

M_2 = Static bending moment due to gravity caused by a 6-*degree* leg angle of inclination (*t-m*)

(b) The legs are to be investigated for any proposed leg arrangement with respect to vertical position, and the approved positions are to be specified in the Operating Booklet. Such investigation should include strength and stability aspects.

(3) Legs in the ocean transit condition are to be designed in accordance with the following (a) to (c). In addition, the approved condition is to be included in the Operating Booklet.

(a) The legs are to be designed for acceleration and gravity moments resulting from the motions in the most severe anticipated environmental transit condition, together with corresponding wind moments.

(b) The legs are to have sufficient strength for a bending moment obtained from the following formula.

$$M_3 + 1.2M_4 \quad (t-m)$$

where :

M_3 = Dynamic bending moment caused by a 15-*degree* single amplitude of roll or pitch at a 10-*second* period (t-m).

M_4 = Static bending moment due to gravity caused by a 15-*degree* leg angle of inclination (t-m).

(c) For ocean transit condition, it may be necessary to reinforce or support the legs, or to remove sections of them.

(4) Legs are to have sufficient strength for the dynamic loads which may be encountered by their unsupported length just prior to touching bottom, and also for the shock of touching bottom while the unit is afloat and subject to wave motions.

(5) The maximum design motions, bottom condition and sea state while lowering legs are to be clearly indicated in the Operating Booklet.

(6) When computing leg stresses, while in the elevated position, the maximum overturning load on the unit, using the most adverse combination of applicable variable loadings together with the loadings as specified in Chapter 4, are to be considered. Forces and moments due to lateral frame deflections of the legs are to be taken into account.

(7) Leg scantlings are to be determined in accordance with a method of rational analysis, to the satisfaction of the Society.

7.2.4 Bottom Mats

1 The consturction of bottom mats is to be designed so that loads transmitted from the leg may be evenly distributed to each part of the mat.

2 The thickness of shell plating and scantlings of stiffeners of the bottom mat without opening to the sea are not to be less than those obtained

U.S.A

CODE OF FEDERAL REGULATION

30 CFR Mineral Resources

Part 250: Oil and Gas sulfer Operation in the Outer Continental Shelf.

rected to act upon the requests, applications, and notices submitted under the regulations in this part, and to require compliance with applicable laws, regulations, lease terms, and OCS Orders so that all operations are conducted in a manner which will protect the natural resources of the OCS. The Director may issue OCS Orders to implement the requirements of the regulations in this part. The Director may issue other orders, either written or oral, to govern lease operations. Oral orders shall be confirmed in writing as promptly as possible. The Director may issue other orders and field rules to govern the development and method of production of a pool, field, or area. Prior to the issuance of OCS Orders and other orders and field rules, the Director may consult with, and receive comments from, lessees, operators, and other interested parties. Before permitting operations on the leased area, the Director may require evidence that a lease is in good standing, that the lessee is authorized to conduct operations, and that an acceptable bond has been filed.

§ 250.11 **Functions.**

(a) The Director, in accordance with the regulations in this part, shall:

(1) Regulate all operations conducted under a lease or permit and shall issue and amend OCS Orders and other orders and field rules as may be necessary and proper in order to supervise operations and to prevent harm or damage to, or waste of, any natural resource (including any mineral deposits in areas leased or not leased), any life (including fish and other aquatic life), property, or the marine, coastal, or human environment.

(2) Require on all new and, whenever practicable, existing drilling and production operations (including the construction and operation of platforms and pipelines) the use of the best available and safest technologies which the Director determines to be economically feasible, wherever failure of equipment would have a significant effect on safety, health, or the environment, except where the Director determines that the incremental benefits are clearly insufficient to justify the incremental costs of utilizing such technologies.

(3) Schedule an onsite inspection, at least once a year, of each facility on the OCS which is subject to any environmental or safety regulations promulgated pursuant to the Act. The inspection shall include all environmental protection equipment and all safety equipment designed to prevent or ameliorate blowouts, fires, spillages, or other major accidents. A lessee shall, on request by the Director, furnish food, quarters, and transportation for Federal representatives to inspect its facilities. Upon request, the lessee will be reimbursed by the United States for the actual costs which it incurs as a result of its providing food, quarters, and transportation for a Federal representative's stay of more than 10 hours.

(4) Conduct periodic onsite inspections without advance notice to the operator of such facility to assure compliance with applicable regulations.

(5) Cooperate with and, when in the Director's judgment it is necessary, consult with or solicit advice from relevant Departments and Agencies of the Federal Government and affected States, executives of affected local governments, and other interested parties.

(6) Identify for those activities under the jurisdiction of the Director those States which are deemed to be affected States as defined in § 250.2(c) of this part.

(b) The Director may prescribe or approve, in writing or orally with subsequent written confirmation, departures from the requirements of OCS Orders and other orders and field rules issued pursuant to paragraph (a)(1) of this section, when such departures are necessary for the proper control of a well, the facilitation of the proper development of a lease, the conservation of natural resources, the protection of life (including fish and other aquatic life) property, or the marine, coastal or human environment.

[44 FR 61892, Oct. 26, 1979; 45 FR 20464, Mar. 28, 1980]

U.S. Dept. of the Interior

Geological Survey

Conservation Division

-Outer Continental Shelf Orders-

Govering oil and gas lease operation

REVISION A (of August 1980)

Effective 9/15/80 the DCM, Offshore Field Operations. When requested, this data shall include sediment and seabed data, e.g., seabed profiles, sediment consistency, allowable bearing and sliding loads, and nearby potential seabed hazards, i.e., sand waves, slumps, and mud slides.

2.1.4 Oceanographic, Meteorological, and Performance Data. Where such information is not readily available, lessees shall collect and report oceanographic, meteorological, and performance data during the period of operations as required by the Deputy Conservation Manager (DCM), Offshore Field Operations. Effective 9/15/80

2.2 Mobile Drilling Units. Applications for drilling from mobile drilling units shall include the following:

a. A listing of the maximum environmental and operational conditions used for the design.

b. A listing of the regional maximum environmental conditions, including wave, wind, current, storm surges, seismic motion, and of the unusual site-specific environmental conditions anticipated to be encountered at the drill site during the drilling operations. Effective 09/15/80 Former Item c (Deleted)

c. Current American Bureau of Shipping Classification, U.S. Coast Guard Certificate of Inspection, or other appropriate classifications, with operational limitations.

2.3 Fixed Drilling Platforms. Applications for installations of fixed drilling platforms or structures, including artificial islands, shall be submitted in accordance with OCS Order No. 8. Mobile Drilling Units which have their jacking equipment removed or have been otherwise immobilized will be considered fixed drilling platforms, and applications shall also be submitted in accordance with OCS Order No. 8.

3. Well Casing and Cementing.

3.1. General Requirements. All wells shall be cased and cemented in accordance with the requirements of 30 CFR 250.41(a)(1). The Application for Permit to Drill shall include the casing design safety factors for collapse, tension, and burst. Wells drilled in areas which are underlain by freshwater aquifers shall have casing programs which are designed to protect the freshwater zones. In cases where cement has filled the annular space back to the ocean floor, upon approval by the District Supervisor, the cement may be washed out or displaced to a depth not exceeding the depth of the structural casing shoe to

Initial edition (of January 1980)

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c. Sediment and seabed data, including seabed profiles, sediment consistency, allowable bearing and sliding loads, and nearby potential seabed hazards, i.e., sand waves, slumps, and mud slides.

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46 CFR: Shipping

Chapter I -Coast Guard Dept. of Transportation

Subchapter I-A Mobile Offshore Drilling Unit

Part 108 -Design and Enquipment

Subpart C -Stabiliy

§ 108.309 Stability on bottom.

Each bottom bearing unit must be designed so that, while supported on the sea bottom with footings or a mat, it continually exerts a downward force on each footing or the mat when subjected to the forces of wave and current and to wind blowing at the velocities described in § 108.311(b)(3).

§ 108.311 Calculation of wind heeling moment (Hm).

(a) The wind heeling moment (Hm) of a unit in a given normal operating condition or severe storm condition is the sum of the individual wind heeling moments (H) calculated for each of the exposed surfaces on the unit; i.e., $Hm=\Sigma H$.

(b) Each wind heeling moment (H) must be calculated using the equation $H=kv^2C_hC_sAh$, where—

(1) H=wind heeling moment for an exposed surface on the unit;

(2) k=0.0623 kilograms (kg-sec^2)/m^4 (0.00338 lb/(ft^2-knots^2));

(3) v=wind velocity of—

Section 3 Foundations and site investigations

Summary

This section gives guidance on foundation design for installations intended to be supported, either permanently or from time to time, on the sea-bed. It also covers the site investigation that should be made and the steps that should be taken to select the soil properties to be used in foundation design.

More often than not the type of foundation to be adopted is decided in the light of existing knowledge of the area and the subsequent site investigation is concentrated on aspects having specific relevance to that type of structure. Foundation design is therefore considered first but it should be clearly understood that design and investigation are interdependent and must be considered together.

3.1 Foundations

3.1.1 General

Most foundations for offshore installations can be classified as either:

(a) piled; or

(b) gravity, including mobile units supported by the sea-bed.

Piled foundations were, until recently, the traditional type, used to support steel-jacket installations. The piles usually take the form of open-ended steel tubes installed either through the legs of the installation or through guides attached to the jacket structure, at or about sea-bed level, and driven to depths below the sea-bed commensurate with the applied loads and the strength of the sea-bed material.

Gravity foundations rely on the weight of the installation to provide stability against the horizontal forces and overturning moments produced by waves, currents and wind and also berthing and other loads with horizontal components.

The sea-bed elements of a restrained buoyant installation may take the form either of slabs or blocks relying on gravity for stability, or of piles, screw piles, self-burying anchors or other devices that mobilise the weight of the sea-bed either by friction or by mechanical means.

3.1.2 Codes of Practice

British Standards Institution CP 2004 Foundations, revised 1972, relates to the foundations of land-based structures but many of the contained recommendations are equally applicable to offshore structures. Other codes have been published or are in a late stage of preparation, dealing specifically with offshore installations, but before applying the recommendations in any code their applicability in specific offshore circumstances should be critically examined.

3.1.3 Design

The performance of the structure and the sea-bed should be considered together.

Foundations should be designed so that under the most severe loading conditions the imposed load does not approach the ultimate bearing capacity, as defined in CP 2004, of the sea-bed and the underlying ground. Further, the allowable bearing pressure, CP 2004, should be selected to ensure that expected long-term settlement is acceptable and will cause no damage either to the installation or to risers, pipelines or other connections.

The factor of safety to be used depends upon the degree of accuracy that can be ascribed to the soil parameters actually used in design. (Many geophysicists and geophysical publications prefer the description 'operational soil parameters', but the term 'design soil parameters' is used here to avoid possible confusion with other meanings of the term common in the industry.) The design soil parameters used should be those most closely approximating to the actual site conditions and which are not necessarily the values measured in the laboratory or by *in-situ* tests: they may be very much lower (reference 1). It is recommended that the factor of safety against bearing capacity failure and against sliding be not less than 1.5, used in conjunction with the design soil parameters and the most severe loading conditions and not less than 2 under conditions likely to occur not more than once, on average, each month.

3.1.4 PILED FOUNDATIONS

Maximum compression, tensile and bending loads on each pile, resulting from all possible loading conditions, including lateral loading, should be considered. The piles should be designed to carry the axial, bending and shear stresses resulting from these loads (reference 2). Consideration should be given to the possibility that long-term cyclic loading, and particularly cyclic horizontal forces, may cause plastic deformations of the soil. If the load-bearing capacity of the pile is being checked during driving by means of a driving formula, due consideration should be given to the effects of pore pressure dissipation. In clays, dissipation of pore pressure after driving can result in a considerable increase of load-carrying capacity due to the increase in horizontal effective stress (reference 3). In some silts and sands, a reduction of pore pressure caused by dilation, as the pile sets up shear stresses in the soil, may temporarily increase the resistance to penetration. A return to the normal pore pressure values after driving may reduce this resistance, which should be checked by re-driving the piles. If the resistance proves to have been reduced, and is not attributable to lifting of the point as described below, consideration should be given to reducing the pile loading, eg by driving additional piles. With end-bearing piles there is a possibility that driving subsequent piles may raise adjacent piles. A check should be made of the levels of the pile heads and, if necessary, allowance made for the loss of end bearing. The load that can be supported by a group of piles may be less than the multiple of the single pile load-bearing capacity and the number of piles in the group. In sand, driving a group of piles may cause permanent consolidation, with a significant increase in the bearing capacity of the group, but in clays a decrease can usually be expected. Where, as is frequently the case, pile groups penetrate successive sand and clay layers, the net effect should be calculated as accurately as possible (reference 4). Lateral loading aspects present special problems (references 5 and 6). During pile-driving continuous records of the number of blows per unit of penetration should be maintained. These records provide a useful correlation between the pile-driving resistance and the soil strength profile.

3.1.5 GRAVITY FOUNDATIONS

The effective pressure between the finished foundation and the sea-bed should be positive over the whole area of contact under the most severe overturning conditions (reference 7). A gravity installation will normally apply loads over a substantial area of the sea-bed and the distribution of the total load should not be assumed to be uniform unless there are specific grounds for so doing. Actual load distribution under any given loading condition will depend upon soil characteristics and response down to depths at least comparable to the dimensions of the foundation. Foundation pressures and stress distribution will also be affected by surface irregularities in the sea-bed. An accurate topographical survey of the sea-bed is necessary and it is desirable, in addition, to make a visual inspection using manned or remotely controlled equipment.

Where the sea-bed is irregular the design should allow for such changes in foundation pressure and stress distribution as may result from the installation being located at any point within planned tolerance limits.

Reasonable estimates for skirt penetration forces can be derived from cone penetration test results and values can be estimated from the shear strength parameters for clays.

The detailed design of a gravity base may have to incorporate provision for grouting to be carried out during installation, both to control levels and to ensure predetermined load distribution.

3.1.6 SETTLEMENT

The problem of settlement is generally associated more with gravity than with piled foundations. The maximum overall settlement should not reduce the distance between the maximum wave crest and the underside of the platform below the clearance specified in 5.2.2. Differential settlements should not be greater than the structure can safely withstand and, to avoid damage to the risers, special provision may be required to allow for differential movement between the structure and the risers.

Settlement may be caused by consolidation and displacement of the soil and by the bedding-down effects of cyclic loading (references 8 and 9).

3.1.7 CYCLIC LOADING

In general, cyclic loading can be expected to increase settlements with a consequent improvement in soil strength although some sensitive soils may be adversely affected, particularly if the design produces large cyclic stress changes in the soil. Typical North Sea sea-bed sand grading is such that, if associated with low density, liquifaction could occur under cyclic loading. Most of the sand deposits so far examined have been found to be densely packed but the *in-situ* density of any deposit should invariably be checked during the course of an investigation.

Analysis of the behaviour of a gravity foundation under cyclic loading is a very complex matter requiring sophisticated analytical techniques and relevant soils data for both static and cyclic loading conditions. In these circumstances consideration should be given to the use of model tests such as centrifugal model tests. Both centrifugal model tests and analytical studies require that the soil sample properties approximate closely to the actual site conditions.

If a gravity structure rocks under cyclic loading a pumping action may become established, sea water being alternately drawn in and forced out below the base. This is a highly dangerous situation, fine material being carried out by the high-pressure flow with consequent loss of positive bearing pressure and progressive deterioration. A skirt penetrating the sea-bed will reduce this hazard.

3.1.8 SEA-BED MOVEMENT

Movement of materials on the sea-bed can be caused by currents and/or wave action and by seepage of gas emanating from below the sea-bed. If the site investigations indicate that there is a possibility of sea-bed movement due to any of these causes, then the effect on the foundations should be considered. Any changes in sea-bed currents which may result from the presence of the structure should also be taken into account. The effect of scour may be alleviated by the addition of a skirt or apron to the foundation, or by the placing of materials around the foundation after the structure has been installed. Proposed methods of alleviating scour and other sea-bed movement should be agreed with the certifying authority.

Special precautions must be taken when carrying out drilling operations below or adajacent to a gravity installation. Cases have occurred where significant quantities of sea-bed material have been displaced or removed either by 'hydraulic explosion' due to the drilling fluid being under excess pressure, or to caving below the tube, fine material being brought up by the circulating drilling fluid.

3.1.9 OPERATIONS MANUAL—FOUNDATION CHARACTERISTICS
For appropriate installations, the Operations Manual should contain relevant particulars of the characteristics of the foundations and, where relevant, their suitability for different sea-bed conditions, bottom penetration aspects and limiting values for scour and other sea-bed movements. Guidance should be given when action may be needed, and instructions on the action to be taken, all relative to the various modes of use, on station, and being placed on or removed from station.

3.2 Site investigation

Information on site conditions can, and should, be built up by combining data from all available sources. A revised edition of British Standards Institution CP 2001 Site Investigations is at present being circulated for comment and is readily available from the BSI. Reference may safely be made to appropriate recommendations in this draft.

3.2.1 BACKGROUND GEOLOGY
Information on the geology of the site may be obtained from adjacent sites, from general surveys such as those made by the Institute of Geological Sciences and from published literature. This information will help in planning the detailed site investigation and in providing an assurance that the findings of the site investigation are consistent with the geological conditions.

3.2.2 GEOPHYSICAL SURVEY
High resolution shallow penetration seismic methods may reveal strata thicknesses and variations in their thicknesses over the general area of the site. When those techniques are used they must be specifically designed to show up the soil conditions. Seismic surveys should be interpreted in the light of one or more boreholes in the survey area.

3.2.3 TOPOGRAPHICAL SURVEY
Knowledge of the configuration of the sea-bed is required to identify potentially unstable slopes, shapes which might affect scour behaviour, etc. At the actual site for the structure a more detailed survey is required to establish surface irregularities, average slopes across the site, presence of rocks or man-made hazards or obstructions. A submersible or other means such as side-scan sonar will be valuable in observing the condition of the sea-bed. The survey should show any seepage of gas from the sea-bed and the presence of pock marks or small craters. Measurements should be made of current flow near the sea-bed.

3.2.4 SUBSOIL INVESTIGATION
This involves the use of *in-situ* testing, boring and sampling and should be carried out at the site finally chosen for the structure. The actual extent and complexity of the investigation depends very much on the consistency and uniformity of the foundation deposits and it should be tailored to the design methods used. Unnecessarily conservative designs may be avoided by making the investigation sufficiently extensive and thorough. The objective of the investigation is properly to describe the soil profile, including soil structure, fabric and discontinuities as well as to establish values for the strength and deformation parameters for the soil.

3.2.5 IN-SITU TESTS
The results from all forms of *in-situ* tests require careful interpretation. Values obtained will be influenced by the type of apparatus, the way it is used and the

type of soil being tested. Rates of strain, which may be difficult to control, can be very important.

In every case the operating principles of the equipment, their potentials and their disadvantages, should be evaluated and taken into account in assessing the accuracy and validity of the results obtained.

Cone penetration tests should be interpreted in the light of the findings by Marsland (1977) (reference 1).

3.2.6 Borings

Boreholes should be made at the site for each structure so that samples of the various strata may be inspected to confirm the soil types. The number and depth of boreholes will depend on the variation of soil type, size and design of foundation and the amount of reliable information already obtained from other investigations made on or near the site by other methods. Guidance is given in the new draft of CP 2001 Site Investigations.

It is important to know accurately the positions of all boreholes.

3.2.7 Sampling

A sufficient number of samples should be taken to ensure that a reasonably continuous profile, composite if necessary, can be obtained from the subsoils encountered. Samples should be as undisturbed as possible. The soil will be disturbed if boring allows excessive stress relief; this can be reduced by the use of drilling mud of sufficient density. The soil will also be disturbed by a sampling tube with an incorrect shape of cutting shoe and/or an excessively large area ratio. The existing stress and stress ratio in the soil will be altered by sampling and this may also seriously affect soil properties measured by subsequent tests made on samples.

On recovery, samples of soil should be either (a) extruded from the sampling tube for examination and/or tested immediately, or (b) suitably stored and transported to a laboratory on land for detailed testing.

The record of the soil profile can be enhanced by a series of colour photographs of undisturbed samples that have been carefully cleaned and split in half without smearing the surface to reveal the colour and texture of the soil strata. Samples to be stored should be kept in their sampling tubes, preferably under an applied stress to minimise swelling and change of structure. Spring-loaded pistons can be fitted to either end of the sample tube and the springs compressed to produce an axial stress in the sample equal to the overburden at the depth from which it was taken.

3.2.8 Soil description

Each sample that is extruded from its sampling tube and split should be described to record its consistency and structure so that an interpretation of its geological history may be made. Special attention should be paid to structural details such as silt and sand partings, fissures, etc., which, though easily overlooked, could play an important role in the behaviour of the soil mass.

Samples that are tested should also be split and carefully examined after the test is complete to add to the overall description of the soil profile.

3.2.9 Laboratory testing

The type of laboratory tests required will depend on the type of foundation and the design method as well as the soil type. In general sufficient classification tests should be carried out to establish the soil type in soil mechanics terms. Deformation and ultimate strength tests should be made to establish soil parameters with the degree of accuracy required by the design. These tests are usually made with a triaxial apparatus which can be used to consolidate the sample under a wide range of stress ratios and cause the stresses to follow a wide variety of paths and measure sample deformation, volume change and/or pore pressure alterations. Oedometers, shear apparatus and other special laboratory equipment may also be used to measure soil properties, depending on the design requirements.

Special tests to measure the deformation properties of soil samples under the action of cyclic loading have been described by Brown, Lashine and Hyde (1975), Moussa (1975) and the Norwegian Geotechnical Institute (1975) (references 10–12). These tests should be made where there is any doubt about the possible behaviour of the soil.

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NORWAY

Norwegian Maritime Directorate 1975

-Subchapter by norwegian Petroleum Directorate --Chapter III Drilling

§ 15 Plassering av boreplattform.

1. Før en oppjekkbar boreplattform plasseres i posisjon for å påbegynne boring, skal havbunnen undersøkes og de nødvendige sikkerhetsforanstaltninger for øvrig tas for at boreplattformen holdes på plass under boringen. Slike undersøkelser skal foretas selv om bunnundersøkelser har vært foretatt i forbindelse med nærliggende boring. Resultatet av disse undersøkelser skal straks sendes til Oljedirektoratet.
Rettighetshaver skal under boring forsikre seg om at forholdene på havbunnen ved boreplattformens ben ikke endres vesentlig.

Ved plassering av boreplattform som holdes i boreposisjon ved ankring skal hvert ankerfeste strekktestes slik at boreplattformen holdes i posisjon under de maksimale miljøpakjenninger som kan forventes i det aktuelle område for den periode boring skal pågå.

2. Boreplattformen skal plasseres i forsvarlig avstand fra andre installasjoner for petroleumsutvinning samt fyr, sjømerker, telegraf- og telefonkabler, rørledninger etc.
I områder hvor slike kabler, rørledninger eller andre undervannsinstallasjoner finnes, kan ankring, jekking og boring ikke påbegynnes for rettighetshaver har foretatt undersøkelse av havbunnen som nøyaktig lokaliserer den undersjøiske kabel, rørledning eller annen installasjon. Selv om samtykke til plassering er gitt, jfr. § 14, kreves spesiell tillatelse fra Oljedirektoratet til ankring, jekking og boring nærmere enn 1 nautisk mil fra slik kabel, rørledning eller annen installasjon, og nærmere enn 2 nautiske mil fra telefon- eller telegrafforsterker. Skade som voldes på kabler, rørledninger eller installasjoner m. m. som er nevnt i de foregående ledd, skal uansett skyld erstattes av rettighetshaveren. Denne bestemmelse begrenser ikke rettighetshaverens eventuelle adgang til regress overfor den entreprenør hvis virksomhet har bevirket skaden.

3. Boreplattformen skal plasseres slik at det gis best mulig beskyttelse for den virksomhet som foregår på plattformen, herunder fortøyning av fartøy, landing og avgang av helikopter.

§ 16 Klargjøring av boreplattform for bore-

Section 15 Precautions When Positioning The Drilling Platform On Location

1. Prior to the placing of a self elevating drilling platform in drilling position, the seabed shall be examined and other necessary safety precautions be taken to ensure that the platform will remain in place during operations. Such seabed inspection shall be carried out even if bottom surveys have taken place in connection with nearby drilling operations. The result of these surveys shall without delay be submitted to the Norwegian Petroleum Directorate.
During drilling, the licensee shall make sure that the conditions of the seabed at the places where the legs of the drilling platform are situated are not substantially changed. When placing an anchored drilling platform in position for drilling, a pretension load test shall be carried out so as to ensure that the drilling platform is kept in position during the maximum environmental loads expected in that particular area during the drilling period.

2. The drilling platform must be placed at a safe distance from other installations for exploitation of petroleum as well as from lighthouses, seabuoys, telegraph and telephone cables, pipelines etc.
In areas where cables, pipelines and other underwater installations exist, anchoring and jacking up of platforms as well as drilling cannot commence until the licensee has undertaken a thorough bottom survey which has located the exact position of the cable, pipeline or other underwater installation.
Even if permission for placing a platform is granted, cf Section 14, special permission from the Norwegian Petroleum Directorate is required before anchoring, jacking up or drilling may take place closer than one nautical mile from cables, pipelines or other installations and less than two nautical miles from telephone or telegraph amplifiers. Damage caused to cables, pipelines or installations etc. as mentioned in the above paragraphs, shall, disregarding who is to blame, be compensated by the licensee. This provision does not limit the licensee's possible right to claim compensation from the contractor whose operations have caused the damage.

3. The drilling platform shall be positioned so as to give maximum protection to the operation, including mooring of vessels and landing and take-off of helicopters.

Section 16 Precautions When Preparing The

INTERNATIONAL

IMCO = Inter-Governmental Maritime Consultative Organization. (Now IMO)

Code for the construction and equipment of mobile offshore drilling units 1980.

2.5 Special considerations for self-elevating units

2.5.1

.1 The hull strength should be evaluated in the elevated position for the specified environmental conditions with maximum gravity loads aboard and supported by all legs. The distribution of these loads in the hull structure should be determined by a method of rational analysis. Scantlings should be calculated based on this analysis, but should not be less than those required for other modes of operation.

.2 The unit should be designed to enable the hull to clear the highest design wave including the combined effects of astronomical and storm tides. The minimum clearance may be the lesser of either 1.2 metres or 10 per cent of the combined storm tide, astronomical tide and height of the design wave above the mean low water level.

2.5.2

.1 Legs should be designed to withstand the dynamic loads which may be encountered by their unsupported length while being lowered to the bottom, and also to withstand the shock of bottom contact due to wave action on the hull. The maximum design motions, sea state and bottom conditions for operations to raise or lower the hull should be clearly stated in the Operating Manual.

.2 When evaluating leg stresses with the unit in the elevated position, the maximum overturning moment on the unit due to the most adverse combination of applicable environmental and gravity loadings should be considered.

.3 Legs should be designed for the most severe environmental transit conditions anticipated including wind moments, gravity moments and accelerations resulting from unit motions. The Administration should be provided with calculations, an analysis based on model tests or a combination of both. Acceptable transit conditions should be included in the Operating Manual. For some transit conditions, it may be necessary to reinforce or support the legs, or to remove sections to ensure their structural integrity.

2.5.3 Structural members which transmit loads between the legs and the hull should be designed for the maximum loads transmitted and arranged to diffuse the loads into the hull structure.

2.5.4

.1 When a mat is utilized to transmit the bottom bearing loads, attention should be given to the attachment of the legs so that the loads are diffused into the mat.

.2 Where tanks in the mat are not open to the sea, the scantlings should be based on a design head using the maximum water depth and tidal effects.

.3 Mats should be designed to withstand the loads encountered during lowering including the shock of bottom contact due to wave action on the hull.

.4 The effect of possible scouring action (loss of bottom support) should be considered. The effect of skirt plates, where provided, should be given special consideration.

2.5.5 Except for those units utilizing a bottom mat, the capability should be provided to pre-load each leg to the maximum applicable combined load after initial positioning at a site. The pre-loading procedures should be included in the Operating Manual.

2.5.6 Deckhouses located near the side shell of a unit may be required to have scantlings similar to those of an unprotected house front. Other deckhouses should have scantlings suitable for their size, function and location.

2.6 Special considerations for column stabilized units

2.6.1

.1 Unless deck structures are designed for wave impact, a clearance acceptable to the Administration should be maintained between passing wave crests and the deck structure. The

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TECHNICAL REPORT

Date	December 31, 1982
VERITAS Report No.	83 - 0904
Subject Group	G3, P3
Department	23
Project No.	~~23-31-02~~
Title of Report	FOUNDATION STABILITY OF JACK-UP PLATFORMS PP2: ANALYSIS METHODS FINAL REPORT, PP2
Approved by	O. Furnes Chief Research Engineer
Client/Sponsor of project	Several Companies, see Foreword
Client/Sponsor ref.	
Work carried out by	Tore J. Kvalstad and Lars A. Kristiansen
Reporters sign.	Tore J. Kvalstad Lars-A. Kristiansen

Summary

This Technical report presents the theoretical background for the analysis methods to be adopted in a parametric study of the factors influencing the foundation stability of Jack-Up platforms. (PP3 of the project)
Analysis methods for penetration and pull-out resistance, stability (lateral, vertical, overturning) under gravity and environmental loads as well as methods for soil-structure interaction are reviewed and commented on in general and the specific methods selected for use in the parametric study are described in detail.
A brief summary of the selected simplified methods for calculation of environmental loads on independent leg and matsupported Jack-Ups are presented as well.

4 Indexing terms

- Jack-up platforms
- Foundation analysis
- Soil-structure interaction
- Foundation capacity

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PREFACE

This report presents the work carried out within part project 2 (PP2) of the Joint Industry Project: 'Foundation Stability of Jack-Up Platforms'. The project is at the time of writing sponsored by the following 14 companies:

Amoco Norway Oil Company
Arco Norway Inc.
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The work presented herein is the result of a study where the main work was carried out during May 1982 to February 1983, and covers a review of available analysis and selection of specific methods for calculation of penetration depth and pull-out resistance, stability under gravity and environmental loads and soil structure interaction effects.

The effects and consolidation and repeated loading on the soil strength and simplified methods for evaluation of environmental loads have been addressed as well.

EXECUTIVE SUMMARY

GENERAL

The purpose of part project 2 (PP2) was to prepare the analysis methods and software required for the broad parametric study to be carried out under PP3. Where necessary improvement of existing or development of new analysis programs was to be performed in order to allow a realistic simulation of soil-structure interaction during installation (penetration of spud-cans and mats) and during severe storm conditions.

Realistic simulation of soil structure interaction required realistic simulation of loads on the structure from wind, waves and current. Further the effect of wave induced pressure on the sea bed was to be evaluated. The horizontal forces and overturning moments act on the individual legs with phase differences and the wave pressure forces on mats and on the sea bed soil are again out of phase with the loading on the legs. The load history during a single wave is thus not easy to overview regarding the definition of a critical load combination for evaluation of foundation stability.

To overcome this difficulty it was decided to develop a quasi-static time step-analysis procedure with load calculation at discrete intervals combined with a simple plain frame structural model which allow the foundation stability to be checked for all possible load combinations in order to define the most critical condition.

The result of this effort were two computer programs **SAM** for Stability Analysis of Mats and **SAIL** for Stability Analysis of Independent Legs. These two programs represents the primary analysis tools for the parametric study in PP3, but separate specialized geotechnical programs have been made available for the investigation of effects like cyclic strength degradation, strength increase of the soil under the footings due to consolidation, etc..

Description of possible failure and interaction mechanisms and methods for calculation of the respective foundation capacity and response is presented in this report. The main items treated in this report are the following:

- Methods for the prediction of penetration/bearing capacity of spud-cans and mats including the punch-through problem

- Methods for evaluation of set-up (consolidation) effects
- Methods for prediction of the bearing capacity and sliding resistance of spud-cans and mats under combined wind-wave-current loading
- Methods for evaluation of sea floor instability due to wave pressure action on the sea bed
- Methods for evaluation of cyclic load effects on the bearing capacity of mats and spud-cans
- Methods for prediction of load-displacement relationship of spud-cans under combined vertical, horizontal and moment loading
- Methods for the analysis of wave-structure-soil interaction

PROGRAM SAIL

The computer program **SAIL** was developed for analysis of foundation behaviour and wave-soil-structure interaction of independent leg type jack-up rigs. The wave/current load calculation is based on linear wave-theory and Morison's equation for wave loads on slender members. These loads plus a static wind force are applied to a plain frame model of a jack-up structure connected to the soil with non-linear springs.

The program allows simulation of the penetration/preload process and provides the user with information about required preload and overturning stability as a function of penetration depth before a time step analysis of the effect of a single wave on the foundation stability is started. Information about load distribution between the legs, degree of fixity of the footing as well as factors of safety against sliding and bearing failure of each leg is presented.

PROGRAM SAM

The computer program **SAM** was developed to allow evaluation of the foundation stability of mat supported jack-up rigs under combined wave-structure-soil interaction. The same wave load approach as in SAIL has been implemented. As the legs of a mat supported rig are connected ridgidly to the mat, the structure is represented by a rigid frame, i.e. no structural analysis is included. In addition to the wave/current forces on the legs the wave load on the mat has been included in the analysis together with wave pressure loading on the sea bed.

The A-shape typical for most mat designs complicates the stablity analysis. Dependent on the relative magnitude of slot and hole compared with the total mat area different types of failure may develop. The program allows analysis of 4 different failure mechanisms;

-sliding of mat along shallow surface under skirt tips
-local bearing capacity failure under most loaded beam of A-mat
-shallow to deepseated overall failure
-deepseated overturning failure of the mat and the underlying soil mass

in a time-step procedure and provides the user with information about the safety against foundation failure for all four mechanisms as a function of time through a single wave. A large range of possible failure surfaces are investigated for each time step. Only clayey soils are treated as the bearing capacity of sandy sea beds is assumed to be sufficient to prevent bearing capacity types of failure.

CONSOLIDATION (SET-UP) ANALYSIS

A stepwise method for evaluation of a possible improvement of the bearing and sliding capacity of jack-up foundations has been described. The method is based on a combination of two finite element programs. **AXIPLN** an axisymmetric or plain strain nonlinear finite element program for analysis of stresses and displacements and **OCEAN2** a finite element program allowing axisymmetric or plain strain, non-linear consolidation analysis.

The method should allow fair estimates of increase in bearing capacity as a function of time after installation and consolidation properties of the soil.

CYCLIC LOADING EFFECTS

Methods for evaluation of cyclic loading effects have been presented in the report. The analysis will be based on the same finite element programs as described above combined with material models describing the degradation of soil strength as a function of cyclic stress levels in the soil mass and the number of load cycles.

1. INTRODUCTION

The analysis of the foundation stability of a jack-up platform during installation (i.e. jacking up and preloading), during operation and during extreme loading conditions as well as evaluation of pull-out resistance will be addressed in the following. The two main types of jack-up foundations, mats and independent legs, requires generally separate treatment although the basic geotechnical theory will apply to both types.

The presented theories and analysis methods must be seen in connection with the proposed overall analysis scheme presented to the second steering committee meeting July 1, 1982. (See Appendix A) and the frame for the parametric study presented at the same meeting (See Appendix B). These two proposals were accepted with a few comments to the value of some of the parameters given in Appendix B.

It has been the aim of PP2 to present a complete analysis package which will enable us to carry out the parametric study in an efficient and rational way (i.e. easy manipulation of input parameters) without too many special effects included. On the other hand the analysis package was also intended to be of help to all participants in their daily work. The work has resulted in two computer programs:

SAIL: Stability Analysis of Independent Legs

SAM: Stability Analysis of Mats

The programs are based on the analysis methods selected for use in the parametric study and covers basically the evaluation of penetration depth due to weight and eventual preloading and evaluates as well the required minimum preload to exceed the maximum combination of gravity and overturning forces on the most stressed foundation of independent leg type jack-ups.

Further the combined effect of environmental and gravity loading on the foundation(s) and for independent leg type platforms also the interaction between the different legs have been included.

Three different soil profiles have been specifically modelled into the computer programs. These soil profiles correspond to the selected profiles shown in Appendix B.

The programs contain also subroutines which allows a simplified evaluation of wave loads on legs and mats. A further description of these methods is given in Chapter 5.

2. DESCRIPTION OF ANALYSIS METHODS FOR EVALUATION OF PENETRATION AND PULL-OUT RESISTANCE

2.1 GENERAL

The prediction of penetration depth of jack-up footings has been addressed by several authors during the last 15 years. Gemeinhardt and Focht (1970), Endley & al. (1981) and Casbarian (1982) treat specifically the prediction of penetration depth for individual leg footings of spud-can shape while Hirst & al. (1976) and Young & al. (1981) present the same problem for mats. Basically all authors conclude that bearing capacity theory with slight modifications can be used to give reasonable estimates of penetration depth for individual leg footings as well as mats.

A jack-up normally moves several times a year from one location to another. Three to ten moves within twelve months is considered to be representative for most of the platforms working as mobile units for exploration and work over operations. A few jack-ups have been used for temporary production and as accommodation platforms and may in these cases stay on one location for a period longer than one year.

Within one to twelve months or more, and even during the preloading period, the soil beneath the platform will experience consolidation settlements under the weight of the platform. These consolidation settlements will tend to increase the shear strength and stiffness of the soil and may have a beneficial effect on the foundation stability. However, when the platform is to be moved the increased soil strength may lead to a considerable increase in pull-out resistance. Efficient 'jetting' or 'suction release' systems will reduce the pull-out problem to a considerable degree.

In this Chapter 2 a general review of available theories and analysis methods that can be used for prediction of penetration resistance and pull-out resistance as well as methods for evaluation of consolidation effects on the strength and stiffness of the soil will be given.

2.2 BEARING CAPACITY METHODS FOR UNIFORM AND LAYERED SOILS

2.2.1 Description of the failure mechanisms

When a jack-up footing is lowered to the sea bed and the weight of legs and hull is gradually transferred to the soil a bearing pressure will develop. The ultimate bearing pressure of the soil depends on the strength parameters of the soil and the shape and dimensions of the footing and is called the bearing capacity of the soil. If the applied bearing pressure exceeds the bearing capacity, the footing will start to penetrate into the soil.

With increasing penetration the local uneveness of the soil will be equalled out and gradually the total footing area comes into contact with the soil. If the bearing capacity still is unsufficient the footing will start to penetrate into the sea bed.

As the shear strength of the soil normally increases with depth below mudline and the weight of the soil adjacent to the footing will lead to an increase in bearing capacity at least down to depth below mudline of say 3 to 5 footing diameters, the bearing capacity will normally increase considerably with increasing penetration depth.

Further the side area of the footing will be subject to side friction, and sooner or later the bearing capacity of the soil will be equal to the applied bearing pressure.

The above described failure mechanism involves a continous squeezing of soil from underneath the footing laterally and upwards around the footing.

For mats the bearing pressure is normally so low that only a few feet penetration of the mat is sufficient to establish equilibrium even in very soft recent delta deposits. Young & al. (1981) presents a case study where the shape and extension of the soil mound forming adjacent to the side of the mat under increased vertical load was measured.

Independent leg footings exerts bearing pressures that are several times higher than for mats. In soft clays penetration of more than 40 m have been reported in extreme cases, while for stiff clays and sandy soils the independent leg may stop after a few meter penetration and the large diameter spud-cans may not even get into full contact.

Deeply penetrating spud-cans leaves an open hole as the footing travels deeper into the soil. The stability of the walls of the hole may represent a critical factor. Sudden caving in of the walls will be equivalent to a sudden increase in load on the footing and may lead to a unexpected additional penetration of the footing.

Another failure mode to be considered is punch-through of stiff crusts either at the top or embedded in softer layers of clay. Stiff crusts may have generated as a results of water level fluctuations with desiccation of the soil surface followed by further sedimentation and subsidence of the area throughout the geological history. Cementation of calcareous sands represents a similar possibility for punch-through failures.

2.2.2 Review of Bearing Capacity Theories

The bearing capacity formula was in its original form first proposed by Terzaghi (1943):

$$q_u = 0.5 \cdot B \cdot \gamma' \cdot N_\gamma + q \cdot N_q + c \cdot N_c \qquad (2.1)$$

where q_u = unit bearing capacity

γ' = unit submerged weight of soil

B = width of footing

D = embedment depth of footing

c' = cohesion of material or undrained shear strength for friction angle $\varphi = 0$

N_γ, N_q, N_c = bearing capacity factors depending on the friction angle of the material.

This expression has through the years been modified and revised by several investigators theoretically and through large and small scale field and model tests. Skempton's (1951) formula for cohesive soils has been used extensively and was recommended by Gemeinhardt and Focht (1970) for prediction of jack-up footings together with a recommendation of using the average shear strength within a one-diameter zone below the bottom of the cylindrical portion of conical spud-cans.

Skempton's expression for the unit bearing capacity of rectangular footings on clay is as follows:

$$q_u = 5.14 \cdot s_u \cdot (1 + 0.2 \cdot \frac{D}{B}) \cdot (1 + 0.2 \cdot \frac{B}{L}) + \gamma \cdot D .... for \frac{D}{B} < 2.5 \qquad (2.2)$$

where q_u = unit bearing capacity of footing on clay
s_u = undrained shear strength of clay
B = width of footing
L = length of footing
D = depth of footing embedment
γ' = unit submerged weight of soil

Brinch Hansen (1970) developed a similar expression for clays:

$$q_u = 5.14 \cdot s_u \cdot (1 + s_{ca} + d_{ca} - i_{ca}) + \gamma' \cdot D \tag{2.3}$$

where s_{ca} = shape factor = $0.2 \cdot (1 - 2 \cdot i_{ca}) \cdot \frac{B}{L}$
d_{ca} = depth factor = $0.3 \cdot arctan \cdot \frac{D}{B}$
approaching 0.47 for
large D/B
i_{ca} = inclination factor

Comparing the two expressions gives nearly coinciding values.

Skemptons and Brinch Hansens expressions were derived for uniform shear strength. A typical feature of soils is that the undrained shear strength increases with depth. The effect of a linearly increasing undrained shear strength was investigated by Davis and Booker (1973). They derived an expression of the following form for a strip footing:

$$q_u = \frac{Q}{B} = F \cdot (5.14 \cdot s_{uo} + \frac{k \cdot B}{4}) \tag{2.4}$$

where F = correction factor dependent on smoothness of footing and the ratio $\frac{k \cdot B}{s_{uo}}$
s_{uo} = undrained shear strength at mudline
k = shear strength gradient
B = width of footing

The expression can be modified for circular or square footings by exchanging the value of 5.14 with 6.2. Further the overburden effect for footings at depth may be included by adding $\gamma' \cdot D$. Figure 2.1 shows the correction factor F plotted vs the ratio of $\frac{k \cdot B}{s_{uo}}$.

Endley & al. (1981) and Young & al. (1981) recommend the use of Davis and Booker's expression if the shear strength increase with depth is high in order to avoid unconservative results.

The effect of overburden pressure will only be present if the hole of a deeply penetrating footing stays open. If the hole caves in the overburden term should be replaced by a corresponding buoyancy term. Casbarian (1982) expresses this term as

$$F_B = \gamma' \cdot \frac{V}{A} \tag{2.5}$$

where F_B = buoyance force
γ' = unit submerged weight of soil
V = volume of penetrating footing
A = bearing area of footing

Casbarian further propose to introduce depth effects in Davis and Booker's expression by introducing Skempton's depth factor.

It is the authors opinion that this may lead to overestimation of the capacity as Skemptons depth factor was derived for uniform shear strength and not for linearly increasing shear strength. However with increasing depth below mudline the error will be minor.

The presence of layered soil complicates the picture considerably. The critical case for jack-up footings is however, the case with a stiff layer overlaying a soft clay which may lead to *punch-through.*

Meyerhof (1974) and Meyerhof & Hanna (1978) presented results of theoretical studies and model test for layered soil with a stiff layer overlaying a soft layer. Further Jacobsen (1977) presented a simplified analysis method based on a fictive foundation placed on the top of the soft layer however, with a correction factor making the load distribution angle dependent on a relative strength of the sand and the clay layers. See Figures 2.2 and 2.3 where the *punch-through* analysis is shown for a general case with stiff clay overlaying soft clay.

Meyerhof and Hanna presents results also for cases with a soft layer overlaying a stiff layer. The failure mode will in this case be a sort of squeezing of the soft layer until the bearing capacity of the stiff layer is approached.

2.2.3 Selection of Analysis Method for Use in the Parametric Study

Based on an evaluation of the different methods presented above Brinch Hansen's method was selected for implementation in the computer programs for evaluation of penetration depth of mats and spud-cans.

Brinch Hansen's method is developed to cover sand (i.e. drained conditions) and clays and includes shape factors, depth factors, and load inclination factors. The method is recommended for evaluation of bearing capacity of gravity platform foundations and is included in *Appendix F, Foundation, Veritas Rules for Fixed Structures (1977)*. A copy of Appendix F1 of VERITAS' rules has been included as Appendix C of this report.

The selection of a representative shear strength in cases with increasing strength with depth is the main problem with this choice. A comparison with Davis and Booker's method indicates that the shear strength at a depth of 0.2 to 0.5 times the width of the footing below the footing base gives reasonable correspondance between the methods.

The stability of the hole generated by a deeply penetrating spud-can may be a critical factor. The problem has been approached by Britto and Kusakabe (1982) and will be used for evaluation of the critical penetration depth where caving in or backfilling of the hole will take place.

The remoulding effect of the penetrating footing on the shear strength of the soil in the wall of the hole is difficult to take into account but may have a significant influence on the stability of the hole.

Analysis of punch-through of stiff top layers is included in program SAIL which treats individual leg footings. This failure mode is considered to be critical and a real problem for several locations both in the Gulf of Mexico and in areas in the Middle East.

The method selected for use was the *fictive footing method* outlined in Figure 2.2 and 2.3. A critical parameter in this method is the selection of the stress distribution angle α. Model tests carried out by Jacobsen (1977) show that a stress distribution angle of 26.5^o from the vertical axis (i.e. an inclination if 1 : 2 (horizontal to vertical)) might overestimate the capacity in cases where the difference in strength between the top layer and the underlying soft clay is small. However, for very soft clays, where punch-through is a critical failure mode the value of 26.5^o was found to give results close to the values mesured in the tests.

Analysis of the effect of a soft layer over a hard layer is investigated as well and included in program SAIL in an approximate way based on a simplified squeezing expression developed by the authors and derived in more detail in Appendix D. See also Figure 2.4. To summarize, the following steps are followed in program SAIL and partly in program SAM for evaluation of penetration depth:

1) Define the soil profile that is to be considered.
2) Evaluate bearing capacity of layer in which the tip of the footing is located, starting at mudline.
3) Check punch-through or squeezing capacity dependent on layering.
4) Increase penetration depth and repeat steps (2 to 4) until bearing capacity exceeds vertical load (preload) to be applied.

2.3 METHODS FOR EVALUATION OF CONSOLIDATION (SET-UP) EFFECTS ON SOIL STRENGTH

2.3.1 General Description of Process

When a jack-up platform is installed the platform weight will be transferred to the sea bed soil. Under and around the platform footing(s) a stress field will be generated which will lead to an immediate generation of excess pore pressure in the soil mass. As these pore pressures dissipate the footing(s) will settle. This is the so called primary consolidation settlement.

The dissipation of excess pore pressures will lead to a gradual increase in effective stresses in the soil, i.e. a better contact between the individual mineral particles will be achieved, and thus a gradual increase in soil strength will necessarily follow.

The strength increase will be of special importance for soft clayey soils where individual footings experience large penetrations during installation and where the sliding stability of mats may be critical. The speed with which this strength increase will take place will be heavily dependent on the consolidation properties of the soil and the drainage conditions.

2.3.2 Methods for Evaluation of Strength Increase

The basic approach for investigation of the possible increase in strength of the soil and thus in the safety against failure is described in the following scheme.

a) Compute initial stress conditions and strength distribution prior to installation.

b) Evaluate increase in total stress field around footing due to platform weights and evaluate bearing capacity prior to consolidation.

c) Evaluate corresponding pore pressure increase.

d) Evaluate dissipation process as a function of time.

e) Assume material law for dependency of undrained strength on effective consolidation stresses and compute increase in soil strength.

f) Evaluate bearing capacity and pull-out resistance at different times during consolidation process.

Point a) can be carried out easily provided a reasonable soil investigation has been carried out. Although the lateral pressure at rest as well as the soil strength may vary within certain limits the initial stress and strength can be considered as rather well defined.

For this study some basic assumptions regarding the relationship between strength and stress will have to be made in order to investigate the relative effect of set-up on pull-out resistance and safety against foundations failure.

Assuming that the coefficient of lateral earth pressure at rest K_0 can be expressed as

$$K_0 = 1 - \sin\varphi \tag{2.6}$$

where φ = friction angle of the soil

the initial stress state can now be expressed as

$$\sigma_1' = \sigma_{z,i}' = \gamma' z \tag{2.7}$$

$$\sigma_3' = \sigma_{x,i}' = K_0 \cdot \gamma' \cdot z \tag{2.8}$$

where $\sigma_{z,i}'$ = vertical effective initial stress
$\sigma_{x,i}'$ = horizontal effective initial stress

The evaluation of the increase in total stress (i.e. point b) is certainly more complicated. Jack-up footings (spud-cans or mats) on soft clay are generally causing large plastic displacements in the soil before equilibrium between platform weight and soil reaction stresses can be achieved.

By combining bearing capacity theory, which allows evaluation of penetration depth, as outlined in Chapter 2.1, with finite element analysis for evaluation of the stress field in the soil mass a relatively good estimate of the increase in total stresses could be achieved.

Admittedly the rather complicated geometry of an A-shaped mat will have to be simplified, but a plane strain analysis of a section crossing the sides of the mat structure is believed to be relatively representative. For a spud-can an axisymmetric approach is considered satisfactorily.

Point c) requires an assumption regarding material behaviour with respect to pore pressure changes in the total stress field.

Two different effects will have to be considered. The first is the pore pressure change caused by a change in the average normal stress $\Delta\sigma_o = \frac{1}{3}(\Delta\sigma_1 + \Delta\sigma_2 + \Delta\sigma_3)$. It is generally accepted that for a saturated material this will cause a change in pore pressure equal to the change in the mean normal stress.

The second effect is the pore pressure change caused by deviatoric stresses or shear stresses. This effect is highly dependent on the density of the material. A normally consolidated (soft) clay and a loose sand will tend to compress when subjected to shear stresses, thus an increase in pore pressure will result, while overconsolidated (stiff to hard clays) and dense sand will tend to delate and thus experience decreasing (negative) pore pressure changes.

For the cases to be studied we feel that the soft clay is the most interesting in this respect. Based on tests by several investigators the pore pressure response can be expressed as follows:

$$\Delta u = \Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3) \tag{2.9}$$

or by separating the deviatoric stresses from the hydrostatic pressure-increase we get:

$$\Delta u = \Delta\sigma_o + \dot{E}\Delta\tau_{max} = \Delta\sigma_o + E\frac{\Delta\sigma_1 - \Delta\sigma_3}{2} \tag{2.10}$$

where Δu = pore pressure increase

$\Delta\sigma_o$ = increase in hydrostatic stress

$\Delta\tau_{max}$ = increase in maximum shear stress

E, A = pore pressure parameters

E and A are dependent on the strain and stress level, the direction of loading (decreasing or increasing) and the load history. For normally consolidated soils the value of E varies in ranges 0 to 2. A value of 0 corresponds to elastic soils, while a value of 1/3 to 4/3 is typical for average stress levels, i.e. for safety against failure in the order of 2 to 3. For failure conditions typical values are in the range 2/3 to 2.

Point d) requires an analytical or numerical method that can handle transient flow in a porous material. Today several finite difference and finite element programs exist that can handle axisymmetric as well as plane problems of this kind.

The input to such a program will be the geometry and boundary conditions, the excess pore pressures and the consolidation properties of the material.

A stepwise procedure will allow the pore pressure dissipation process to be followed and thus allow an estimate of the increase in effective stresses as a function of time after installation.

Point e) is related to the assumptions made under point c) regarding the choice of pore pressure parameters for estimation of excess pore pressures after installation. The shear strength of a cohesionless, frictional material after partial or full set up can be estimated as

$$s_u = \frac{\sin\varphi}{1+(2A_f-1)\cdot\sin\varphi}\cdot\frac{(\sigma'_1-\sigma'_3)}{2} \tag{2.11}$$

Assuming now a normally consolidated clay typical values for A_f will be in the range 0.7 to 1.3 and typical values for φ will be in the range 25 to 30 degrees. This leads to a typical range of s_u:

$$s_u = (0.25-0.42)\cdot\frac{(\sigma'_1+\sigma'_3)}{2} \tag{2.12}$$

Comparing with the assumptions to be made under point a) initial stress and strength conditions this leads to the following relationship between effective overburden pressure and undrained shear strength:

$$\sigma'_3 = (1-\sin\varphi)\cdot\sigma'_v \tag{2.13}$$

and thus:

$$s_u = (0.20-0.30)\cdot\gamma' z \tag{2.14}$$

Assuming $\gamma = 8 \ kN/m^3$ we finally get

$$s_u = (1.6 - 2.5) kN/m^3 \cdot z \tag{2.15}$$

The last step is point f) which can be somewhat difficult. Normally bearing capacity formulas are based on uniform strength, either constant or linearly increasing with depth. Dependent on the strength distribution evaluated under point e) it has to be decided whether bearing capacity methods can be used or not. If not there is a possibility of using finite element analysis to estimate the bearing capacity. However, a finite element analysis of large plastic deformations requires generally non-linear geometry analysis, but even a constant geometry program with a non-linear material model can give reasonable upper limits for bearing capacity.

To allow a consistent evaluation of strength increase the finite element analysis could also be used for calculation of failure loads under point b).

2.3.3 Available Computer Programs

The analysis of the strength increase under jack-up foundations and the time dependency can be estimated roughly by making course assumptions and using available analytical solutions for the stress distribution and consolidation process.

However, to avoid some of the uncertainties involved in the very simplified assumptions that necessarily have to be made in such an approach a more comprehensive analysis procedure is proposed.

The steps d) to f) described in the previous section 2.3.2 can be analysed with the two finite element programs **OCEAN 2** and **AXIPLN**.

OCEAN 2 (Rahman, 1977) is a program written for the analysis of pore pressure response under offshore gravity structures subjected to wave loading. The program based on a finite element formulation and allows simultaneous evaluation of pore pressure generation and dissipation. The program was originally developed for analysis of axisymmetric cases. However, minor changes is needed to allow plane strain (2-dimensional) problems to be analysed.

AXIPLN (Whithiam, 1978) is a finite element program for axisymmetric or plane strain analysis of soil structure interaction. The program uses a non-linear (hyperbolic) material model based on the model developed by Duncan & Chang, (1970). Basically the program AXIPLN was developed for analysis of pile-soil interaction, but AXIPLN is in fact based on a general purpose soil-structure interaction analysis program SOIL-STRUCT developed by Clough.

AXIPLN allows stepwise application of loads and includes an iterative approach for calculation of stresses and displacements within each load step.

The program has been modified by Kvalstad and Ronold to allow a combined drained and undrained analysis, a feature that is of a certain value when problems involving consolidation under static loads combined with wave loading (i.e. short term undrained) are to be analysed.

The modified version contains automatic generation of undrained shear strength when changing from drained to undrained load steps.

The two programs together allow all steps a) through f) described in Section 2.3.2 to be evaluated. The number of cases to be studied will, however, necessarily have to be limited due to considerable computer costs and the considerable effort needed to feed data between the two programs which have been developed for different purposes and thus cannot be coupled directly.

The two programs are university programs and do thus not have the documentation and input-output features normally developed for commercial program systems.

2.4 EVALUATION METHODS FOR PULL-OUT RESISTANCE

The pull-out resistance of objects completely or partly penetrated into the sea bed consists primarily of the submerged weight of the structure, the friction forces on the sides of the embedded structure and the adhesion or suction forces on the base.

For deeply penetrated spud cans in soft clayey soils instability of the hole generated during penetration, i.e. backfilling, will increase the pull-out resistance by the submerged weight of the backfill and eventual side forces on the backfill volume of soil following the spud-can during pull out.

Further the reconsolidation time between installation and pull-out will lead to an increase in the undrained shear strength of the soil surrounding the footing as described in Section 2.3.

The side friction forces can be expressed as:

$$S = A_{ss} \cdot \alpha \cdot s_{uss} + A_{sb} \cdot \alpha \cdot s_{usb} \tag{2.16}$$

where S $=$ total side resistance on footing and backfill

A_{ss} $=$ side area of embedded part of footing

A_{sb} $=$ side area of lifted backfill

α $=$ strength reduction factor

s_{uss} $=$ undrained shear strength of clay along side of footing at time of pull-out

s_{usb} $=$ undrained shear strength of backfill at time of breakout

For calculation of base adhesion or suction the same type of formulae as for bearing capacity can be used as described by Byrne & Finn (1978), Vesic (1971) and Ninomiya & al. (1972).

However, certain modifications regarding depth effects will be required as the failure surfaces developing in the soil during pull-out will differ somewhat from the failure surfaces developing during penetration. The presence of the seabed will influence the bearing capacity to a greater relative depth of a footing during pull-out than during penetration. However, for surface footings under rapid loading the model tests performed by Byrne & Finn (1978) indicate that bearing capacity theory could be used without any modifications.

However, where suction releasers or jetting systems operate properly the base adhesion will be reduced considerably. In case of malfunctioning of these systems the ultimate base adhesion will increase considerably and can best be described with the above referenced methods.

The base adhesion is certainly also dependent on the time. For soft clays and base dimensions in the range of 10 m or more it is not expected that significant drainage can develop within a time span acceptable for a rig mover. The pull-out of mats and spud-cans in this type of material should most probably be analysed completely undrained.

For more silty and sandy soil types the permeability is considerably higher than for clays. By keeping the base under constant tension or slowly increasing tension, infiltration of water will take place, reducing the underpressure in the soil and thus reducing the strength gradually until pull-out takes place.

3. STABILITY ANALYSIS OF JACK-UP FOOTINGS UNDER COMBINED GRAVITY AND ENVIRONMENTAL LOADING

3.1 GENERAL

The following section outlines and discusses in general the available analysis methods for evaluation of the safety against foundation failure of jack-up platforms and specifically describes the procedures adopted in the two programs SAIL and SAM.

Further the effect of pore pressure generation due to cyclic loading transferred to the seabed either through the jack-up footing(s) or directly due to wave pressure on the seabed will be discussed and the procedure chosen for the parametric study will be described in some detail.

3.2 STABILITY OF INDEPENDENT LEG FOOTINGS (SPUD-CANS)

3.2.1 Bearing Capacity Methods for Inclined Eccentric Loading and Layered Soils

Reference is made to Section 2.2. However, most of the methods described there do not include the effect of horizontal loading on bearing capacity.

Brinch Hansen (1970) introduces load inclination factors and shape factors which are a function of the horizontal load level. (See Appendix C.)

Janbu (1975) introduces a roughness ratio, r, to consider the mobilization of soil strength along the interface between the foundation base and soil. Janbu's N_c value is a function of the roughness ratio r, which again varies with load inclination.

Most of the bearing capacity calculations for onshore structures involve small horizontal loads compared to vertical loads. However, for offshore structures under extreme storm conditions the environmental horizontal loads are of a greater magnitude. For a jack-up placed at large water depth the horizontal loads could be up to one quarter of the vertical load on the leg with less vertical stress level, during a storm. This reduces the bearing/sliding capacity of the soil. See Fig. 3.1. Usually the environmental loads also generate a moment of the foundation which increases the stress level even more. See Fig. 3.2 and Fig. 3.3.

To include most of the effects mentioned above we have chosen Brinch Hansen's formula in our calculation procedure. This formula is internationally accepted and was chosen for geotechnical calculations in DnV's rules for the design construction and inspection of offshore structures.

There are of course different ways to do the bearing capacity calculations as mentioned earlier in this report, but Brinch Hansen's formula was found to be the most suitable for this investigation.

Gravity and environmental forces create vertical loads, horizontal loads and moments on the foundation. Both horizontal load and moment reduces the vertical bearing capacity. Figure 3.1 shows how the ultimate vertical bearing capacity is reduced as a function of increasing horizontal forces.

To account for moment effects the *effective area* concept has been introduced, reducing the total area so that the resulting inclined force acts in the center of gravity of the reduced area, i.e. the effective area.

The effective area concept is only valid if the footing is allowed to rotate. For independent leg type platforms the rotation will be dependent on the flexural rigidity of the legs and will in any case be restricted to relatively small values. As soon as failure starts to develop under the most stressed edge of a footing the footing will start to rotate and the moment transferred to the soil will gradually stop to increase.

With further increase in vertical load the full bearing capacity will be approached leading to a more uniform stress distribution over the footing area. This will reduce the possible eccentricity of the reaction force and thus decrease the reaction moment from the soil.

For surface footings the horizontal force will have to be transferred to the soil through shear between the base of the footing and the soil. This is accounted for in Brinch Hansen's method by introduction of load inclination factors.

For embedded footings the passive earth pressure against the front and the shear forces along the sides of a footing will increase the lateral capacity compared with a surface footing. This is normally accounted for by separate calculation of these forces. The bearing capacity is then calulated using a horizontal force reduced by the additional capacity due to embedment.

3.2.2 Analysis of Sliding Resistance and Lateral Resistance of Deeply Embedded Spud-Cans and Legs

From a geotechnical point of view the shape of a spud-can is more complicated than a flat-based circular or square foundation. We have therefore introduced a simplified model as shown in Fig. 3.3. This model is used for calculation of sliding resistance along the base and lateral resistance for all degrees of penetration.

In the computer program SAIL two-layer soil profiles have been incorporated and the following evaluation refers to the methods selected for SAIL.

In SAIL the base area and side area of the spud can are calculated for the actual penetration depth. If the spud-can is crossing the interface between the upper and the lower layer the base and side areas within each layer are calculated. The lateral resistance is then assumed to be composed of the sliding resistance of the base area(s) and the the passive (lateral) earth resistance against the side area(s).

The generally complicated geometry of a spud can is simplified as shown in Figure 3.3. The lateral resistance is then calculated as:

$$F_{h,ult} = F_{h,side} + F_{h,base} \tag{3.1}$$

$$F_{h,side} = A_{side} \cdot N_p \cdot s_{u,side} + A_{side} \cdot p_u \tag{3.2}$$

$$F_{h,base} = (A_{base} \cdot s_{u,base} + F_v \cdot \tan\varphi) \cdot RR \tag{3.3}$$

where A_{side} = side area

N_p = bearing capacity factor for lateral earth pressure in clay, see Eq. (3.4) below

$s_{u,side}$ = undrained shear strength relevant for side area

$s_{u,base}$ = undrained shear strength relevant for sliding at the base

p_u = passive earth pressure in cohesionless soil (see expression below)

F_v = vertival force at base area

φ = friction angle of cohesionless soil

RR = roughness ratio between base and soil

The bearing capacity factor N_p can be expressed as

$$N_p = 2 + (N_{p,max} - 2) \cdot \frac{z_s}{5 \cdot D} \tag{3.4}$$

where $N_{p,max}$ = bearing capacity of deeply embedded objects, in the analyses with SAIL assumed equal to 10.0

z_s = average penetration depth of side area into clay layer

D = width of side area

The unit passive earth pressure in cohesionless material can be expressed as

$$p_u = \gamma \cdot z \cdot 3 K_p + (K_0 \cdot N_q - 3 K_p) \cdot \frac{z_s}{6 \cdot D} \tag{3.5}$$

where γ = submerged unit weight of soil

z = average depth of side area in sand layer relative to sea bed

$K_p = \dfrac{1+\sin\varphi}{1-\sin\varphi}$ = passive earth pressure coefficient

$K_0 = 1 - \sin\varphi$ = coefficient of earth pressure at rest

N_q = Reissner's bearing capacity coefficient

z_s = average penetration of side area into sand layer.

z_s equals z when sand at top and equals
z minus thickness of top layer if sand in lower layer.

D = width of side area

If a spud can is partly penetrated into the lower layer the contributions to base and side resistance from both layers are added.

Truss type legs are assumed to contribute only to a very limited degree to the lateral resistance of a leg and this resistance has not been considered in SAIL.

To include leg capacity for special designs of pile type or similar legs the height of the spud can can be increased to allow an approximate calculation of the total lateral resistance.

3.3 STABILITY OF MAT SUPPORTED JACK-UP PLATFORMS

3.3.1 General Description of Problem

The large foundation areas of the common mat types reduces penetration of the mat to a few feet even in very soft soils. The mat foundation will thus always have to be treated as a surface (or at least close to surface) footing. and is thus very similar to the gravity platforms placed in the North Sea from a geotechnical point of view.
The basic problems that may develop with respect to stability are listed below:

a) Sliding failure along very shallow, nearly horizontal failure surface direct under the mat due to horizontal forces from waves and wind.

b) Local bearing capacity failure under the edge of the platform caused by combined action of horizontal forces and overturning moments. Dependent on the shape of the mat different failure modes may develop. A mat with a small hole may fail along failure surfaces that start out almost horizontally under the least loaded side of the mat and then goes deeper under the the most loaded side. A mat with a large hole and slot area may develop a nearly vertical bearing capacity type of failure under the most loaded beams and thus sink in and tilt.

c) Deepseated overall overturning failure involving a large soil mass due to overturning moment on the platform and wave pressure on the seabed.

d) Scour and undermining of mat due to combined wave and current induced material transport and/or pumping effect of the platform itself when subjected to wave loading.

3.3.2 Shallow Bearing Capacity Failure under Edge

With increasing load inclination and load eccentricity the failure surface may develop rather shallow. The strength distribution with depth will also significantly influence the shape of the failure surface. In a soil with low strength at mudline and a relative strong increase in strength with depth it is obvious that the critical failure surface will be shallower than in the case with a uniform strength and for high load inclinations pure sliding may be critical. This effect is clearly more important for large footing dimension than for smaller footing dimensions.

A review of available bearing capacity methods revealed that they all have certain limitations. The most flexible method described is probably Brinch-Hansen's (1970) which allows inclined, eccentric loading to be taken into account and also specifies shape factors and depth factors for treatment of strip to circular footings at mudline or embedded.

However, Brinch Hansen's method does not take soils with increasing strength into account. This is on the other hand treated by Davis and Booker (1971), but there limited to vertical load on strip footings at mudline.

Lauritzen and Schjetne (1976) presented their limit method which has been used in the design of concrete gravity platforms. Janbu's generalized method of slices (1973) is also a possible solution for this type of analysis.

For the special purpose of a parameter study a closed form solution would be a practical way of reducing the number of variables to be fed into a stability analysis of this type, and a method is presented in the following which allows a fast and simple variation of failure surfaces in order to determine the critical surface approximately and the corresponding minimum factor of safety against shallow bearing capacity failure.

The method is a two-wedge limit method. A cylindrical failure wedge is assumed to develop under the mat. The center of the cylinder is assumed to be either below the left edge or above the right edge of the mat. To the right of the mat a passive wedge is pushed along a slip plane inclined to the horizontal.

A vertical shear force is assumed to develop between the passive wedge and the cylindrical wedge as the cylindrical wedge is pushed downwards relative to the passive wedge. Figures 3.6 and 3.7 show the forces and the geometry of the two investigated failure surfaces.

The shear strength of the soil is assumed to increase linearly from s_{u0} at mudline with a shear strength gradient k. Typical values for s_{u0} is 2 to 10 kPa for very soft, under-consolidated to normally consolidated soils, and the corresponding shear strength gradient will most probably lie in the range 0.5 to 2.0 kN/m^3.

The safety against failure is defined as the ratio between the resisting moment and the overturning moment and the minimum value is found by varying the radius R over a certain range. When R approaches infinity this case converges towards pure sliding.

A more detailed presentation of the definition of the different components of the resisting moment is presented in Appendix E.

The method developed has been checked against Brinch Hansen's formula for uniform soil strength for various ratios of footing length to width and for various load inclination angles with good correspondence for all cases. The maximum deviation found is about 18% overestimation of capacity for pure vertical load, a case for which the method is not specifically effective. For ratios $\frac{F_u}{F_v}$ greater than 0.1 there is good correspondence and generally Brinch Hansen's method gives slightly higher values.

Compared with Davis and Booker's method the chosen approach seems to give values some 20 to 25% higher. However, compared with other methods like Skempton's and Brinch Hansen's with assumptions for average strength some $\frac{1}{2}B$ to $\frac{2}{3}B$ below the footing the proposed method is generally much closer to Davis' and Booker's solution than the other methods. Again it must be underlined that the method was not developed with the intention to evaluate bearing capacity under pure vertical loading. As soon as a certain load inclination is present it is believed that the method will give a very good estimate of the safety against foundation failure for uniform as well as linearly increasing soil strength and for strip footing as well as square footings. The method converges towards the correct solution for sliding for increasing load inclinations.

The described method for evaluation of the bearing capacity of mats under inclined, eccentric loading has been incorporated in the program SAM in the subroutine, SHBEAR.

3.3.4 Deepseated Bearing Capacity Failure

Deepseated bearing capacity failure will mainly be a problem in soft, underconsolidated clays when the platform is subjected to a combination of large overturning moments on the structure and a wave pressure on the sea bed as illustrated in Figure 3.8

The failure surface may be approximated by a cylinder. The rotation centre of the most critical failure surface will depend on the wave period, the water depth, the geometry of the structure and also on the strength distribution with depth. A shallower failure surface may be more critical where a stiffer layer is found at some depth below the mat or the soil's shear strength increases considerably with depth.

The basic approach is to vary the center and the radius of the failure surface within reasonable limits and in this way evaluate the most critical geometry and the corresponding safety against deepseated overturning failure.

In Figure 3.9 a general situation is outlined showing all forces acting and a simplified assumption for the shape of the failing soil mass with the resisting shear stresses indicated. The derived expressions for resisting moments are presented in Appendix F.

The described approach for evaluation of safety against deepseated bearing capacity failure has been incorporated in the computer program SAM in subroutine DEBEAR where a large number of possible sliding surfaces are evaluated for 21 time steps through each wave period investigated in order to determine the critical surface and the corresponding critical loading conditions.

3.4 METHODS FOR EVALUATION OF CONSOLIDATION AND REPEATED LOADING EFFECTS ON THE STABILITY

3.4.1 General

In Section 2.2 a method for evaluation of consolidation effects under static loading conditions has been outlined. The method will be used to at least qualitatively describe the beneficial effects of consolidation on the bearing capacity of a footing and may give some additional information on the effect of variable preload duration as well as variable preload level.

The cyclic loading due to passing waves may have a very significant effect on the stability of jack-up platforms and the surrounding seafloor. This problem area has been addressed by numerous authors during the last 15 years and a considerable research effort has concentrated on improving the understanding of soil behaviour under cyclic, wave induced loading.

Most of this research has been carried out in connection with gravity platform foundations in the North Sea, but underwater slides and local seafloor instability observed in connection with hurricanes in the Gulf of Mexico has triggered several investigations on this problem area as well.

The analysis of problems of this type must be characterized as extremely complex as it involves wave-structure-soil interaction with time-dependent and non-linear material parameters. Within this study it is therefore necessary to restrict the analysis to cover some well-defined cases and to make a series of simplifying assumptions. The aim of the analysis must be to quantify possible effects on the foundation stability.

In the following a brief description of the main parameters and assumptions involved in evaluations of repeated loading effects will be given and finally a selected analysis scheme is presented.

3.4.2 Effect of Cyclic Loading on Soil Behavior

Soils subjected to variations in stress will react with tendencies to change volume. For soft clays and loose sands the volume will tend to decrease while for very stiff, over-consolidated clays and very dense sand shear stress variations may tend to increase the volume.

For saturated soils, as is the case offshore, the effect of volume changes will induce changes in the pore water pressure. Soft clays and loose sands will thus experience an increase in pore water pressure which will lead to strength reduction. As the pore pressure increase is dependent on the intensity of shear stress variations, (and this may vary considerably around a footing), an uneven pore pressure field will develop and a seepage process is started. This seepage will gradually lead to dissipation of the induced pore water pressures, i.e. the consolidation process is started.

The large difference in permeability of sand and clay will lead to different consequences of pore pressure generation. In thicker strata of sand the seepage direction will be predominantly in direction of the seafloor, i.e. a vertical pressure gradient will develop.

The relative high permeability of sands will allow water from the deeper parts of the sand layer to move upwards and add to the effect of pressure generation in the upper part. The vertical gradient may in extreme cases be so high that the upper layers are brought in suspension, i.e. liquefaction occurs. This phenomen has been observed in connection with earthquake induced repeated loading on many occasions and has been subject to extensive research for many years. Even if total liquefaction does not occur a considerable reduction in strength of the soil and thus in the bearing capacity may occur.

The low permeability of clays will not allow accumulation of pressure gradients and liquefaction of the material as in the case of sand. The repeated loading effect will be more local and restricted to the soil elements where high shear stress variations are present. In this case the soil will experience strength degradation in the most stressed zones and even if a dissipation process will be present the redistribution of pore pressures will most probably take place too slow to introduce significant accumulation of pressure gradients towards mudline. The cohesive nature of clays has further a preventing effect on complete loosening of the grain structure which eliminates liquefaction.

The approach for analysis of these phenomena will therefore be somewhat different for sand and clay. In sands the pore pressure generation is the essential parameter. Further the permeability is so high that considerable dissipation of generated pore pressures will have to be considered. In clays the local strength degradation will be the main factor of influence. The permeability can be considered to be so low that for storm durations of 6 to 24 hours no significant dissipation will take place.

3.4.3 Pore Pressure Increase and Liquefaction Characteristics of Sands

A long series of laboratory investigations on the behaviour of sands subjected to cyclic loading have been made. These studies have very often been earthquake related and concentrated on describing the relationship between stress intensity, expressed as a stress ratio $\frac{\tau_c}{\sigma'_{v,0}}$ i.e. the ratio of the applied cyclic shear stress to the vertical effective normal stress, and the number of cycles required to cause liquefaction. The gradual development of pore pressure vs number of cycles is more important for wave loading and has been investigated in connection with gravity platform foundations in the North Sea.

The interpretation of the data have been done in different ways by different investigators, however, the common observation is that during cyclic loading the pressure increase per cycle is high during the first cycles, then it tends to decrease, but as liquefaction is approached the increase per cyclic again increases as shown in Figure 3.10.

The shape of these pore pressure generation curves can be represented by the following expressions according to Martin (1975):

$$\frac{\Delta u}{\sigma_v'} = \frac{1}{2} + \frac{1}{\pi} \cdot \sin^{-1} \left[2 \cdot \left| \frac{N}{N_l} \right|^{\frac{1}{\Theta}} - 1 \right] \tag{3.4}$$

where Δu = generated pore pressure corresponding to the number of cycles applied

N = number of cycles applied

N_l = number of cycles required to cause liquefaction

σ_v' = initial effective stress

Θ = empirical constant

The described pore pressure generation model is a function of the number of cycles required to cause liquefaction N_l. Based on constant amplitude tests (triaxial and simple shear) the common procedure has been to plot the cyclic shear stress level $\frac{\tau_c}{\sigma_v'}$ vs number of cycles causing liquefaction. Typical curve shapes for sand are shown in Figure 3.11.

3.4.4 Strength Degradation of Clay due to Cyclic Loading

A large research project performed during 1975 and '76 improved the general knowledge about cyclic load effects on clay considerably. Andersen (1976), NGI (1976).

The *strain contour method* developed by Andersen, NGI has been used extensively for checking the stability of gravity platforms in the North Sea. Foss, Dahlberg and Kvalstad (1978) introduced a method to assess the risk of *Failure in Cyclic Loading* based on use of the *strain contour method* to predict the gradual development of cyclic shear strains.

Basically the method allows the cyclic shear strain in soil elements due to the accumulated effect of all the waves in a storm to be predicted, and by varying the storm intensity the failure value characterized by large uncontrollable strains can be determined. The 'cyclic strength' is mainly a function of the number of cycles (i.e. the duration of storms) and the stress level $\frac{\tau_c}{s_u}$ where τ_c is the cyclic shear stress and s_u the initial undrained shear strength of the clay.

3.4.5 Characterization of Storm Loading

In a storm the occurence of individual waves is random with small and large waves interspersed. For practical reasons, however, cyclic tests and soil materials are normally carried out as constant amplitude tests.

The relationship between constant amplitude tests and random variations caused by waves has been discussed by Seed (1971), Andersen (1978) and Kvalstad and Dahlberg (1980).

The method of an *equivalent number of cycles* developed by Seed and the *wave parcel* approach taken by Andersen are quite similar and have been used with good results for simulation of random wave loading. Andersen's approach was verified to be reliable by Kvalstad and Dahlberg through a series of comparable wave parcel and random amplitude triaxial cyclic tests.

3.4.6 Analysis Methods Selected for PP3

Evaluation of the effect of cyclic loading on the stability of jack-up footings can be carried out in the following way:

a) Determine initial stress conditions in soil by assuming at rest conditions.

b) Determine stress change due to installation of jack-up.

c) Evaluate static capacity.

d) Use programs SAIL or SAM to determine loads on footing and seabed for various wave heights and periods.

e) Select a few characteristic load levels and determine stress changes in soil surrounding footing, evaluate safety without degradation effects.

f) Determine pore pressure generation /dissipation for sand or *cyclic strength* for clay.

g) Evaluate safety reductions induced by repeated wave loading.

Points a), b), c) and e) can be analysed by means of the Finite Element Program AXIPLN.

Points c), d) and g) can be evaluated by means of SAIL and SAM.

Point g) needs input of reduced average strength values and will only give a very rough indication of safety reductions.

Point f) can be analysed with the FEM consolidation program OCEAN 2 for sand by means of the program RELOC for clay.

The above described analysis scheme is rather complex and time consuming and will allow only a limited number of cases to be analyzed under PP3.

4. SOIL-STRUCTURE INTERACTION ANALYSIS

4.1 GENERAL

The methods for evaluation of penetration resistance and foundation stability are mainly based on plasticity theory. The footings are treated as rigid elements and the flexibility of the jack-up structure is not taken into consideration.

Mat supported jack-ups are resting on a steel structure of large dimensions. The design of the mat has to be based on some assumptions regarding the distribution of contact stress. For soft clays a uniform distribution under weight alone and a linear distribution of contact stress under wave loading is a reasonable assumption. However, for stiffer clays and sandy/gravelly soils, the local uneveness of the seabed may lead to local consentrations of reaction stress. The presence of obstructions like boulders or steel scrap represent as well risks to the mat.

The legs of a mat supported structure are all fixed to the mat and the support condition at the lower end of the leg can be assumed as fixed.

For independent leg platforms this is not the case. The individual legs are subject to different loads and the support condition may vary from nearly fixed to pinned dependent on soil type and load level as well as the shape and dimensions of the footing. The description of the support conditions of individual legs is thus a difficult task which necessarily will have to involve several simplifying assumptions.

Further the flexibility of the legs which is mainly a function of the leg design and the water depths will influence the load distribution between the legs and the effect of rotational stiffness of the individual footings may play an important role in the interaction between the legs.

The methods selected for use in PP3 to study soil-structure interaction effects are described in the following.

4.2 ANALYSIS METHODS FOR EVALUATION OF CONTACT STRESS DISTRIBUTION UNDER MATS AND SPUD-CANS

4.2.1 Mats

The large dimensions of mat structures and the relative long span between the legs indicates that the mat structure might be sensitive to uneven distribution of contact stress. In stiff clays and dense sand only a very limited contact area is required to produce sufficient bearing capacity.

Uneven sea bed and scouring may reduce the contact area to a minimum which is required to give sufficient bearing capacity. The distribution of the local contact stresses has, however, to fullfill the overall equilibrium requirements.

Bearing capacity theory as outlined in Section 2.2 can be used for evaluation of possible local contact stress concentrations.

Boulders or debris on the sea bed can be critical for the most structures. Boulders and debris may be squeezed down into the soil by the mat, but the local contact stresses can be very high where the sea bottom consists of dense sand or stiff clay. Again the bearing capacity method can be used for evaluation of the penetration resistance of such obstacles dependent on size, soil strength and soil layering.

4.2.2 Spud-Cans

Spud-cans are general designed conservatively and are capable of taking high local stress concentrations. The conical shape of the more recent designs secures in most cases a centric contact. However, the presence of debris and boulders on the sea floor represents a possible hazard on locations where the soil has a high shear strength.

4.3 LOAD-DISPLACEMENT RELATIONSHIPS FOR SPUD-CAN - SOIL INTERACTION

4.3.1 Hyperbolic Load-Displacement Relationship

During penetration the spud-cans produce a continous failure until the bearing capacity of the soil exceeds the vertical load. After reducing the load from preload level to normal operating vertical load the foundation will be subjected to additional combined load effects from overturning moment and horizontal shear force dependent on the environmental conditions.

There exists numerous records of load tests performed on footings of various dimensions and under various types of loads. Generally it seems that the load-displacement curves can be simulated closely by hyperbolas.

To evaluate the influence of nonlinear soil support against the spud-cans on the total distribution of forces in the legs we have introduced three hyperbolic springs on each leg as shown in Figure 4.1

The expression for the hyperbolic load-displacement curve is:

$$F = \frac{\delta}{\dfrac{1}{K_i} + \dfrac{\delta}{F_u}} \tag{4.1}$$

where F = load or moment on footing
δ = displacement or rotation
K_i = initial spring stiffness
F_u = ultimate load or moment capacity

The tangential stiffness can be expressed very simply as:

$$K_t = K_i(1 - F/F_u)^2 \tag{4.2}$$

4.3.2 Initial Spring Stiffness

The initial spring stiffnesses have been assumed to be comparable to elastic conditions and the following expressions have been extracted from Veritas Rules for the design, construction and inspection of offshore structures (1977), Appendix G, Dynamic Analysis:

Mode of vibration	Spring coefficient K
Vertical (z)	$K_z = \dfrac{4GR}{(1-\nu)}$
Horizontal (x)	$K_x = \dfrac{8GR}{(2-\nu)}$
Rocking (Θ)	$K_\Theta = \dfrac{8GR^3}{3(1-\nu)}$

where R_0 = radius of foundation in contact with soil
G = shear modulus of soil
ν = Poisson's ratio

Effect of Embedment and Two-Layer Systems on Spring Stiffnesses

The above elastic solutions are valid for circular footings on the surface of an elastic half-space. With increasing penetration depth and for layers of finite depth the above expressions tend to underestimate the elastic spring stiffnesses.

Elastic solutions are available for vertical and rotational spring stiffnesses of circular footings on the surface of a layer of finite thickness. This corresponds roughly to soil profile no. 1 with a soft clay overlaying a stiff clay or a dense sand layer. An evaluation of such solutions presented by Poulos and Davies (1973) shows that the vertical stiffness increases considerably with decreasing layer thickness.

The effect of layer thickness on the rotational and lateral springs is much less significant than for the vertical spring.

The effect of embedment is also of a certain importance. With increasing embedment depth and with increasing height of the spud-can there is a clear increase in spring stiffnesses compared with the surface values (Tassoulas & Kausel, 1983).

All these effects tend to increase the spring stiffnesses beyond the surface values. It is not possible within the scope of work of this project to incorporate details from the very complex analysis required for calculation of stiffnesses of embedded spud-cans. However, these effects can be evaluated approximately by varying the shear modulus of the soil and in this way simulate increased or decreased stiffnesses in order to see the effect on the interaction between soil and structure.

4.3.3 Ultimate Capacities

The ultimate capacities to be inserted in the different expressions for the three springs will be evaluated by use of relevant expressions for bearing and sliding capacity.

The *vertical capacity* is found by using Brinch Hansen's (1970) bearing capacity formula as outlined in section 2.2 and in section 3.2.1 taking the effect of load inclination and load eccentricity into account.

The *horizontal capacity* is found by using the procedure described in Section 3.2.2.

The *moment capacity* is found by calulating the maximum possible eccentricity that can be combined with the existing combination of vertical and horizontal forces on the footing. It can be shown that this lead to the following value for the ultimate eccentricity e_u:

$$e_u = \frac{1}{2}\frac{\pi R^2}{\sqrt{A}}(1 - F_v/F_{vu}) \tag{4.3}$$

where R = radius of contact area
A = total contact area of footing
F_v = Vertical force
F_{vu} = Total bearing capacity when eccentricity equal 0

The corresponding ultimate moment capacity will then be:

$$M_u = F_v \cdot e_u \tag{4.4}$$

4.3.4 Comparison with Finite Element Analysis

The above described method based on hyperbolic springs was checked against analyses with the finite element program AXIPLN which contains a nonlinear hyperbolic stress-strain relationship very similar to the spring-displacement relationship adopted for the soil - spud-can interaction.

A few runs have been made comparing the hyperbolic model with FEM results. The results of a sample problem is shown in Figure 4.2 where the displacement vs load curves are shown for both analysis methods.

The footing investigated was a strip footing to allow comparison with the plain strain FEM analysis. The footing was given a width of 14m and was placed at the surface of a soil with a linearly increasing undrained shear strength starting from 41 kPa at mud-line and increasing with 2 kPa per m depth. The finite element mesh is shown in Figure 4.3.

The initial shear modulus of the soil was assumed to be 300 times the undrained shear strength for both types of analysis so the results are comparable although it is always difficult to compare lumped stiffness parameters with stiffness parameters of soil element in a volume of soil subjected to highly variable stresses and strains.

The load was applied in several steps starting with two vertical load increments each of 1336 kN/m' followed by 7 load increments of 320 kN/m' vertical load, 50 kN/m' horizontal load and moment increments of 150, 140, 135, 130, 125, 100 and 50 kNm/m'. After load increment No. 6 failure was obtained with the Brinch Hansen bearing capacity model and the hyperbola reached the assymtotic value in the non-linear spring model.

Comparison of the results showed that the vertical spring was represented well by the hyperbolic model, see Fig. 4.4. The horizontal spring was initially too stiff and the rotational spring too soft. The combined effect of horizontal and vertical force was clearly underestimated. By coupling the horizontal and vertical deformations by the expression below the horizontal load-displacement curve compared much better with the FEM results.

$$\delta h = \delta h(F_h) + \delta v(F_v)\sin(\alpha) \tag{4.5}$$

where $\delta h'$ = combined horizontal displacement

$\delta h(F_h)$ = horizontal displacement due to horizontal load
$\delta v(F_v)$ = vertical displacement due to vertical load
α = load inclination angle

The above combination of vertical and horizontal displacements reflects the fact that failure occurs along a failure surface in the soil and not as an overall softening in direction of the inclined load.

The reason for the too soft moment response, computed by the hyperbolic spring model, is believed to be caused by a similar effect. The moment capacity as determined for combined vertical, horizontal and moment loading by means of Brinch-Hansen's formula may be somewhat underestimated, because the corresponding failure surface is entirely different from that governing the moment capacity for pure moment loading, i.e. the failure surface goes through a different part of the soil volume. Hence it is believed that the soil that provides the moment capacity for pure moment loading may still contribute somewhat to the moment capacity also when combined loading is assumed. This contribution is not included in the moment capacity estimated by Brinch-Hansen's formula, and thus a too low rotational stiffness may be the result because the stress level in moment loading is overestimated. On the other hand, one should still keep in mind that the kind of finite element analysis performed by means of AXIPLN tends to underestimate the displacements at high stress levels, and this may to some extent counteract the afore-mentioned effects.

The hyperbolic stiffness model could be improved in several ways. This is however, in the authors opinion not the essential point. The comparison has shown that a hyperbolic model based on lumped, elastic springs and bearing capacity theory taking the combined effect of vertical, horizontal and moment loading into account leads to reasonable estimates of stiffness, and that the use of this simple method yields results rather close to the very comprehensive and time consuming finite element method.

4.4 ANALYSIS METHODS FOR WAVE-STRUCTURE-SOIL INTERACTION

4.4.1 General

Wave loading on the legs of a jack-up platform causes an overall overturning moment and an overall horizontal force on the platform.

For the individual legs this leads to load variations. The horizontal force and the overturning moment are generally not in phase, leading to a complex combination of vertical force, horizontal force and rotating moment in the foundation. Combined with the nonlinear load-displacement behaviour of soils which can be very pronounced for high load levels typical for the footings of independent leg jack-ups the analysis of wave-structure-soil interaction turns out to be very complex unless pinned conditions are assumed at the bottom of the legs. This is conservative for most cases, and there is today a trend going in direction of including some fixity at the seabed for large diameter spud-cans.

4.4.2 Proposed Analysis Method

The proposed analysis method presented to the Steering Committee as shown in Appendix A was the basis for the development of a computer program that is capable of doing the overall analysis of wave-structure-soil interaction. The aim was not to develop a design tool for jack-ups regarding the load calculation and the structural system, but to develop an analysis tool which allows a simulation of soil-structure interaction under realistic load combination and with a realistic simulation of structural stiffness.

The program SAIL is the result of this effort. The program contains a simplified plane frame representation of the structure, a wave load generation routine based on linear wave theory as outlined in Section 5 and hyperbolic springs at the bottom of the legs simulating the non-linear behaviour of the soil. Loads are applied incrementally in small steps and the stiffnesses of the soil springs are corrected for each step. A preliminary and brief description of the program SAIL is given in Appendix G.

5. SUMMARY OF METHODS FOR CALCULATION OF ENVIRONMENTAL LOADS

The following summary is mainly intended to give the background for the selection of analysis methods for wind, wave and current forces implemented in the computer programs SAIL and SAM. A thorough discussion of the hydrodynamic aspects and specific problems related to calculation of wave and current forces is not within the scope of the project and the same applies to the wind forces.

5.1 WIND

5.1.1 General

Within this study wind forces will only be considered in a very simplified way. The wind force will be treated as a static force acting at a given height above the still water level.

5.1.2 Choice of Reference Wind Speed

Two kinds of wind speed will normally have to be considered, the sustained wind speed and the gust wind speed. The sustained wind speed is defined as the average wind speed during a time interval of 1 minute while the gust wind speed is defined as the average wind speed during a time interval of 3 seconds.

The normal reference for wind speed is the sustained wind speed at ten meters above the still water level v_{10}. The rules of the classification societies requires normally 100 years sustained wind speed to be considered in design.

For sheltered locations $v_{10} = 40\ m/sec$ is recommended, and with increasing roughness of the environment values up to 55 m/sec are specified for extreme areas. For the North Sea 50 m/sec is considered in design.

5.1.3 Wind Speed vs. Height

The wind speed increases with increasing height above sea water level. Several different wind speed profiles are in use, however, the differences between these different profiles are minimal.

In Veritas' Rules for Mobile Offshore Structures the following expression is given for sustained wind speed.

$$v(z) = v_{10} \cdot \sqrt{0.93 + 0.007z} \quad \text{for} \quad z < 150m \tag{5.1}$$

where v(z) = sustained wind speed at height z metres above the still water level

v_{10} = sustained wind speed at 10 metres above still water level (reference value)

for gust wind speeds Veritas' Rules specifies the following expressions:

$$v(z)_{gust} = v_{10} \cdot \sqrt{1.53 + 0.003z} \tag{5.2}$$

which corresponds to a *reference gust speed* which is 25% higher than the sustained wind speed. See also Figure 5.1.

Since most of the deckhouse area, derrick structures etc. are located some 20m above SWL the *average* sustained wind speed will effectively be increased some 1.5 to 2.0 m/s beyond the reference value v_{10}.

5.1.4 Wind Forces

Wind forces are a function of the wind pressure and the area of the structure subjected to the wind pressure. The wind pressure may be calculated as:

$$q = \frac{1}{2}\rho v^2 \tag{5.3}$$

where q = basic wind pressure

ρ = the mass density of air, normally taken as 0.012 $kNs^2/m4$
v = wind velocity

The wind force F_w on a structural member or surface can be calculated as

$$F_w = C_s q A \sin\alpha \tag{5.4}$$

where F_w = wind force
C_s = the shape coefficient
q = the basic wind pressure
A = Projected area member normal to the direction of the force
α = angle between the direction of the end and the axis of the member or surface

The wind force can thus be expressed as

$$F_w = \frac{1}{2}\rho C_s v^2 A \sin\alpha \tag{5.5}$$

The basic difference between most rules and regulations lies in the choice of C_s values of the differently shaped structural members like cylinders, flat bows, box girders etc. and for structures on the deck like deck houses and drilling equipment.

Further solidification effects is areas with closely spaced members have to be considered as this will increase the wind force. On the other hand shielding effects need to be taken into account in order to limit the conservatism in the local assumptions.

Wind tunnel tests on carefully built models may be the most reliable way to take these effects into account provided the model laws are fullfilled.

It is not the objective nor the intention of this study to go into details regarding the evaluation of wind loads. The basic approach is to establish a relevant load level for a parametric study and a connection between the wind speed and the load.

Several design calculations have been reviewed and based on this review the following simplified expression was derived:

$$F_w = 0.002 A_{hull} v^2_{hull} \quad (in \quad kN) \tag{5.6}$$

where A_{hull} = the projected hull area of the jack-up normal to the wind direction excluding deck houses, derrick and legs (in m^2).

v_{hull} = the average sustained wind speed acting at the hull (in m/s).

The wind force lever arm is certainly dependent on the distribution of areas, leg length above deck, height of derrick, airgap etc. To simplify it was decided to use values for wind lever arms as outlined in Appendix B.

5.2 CURRENT

5.2.1 General

The water current is normally considered to consist of two components, the tidal current and the wind generated current.

5.2.2 Current Speed vs. Depth

The variation of current velocity with depth is not very well understood and is highly dependent on the local conditions. The following distribution is proposed in Veritas Rules for mobile structures.

$$v(z) = v_1(z/d)^{\frac{1}{7}} + v_2(z/d) \tag{5.7}$$

where v(z) = total current velocity at a distance z above the sea bottom.

v_1 = tidal current velocity at still water level
v_2 = wind generated current at still water level
d = water depth

The above expression is considered to be a conservative approach. However, there exists differing views on the distribution of current velocity with depth and the recommendations vary somewhat between the different rules and regulations.

The distributions of the tidal induced and the wind induced components are shown in Figure 5.2.

5.2.3 Choice of Current Velocity

The tidal current velocity to be used in conjunction with extreme weather conditions is in Veritas Rules specified to be taken not less than 0.5 m/sec. The tidal current velocity is certainly heavily dependent on the local conditions but may as an example in the North Sea area vary in the range 0.5 to 1.5 m/sec.

The wind-generated current velocity is recommended to be taken as 1 percent of the reference sustained wind speed v_{10} i.e. for wind speeds varying between 40 and 55 m/sec an average of about 0.5 m/sec should be considered for wind generated current speed at surface.

5.2.4 Current Forces

The current-induced drag forces on submerged, structural members can be calculated according to Morison's equation. More details about Morison's equation is given in Section 5.3. For the work in this project the current velocity will be superimposed on the wave-induced particle velocities. Current forces on spud-cans and mats will not be considered.

5.3 WAVES

5.3.1 General

There are basically two alternative approaches to the design of offshore structures, the 'design wave' approach and the 'spectral and probabilistic' approach. Which is appropriate depends on the type of structure, its size and location, and in particular, on its most likely mode of failure.

For this study it was decided to concentrate on one side on the installation problems (neglecting environmental loads) and on the other side the extreme storm conditions. It was further decided to consider static analysis only. On this basis the design wave method is straight foreward. The only difficulty might be to choose an appropriate design wave period. However, the wave period is one of the parameters to be varied during the parametric study and the programs allow for a variation of the wave period within a range with a selected step in period.

Several non-linear theories are suitable for the design wave approach, however, as the objective of the study is related to the foundation stability it was decided not to include more sophisticated methods for calculation of wave forces, and linear (Airy) wave theory was selected.

For the three different water depths selected in the *Proposal for Parameter Ranges* (see Appendix B) the validity of linear theory was checked against the criteria:

$$a) \quad \frac{L^2 H}{d^3} << 100 \tag{5.8}$$

b) wave steepness H/L is to be small

where L = wave length
H = wave height
d = water depth

according to Longuet-Higgins (1956).

For water depths of 30, 60, 90 and 110 m and for maximum wave periods of 17 seconds Equation (5.8) was evaluated and gave values of 40, 11, 5 and 4.3 respectively i.e. all values well below 100 and acceptable for this purpose.

The wave steepness might be more critical in this respect. There is, however, a limit for steepness included in the subroutines in SAM and SAIL calculating wave forces. The expression gives a maximum wave height dependent on the wave length as follows:

$$H_{max} \leqslant \frac{L}{5.4+0.023L} \quad \text{if} \quad L < 200m, \text{ then } H_{max} = L/10 \qquad (5.9)$$

If the user specifies a larger wave height than H_{max} the program authomatically reduces the wave height to H_{max}. Equation (5.9) gives a maximum steepness of 1/10 for wave lengths less than 200 m and the wave steepness decrease below 1/10 for longer waves.

To conclude it can be said that the linear wave theory might be a simplification for the shallowest case (i.e. water depth 30m) to be analyzed. However, it is felt that for this type of study this simplification can be accepted.

For the study the following expressions have been adopted from Hallam & al.,CIRIA Report UR8 (1977).

$$\eta(x,t) = \frac{1}{2} H \cos 2\pi \left[\frac{x}{L} - \frac{t}{T} \right] \qquad (5.10)$$

where $\eta(x,t)$ = free surface at position x at time t
H = wave height
x = x-coordinate
L = wave length
T = wave period
t = time

The zero of the x-axis is located at the crest of the wave.

The wave number k is:

$$k=\frac{2\pi}{L} \tag{5.11}$$

And the implicit expression for the wave length L is

$$L=\frac{gT^2}{2\pi}\tanh\frac{2\pi d}{L} \tag{5.12}$$

where d = water depth

The horizontal water particle velocity u and the horizontal acceleration $\dot{u}$ are defined as:

$$u(x,z,t)=\frac{\pi H}{\ldots}\left|\frac{\cosh(2\pi(z+d)/L)}{\ldots}\right|\cdot\cos 2\pi\left|\frac{x}{\ldots}-\frac{t}{\ldots}\right| \tag{5.13}$$

$$\dot{u}(x,z,t)=\frac{2\pi^2 H}{T^2}\left|\frac{\cosh(2\pi(z+d)/L)}{\sinh(2\pi d/L)}\right|\cdot\sin 2\pi\left|\frac{x}{L}-\frac{t}{T}\right| \tag{5.14}$$

and the *dynamic pressure* can be expressed as:

$$p(x,z,t)=\frac{1}{2}\rho gH\left|\frac{\cosh(2\pi(z+d)/L)}{\cosh(2\pi d/L)}\right|\cdot\cos 2\pi\left|\frac{x}{L}-\frac{t}{T}\right| \tag{5.15}$$

Note that the vertical z-axis is to be taken positive upwards from still water level, i.e. at sea bed z = - d.

Only the horizontal components of velocity and acceleration are considered as vertical legs only have been accounted for in the program.

5.3.2 Choice of Wave Heights and Wave Periods

The wave heights to be used in conjunction with design for extreme weather conditions are certainly dependent on the offshore areas in which a jack-up is planned to operate.

The different classification societies and the national authorities have issued rules and guidelines for the choice of design wave heights and corresponding ranges of wave periods.

As outlined earlier the maximum wave height is dependent on the water depth and the wave period or wave length. The range of wave heights and wave periods selected for this study are presented in Appendix 2 and was subjected to discussion during Steering Committee Meeting No. 2.

5.3.3 Wave Forces on Legs

The calculation of wave forces acting on the legs of jack-ups has been based on Morison's equation:

$$F_{wave} = F_D + F_I \tag{5.16}$$

where F_{wave} = the total wave force on a long slender member

F_D = the drag component

F_I = the inertia component

The drag force on a vertical leg and be expressed as:

$$F_D = \int_0^{\eta} \frac{1}{2} \rho C_D D_D u \mid u \mid dz \tag{5.17}$$

where ρ = the mass density of sea water

C_D = a drag coefficient

D_D = effective width of member for drag

u = horizontal water particle velocity,
wave-induced plus current velocity

η = distance from sea bottom to water level

The inerta force can be expressed as:

$$F_I = \int_0^{\eta} \rho C_I \dot{u} \frac{\pi D_I^2}{4} dz \tag{5.18}$$

where C_I = inertia coefficient

$\dot{u}$ = horizontal water particle acceleration

D_I = effective diameter for inertion

The effective widths D_D and D_I for drag and inertia respectively were introduced to allow simplified evaluation of truss type legs. For cylindrical legs $D_D = D_I =$ the diameter of the leg, however, for truss type legs the drag and inertia forces on all members will have to be added. D_D and D_I represent single cylindrical members giving the equivalent drag and inertia forces, i.e. giving the same projected area and the same total volume per unit leg length as the sum of all truss members over the same unit length.

No allowance for variation of D_D and D_I over the leg length has been included in the programs.

The drag coefficient for circular cylinders varies within a range 0.5 to 1.2 depending on roughness, Reynold's number, wave theory used, etc. For Reynold's numbers larger than $3 \cdot 10^6$ the drag coefficient should as a minimum be taken as 0.7.

The inertia coefficient for long slender circular cylinders can be taken equal to 2.

5.3.4 Procedure adopted in computer programs

Within the computer programs SAIL and SAM the leg is divided in 10 equal segments from the water level down to the sea bottom. The water particle velocity and acceleration is calculated in the middle of each segment and multiplied by the corresponding area or volume as well as the relevant constants to arrive at drag and inerta forces for each segment.

The total drag and inerta forces and moments relative to sea bed are calculated by summing up the forces and the corresponding overturning moments on each segment.

The procedure is continued for all legs (i.e. 3 or 4) and the total horizontal force and overturning moment caused by waves and current is calculated.

A range of wave periods can be selected and the program divides the wave period in 20 time increments and evaluates the drag, inerta and total force and moment for all 20 timesteps.

5.4 WAVE PRESSURE ON MATS AND SEA BED

The wave pressure acting on the mat surface vertically and laterally may give rise to considerable horizontal forces and overturning moments. The wave pressure is also acting on the sea bed and will introduce additional shear stress in the soil mass which needs to be considered.

The *dynamic component* of the wave pressure can be expressed as:

$$p = \frac{1}{2}\rho g H \left| \frac{\cosh(k(z+d))}{\cosh(kd)} \right| \cdot \cos 2\pi \left| \frac{x}{L} - \frac{t}{T} \right| \qquad (5.19)$$

By integrating this pressure over the surface of the mat it is possible to calculate the total horizontal force and the overturning moment.

The mat structure will in the program SAM be represented by the simplified structure shown in Figure 5.4 The vertical force F_v over the horizontal surface at height TMAT above the sea bed can be calculated by the following integral:

$$F_v = \int_A p(x,TMAT,t)\cdot dA \tag{5.20}$$

inserting Eq. 5.19 in the above equation gives for waves travelling in the x-direction:

$$F_v = \frac{\gamma_w H\cdot L\cdot TOTL}{4\pi}\cdot\frac{\cosh k\cdot TMAT}{\cosh(kd)}\cdot\left[\sin 2\pi\left[\frac{x_2}{L}-\frac{t}{T}\right] \quad -\sin 2\pi\left[\frac{x_1}{L}-\frac{t}{T}\right]\right] \tag{5.21}$$

By calculating the vertical force on the total area and subtracting the forces on the slot and hole areas which are acting directly on the sea bed the total resultant vertical force can be calculated. For waves travelling in the y-direction the corresponding expression is developed.

The overturning moment acting on the horizontal surface of the mat relative to the y-axis will be

$$M_y = \int_A p(x,TMAT,t)\cdot x\cdot dA \tag{5.22}$$

Inserting Eq. 5.19 into the above expression and integrating leads to the following expression for an area as shown in Figure 5.4.

$$M_y = \frac{\gamma_w H\cdot L\cdot TOTL}{4\pi}\cdot\left[\frac{\cosh hkTMAT}{\cosh kd}\right]$$

$$\cdot\left[\left[\frac{L}{2\pi}\cos 2\pi\left[\frac{x_2}{L}-\frac{t}{T}\right]+x_2\sin 2\pi\left[\frac{x_2}{L}-\frac{t}{T}\right]\right]-\left[\frac{L}{2\pi}\cos 2\pi\left[\frac{x_1}{L}-\frac{t}{T}\right]+x_1\sin 2\pi\left[\frac{x_1}{L}-\frac{t}{T}\right]\right]\right] \tag{5.23}$$

Correspondingly the moments due to pressure on the hole and slot areas are subtracted from the moment acting on the total area in order to define the net resulting moment.

Similar expressions for the moment about the x - axis for waves travelling in the y - direction have been developed.

the pressures acting on all sides of the mat. The shape of the mat will change the horizontal water particle velocities around the mat to some extent. An added mass term should probably be considered. The dimensions of the hole and the slot are of the same order as the height and width of the mat fingers and beams. Shading effects will therefore be present and the selection of an average inertia coefficient is thus not easy. With the low height to length ratio and also a relative low height to width ratio the inertia coefficient will probably lie in the range 1.1 to 1.4 with the higher values for a large slot and hole area and the smaller values for a mat without hole or slot.

It is again stressed that the authors are not experts in hydrodynamics and and that the above is our best interpretation of more general literature about this subject.

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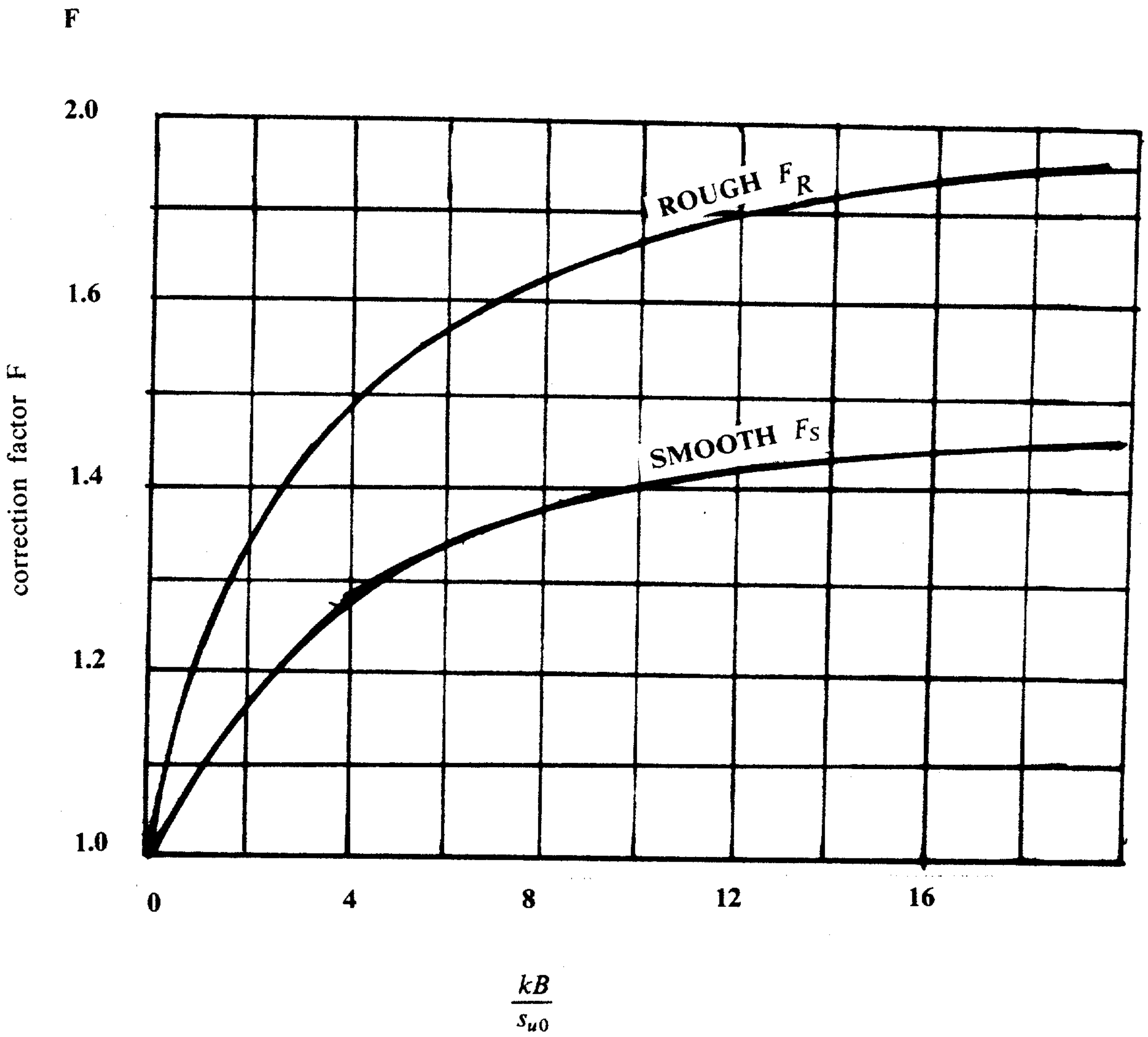
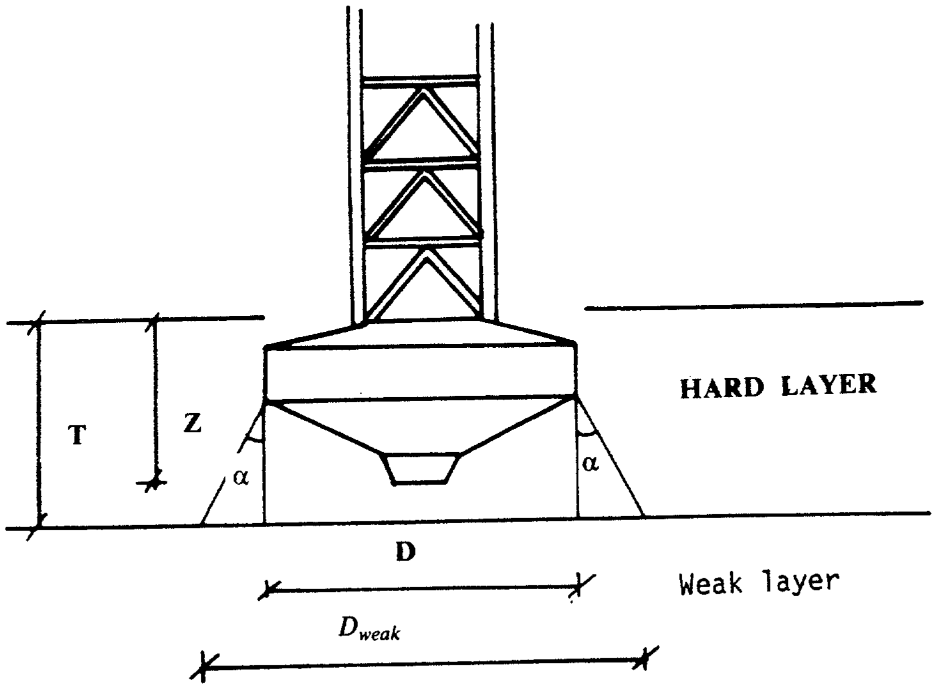
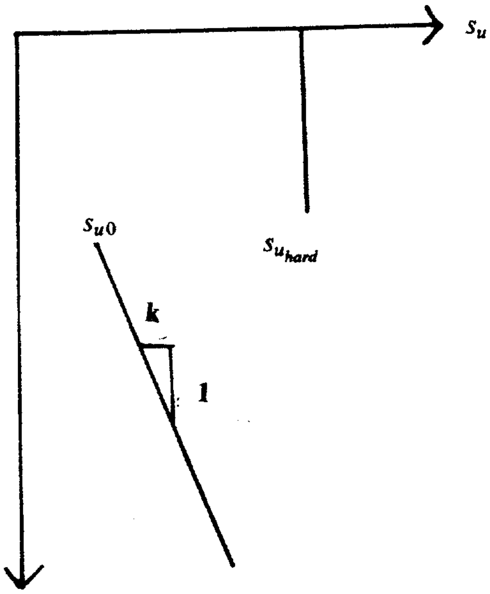


Figure 2.1 Davis and Booker's correction factor F vs. the ratio $\frac{kB}{s_{uo}}$





$$D_{weak} = D + 2 \cdot (T - Z) \cdot \tan\alpha$$

$$A_{0_{weak}} = \frac{\pi}{4} D_{weak}^{2}$$

where T = **Thickness of upper layer**

Z = **Penetration depth**

α = **stress distribution angle**

D = **Diameter of penetrated part of footing**

D_{weak} = **Fictive foundation diameter in weak layer**

$A_{0_{weak}}$ = **Foundation area corresponding to** D_{weak}

Figure 2.2 Model for *punch-through* failure

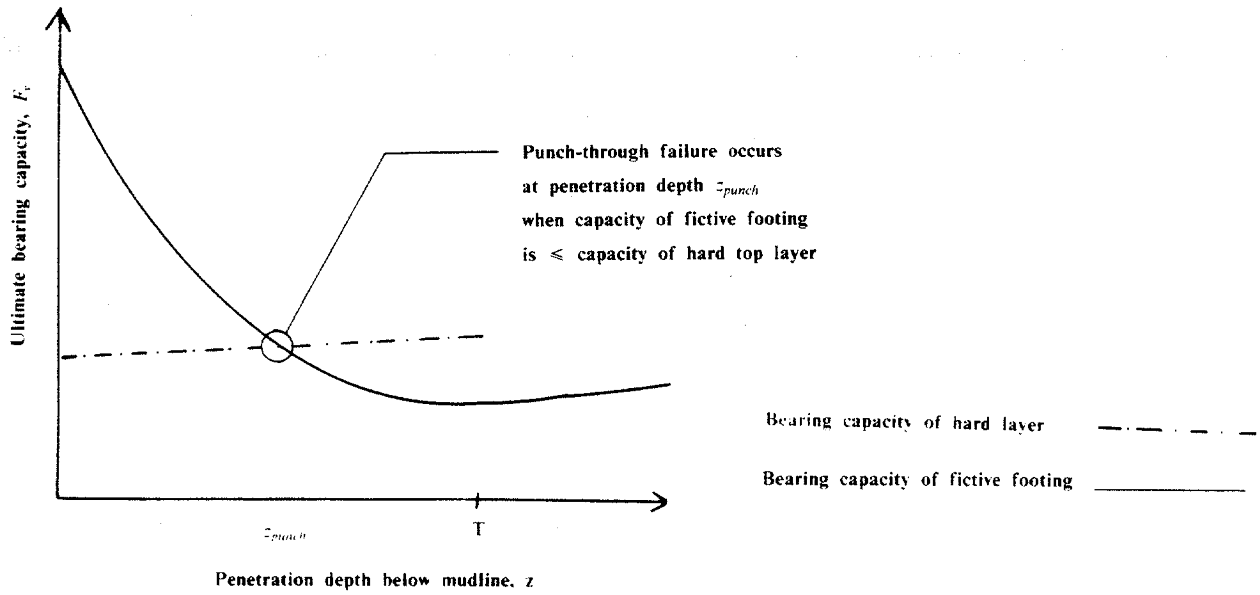


Figure 2.3 *Punch-through* analysis shown for a general case with stiff clay overlaying soft

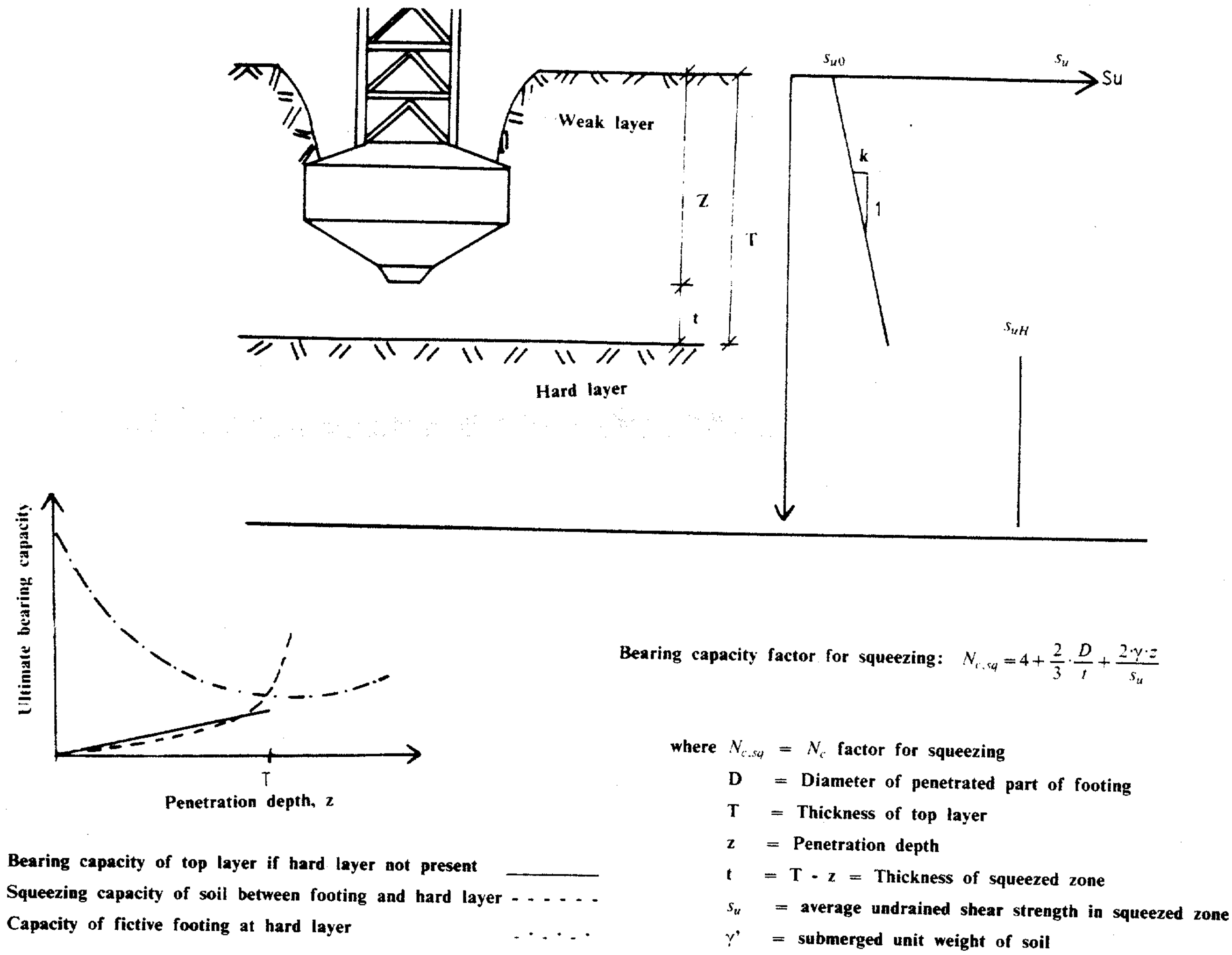


Figure 2.4 *Squeezing* analysis shown for a general case with a weak layer over a hard layer

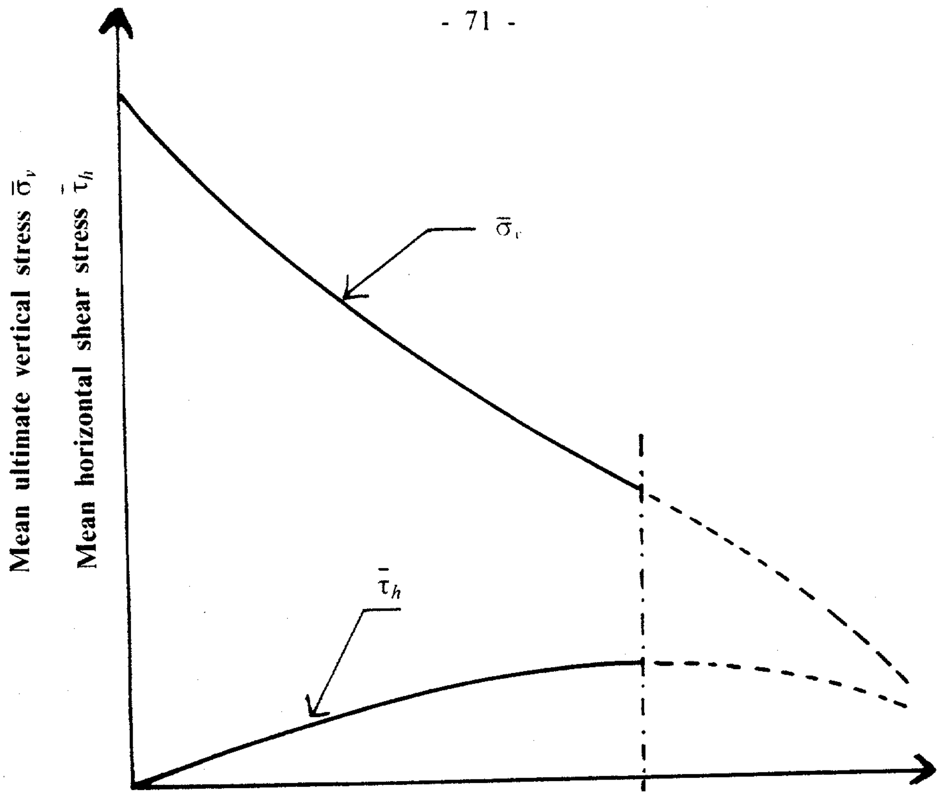
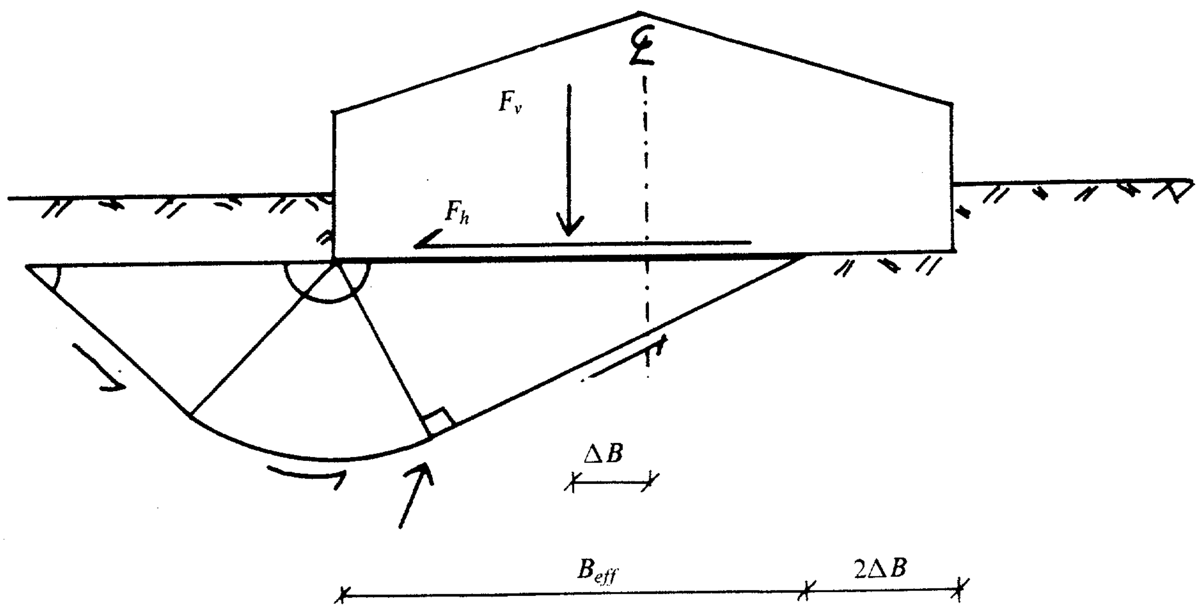


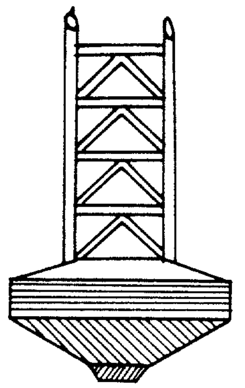
Figure 3.1 Bearing capacity vs. load inclination (from Janbu, 1975)



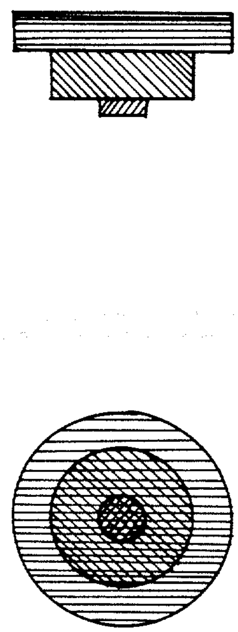
B_{eff} = **'effective width of foundation area**

ΔB = **eccentricity of vertical load component**

Figure 3.2 Reduction from full to *effective* area for footing under moment loading

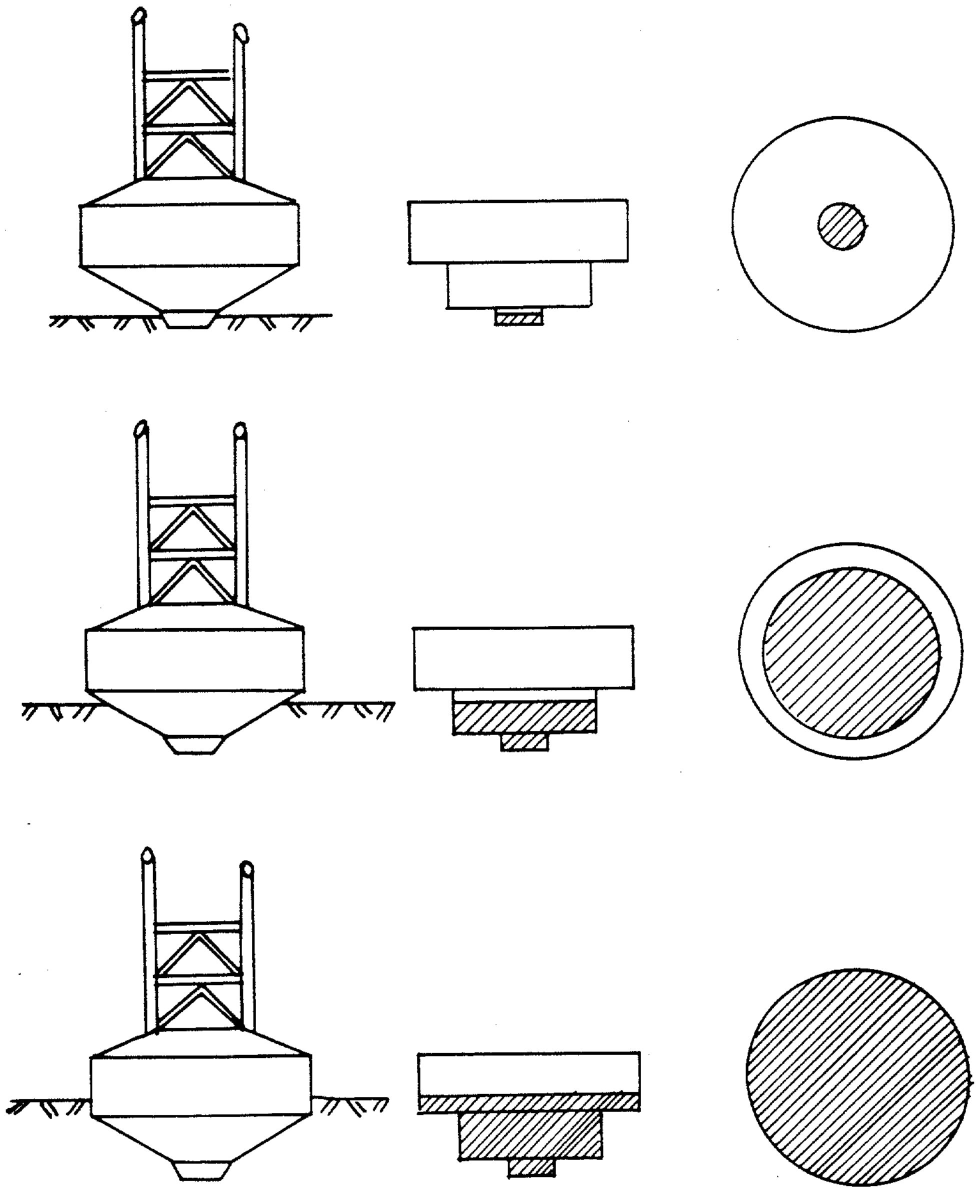


A) Idealized spud can model for penetration analysis



B) Simplified idealization for analysis of sliding and lateral resistance

Figure 3.3 a) Spud-can model for penetration analysis
b) Idealized model for analysis sliding and lateral resistance



side view of idealized penetrated spud can | **area for lateral resistance** | **area for sliding resistance**

Figure 3.4 Various degrees of spud-can penetration giving gradually increasing lateral resistance in uniform soil

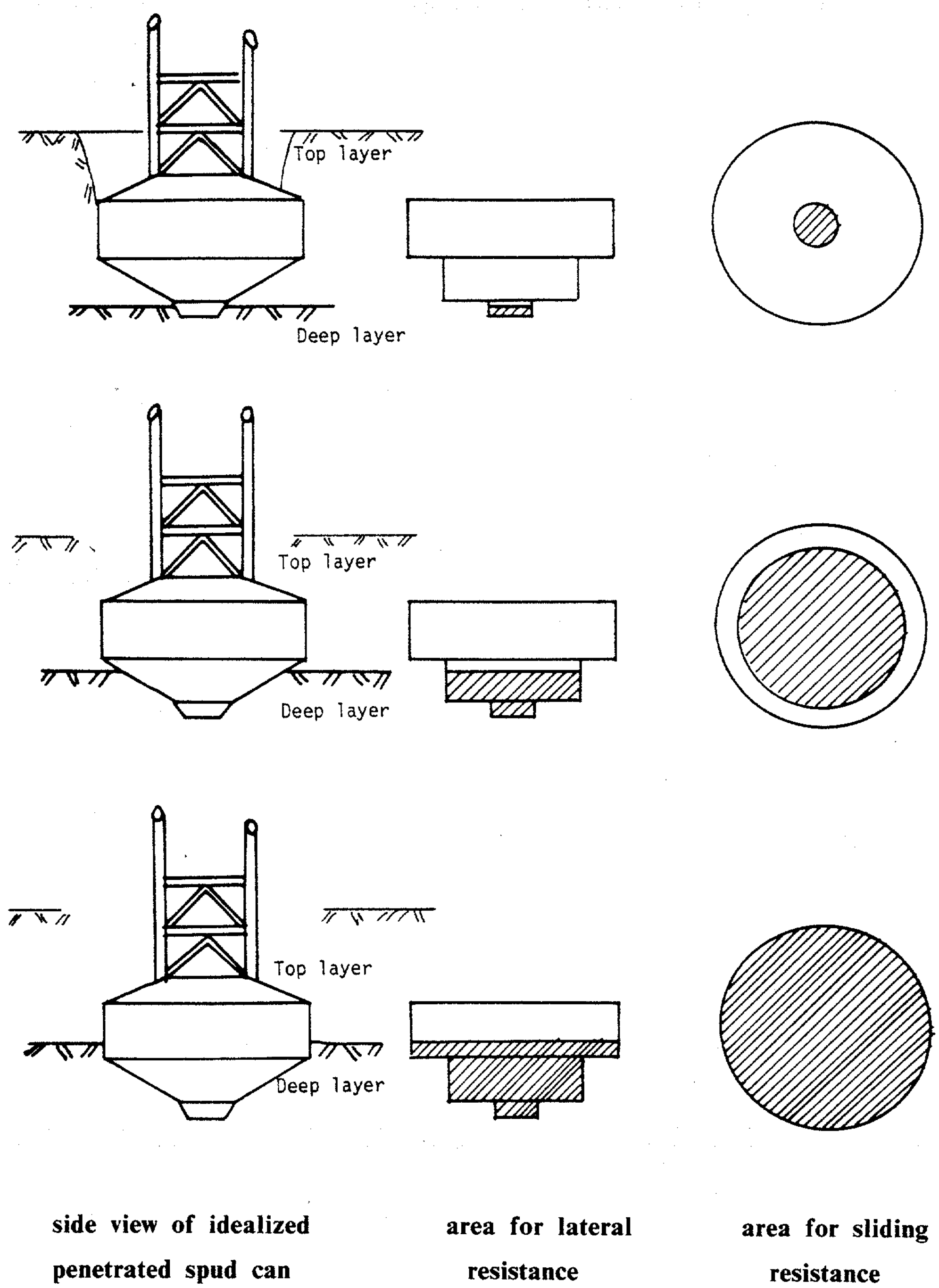
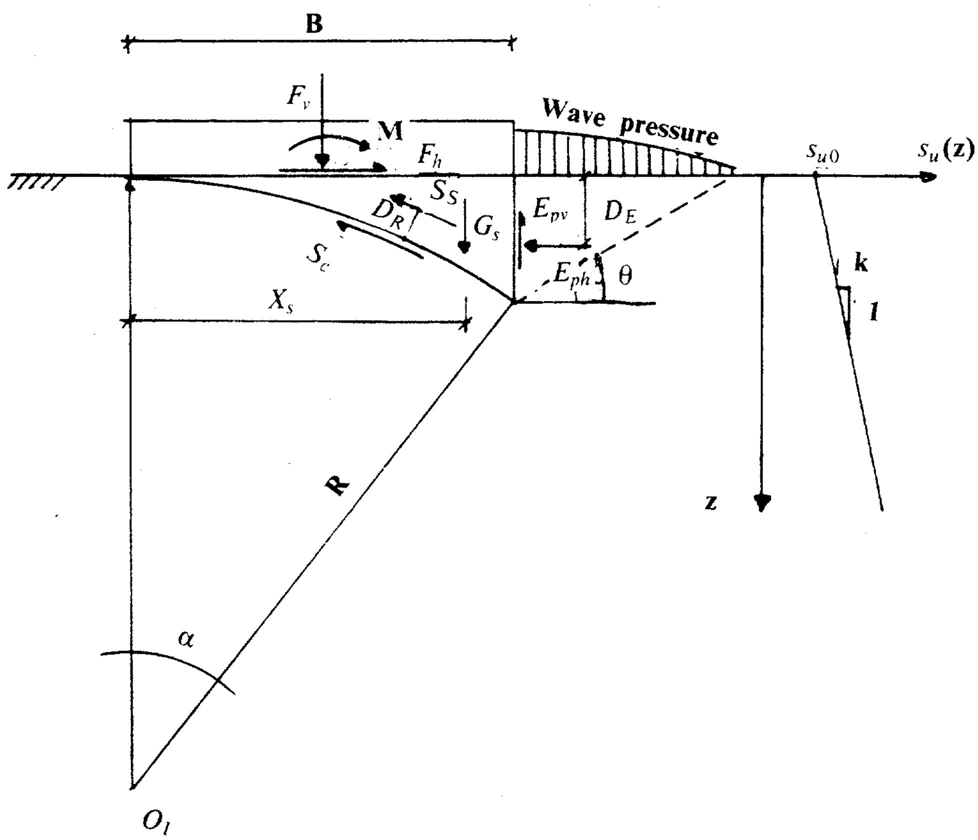


Figure 3.5 Deeply embedded spud-cans in layered soil



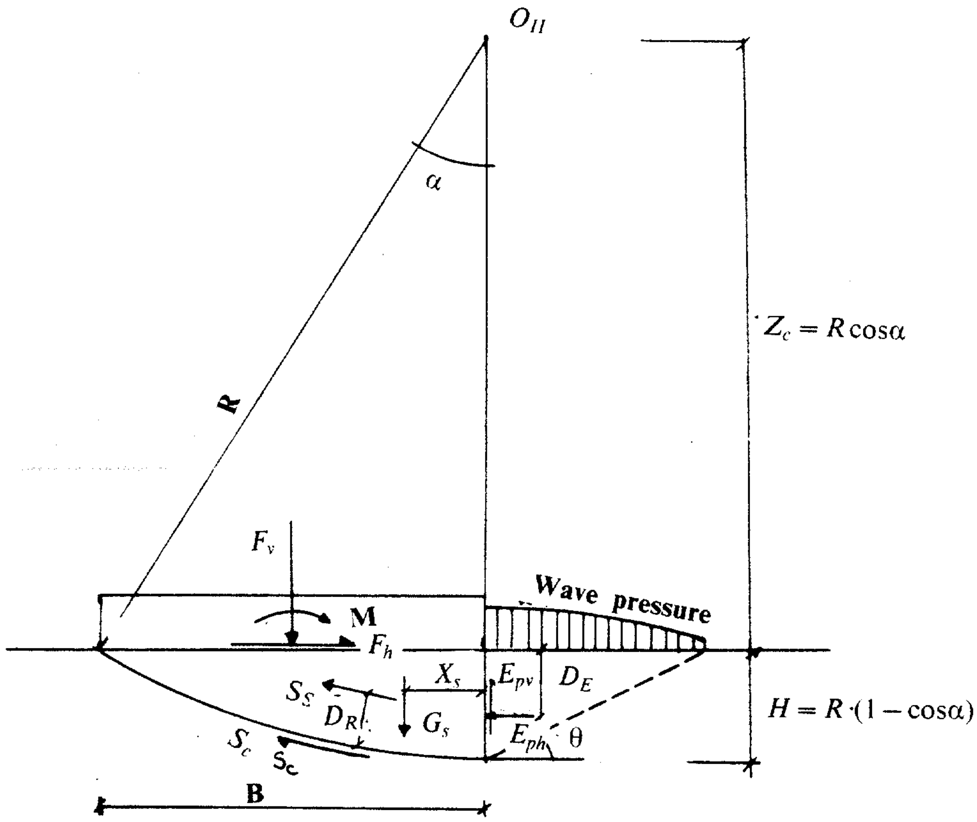
Overturning moment about O_l:

$$M_{O_l} = F_v \cdot (\frac{B}{2} + \frac{M}{F_v}) + F_h \cdot R + G_s \cdot X_s$$

Resisting moment about O_l:

$$M_{R_l} = S_c \cdot R + 2 \cdot S_s \cdot (R + D_R) + E_{pv} \cdot B + E_{ph} \cdot (R - D_E)$$

Figure 3.6 Shallow bearing capacity failure for mat, center of cylindrical wedge under left corner



Overturning moment about O_{II}:

$$M_{O_{II}} = F_v\cdot\left(\frac{B}{2}-\frac{M}{F_v}\right)+F_h\cdot R\cdot\cos\alpha + G_s\cdot X_s$$

Resisting moment about O_{II}:

$$M_{R_{II}} = S_c\cdot R + 2\cdot S_s\cdot(R-D_R)+E_{ph}\cdot(R\cos\alpha - D_E)$$

Figure 3.7 Shallow bearing capacity failure for mat, center of cylindrical wedge above right corner

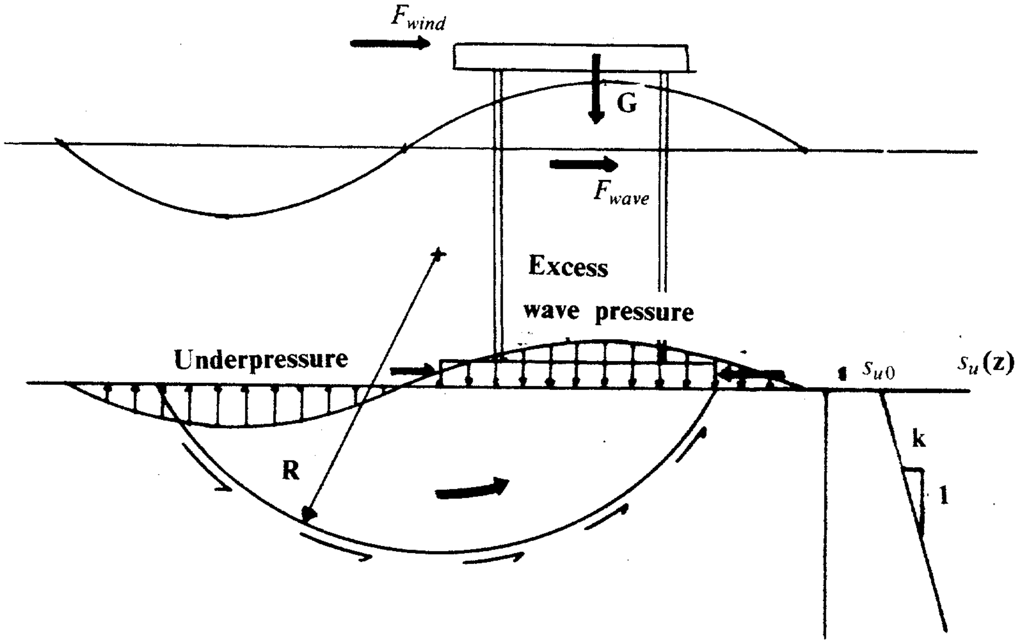
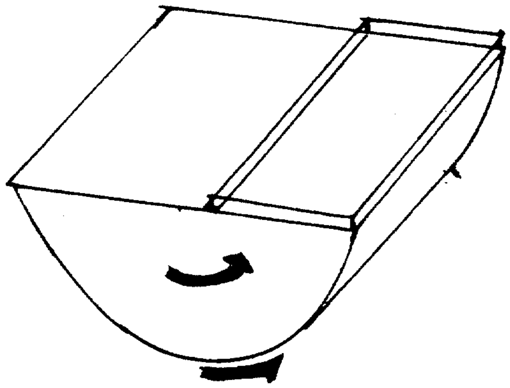
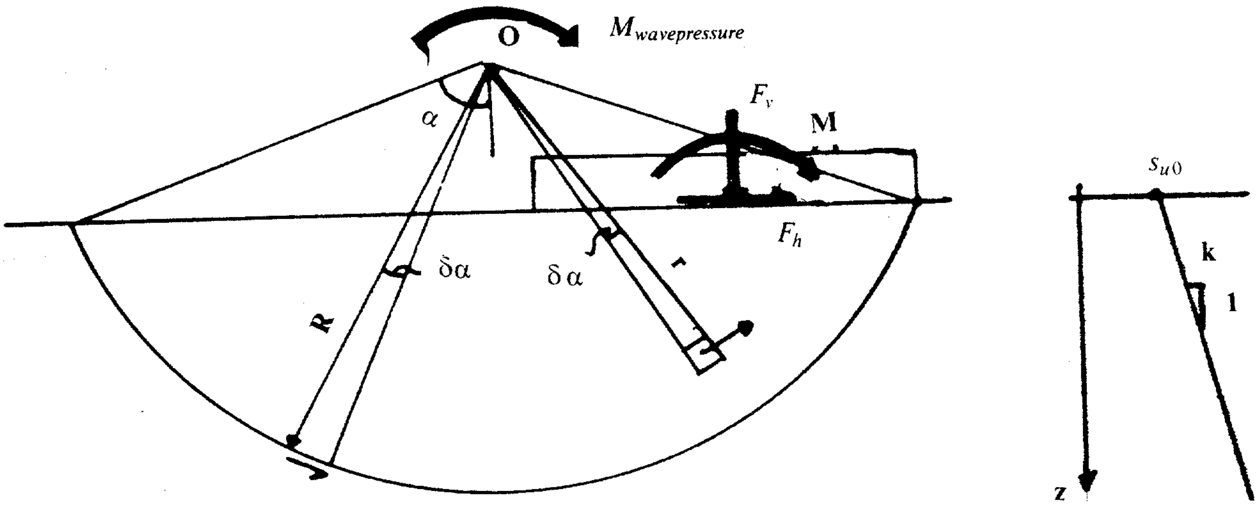


Figure 3.8 Deepseated bearing capacity failure for mat





Resisting moment on end surfaces:

$$M_e = s_{u0} \cdot R^3 \cdot \left[\frac{4}{3} \cdot \alpha + \frac{kR}{s_{u0}} \cdot \sin\alpha - \frac{4}{3} \frac{kz_c}{s_{u0}} \alpha \right]$$

$$- s_{u0} \cdot z_c{}^3 \cdot \left| \frac{2}{3} - \frac{1}{6} \cdot \frac{kz_c}{s_{u0}} \right| \cdot \left| \frac{\sin\alpha}{\cos^2\alpha} + \ln \left| \frac{1+\tan\frac{\alpha}{2}}{1-\tan\frac{\alpha}{2}} \right| \right| \quad \text{for } \alpha < \frac{\pi}{2}$$

if $\alpha = \frac{\pi}{2}$ **then** $M_e = s_{u0} \cdot R^3 \cdot \left| \frac{4}{3} \cdot \frac{\pi}{2} + \frac{kR}{s_{u0}} \right|$

Resisting moment on cylinder surface:

$$M_c = 2 \cdot L \cdot R^2 \cdot s_{u0} \cdot \left| (1 - \frac{kz_c}{s_{u0}}) \cdot \alpha + \frac{R \cdot k}{s_{u0}} \cdot \sin\alpha \right|$$

Figure 3.9 Cylindrical wedge with external forces and shear resistance on cylinder surface and end surfaces

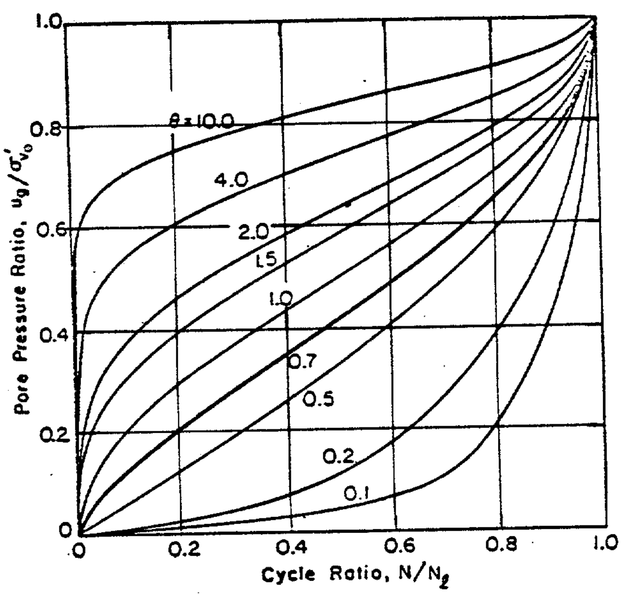


Fig. 1-2 RATE OF PORE PRESSURE GENERATION

Figure 3.10 General shape of pore pressure generation curves vs number of cycles for various stress levels

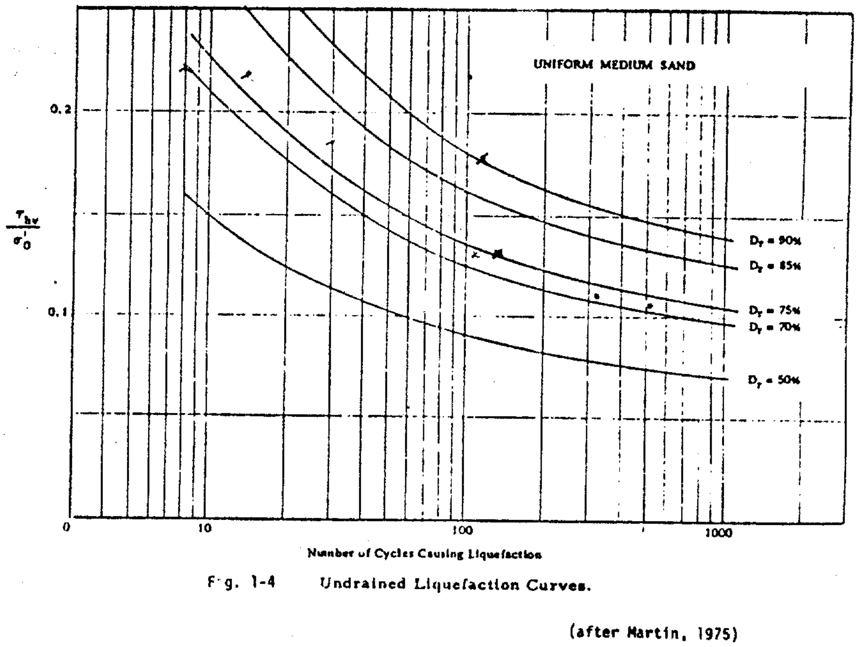


Figure 3.11 Typical shapes of curves showing number of cycles to initial liquefaction vs cyclic stress ratio for sands varying density

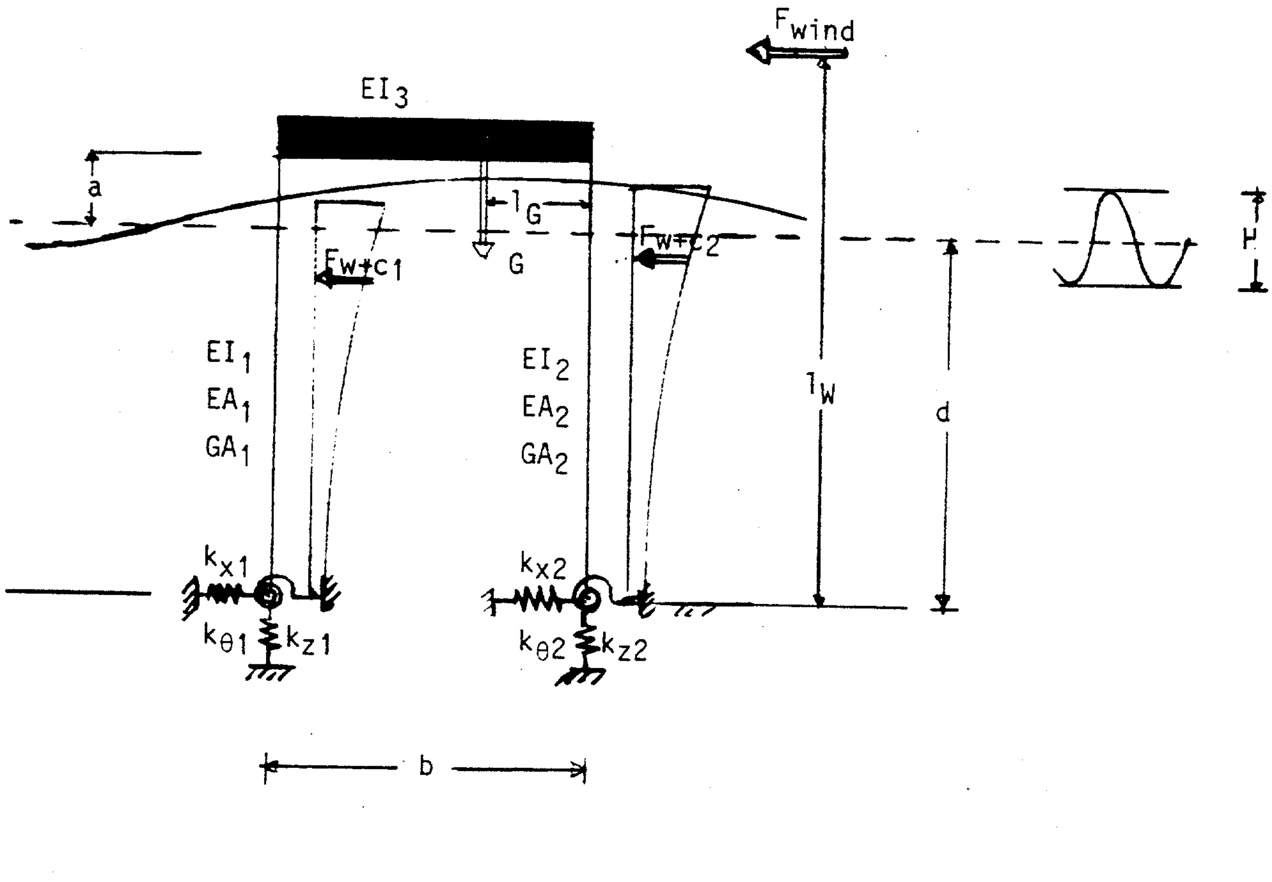


Figure 4.1 Plane frame model with springs simulating the fixity of the leg

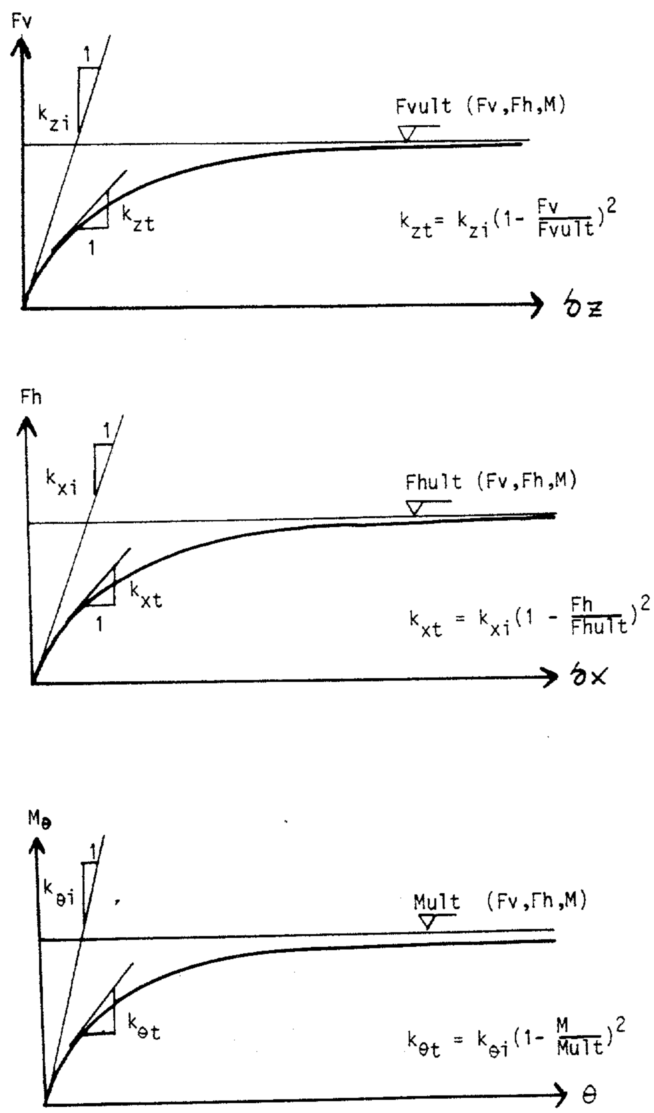


Figure 4.2 Nonlinear, hyperbolic load-displacement relationship for vertical and horizontal forces as well as overturning moment

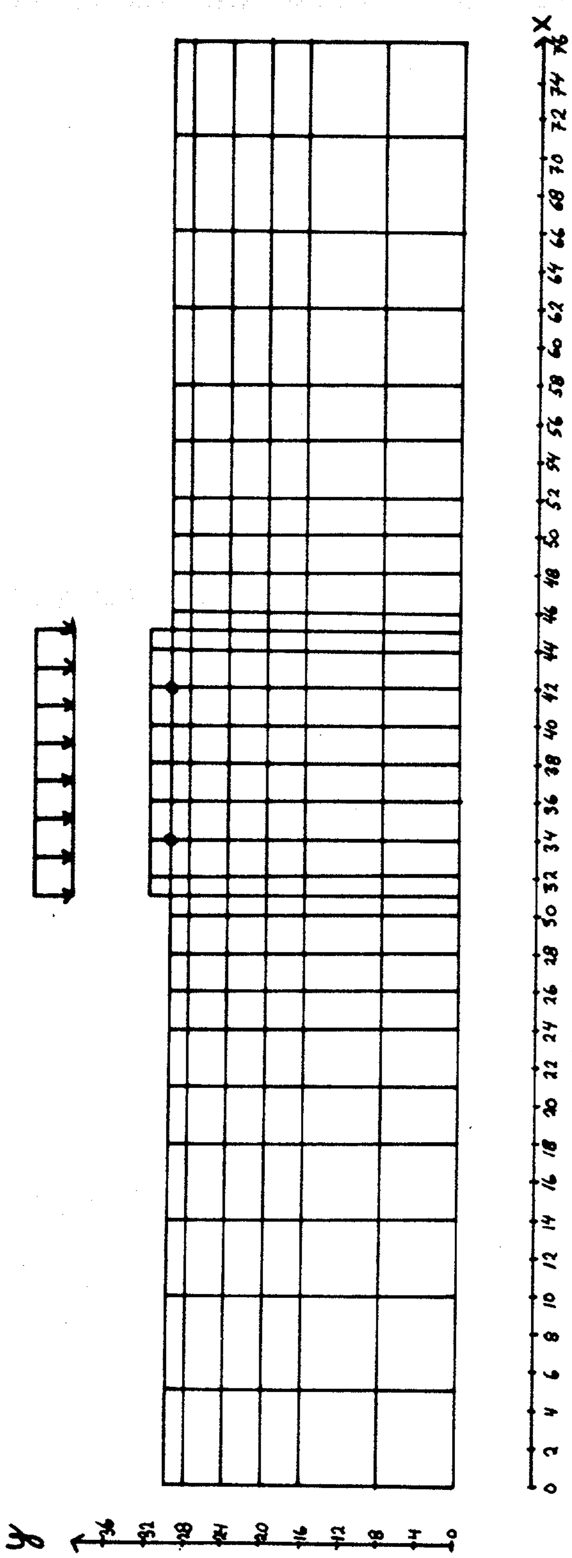


Figure 4.3 Element Mesh used for nonlinear FEM program AXIPLN.

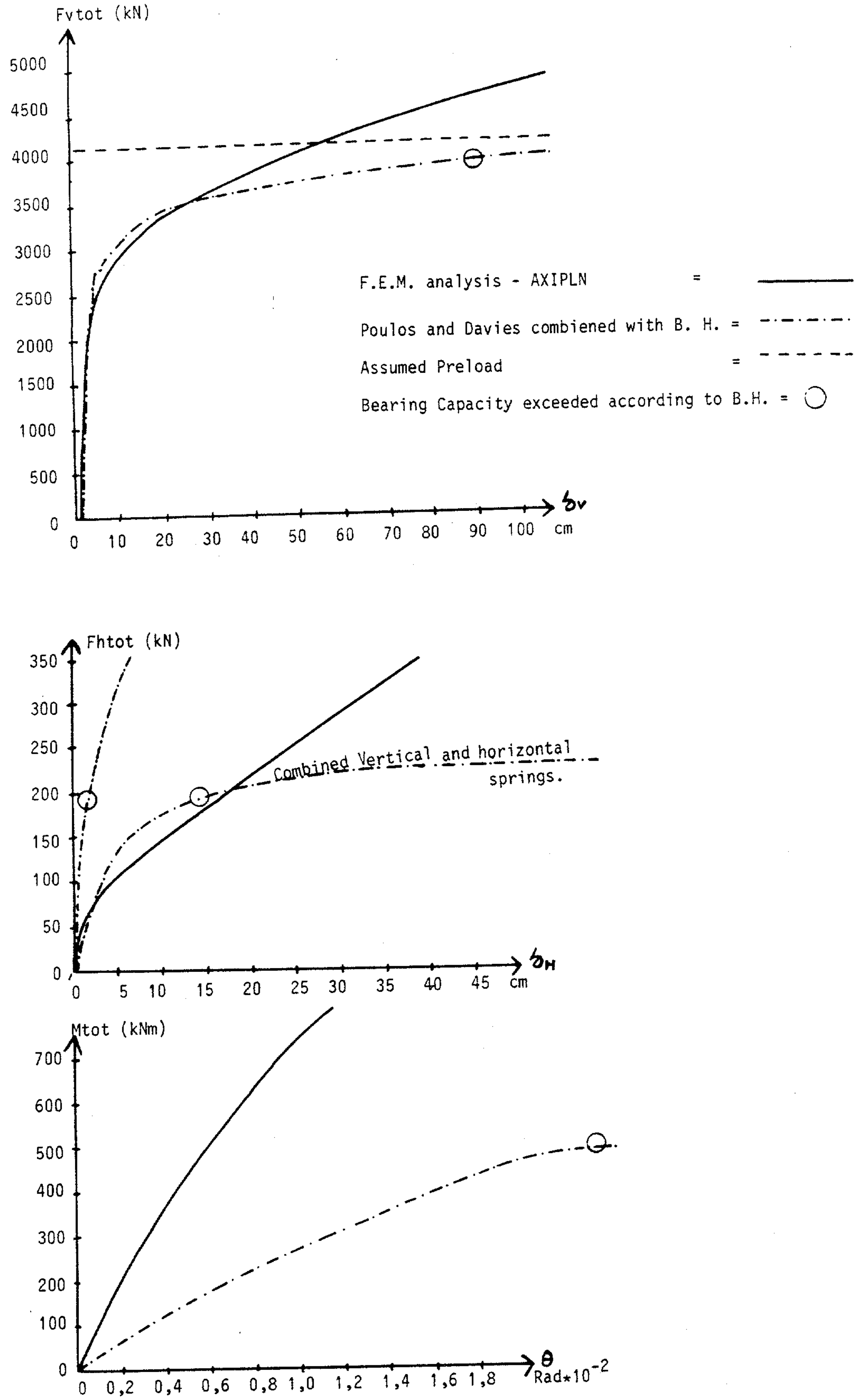


Figure 4.4 Comparison between hyperbolic model and analysis with nonlinear FEM program AXIPLN

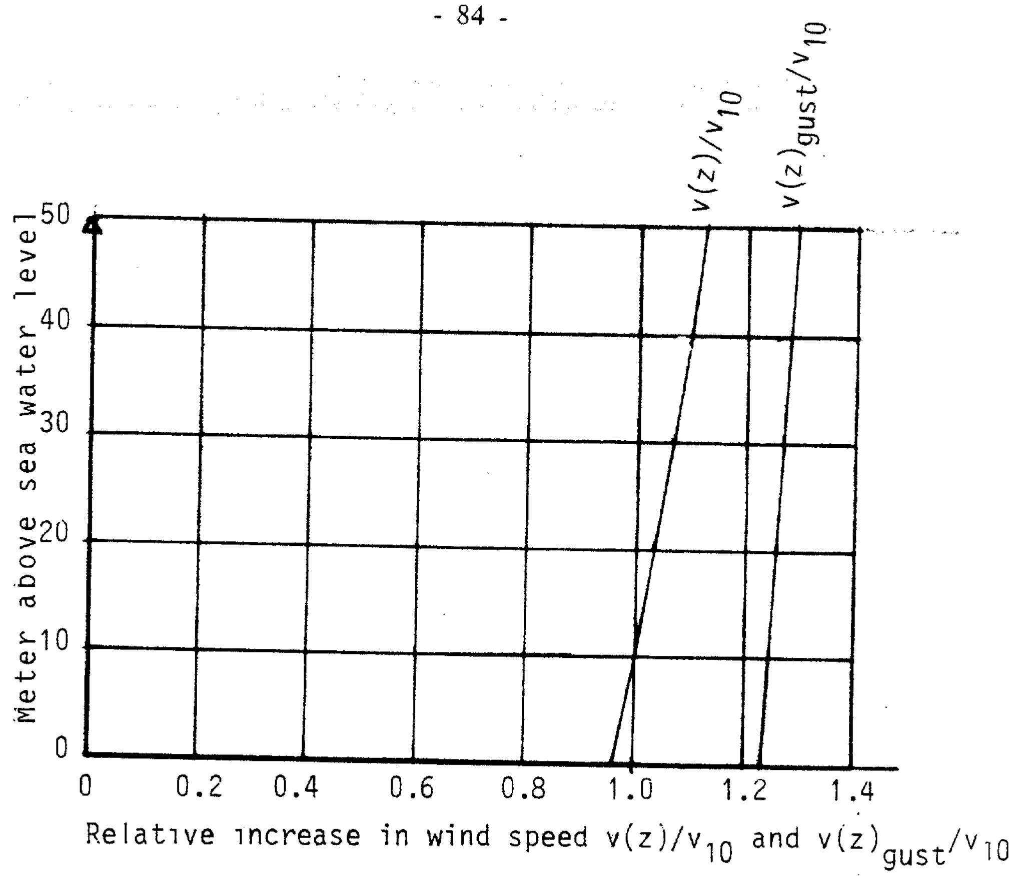


Figure 5.1 Wind velocity vs height

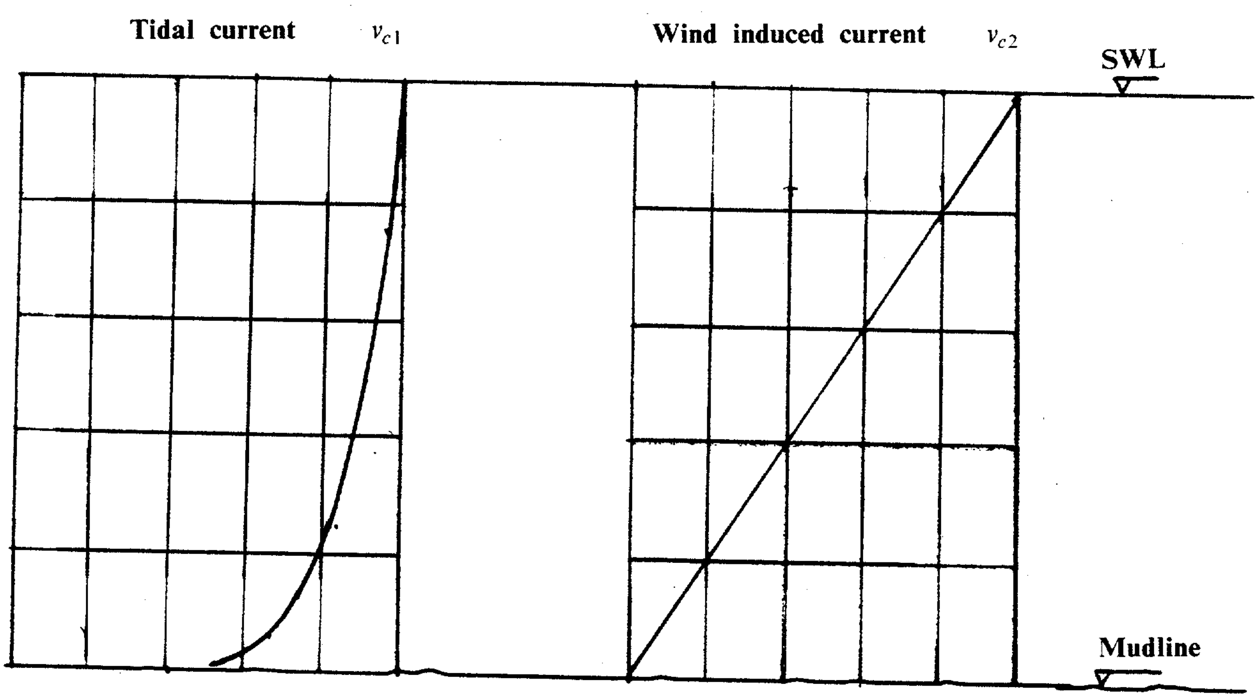


Figure 5.2 Distribution of tidal induced and wind generated current vs depth

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TECHNICAL REPORT

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VERITAS Report No.	83-0905
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Project No.	233103
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Approved by	O. Furnes Chief Research Engineer
Client/Sponsor of project	Several Companies, see Preface
Client/Sponsor ref.	
Work carried out by	Lars A. Kristiansen Tore J. Kvalstad Knut O. Ronold
Reporters sign.	Lars A. Kristiansen Knut Olav Ronold

Summary

This technical report presents the results of a comprehensive parametric study of the foundation stability and overall behaviour of jackup platforms. Emphasis has been laid on evaluation of relative effects of variations in governing soil and geometry parameters, and critical conditions and failure modes have been determined. Mat-supported platforms as well as platforms with independent legs have been addressed, and the study has been based on theory described in the report for PP2 of the project.

4 Indexing terms

- JACKUP PLATFORMS
- FOUNDATION STABILITY
- SOIL-STRUCTURE INTERACTION
- PARAMETRIC STUDY

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October 23, 1984	1	121

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PREFACE

This report presents the work carried out within Part Project 3 (PP3) of the joint industry project "Foundation Stability of Jackup Platforms". The project is at the time of writing sponsored by the following 14 companies:

Amoco Norway Oil Company
Arco Norway Inc.
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Chevron Oil Field Research Company
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The work presented herein is the result of a study where the main work was carried out during February to December 1983 and covers a comprehensive parameter study for determination of the relative effects of variations in important parameters on the foundation stability and overall behaviour of jackup platforms. Methods and analysis tools reviewed and presented in Part Project 2 have been applied for the performance of the study.

EXECUTIVE SUMMARY

GENERAL

The purpose of Part Project 3 (PP3) was to determine the relative effect of different parameters on the foundation stability and overall behaviour of a jackup platform. This has been achieved through a comprehensive parametric study where a range of mat-supported and independent-leg-supported jackup platforms has been analyzed regarding

- penetration depth of mats and spudcans
- pull-out resistance
- lateral, vertical and overturning stability of the jackup foundations
- soil-structure interaction

A broad range of environmental conditions has been covered with water depths ranging from 30 to 100 m. The work has concentrated on stability evaluations during installation and under extreme weather conditions.

The soil conditions have in general been varied over a broad range, however, soil conditions that were likely to be critical have been examined in more detail.

The parametric study has to a large extent been carried out by means of the computer programs SAM (Stability Analysis of Mats) and SAIL (Stability Analysis of Independent Legs). These two programs represent the primary analysis tools, while separate specialized geotechnical programs as well as hand calculations have been applied for evaluation of special effects.

It must be emphasized that a number of assumptions had to be made in order to reduce the possible number of variations in the parametric study. These assumptions may at least to some extent limit the general validity of the findings. Furthermore, the approach was to vary one parameter at a time in order to evaluate its relative effect on the foundation response (i.e. safety factors, penetration depths etc.). Due to non-linear effects and coupling between some of these parameters, a simultaneous change of parameters may be more critical than can be seen from the results of the parametric

study. Nevertheless, it is felt that the conclusions summarized in the following are of general validity for a relatively wide range of boundary conditions.

JACKUP PLATFORMS WITH INDEPENDENT LEGS

Penetration of independent legs

The penetration depth of spudcans is governed by the shear strength of the soil, the weight of the platform with preload, and the geometry of the spudcans. Of these parameters the undrained shear strength of the soil and its variation with depth is by far the most uncertain. The use of different bearing capacity formulae did not reveal any significant influence on the results. A number of design charts for evaluation of penetration depth in soft clays have been developed (Appendix B).

The stability of the hole generated around a deeply penetrating spudcan is found to be a parameter of concern, as caving-in of the side wall may lead to a sudden additional load on top of the spudcan and thus a sudden additional penetration.

Punch-through failure is known to be critical. A broad study of this problem was carried out and showed that the top layer thickness is a very important parameter together with the undrained shear strength of the underlying soft clay.

Stability under extreme loading

The study concludes that jackup platforms with independent legs will have a sufficient *safety against sliding failure* for all practical cases provided adequate preloading is performed. Only one case may cause problems; a soft clay of limited thickness over a hard soil stratum combined with flatbased spudcans may be critical under extreme environmental loading.

The *safety against bearing capacity failure* (i.e. safety against additional penetration) during extreme loading conditions is heavily dependent on the preloading level and the soil conditions. The most critical soil conditions are thin strata with sand or stiff clay overlying a weaker zone of clay, i.e. the punch-through situation, but soft clays may as well be critical as a considerable extra penetration may develop in cases where insufficient preloading has been carried out. The effect of consolidation is considered to be

small and requires in any case a considerable amount of time after installation and should thus not be relied upon in this connection.

MAT-SUPPORTED JACKUP PLATFORMS

Penetration of mats due to platform weight

The result of the parametric study show that mat penetration will only take place in very soft underconsolidated soils, and even here the penetration depth will be limited to a few feet.

The consolidation settlements in soft clays have been estimated to be in the range 0.3 to 0.6 m for a mat with an average contact stress of 20 kPa. However, for normal drilling operations only a limited part of this settlement will occur due to limited time, say 2-3 months per location.

Skirt penetration problems are predicted for stiff clays and dense sands, and in stiff clays pull-out of skirts may represent a problem.

Stability under extreme loading

Sliding failure is generally found to be the most critical failure mode for A-shaped mats similar to the ones investigated. The shear strength of the soil in the first meter below mudline is controlling the factor of safety, and it is clearly shown that sliding represents a problem for mat-supported jackup platforms operating on very soft soils and under rough weather conditions. Uncontrollably large lateral displacements may develop.

Local failure under the most stressed edge of a platform may develop and is a failure mode that has to be considered in deeper waters where the overturning moment due to environmental forces will develop high edge stresses. It is, however, shown that through sinkage of parts of the mat into the soil a stabilizing moment will develop due to increased bearing capacity and buoyancy within the soil mass. Tilt, but not complete failure, is predicted.

In shallow waters and for soft clays showing little increase in strength with depth below mudline the *deepseated failure* mode will be the most critical. Complete overturning and capsize are predicted to be the result of this type of failure.

1. INTRODUCTION

It has been the aim of PP3 to carry out and present a comprehensive parameter study of the foundation stability and overall behaviour of jackup platforms with emphasis laid on evaluation of relative effects of variations in governing soil and geometry parameters.

The foundation stability of jackup platforms has been addressed for various important situations covering installation, operational phase and extreme loading conditions as well as pull-out. Independent-leg-supported jackup platforms have naturally been distinguished from mat-supported jackup platforms in the parameter study, and special topics like effects of consolidation and cyclic loading on soil strength have also been dealt with, emphasizing that soil parameters are not necessarily always stationary parameters.

Discrepancies between tentative and final contents of the parameter study are related to our findings during the project with respect to which topics are the most important for the foundation stability. Topics which have been assessed to be of great importance may have been dealt with in more detail than originally planned. And other topics which originally were intended to be studied may have appeared to be impossible to quantify or handle in a computational way, like e.g. scour, and have therefore not been included. Carrying out the parameter study we have thus had to be selective, concentrating on topics which are assessed to be most critical and important for the foundation stability of jackup platforms.

The analyses carried out as a significant part of the parameter study are based on theories and analysis methods which are presented and described in the PP2 report. Use of computer programs has formed a substantial means for these analyses with the SAIL and SAM programs being the predominating tools for most of the analyses and with programs like AXIPLN, OCEAN2 and RELOC applied for more special calculations. For description of all applied computer programs, reference is made to the appendices of the PP2 report.

No special case study has been performed as no case record has been available to the project.

2. DEFINITION OF PARAMETER RANGES TO BE ANALYZED

2.1 INDEPENDENT-LEG-SUPPORTED JACKUP PLATFORMS

2.1.1 Variation in Structural and Foundation Geometry

Some jackup rigs are designed for particular locations with special considerations taken regarding overturning stability, capacity for preload and variable load. The three jackup models selected for the parameter studies within this project are however all designed for world-wide operation, and these three models are briefly outlined in Figs. 2.1-2.3.

The three selected jackup models are used for the parameter studies for independent-leg-supported jackup platforms, and basic dimensions for the corresponding spudcan foundation are given in Table 2.1, whereas a sketch of the basic spudcan shape is shown in Fig. 2.4. Fig. 2.5 indicates spudcan sizes for the three jackup models, and Fig. 2.6 shows the selected plane frame model for analysis of independent-leg-supported jackup platforms with dimension, load and stiffness notations marked out.

2.1.2 Variation in Gravity and Environmental Loads

Reference is made to Table 2.2. The platform weight includes weights of fixed equipment and submerged legs with water-filled spudcans. Ballast water for preloading and variable load are not included.

The variable load for all three platforms equals 20% of the platform weight and is not varied. The preload capacity (ballast tanks capacities) is set equal to 30% of the platform weight for Model No. 1, 40% for Model No. 2 and 50% for Model No. 3. The variation in platform weight is 20% for all models, while the variation in preload weight is 20% for Model No. 1 and 50% for Model No. 2 and No. 3.

The following basic wave periods are selected for the three models:

Model	Basic wave period
No. 1	T=13 sec.
No. 2	T=15 sec.
No. 3	T=16 sec.

The basic value for maximum wind speed is set to 50 m/s for all models.

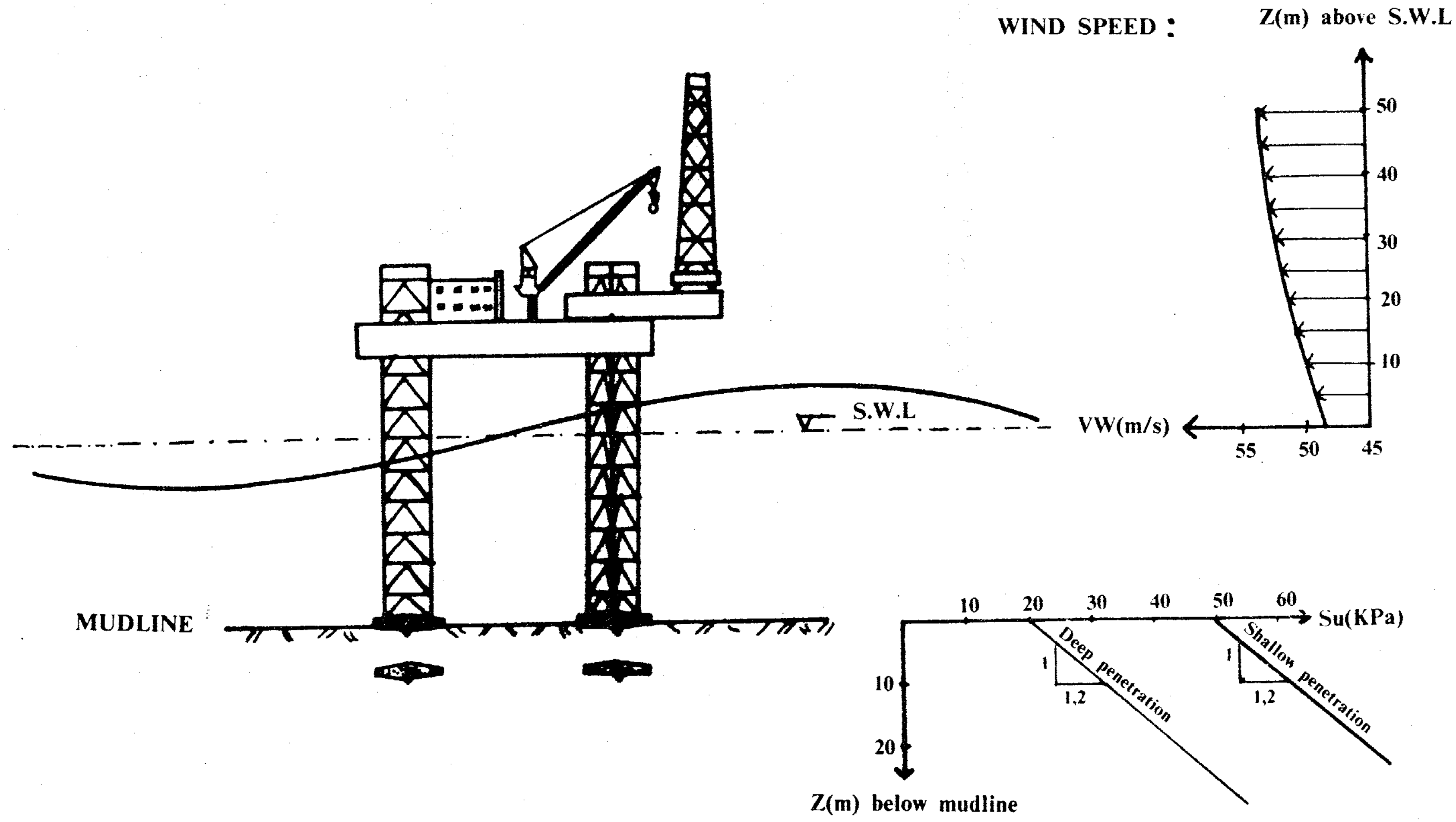


Fig. 2.1 Jackup platform Model No. 1 for parametric study.

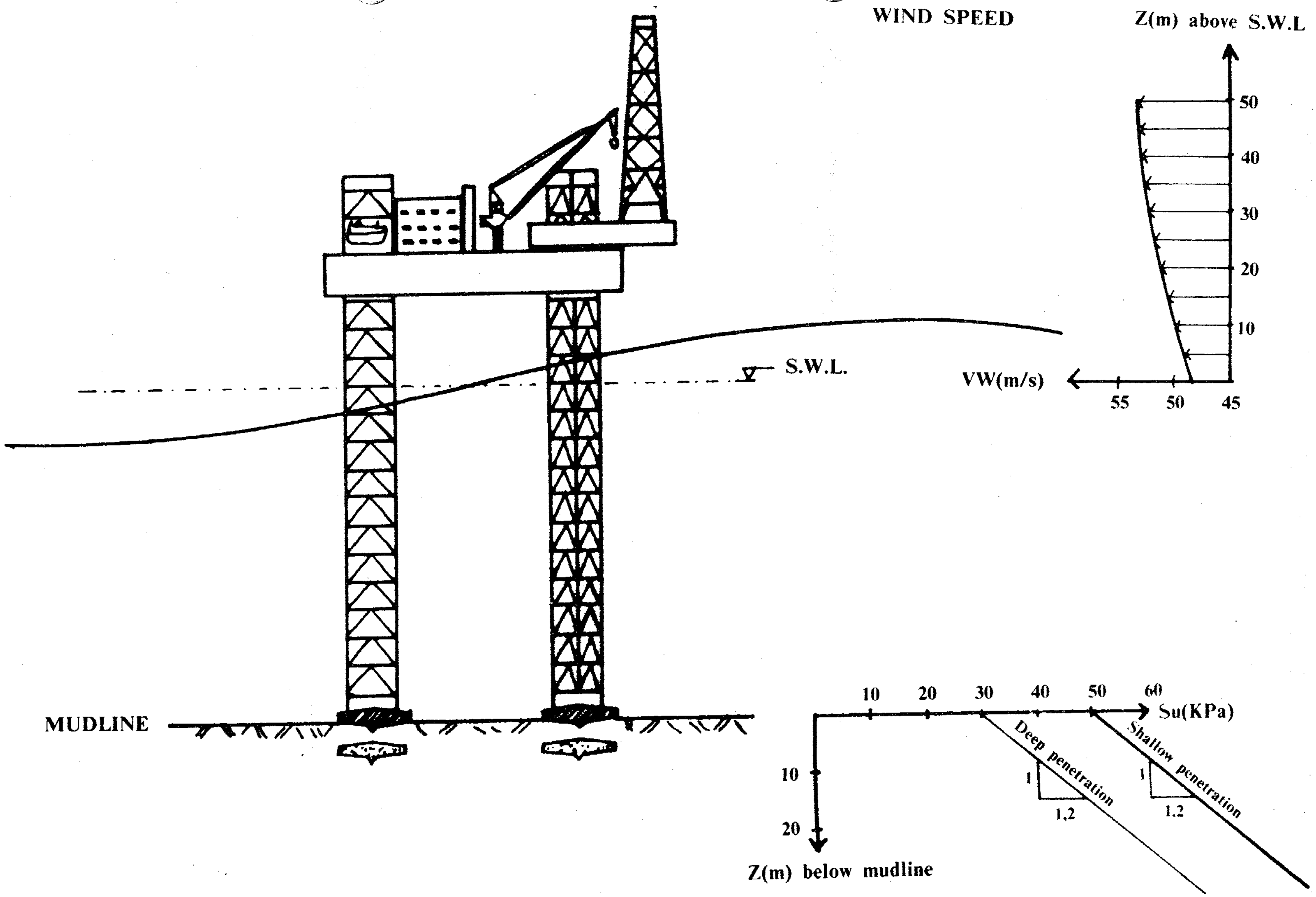


Fig. 2.2 Jackup platform Model No. 2 for parametric study.

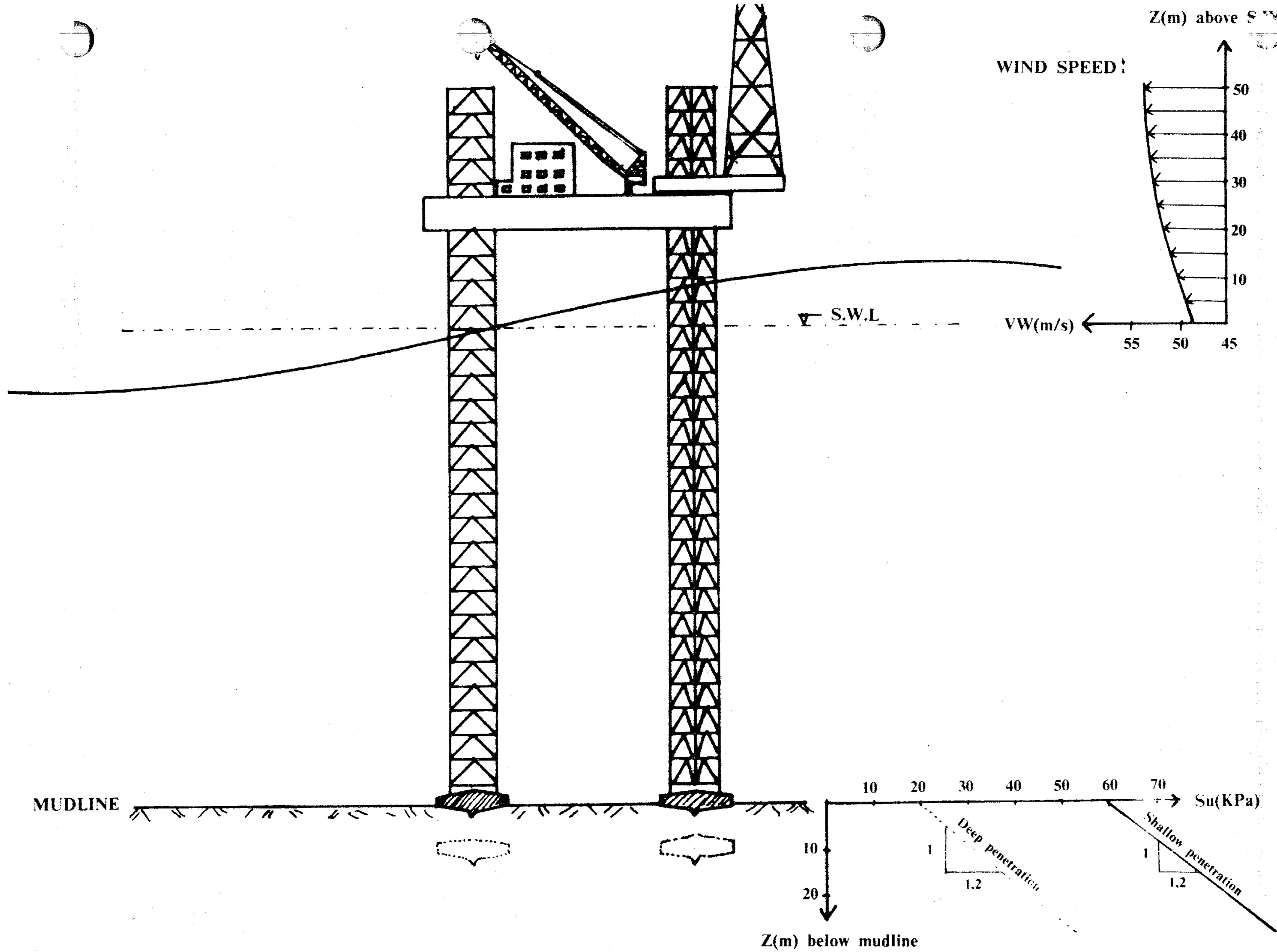


Fig. 2.3 Jackup platform Model No. 3 for parametric study.

Table 2.1 Basic parameters for three different jackup platform models.

	Model No. 1	Model No. 2	Model No. 3
Wave data:			
Wave Period, T=	13 sec	15 sec	16 sec
Wave Length, WL=	196 m	299 m	373 m
Max.Waveheight, HWMAX=	15 m	20 m	25 m
Footing data:			
(See figure of spud-can below).			
Top height, HTOP=	0.5 m	0.6 m	0.75 m
Height of vertical spud-can wall, HSID=	1.35 m	1.6 m	2.0 m
Height of cone, HCON=	0.5 m	0.5 m	0.75 m
Height of tip, HTIP=	0.5 m	0.6 m	1.0 m
Total height, HTOT=	2.85 m	3.3 m	4.5 m
Tip diameter, DTIP=	1.0 m	1.25 m	1.75 m
Total diameter, DTOT=	11.0 m	13.0 m	15.0 m

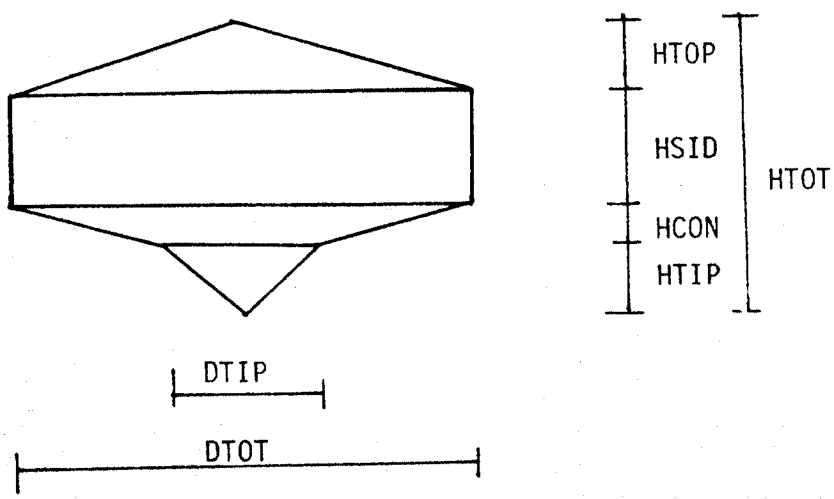


Fig. 2.4 Basic spudcan shape.

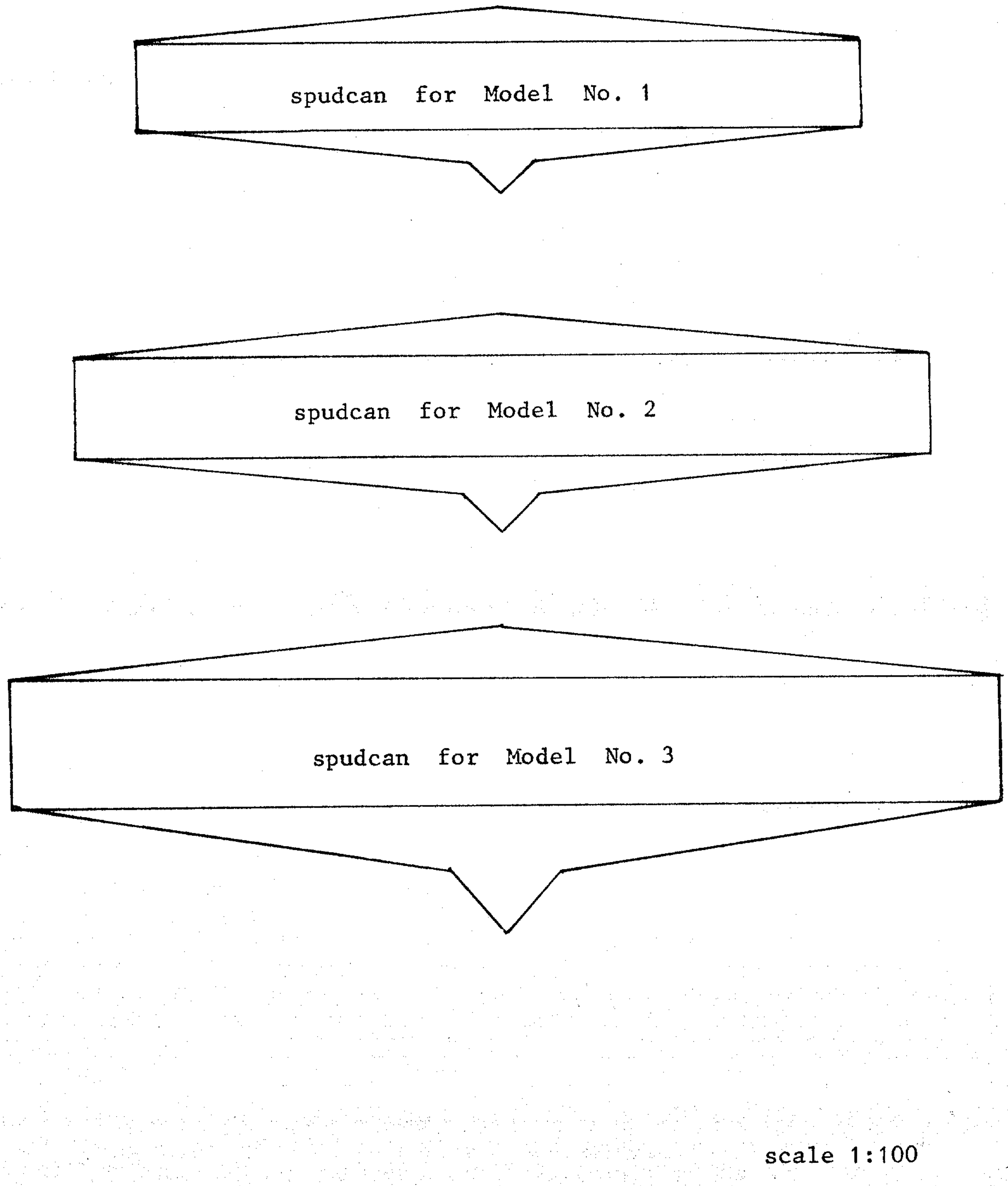


Fig. 2.5 Sketch of spudcans for platform Models Nos. 1, 2 and 3.

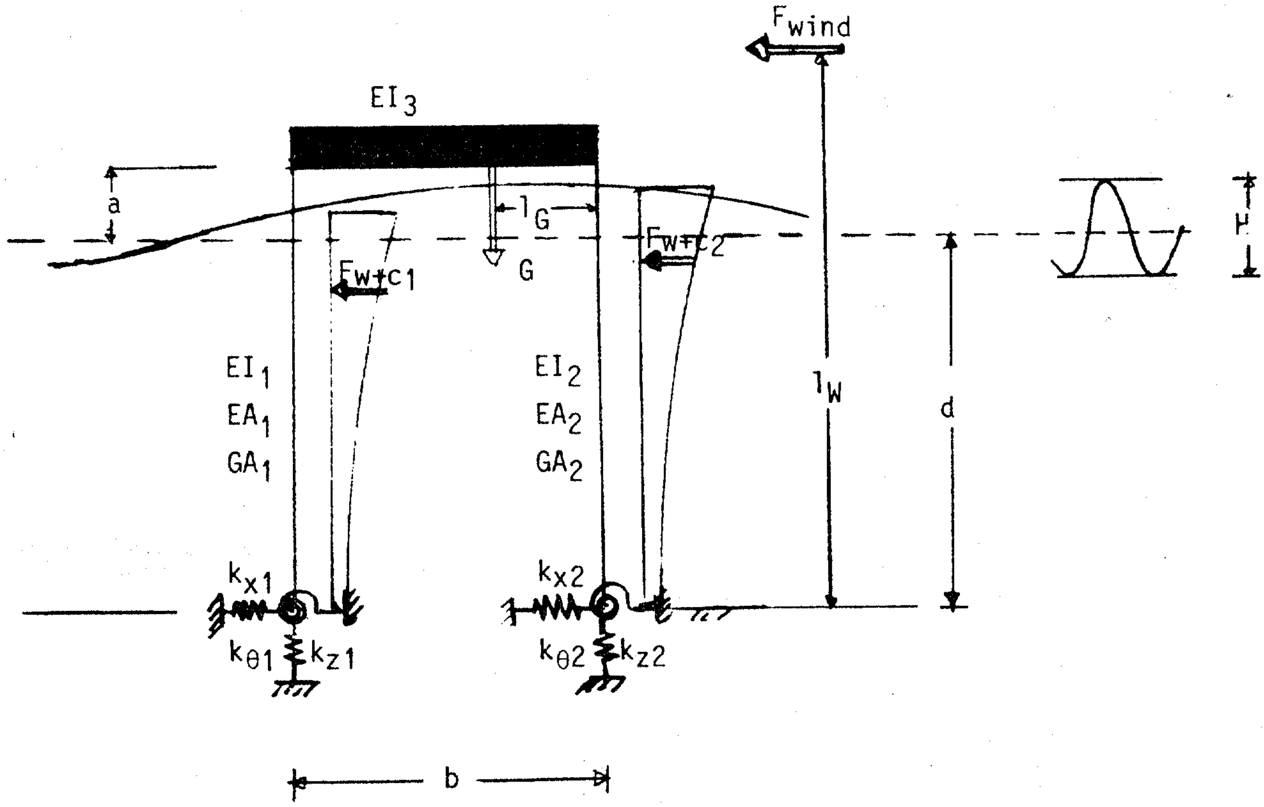


Fig. 2.6 Selected plane frame model for analysis of independent-leg-supported jackup structure with basic dimensions, loads and stiffnesses.

Table 2.2 Wave and footing data for basic cases for parametric study of three different jackup platform models.

		MODEL 1	MODEL 2	MODEL 3	Variation in parameter study
Water depth	d(m)	30.0	60.0	100.0	-
Max. Wave height	H(m)	15.0	20.0	25.0	$\pm$ 20%
Wave period range	T(s)	9-17	10-20	11-20	
Max. Wind speed (1 min. sustained)	v_w(m/s)	50.0	50.0	50.0	$\pm$ 10%
Max current speed (tidal + wind induced)	v_c(m/s)	1.0	1.0	1.0	$\pm$ 100%
Air gap (assumed 0.8H)	a(m)	12.0	16.0	20.0	-
Distance between fore and aft legs	b(m)	33.0	37.0	47.0	$\pm$ 10%
Platform weight	WG(MN)	60	90	120	$\pm$ 20%
Leg weight	WGLEG(MN)	4.5	6.5	9.75	-
Ballast Water for preloading	PRE(MN)	18	36	60	$\pm$ 20-50%
Variable load	WGV(MN)	12	18	24	-
Projected area (hull) A hull	$A_w(m^2)$	250	400	530	
Lever arm for wind force (d + a + 12m)	l_w(m)	54.0	88.0	132.0	$\pm$ 5%
Number of legs		3 or 4	3 or 4	3	
Drag coefficient	C_D	0.7	0.7	0.7	-
Inertia coefficient	C_I	2	2	2	-
Effective diameter of leg for drag	D_D(m)	5.0	5.0	5.0	$\pm$ 20%
Effective diameter of leg for inertia	D_i(m)	2.0	2.0	2.0	$\pm$ 20%
Leg stiffness, bending	EI (MNm^2)	$0.8 \cdot 10^6$	$1.3 \cdot 10^6$	$2.0 \cdot 10^6$	$\pm$ 20%
Leg stiffness, axial	EA (MN)	$1.0 \cdot 10^5$	$1.3 \cdot 10^5$	$2.2 \cdot 10^5$	$\pm$ 20%
Leg stiffness, shear	GA (MN)	$2.0 \cdot 10^3$	$2.0 \cdot 10^3$	$2.0 \cdot 10^3$	$\pm$ 20%

2.1.3 Variation in Soil Parameters and Stratification

Soil profiles and soil parameters are selected appropriately from case to case in order to obtain interesting parameters for the various parameter studies, i.e. profiles and parameters which for each considered case give critical conditions with factor of safety close to 1.0.

The soil profiles and soil parameters themselves are to some extent presented in connection with the respective parameter studies and are therefore not reproduced here. However, the soil profiles used for the general parameter studies for independent-leg-supported jackup platforms in Chapter 3 and 5 of this report are all based on the three idealized soil profiles that appear from Appendix B of the PP2 report. These profiles are presented in Figs. 2.7 to 2.9 with basic spudcan locations marked out. Penetration curves, i.e. curves of penetration depth vs. bearing capacity, are presented for the different considered soil profiles in Figs. 2.10-2.12.

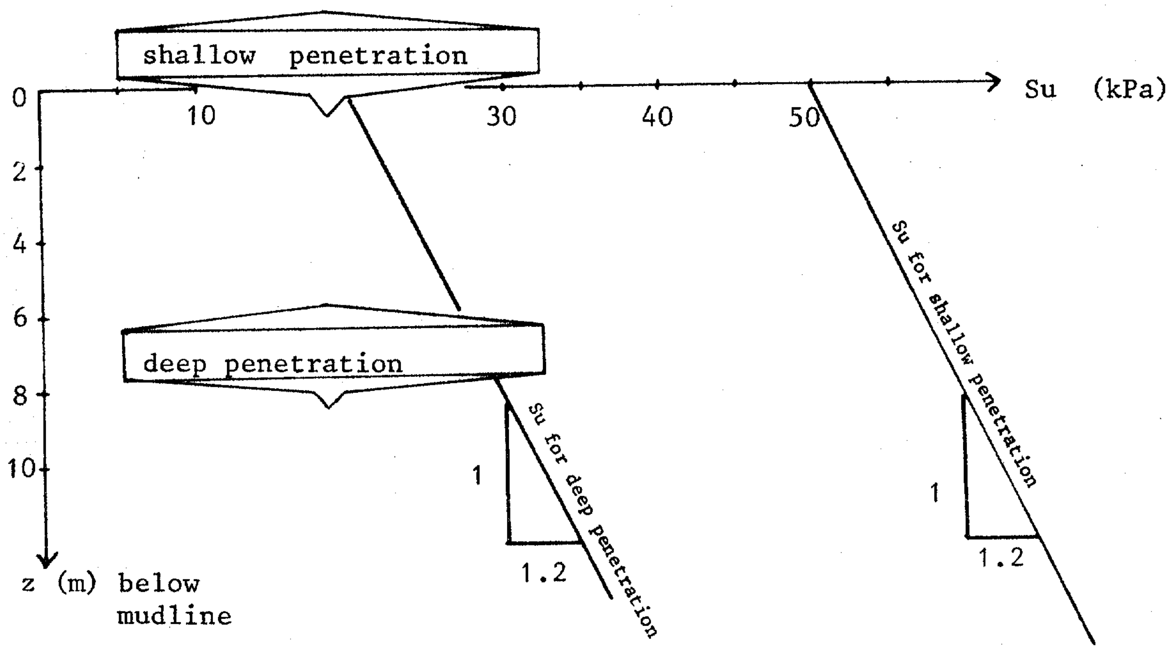


Fig. 2.7 Soil profiles and spudcans for jackup Model No. 1, shallow and deep penetration.

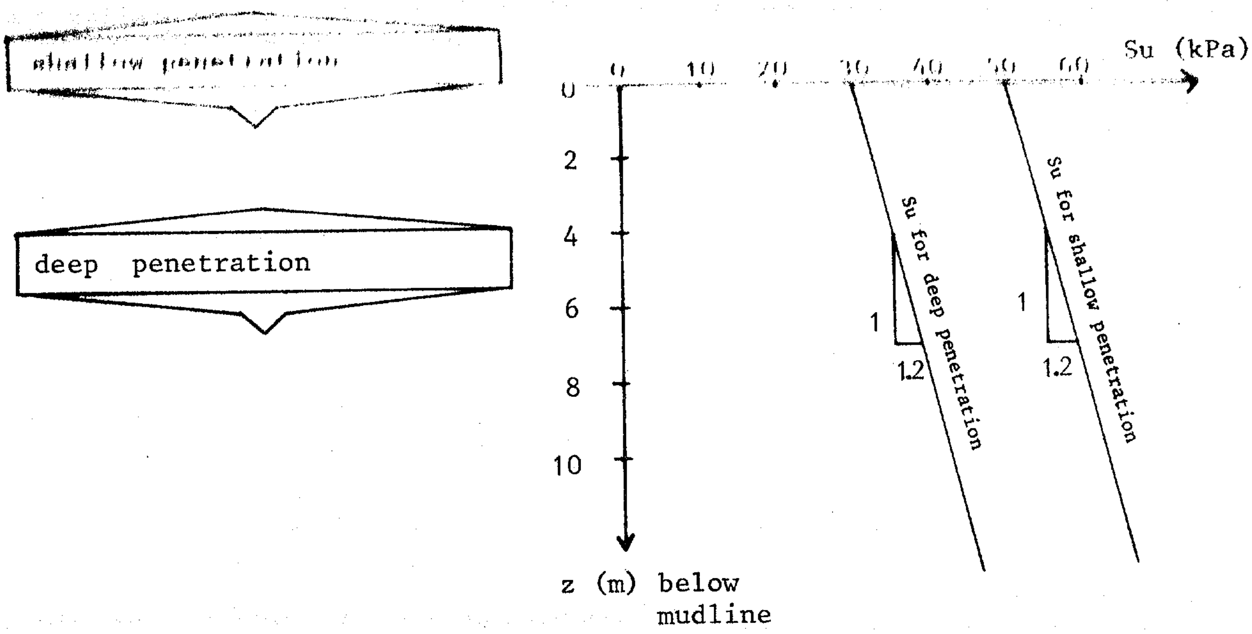


Fig. 2.8 Soil profiles and spudcans for jackup Model No. 2, shallow and deep penetration.

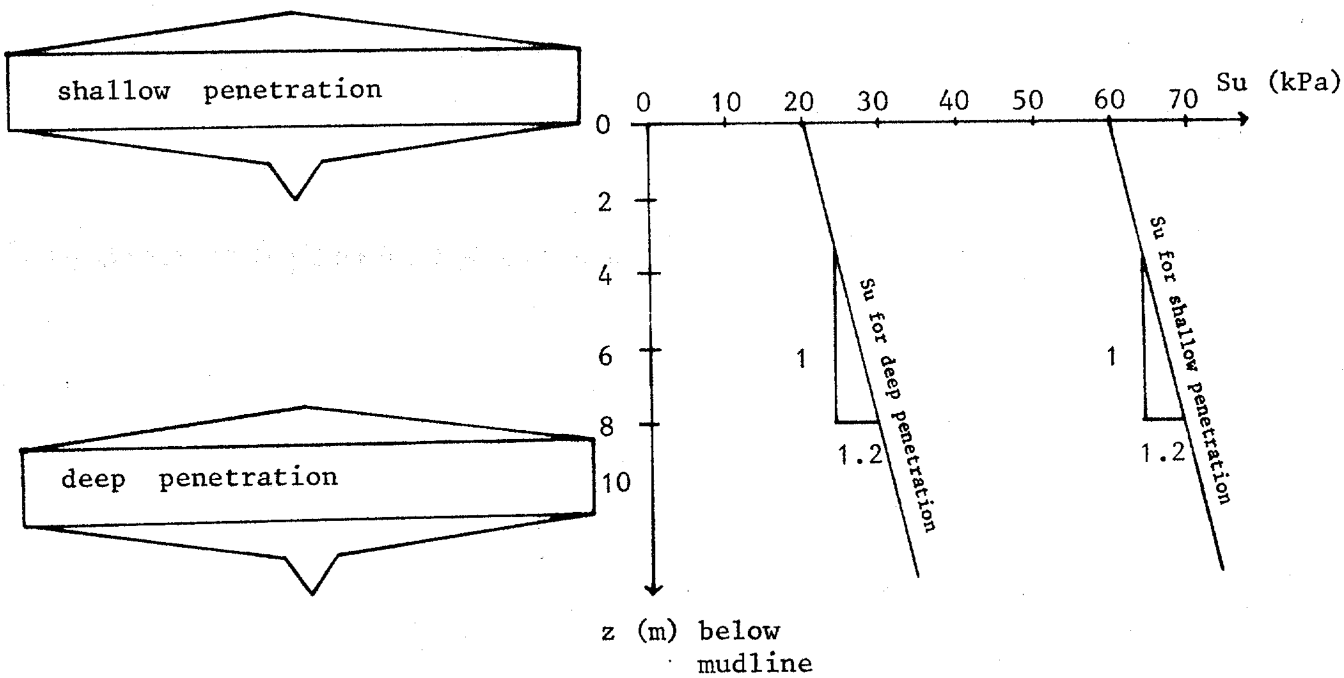


Fig. 2.9 Soil profiles and spudcans for jackup Model No. 3, shallow and deep penetration.

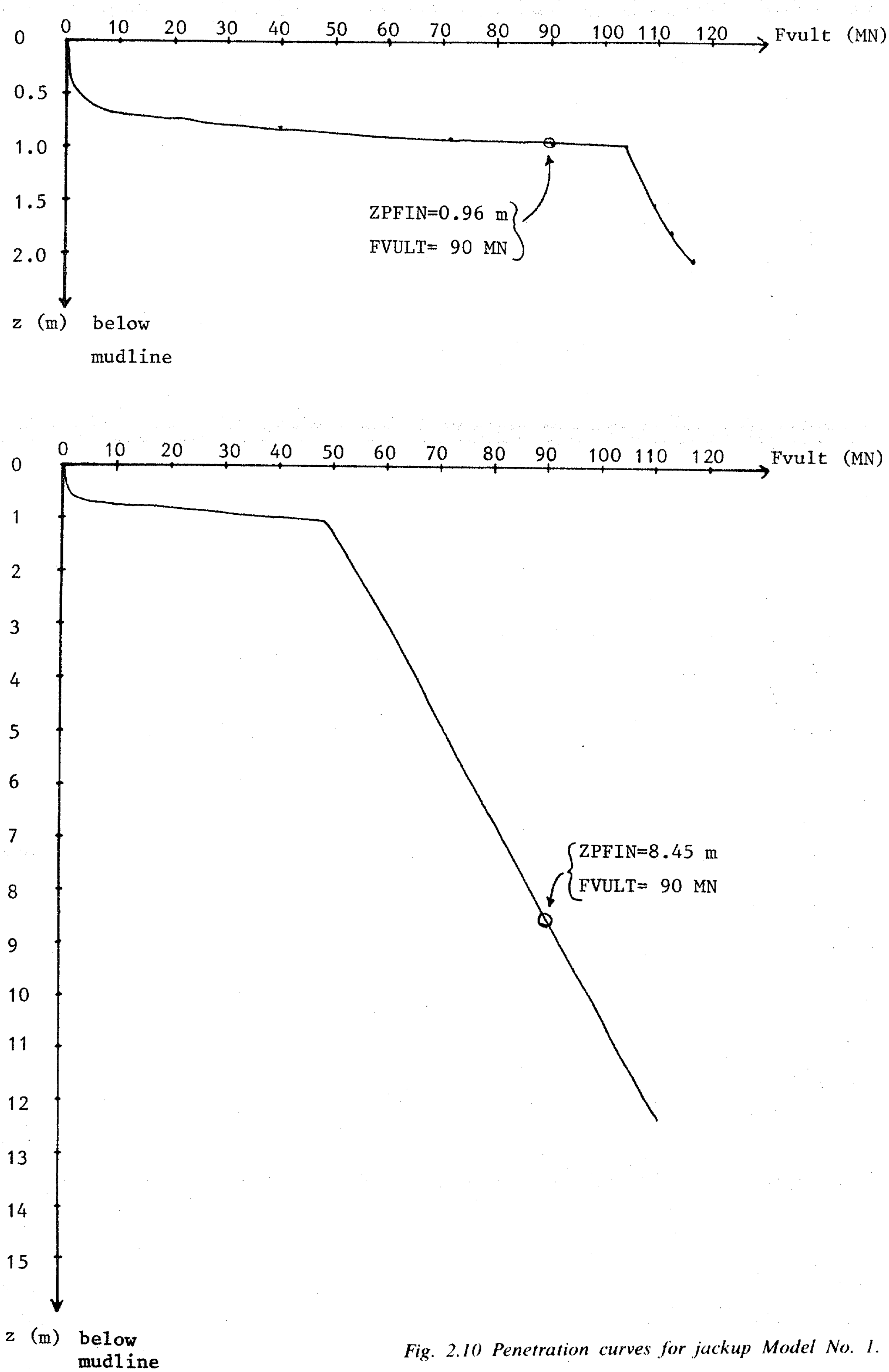
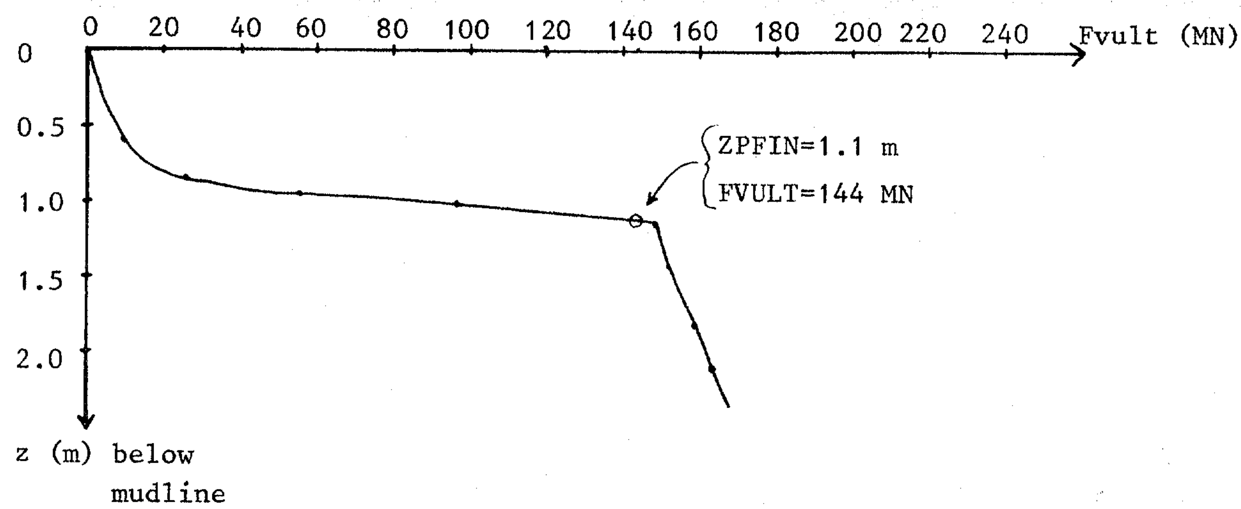


Fig. 2.10 Penetration curves for jackup Model No. 1.



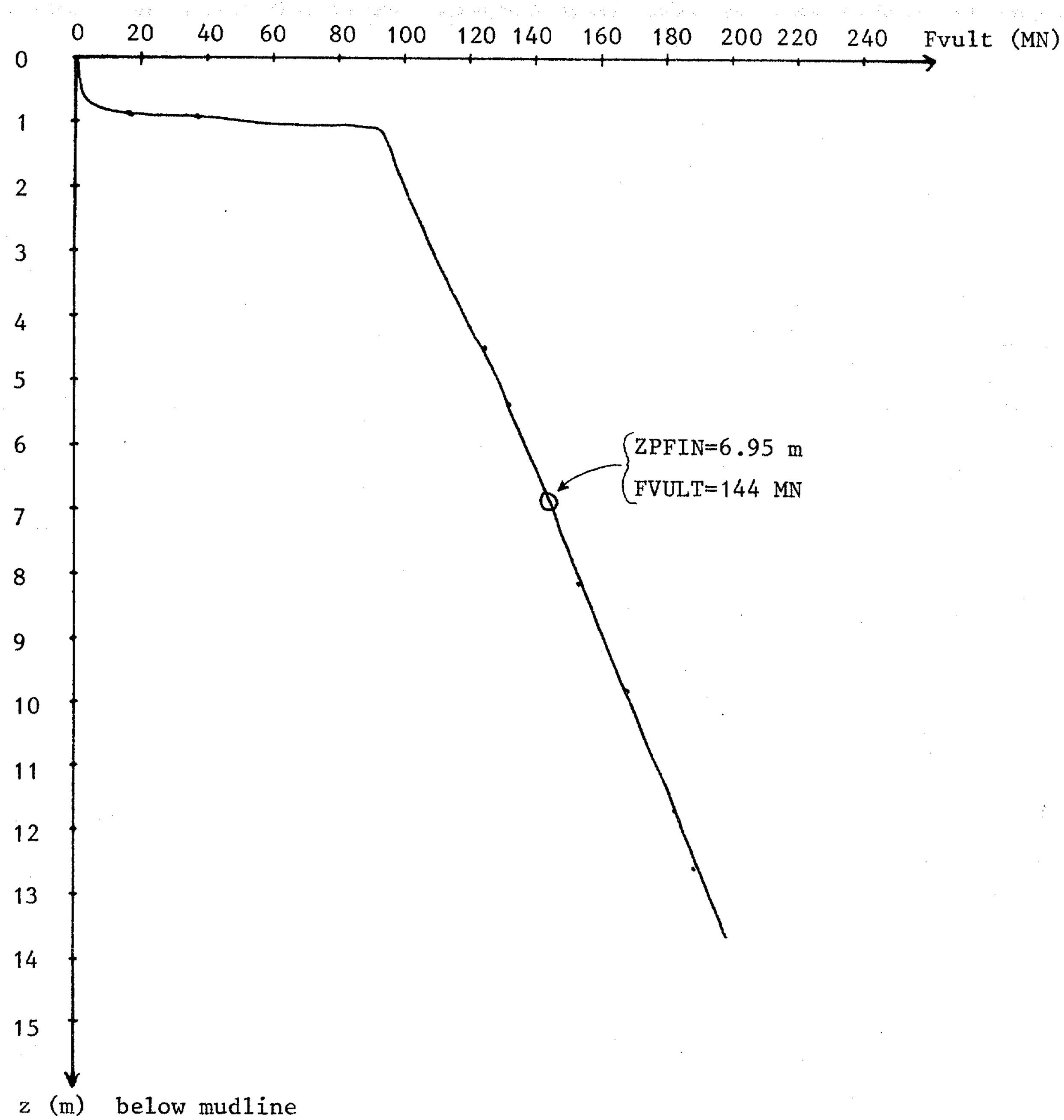
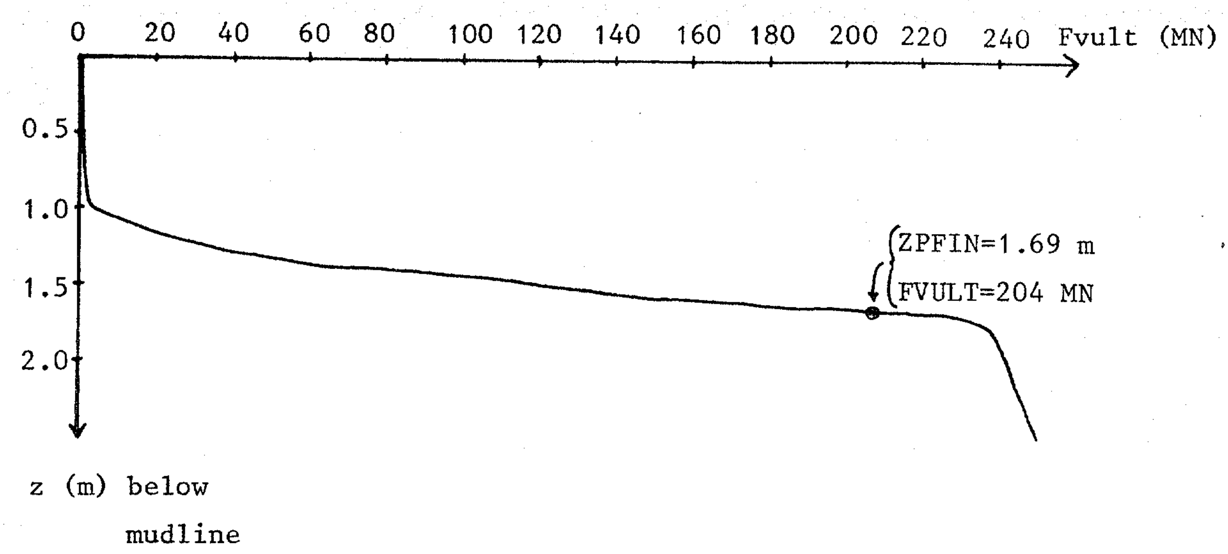


Fig. 2.11 Penetration curves for jackup Model No. 2.



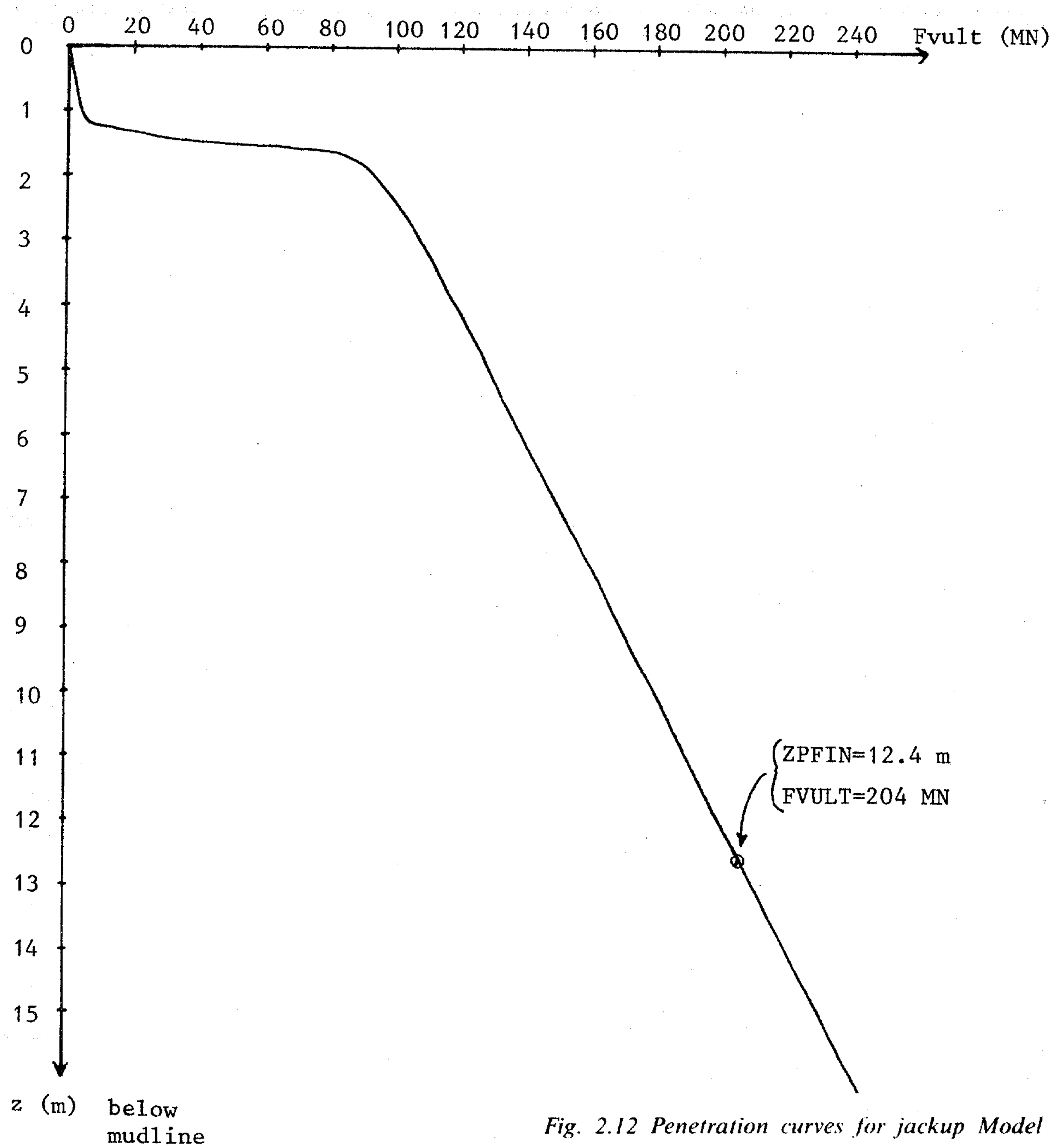


Fig. 2.12 Penetration curves for jackup Model No. 3.

2.2 MAT-SUPPORTED JACKUP PLATFORMS

2.2.1 Variation in Structural and Foundation Geometry

Three different mat supported models were selected for the parametric evaluation. As can be seen from Figure 2.13 water depths of 30 m, 60 m and 100 m were selected. All models were equipped with cylindrical legs and A-shaped mats with slot and hole.

As the mat geometry is rather complex, the number of variations had to be limited. In addition to the basic versions shown in Figure 2.13 a small and a large mat alternative was investigated. The geometries of the small and the large mats for the three models are shown in Figure 2.14. The basic differences pertain to the area and the width of the fingers. Furthermore, the height of the mats has been varied with +/- 0.5 m to evaluate the effect of the increased horizontal wave force on the mat, and finally the skirt height has been varied with +/- 0.2 m to evaluate the effect on the sliding resistance. See also Table 2.3.

2.2.2 Variation in Gravity and Environmental Loads

The submerged weights of the platform models were chosen to 30 MN, 40 MN and 50 MN for the three models, respectively, and for each model the weight was varied with +/- 5 MN. No preloading capabilities were considered for the mat supported platforms.

The parameters controlling the total environmental loads were varied as outlined in Table 2.3. The "basic case" periods of 13, 14 and 16 seconds were selected based on some trial runs and give minimum or close to minimum foundation safety factors.

The current speed is composed of two components, the tidal induced speed and the wind induced speed, respectively. At sea water level the same speed was chosen for these two current components. The distributions with depth have been outlined in the previous report on PP2.

2.2.3 Variation in Soil Parameters and Stratification

The critical soil conditions regarding foundation stability of mat-supported jackup platforms is generally soft clay. The mat-supported units are specifically designed to cope with soft to very soft soils where considerable penetration depths are inevitable for

independent-leg-supported jackup platforms.

Accordingly, most emphasis has been put on the evaluation of foundation stability of mat-supported units on soft clays. In order to allow the three models to be compared directly the same soil strength profile was chosen for all models as the "basic case". This profile starts with an undrained shear strength of 10 kPa at mudline and increases with 1.2 kPa per m below mudline. Harder clays and sands represent no stability problem with respect to sliding and bearing capacity and have not been evaluated in this respect.

Erosion of sandy sea beds under combined current and wave action represents a severe hazard to mat-supported platforms in shallow waters. Erosion predictions can hardly be made with much confidence and this area has not been treated within the study.

Skirt penetration problems and uneven distribution of soil reaction stresses against the mat may develop on locations with hard clays, sands or gravel. These problems are treated on a more general basis and no specific soil profile has been selected as a basic case.

2.3 UNEVENNESS AND SLOPE OF SEABED

The seabed is usually flat, but local unevennesses may still be present. However, local unevennesses in soft clay are not of any interest, while local unevennesses in sand may be of importance for mat foundations. The effect of an uneven sea bed on the soil reaction forces against mats and the effect of a sloping seabed on the stability of a mat are both treated on a general basis.

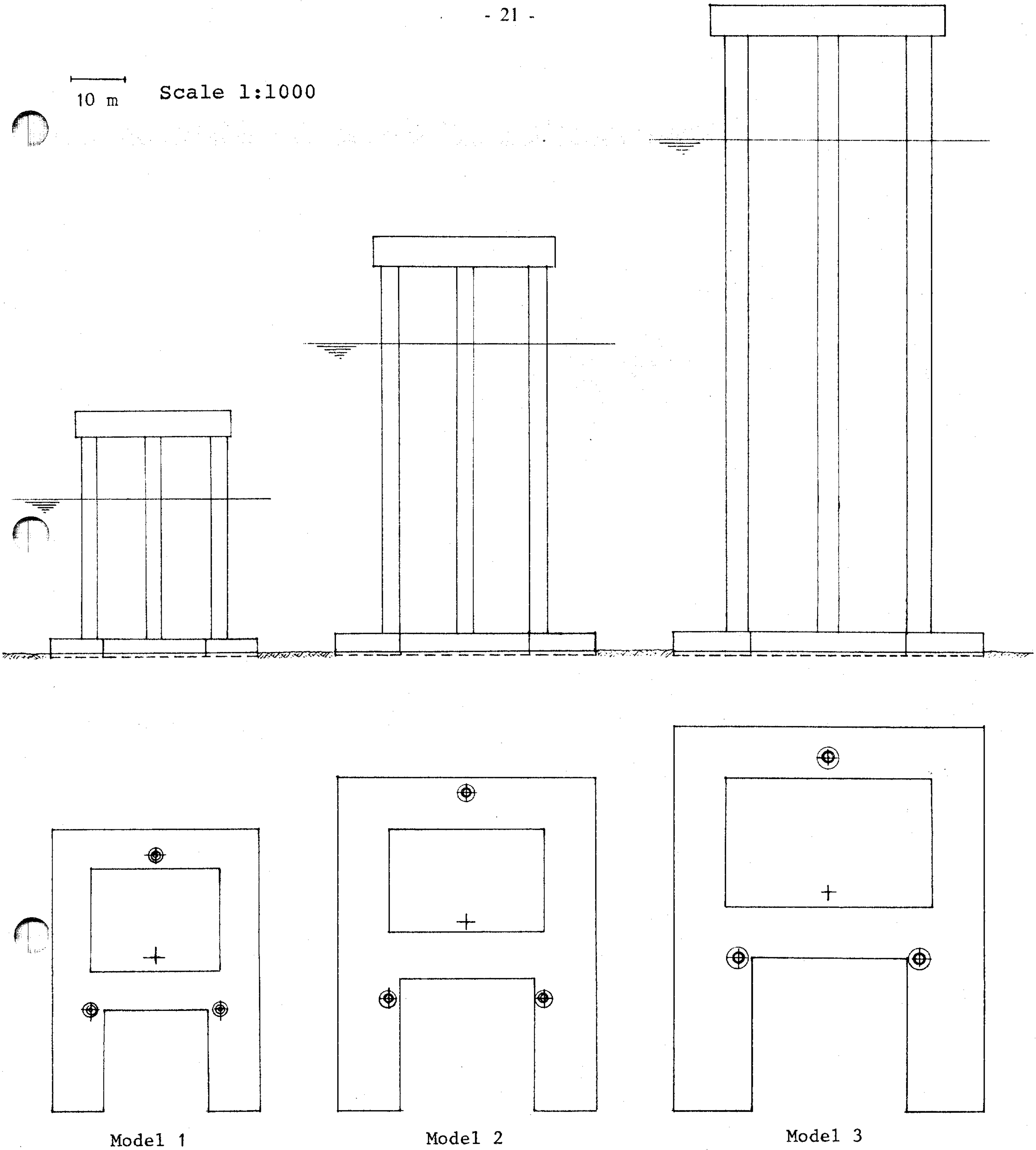


Fig. 2.13 Geometry and dimensions of platform models selected for the parametric evaluation of mat supported platforms.

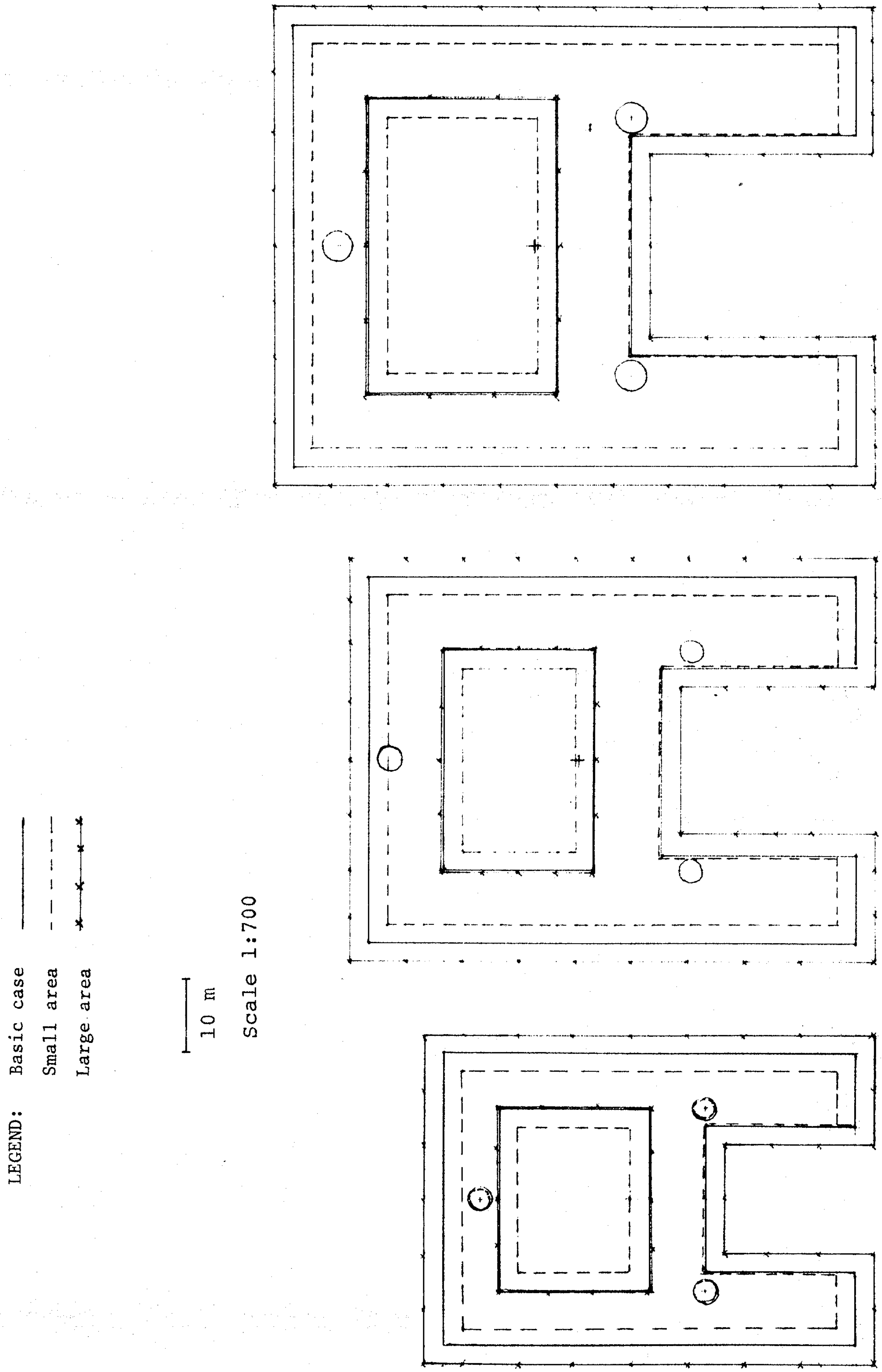


Fig. 2.14 Variation of mat geometry. Small, basic and large mat areas.

Table 2.3 Parameters and parameter variations for the three considered mat-supported jackup platform models.

PARAMETERS	MODEL 1		MODEL 2		MODEL 3	
	Basic Case	Variation	Basic Case	Variation	Basic Case	Variation
PLATFORM WEIGHT						
Subm. pltf. weight, kN	30000	25000 35000	40000	35000 45000	50000	45000 55000
ENVIRONMENT						
Water depth, m	30	-	60	-	100	-
Wave height, m	15	12 18	20	16 24	25	20 30
Wave period, s	13	9 11 15 17	14	10 12 16 18 20	16	10 12 14 18 20
Current speed, m/s	0.5+0.5	0.0+0.0 1.0+1.0	0.5+0.5	0.0+0.0 1.0+1.0	0.5+0.5	0.0+0.0 1.0+1.0
Wind speed, m/s	50	45 55	50	45 55	50	45 55
Drag coeff., -	0.7	0.6 0.8	0.7	0.6 0.8	0.7	0.6 0.8
Inertia coeff., -	2.0	1.8 2.2	2.0	1.8 2.2	2.0	1.8 2.2
MAT GEOMETRY						
Skirt height, m	0.5	0.3 0.7	0.5	0.3 0.7	0.5	0.3 0.7
Mat thickness, m	3.0	2.5 3.5	3.5	3.0 4.0	4.0	3.5 4.5
Mat area and shape	-	see Figure 2.14	-	see Figure 2.14	-	see Figure 2.14
SOIL DATA						
Shear strength at mudline, kPa	10	5 7 12 15	10	5 7 12 15	10	5 7 12 15
Shear strength gradient k, kPa/m	1.2	0.3 0.6 1.5 1.8	1.2	0.3 0.6 1.5 1.8	1.2	0.3 0.6 1.5 1.8
Subm. unit weight of soil, kN/m**3	7.0	6.0 8.0	7.0	6.0 8.0	7.0	6.0 8.0

3. PENETRATION PHASE AND PULL-OUT PHASE

3.1 EVALUATION OF PENETRATION DEPTHS FOR MATS

3.1.1 General

When a mat-supported jackup platform is set down on the seabed the typical average contact stress will be in the range 15 to 30 kPa (0.3 to 0.6 ksf). The numbers may vary somewhat from one design to another, but it is believed that this range is typical and representative for most of the mat-supported jackup platforms operating today.

The mat height varies in the range 2 to 4 m, and the fingers and cross beams of a typical A mat have widths in the order of 10 to 15 m.

There also exists other mat shapes ranging from squares to rectangles with a center hole varying from nothing to about 40 percent of the total area.

As the mat is set on the seabed and the hull is gradually jacked out of the water, the contact stress gradually increases and the soil strength is mobilized. If the seabed is uneven, the local unevennesses of the seabed will be gradually flattened out until a sufficient bearing capacity is mobilized. In very soft clays the full contact area may develop and then the mat sinks in, gradually mobilizing buoyancy within the soft clay. In stiffer clays and sandy soils only a part of the base area will be needed to mobilize sufficient bearing capacity.

Theoretically the total settlement comprises the following components:
(a) Plastic penetration if the bearing capacity is exceeded.
(b) Elastic immediate settlements.
(c) Consolidation settlements.
(d) Long-term creep
(e) Cyclic induced accumulation of settlements.

For practical purposes the plastic penetration will be of main interest, and in some cases the consolidation settlements may have to be considered. Cyclic induced accumulation of settlements may develop under extreme loading conditions, however, there exists today no practical method for evaluation of the magnitude of this settlement

component. In the following only plastic penetration caused by exceedance of bearing capacity and consolidation settlements will be treated as elastic immediate settlements and long-term creep settlements are considered to be of little importance.

3.1.2 Plastic Penetration (Exceedance of Bearing Capacity)

The penetration depth of mats will practically only be of interest in soft clays and is best estimated by means of bearing capacity expressions as developed by Brinch-Hansen, Skempton, Booker and Davies and others.

To illustrate the typical range of penetration depths to be expected for very soft clays, Brinch-Hansen's formula has been selected:

$$q_{ult} = s_u \cdot (1 + s_{ca} + d_{ca}) + \gamma' \cdot D$$

$$s_u = s_{u0} + k \cdot (D + \frac{B}{2})$$

$$s_{ca} = 0.2 \cdot \frac{B}{L}$$

$$d_{ca} = 0.3 \cdot arctg(\frac{D}{B}) \approx 0.3 \cdot \frac{D}{B} \text{ for } \frac{D}{B} \leqslant 0.3$$

where:

s_u = undrained shear strength at a representative depth below the bottom of the mat, here assumed to be B/2.

s_{u0} = undrained shear strength at mudline

k = shear strength gradient

N_c = bearing capacity factor = 5.14

B = representative width of the mat elements, dependent on the shape of the mat

D = penetration depth of mat

L = typical length of mat elements

γ' = submerged unit weight of soil

s_{ca} = shape factor

d_{ca} = depth factor

For typical mat shapes B/L varies in the range 0.1 to 0.3 and for penetration depths less than the typical height of a mat the ratio D/B will vary in the range 0 to 0.3.

This gives a shape factor s_{ca} in the range 0.02 to 0.06 and a depth factor d_{ca} of 0.0 to 0.09. Thus the expression $N_c^* = N_c \cdot (1 + s_{ca} + d_{ca})$ will vary in the range 5.24 to 5.9.

In this way a rather rough, but normally sufficiently accurate estimate of the pentration depth of a mat due to platform weight can be made.

Fig. 3.1 shows a diagram for estimation of penetration depth based on soil data, mat geometry and platform weight. The upper part of the diagram determines how much of the contact pressure that cannot be taken by mobilization of soil strength without penetration (assuming $N_c^* = 5.3$ as an average), while the lower part determines the penetration depth required for mobilization of buoyancy in soil and extra shear strength development due to mat penetration, provided that the undrained shear strength increases from s_{u0} at mudline with a gradient k with depth below mudline.

Sample problem:

Rig weight = 40 MN

Net area of mat = 2000 m^2

Average contact stress = q = $\dfrac{40000kN}{2000m^2} = 20$ kPa

$$\left.\begin{array}{l} \textit{Typical width } = B = 10 \ m \\ \textit{Typical length } = L = 50 \ m \end{array}\right\} \rightarrow \frac{B}{L} = 0.2$$

$s_{u0} = 2$ kPa

k = 0.2 kPa/m

$s_{u0} + k \cdot B/2 = 3.0$ kPa

$$\left.\begin{array}{l} \gamma' = 5 \ kN/m^3 \\ \textit{Assuming } N_c^* = 5.3 \end{array}\right\} \rightarrow \gamma' + k \cdot N_c^* = 5.0 + 1.1 = 6.1 \ kN/m^3$$

This gives as shown in Fig. 3.1 a penetration depth of approximately 0.7 m. A reevaluation of N_c^* with this penetration depth gives $N_c^* \approx 5.4$ and thus a slightly smaller penetration depth of about 0.6 m as the final answer.

The diagram and the sample problem clearly show that mat penetration caused by exceedance of the soil's bearing capacity will only take place in very soft underconsolidated soils as those found in delta areas.

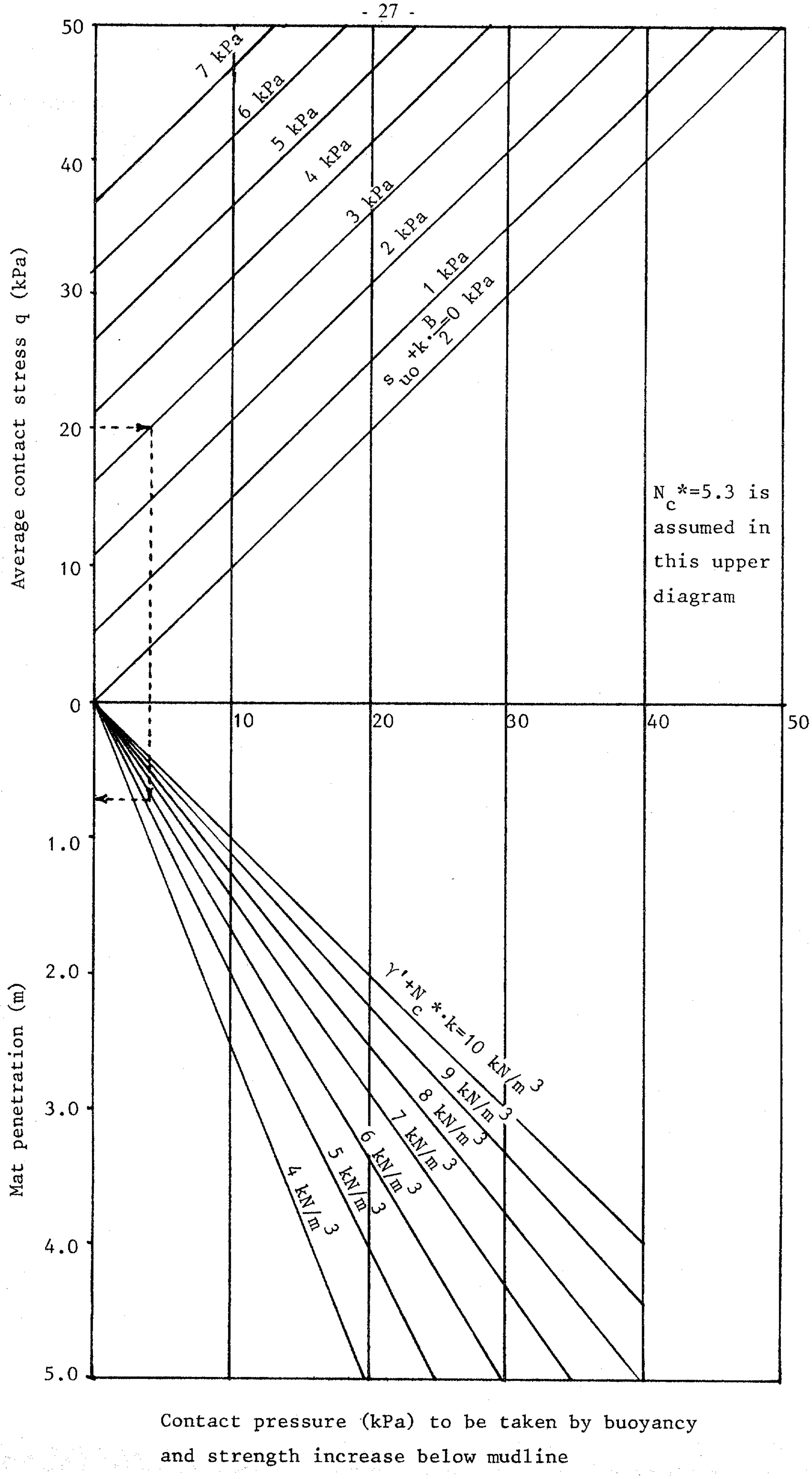


Fig. 3.1 Average contact stress and mat penetration vs. contact pressure to be taken by buoyancy and strength increase below mudline

As an extreme case, a soil with a submerged weight of 5 kN/m^3 and no shear strength at all (which is impossible) will prevent a mat of 3 m height and with a contact pressure of 15 kPa from complete embedment, and a uniform shear strength of about 3 kPa is sufficient to prevent the same mat with 30 kPa contact pressure from complete embedment.

The softest soils known to the authors are found in the Mississippi Delta areas. The South Pass and the West Delta area consist of recent delta deposits having an undrained shear strength at mudline of 2 to 5 kPa and a shear strength gradient of 0.2 to 0.6 kPa/m. Even the softest combination will prevent a mat with 30 kPa contact pressure from complete embedment.

3.1.3 Consolidation Settlements

The consolidation settlement occurring under a mat on a soft clay can be estimated as follows:

- assume a stress distribution of 1:2 under a strip footing of 10 m width
- assume an average contact stress of 20 kPa
- assume the confined compression modulus M of the soil to be linearly increasing with increasing consolidation stress σ_v';
  for soft plastic clays typical values will be in the range
  $M = 10$ to $15\ \sigma_v'$

This approach allows a simplified estimate of the consolidation settlement of an average type of mat on a soft soil.

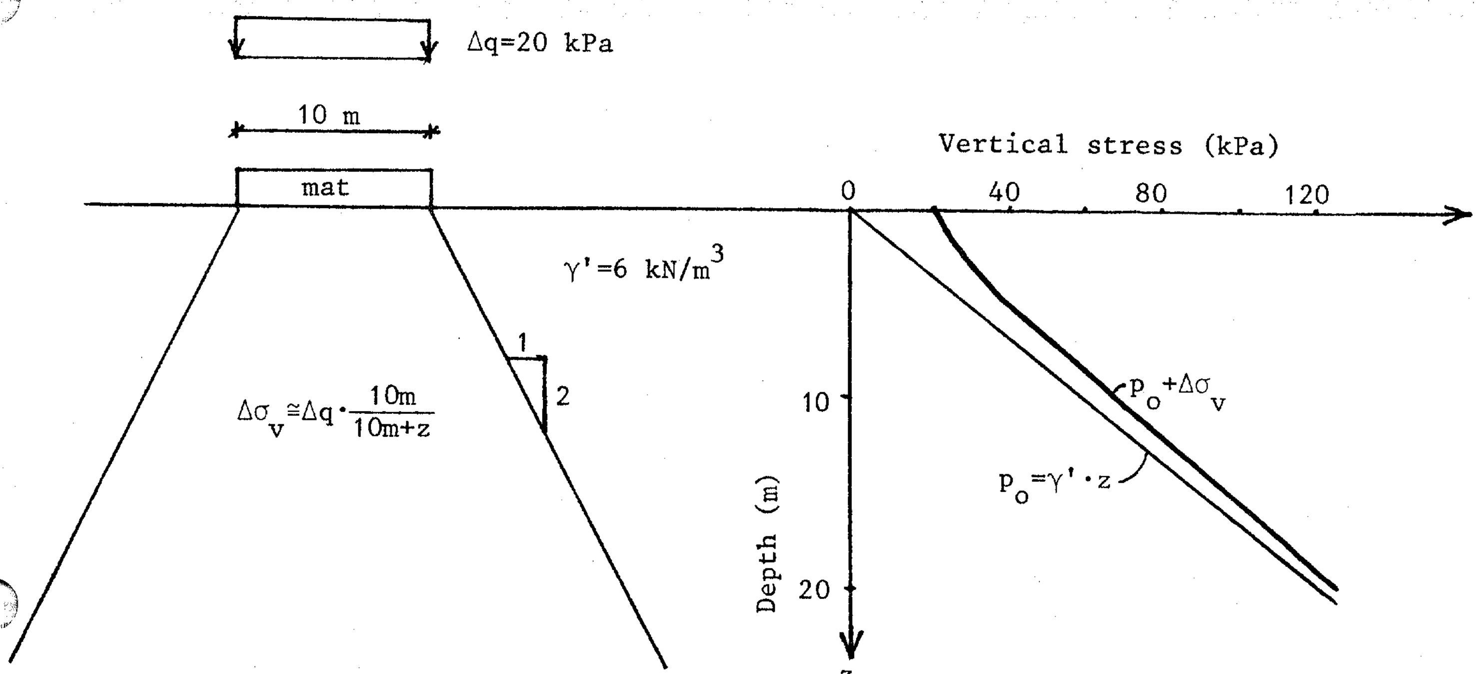


Fig. 3.2 Assumed stress distribution for settlement calculation.

Fig. 3.2 shows the assumed stress distribution, the existing vertical consolidation stress prior to mat installation and the increase in vertical stress caused by the mat installation. In Table 3.1 the settlement has been integrated down to a depth of 2 times the width of the footing. As can be seen, the estimated consolidation settlement is in the order of 0.3 to 0.6 m. However, for soft clays the coefficient of consolidation is generally so low that only a limited part of the total consolidation settlement will take place.

Reference is made to Section 3.7 for more information about effects of consolidation settlements.

z (m)	Δz (m)	p_o' (kPa)	$\Delta\sigma_v'$ (kPa)	$p_o' + \frac{\Delta\sigma_v'}{2}$ (kPa)	M (kPa)	ΔS (m)
0-2	2	6	18	15	150 to 300	0.24 to 0.12
2-4	2	18	15	26	260 to 520	0.12 to 0.06
4-6	2	30	13	37	370 to 740	0.07 to 0.04
6-8	2	42	12	48	480 to 960	0.05 to 0.03
8-10	2	54	11	59	590 to 1180	0.04 to 0.02
10-15	5	75	9	79	790 to 1580	0.06 to 0.03
15-20	5	105	7	108	1080 to 2160	0.03 to 0.01
Total settlement $\Sigma\Delta S$						0.61 to 0.31

Table 3.1 Settlement calculation.

3.2 SKIRT PENETRATION RESISTANCE FOR MATS

3.2.1 General

Mats are normally provided with skirts to improve sliding resistance and reduce effects of scour and hydraulic pumping effects under rocking motions.

Typical skirts are 2 to 3 ft high and extends along the total outer and inner periphery of the mat structure. In order to give the skirts a certain flexural rigidity, corrugation of the skirts or welding of stiffeners to them is normally carried out.

Assuming skirts of 2 ft and 3 ft for the three jackup platform models evaluated regarding stability and assuming skirt tip thicknesses in the order of 3" to 8" gives the following tip and side areas:

	Model 1	Model 2	Model 3
Side area A_s	360 to 540 m^2	458 to 688 m^2	552 to 828 m^2
Tip area A_t	24.3 to 65.0 m^2	29.1 to 77.6 m^2	35.1 to 93.4 m^2

The submerged weights of the three models have been assumed to be 30, 40 and 50 MN, respectively.

3.2.2 Penetration Resistance in Cohesive Soils

The penetration resistance of skirts are considered to be composed of tip resistance and side resistance.

The tip resistance can be expressed as

$$Q_t = A_t \cdot N_c \cdot s_{ut}$$

where
A_t = tip area of skirt
N_c = bearing capacity factor (≈ 8)
s_{ut} = undrained shear strength at skirt tip level

The side resistance can be expressed as

$$Q_s = A_s \cdot \alpha \cdot s_{u,av}$$

where
A_s = side area of skirt (inner and outer area)
α = adhesion factor
$s_{u,av}$ = average undrained shear strength at skirt

Penetration problems may develop in stiffer (overconsolidated) clays. The three mat models with skirt lengths of 2 ft and 3 ft and with skirt thicknesses of 3" and 8" have been evaluated in order to determine critical values for undrained shear strength.

The undrained shear strength is assumed to be constant with depth within the 2 to 3 ft considered for skirt lengths. Furthermore, the adhesion factor α is assumed to be in the order of 0.3 to 0.5 for mat installation. These assumptions lead to the following critical values for the undrained shear strength.

Skirt length (ft)	Skirt thickness (inches)	Critical undrained shear strength (kPa)					
		Model 1		Model 2		Model3	
		$\alpha=0.5$	$\alpha=0.3$	$\alpha=0.5$	$\alpha=0.3$	$\alpha=0.5$	$\alpha=0.3$
2	3	80	100	87	108	90	112
2	8	43	48	47	53	49	55
3	3	65	84	69	91	72	94
3	8	38	44	41	48	43	50

Table 3.2 Penetration resistance of skirts in cohesive soils, critical values of undrained shear strength

3.2.3 Cohesionless Soils

Skirt penetration resistance in cohesionless soils can be evaluated by means of the following formulae which were developed based on full scale measurements on gravity platforms with steel skirts installed in the North Sea.

Tip resistance:

$$Q_t = A_{tip} \cdot K_t \cdot q_{c,t}$$

where

A_{tip} = tip area of skirt

$q_{c,t}$ = cone penetration resistance at skirt tip level

K_t = empirical factor

Side resistance:

$$Q_s = A_{side} \cdot K_s \cdot q_{c,av}$$

where

A_{side} = side area of skirt

K_s = empirical factor

$q_{c,av}$ = average cone resistance

Based on experience, the empirical factors K_t and K_s are found to vary in the ranges 0.3 to 0.6 and 0.001 to 0.003, respectively. The lower values pertain to situations where piping occurs due to overpressure in the skirt compartments.

Typical ranges for q_c values at 0.6 to 0.9 m depth are as follows:
Dense sand: q_c = 3 to 8 MPa
Medium sand: q_c = 0.6 to 3 MPa
Loose sand: q_c = 0.2 to 0.6 MPa

For the considered skirt configurations and platform weights refusal values for q_c as indicated in Table 3.3 below can be found:

Skirt length	Skirt thickness	Critical cone resistance (MPa)		
(ft)	(inches)	Model 1	Model 2	Model 3
2	3	1.9 to 4.1	2.1 to 4.3	2.2 to 4.5
2	8	0.75 to 1.5	0.83 to 1.7	0.87 to 1.8
3	3	1.9 to 3.8	2.0 to 4.2	2.1 to 4.4
3	8	0.74 to 1.5	0.82 to 1.7	0.85 to 1.7

Table 3.3 Penetration resistance of skirts in cohesionless soils, critical values for cone resistance.

The critical cone resistance values in Table 3.3 indicate that penetration problems (refusal) may occur in medium dense to dense sands. The tip resistance is the major component and the penetration ability of the mats is thus heavily dependent on the thickness of the skirt tip.

3.3 PULL-OUT RESISTANCE OF MATS

The pull-out of mat-supported jackup platforms is heavily dependent on an efficient suction release system. If the suction release system is malfunctioning, enormous suction forces may develop. The suction capacity of the soil is of the same order as the bearing capacity, and with mat areas in the range 1000 to 3000 m^2 the suction capacity will be in the order of

$$\left.\begin{array}{l} Q_{suc} = (1000 \ to \ 3000 \ m^2)\cdot N_c \cdot s_u \\ s_u = 3 \ to \ 40 \ kPa \ (soft \ to \ stiff \ clay) \\ N_c = 5.3 \end{array}\right\} \rightarrow Q_{suc} = \left|\begin{array}{l} 15 \ to \ 50 \ MN \ in \ soft \ soil \\ 200 \ to \ 600 \ MN \ in \ stiff \ clay \end{array}\right.$$

However, provided that the suction release system is working, the pull-out resistance is limited to the skirt pull-out resistance. Due to setup effects (i.e. consolidation of excess

pore pressures) the side friction in cohesive soils may increase considerably during a period of two to three months. Adhesion coefficients of 1.0 may very well develop. In stiff clays where the skirt penetration resistance during installation is close to the submerged weight of the platform, considerable pull-out forces may develop.

The pull-out resistance in sand is usually small compared with that in clay, so it will be most critical to evaluate the pull-out resistance for a platform located on a clay deposit. The tip resistance is neglected for simplicity, so that only the side resistance's contribution to the pull-out resistance is considered in the following. Assuming now an α value of 1.0 and applying the calculated critical undrained shear strengths from Table 3.2, the pull-out resistance is found to vary within the range 15 to 45 MN for the small Model 1 platform type and 30 to 90 MN for the larger Model 3 platform type.

This means that overpressure has to be applied in the skirt compartments in order to overcome the skirt pull-out resistance in stiff clays.

3.4 PENETRATION OF INDEPENDENT LEGS WITH OR WITHOUT SPUDCANS

3.4.1 General

The penetration phase of a jackup platform supported by independent legs may cause some serious problems to occur. The most common problem types are:

(1) too little penetration
(2) too much penetration
(3) punch-through problems

The punch-through problems are described and discussed separately in Section 3.6, while the two other problems are dealt with in further detail below:

The footing width for legs without spudcans usually ranges from 8 to 12 feet. For legs with spudcans the range is from 25 to 46 feet. The maximum reported penetration is about 90 feet.

If the sum of air gap, water depth and actual penetration depth exceeds the available leg length, it becomes necessary to replace the topical platform by a platform with longer legs or a larger foundation area.

The penetration depth can be predicted by means of a bearing capacity formula when the necessary soil information is available. The most common bearing capacity formulae are:

(1) Skempton's formula.
(2) Meyerhof's formula.
(3) Brinch-Hansen's formula.

The penetration depth may become too little in case the seabed is covered by a hard top soil, e.g. dense sand or gravel, in which only the very tip of a spudcan footing can be expected to be embedded. Such a spudcan-soil configuration will not offer any rotational foundation stiffness for support of the corresponding platform leg, which hence will behave in a pinned manner and in some cases cause the topical platform conditions to become critical.

The problem with too much penetration is limited to weak and soft soil types, as stiff clays and sands will only produce shallow penetrations, provided that punch-through is not critical. The deep penetration study therefore concentrates on the idealized soil profile No. 1, i.e. soft clay overlaying a harder layer of clay or sand. Fig. 3.3 below shows a typical spudcan in clay, selected as an example case, and with properties listed.

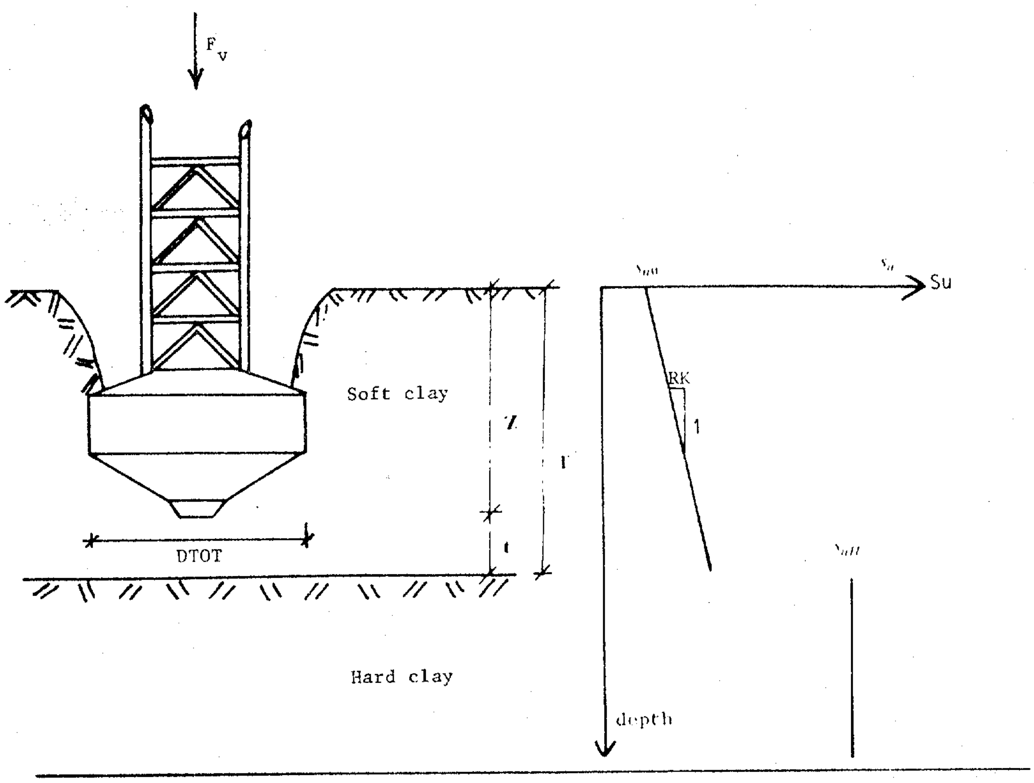


Fig. 3.3 Spudcan in clay.

$s_{u0} = 5$ kPa

k = 1.2 kPa/m

$\gamma' = 7\ kN/m^2$

$F_v = 33250$ kN

$D_{tot} = 11$ m

$A_{tot} = 95\ m^2$

where

F_v = applied vertical load

s_{u0} = undrained shear strength at mudline

k = rate of increase in undrained shear strength with depth

γ' = submerged unit weight

D_{tot} = total diameter of spudcan base

A_{tot} = foundation area for DTOT

$q_v = \dfrac{F_v}{A_{tot}}$ = necessary bearing capacity

In the following, the penetration depth for the considered spudcan is predicted by application of the three most common bearing capacity formulae. First an open hole above the footing is assumed, and then the stability of this open hole is considered together with the effect of backfilling. The bearing capacity formulae are reproduced below, and the determined penetration depth for the spudcan is referenced for each of them:

(1) Skempton's bearing capacity formula for a circular footing:

$$q_u = 6.0 \cdot \overline{s_u} \cdot (1.0 + \frac{0.2D}{B}) + \gamma' \cdot D$$

For description of this formula see the PP2 report. Use of Skempton's formula for the selected case referenced above gives a prediction of 17.0 m for the spudcan penetration.

(2) Meyerhof's bearing capacity formula for a circular footing:

$$q_c = N_{cr} \cdot s_u + \gamma' \cdot D$$

where

q_c = ultimate bearing pressure.

N_{cr} = resulting bearing capacity factor for a circular foundation. $N_{cr,min} = 6$ for $\frac{D}{2r} = 0.0$ and $N_{cr,max} = 9$ for $\frac{D}{2r} = 1.7$. See Fig. 3.4.

r = radius of circular foundation.

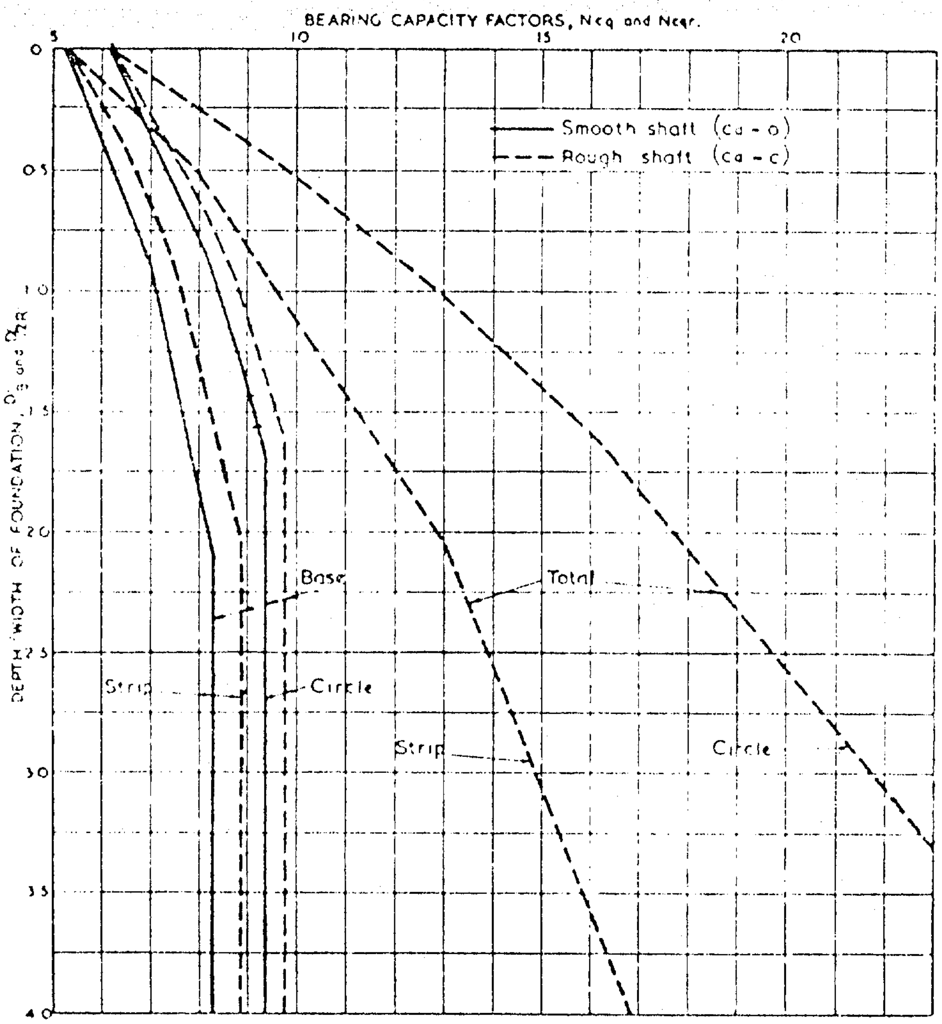


Fig. 3.4 Bearing capacity factors for strip and circular foundations in purely cohesive material.

s_u = average shear strength of clay taken at a depth of B/2 below the footing
γ' = submerged unit weight
D = depth of foundation area below surface

Meyerhof's formula predicts a penetration of 16.8 m for the considered spudcan. Reference is made to Meyerhof (1951).

(3) Brinch-Hansen's bearing capacity formula for a circular footing:

$$q_d = N_c \cdot c_d \cdot (1 + s_{ca} + d_{ca} - i_{ca})$$

Reference is made to Appendix C of the PP2 report. Brinch-Hansen's formula predicts a penetration of 17.6 m for the considered spudcan.

This example indicates that the choice of bearing capacity formula has minor effects on the predicted penetration depth, as long as one applies one of the most common

formulae. It is more important to determine the correct soil parameters for input to these formulae, as these properties have a considerable effect on the ultimate bearing pressure as well as on the predicted penetration depth.

To find the accuracy of the bearing capacity formulae, a comparison has been performed between Brinch-Hansen's formula and a more sophisticated finite element method. The finite element program AXIPLN was used for this purpose, and element mesh and input data for the selected example case are shown in Appendix A.

The finite element analysis consisted of excavation of the foundation hole, corresponding to penetration to the topical depth, and followed by loading of the foundation in five incremental steps up to 130% of the bearing capacity predicted by Brinch-Hansen's formula.

In Appendix A sketches indicate how the failure develops from Load Step No. 3 to Load Step No. 5. The finite element program predicts failure to occur during Load Step No. 5, i.e. at a 10% higher load than that predicted by Brinch-Hansen's bearing capacity formula.

3.4.2 Penetration prediction by diagram with normalized axis

To make a quick estimate of penetration depth and to see the influence of the different parameters, a bearing capacity diagram can be very useful. We have therefore applied and solved Brinch-Hansen's formula for varying soil properties and spudcan geometries, and the results are presented by a series of diagrams in Appendix B, where also the equation that forms the base for the diagram is derived from Brinch-Hansen's formula.

The use of the diagrams is demonstrated by an example in Section 3.4.4.

3.4.3 Stability of the foundation hole and the effect on predicted penetration

Deeply penetrated spudcans leave an open hole above the footing. The stability of the wall of this hole may be critical for the penetration depth. If the wall fails and soil slices fall down on the footing this will increase the load on the footing and may lead to a sudden additional penetration of the footing.

Based on the paper "Stability of Axisymmetric Excavation in clays" (Britto and Kuakabe, 1982) we have constructed a diagram for estimation of a critical depth for a foundation hole. When the penetration depth for the foundation base minus the height of

the vertical spudcan side exceeds the critical depth, the foundation hole will probably become backfilled due to failure of the soil wall.

The value for critical depth D_{crit} found from the diagram must be considered as an upper bound value. Because of the remoulding and disturbance of the soil during preloading, wall failure may occur at a somewhat more shallow depth than D_{crit} The diagram for estimation of D_{crit} is reproduced in Appendix C.

As stated above the wall of the foundation hole will probably fail if D_{crit} is equal to or greater than the penetration depth of spudcan base minus height of spudcan side. To include the effect of this in the penetration prediction one must add the weight ΔF_v of the backfilled soil to the preload, i.e.

$$\Delta F_v = \gamma' \cdot \pi \cdot r^2 \cdot (z-h)(1-(0.5\frac{(z-h)}{r}+1)^{-2})$$

This expression for ΔF_v is derived in Appendix D, where also the explanation to the applied notation can be found.

3.4.4 Use of diagram for penetration prediction

Diagrams for penetration prediction are shown in Appendix B. For a topical case one has to apply the diagram drawn for the actual h/r ratio. r is radius of exposed foundation base. For soft clays with s_{u0} less than 40 kPa h is usually equal to the total height of the spudcan side, while for stiffer clays h is the height of the penetrated part of the spudcan side. For legs without spudcans h should be considered as penetrated leg length.

The input to the diagrams consists of the following dimensionless ratios:

$\frac{h}{r}$, $\frac{q_0}{s_{u0}}$, $\frac{kr}{s_{u0}}$, $\frac{\gamma' r}{s_{u0}}$

For notations see Appendix B.

As described in Appendix B each diagram consists of two parts. To find the total bearing capacity one must add the contribution Q1 and Q2 from the right and the left side of the diagram, respectively. Q2 will be identical for all h/r ratios, provided the final penetration depth is unchanged, while Q1 will vary with varying different h/r ratio.

Applied vertical load divided by s_{u0} should be equal to Q1+Q2. The z/r ratio from the vertical axis will give the penetration depth for the spudcan base.

The output from the diagram applied for the topical h/r ratio consists of the z/r ratio, where z is the penetration depth of the spudcan base, and r is still radius of exposed foundation base.

Below, an example case is presented in order to show combined use of the two diagram types of Appendix B, namely the diagram for penetration prediction and the diagram for stability of foundation hole:

Soil and geometry properties for the example case are as follows:

$s_{u0} = 5$ kPa

r = 5 m

k = 1.2 kPa/m

h = 2 m

$\gamma' = 7\ kN/m^3$

q = 240 kPa

The input to the penetration diagram becomes:

$$\frac{h}{r} = 0.4,\ \frac{q_0}{s_{u0}} = 48,\ \frac{kr}{s_{u0}} = 1.2,\ \frac{\gamma' r}{s_{u0}} = 7$$

Procedure for use of the penetration diagram for this example:

(1) find proper diagram for h/r = 0.4

(2) take a ruler and measure the distance for $\frac{q}{s_{u0}} = 48$

(3) displace the ruler upwards parallel to the $\frac{q}{s_{u0}}$ *axis*

(4) watch the value for $\frac{kr}{s_{u0}}$ and $\frac{\gamma' r}{s_{u0}}$

(5) when the distance measured in (2) equals the distance between the two values mentioned in (4), read value on z/r axis

For the considered case, z/r becomes 2.2, and the predicted penetration thus becomes z = 11 m.

The stability of the foundation hole must be checked for this penetration depth:

Input to stability diagram:

$$\frac{kr}{s_{u0}} = 1.2,\ \frac{\gamma' r}{s_{u0}} = 7$$

Output from stability diagram:

$$\frac{(z-h)}{r} = 1.9 \rightarrow z_{crit} = 1.9r + h = 11.5\ m$$

This result indicates that if the penetration is greater than 11.5 m, the wall will fail and drop into the foundation hole. And if the penetration is less than 11.5 m, the foundation hole will be stable. z_{crit} is the upper bound value for a stable foundation hole for this type of spudcan (h = 2 m). Due to disturbance and remoulding of soil surface in foundation hole, the soil wall could possibly fail for the determined penetration depth of z = 11 m.

The increased penetration due to failure of the soil wall may be estimated in the following way:

(1) Calculate additional load on footing due to failed soil mass; reference is made to Appendix D:

$$\Delta F_v = \gamma' \cdot \pi \cdot r_o^2 \cdot (z-h)(1-(0.5\frac{(z-h)}{r_0}+1)^{-2})$$

Use of this formula leads to:

$$\Delta F_v = 3576\ kN$$

$$\Delta q_0 = \frac{\Delta F_v}{A} = 37.6\ kPa$$

(2) Calculate new penetration depth:

The input to the penetration diagram consists of:

$$q^0 = q_0' + \Delta q_0 = 240 + 38 = 278\ kPa$$

$$\frac{q^0}{s_{u0}} = \frac{278}{5} = 56$$

$$\frac{\gamma' r}{s_{u0}} = 7$$

$$\frac{kr}{s_{u0}} = 1.2$$

$$\frac{h}{r} = 0.4$$

The output from the penetration diagram gives:

$$\frac{z}{r} = 2.65 \rightarrow z = 13.3m$$

The conclusion from this example will be that wall failure in the foundation hole will produce an extra penetration of approximately 2.3 m, corresponding to an increase of 21%. If the k value had been smaller, the increase would have been more than 21%.

3.5 PULL-OUT RESISTANCE OF INDEPENDENT LEGS WITH OR WITHOUT SPUDCANS

Reference is made to Section 2.4 of the PP2 Report.

The bearing capacity obtained by preload can be divided in base resistance and side wall resistance. Model tests have shown that the pull-out capacity of the base can be up to 13% bigger than the bearing capacity. To estimate the necessary pull-out force one can use the following expression:

$$F_{pull-out} = 1.13 q_{v,base} A_{base} + A_{ss} \alpha s_{u,ss} + A_{sb} \alpha s_{,sb}$$

in which $F_{pull-out}$ = maximum necessary pull-out force for one leg
$q_{v,base}$ = ultimate bearing capacity for base
A_{base} = exposed foundation area of base
A_{ss} = side area of embedded part of footing
α = strength reduction factor
$s_{u,ss}$ = undrained shear strength of clay along side of footing at time of pull-out
A_{sb} = side area of lifted backfill
$s_{u,sb}$ = undrained shear strength of backfill at the time of break out

The force estimated by the formula above must be considered as an upper bound value. Many jackup platforms do not have enough buoyancy to establish the necessary pull-out force for deeply penetrated spudcans, especially if the foundation hole is back-filled. To solve this problem many jackup platforms have installed suction releasers or jetting systems.

Suction releasers are low pressure/large volume systems, and jetting systems are high pressure/small volume systems. These systems are most efficient if they are mounted in the spudcan base. Their contribution to reduce the pull-out resistance produced at the base is for normally designed spudcans much more important than their contribution to reduce the resistance produced at the side of the spudcan.

We do not have any experience with suction releasers or jetting systems and would therefore not recommend any of the two systems right away. But from a pure geotechnical judgment we believe a suction release system could be preferable in clay, while a jetting system could be suitable for sand. In sand, only the base (cone and tip) of normally designed spudcans will penetrate into the soil and there will therefore usually not be any pull-out problems involved with such soils.

At large water depths it may be necessary to use a jetting system to obtain an increased penetration depth if the seabed consists of sand. The jetting system causes liquefaction of the sand and will allow for a deeper penetration and thereby an improved rotational stiffness of the footing. The soil has very often an increasing strength with depth, and an increased penetration depth will therefore improve the foundation stiffness for all degrees of freedom.

A general problem for suction releasers and jetting systems is that they are easily blocked by soil intrusion in the nozzles. This should be kept in mind when such systems are installed.

3.6 PUNCH-THROUGH OF LAYERED SOILS

3.6.1 General

Punch-through is the most severe failure mode for a jackup platform and may occur when the platform's foundation is resting in a relatively stiff soil, underlain by a softer soil. The consequences of punch-through mainly depend on the difference in strength between the hard top layer and the soft deep layer, but also on the rate of strength increase with depth in the soft layer. Now, assume that a situation occurs (e.g. a storm) where the bearing capacity is not any longer sufficient: If a small additional penetration is enough to achieve the necessary bearing capacity for the new situation, punch-through will usually not cause any problems. But, if a large additional penetration is required and takes place e.g. during preloading with maximum load on board, it may cause enormous damages to the legs and in some cases even capzise the rig.

The damages and troubles can be reduced if the understanding of the failure mechanism is improved and the necessary soil data are available. Before moving to a specific location the "punch-through stress level" can be predicted and compared with planned

preload weight. If punch-through is likely to be governing, various measures to avoid its occurrence may be put to use: The most important parameters in this context are the foundation area and the total weight of the platform. Reduction in total weight, provided that the safety against overturning is still satisfied, and increase in foundation area will both improve the safety against punch-through failure. The ballast weight (for three leg type platforms) and load level (for four leg type platforms) during preloading are meant to apply at least the same vertical load to the footing as the one occurring during design storm conditions. By taking into account the degree of fixity for the footing in the actual soil, the preload weight may be reduced, and this in turn will improve the safety against punch-through failure.

3.6.2 Results of finite element analyses for layered soils and comparison with punch-through formula

Reference is made to Appendix E for a detailed description of a formula for estimation of punch-through capacity, and to Appendix F for the finite element analyses that have been carried out. All data for the example case selected as a base for the finite element analyses are presented in Appendix F.

Both sand and stiff clay overlaying soft clay have been considered in the finite element analyses. The same element mesh and loading procedure have been used in both cases, and the main goal has been to determine the failure patterns for punch-through and evaluate the punch-through capacity.

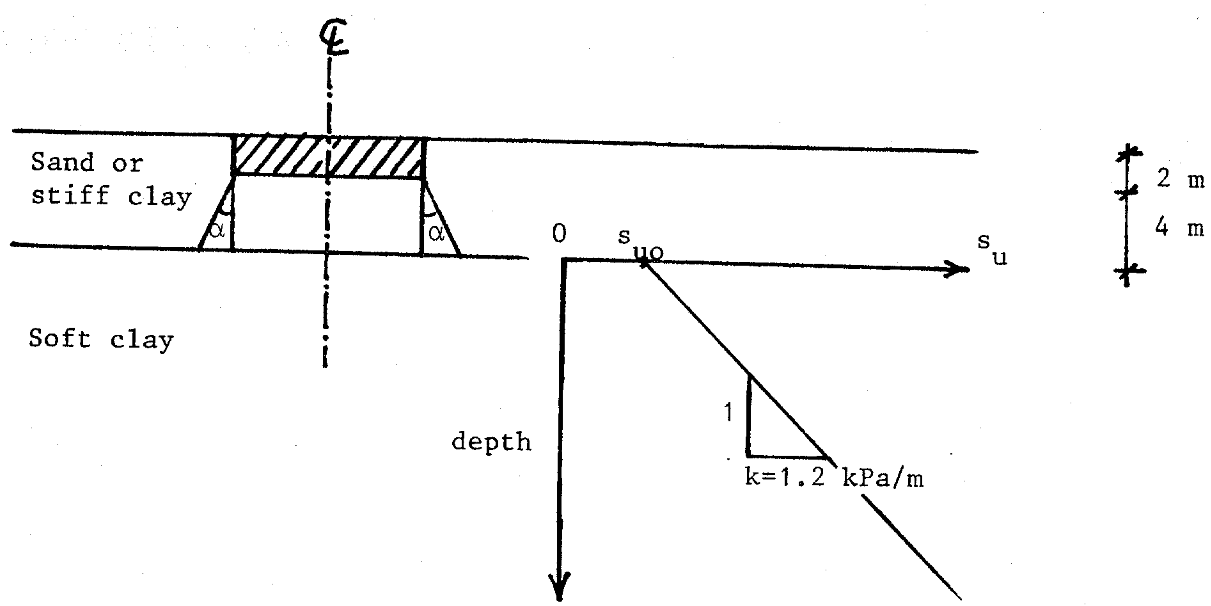


Fig. 3.5 Spudcan in hard layer overlaying a soft clay.

For both considered soil profiles the finite element analyses indicated that the failure starts at the edge of the footing in the hard soil layer and spreads out into the top of the soft soil layer. Below the footing, the soft soil in the deep layer is pressed down and sideways and lifts up the hard layer in a zone between 2 and 5.5 footing radii from the center of the footing. The hard soil within a distance of one radius from the footing is displaced downwards together with the footing itself, and the largest soil displacements are observed to occur near the footing.

The results of the finite element analyses were interpreted in terms of ultimate foundation pressure q_{ult}, i.e. the punch-through capacity. For the top layer consisting of stiff clay, $q_{ult} \approx 200$ kPa was found to occur at a vertical displacement of 5 cm. For the top layer consisting of sand, $q_{ult} \approx 160-190$ kPa was found to occur at a displacement of 7 cm.

These two capacities are also calculated by the punch-through formula. For top layer of stiff clay $q_{ult} = 190$ kPa was calculated, and for top layer of sand $q_{ult} = 175$ kPa was found. These calculated values compare well with the results of the finite element analyses referenced above. This is further visualized by the load-displacement diagram for the considered spudcan in Fig. 3.6, where calculated values appear to correspond well with the ultimate resistances defined at the breakpoints of the respective load-displacement curves found by the finite element analyses.

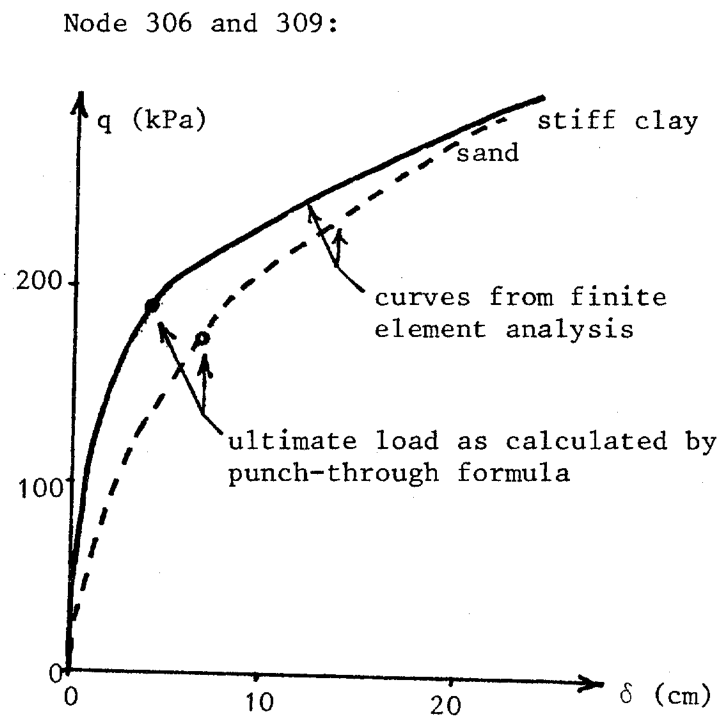


Fig. 3.6 Load-displacement diagram for spudcan as determined from finite element analysis.

3.6.3 Results of parametric study on safety against punch-through failure

For a detailed description reference is made to Appendix G.

Two types of soil profiles have been examined:
Soil profile A: Stiff clay overlaying soft clay.
Soil profile B: Sand overlaying soft clay.

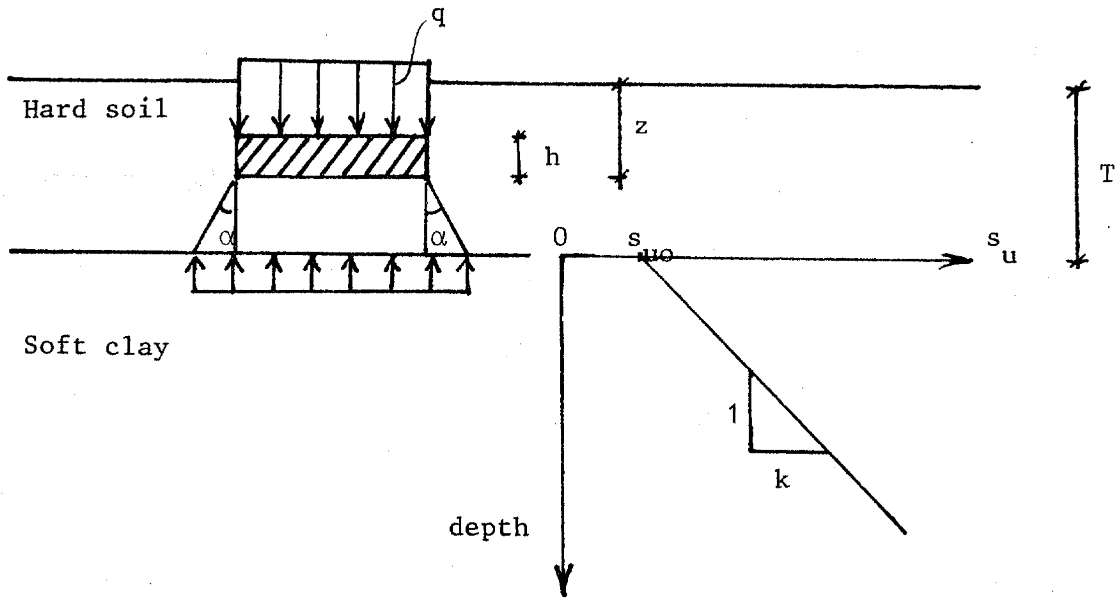


Fig. 3.7 Typical punch-through situation.

Jackup platforms supported by spudcans will normally not penetrate in sand. Therefore soil profile B has only been studied for a surface footing, while soil profile A has also been studied for a penetrated footing.

The failure mechanism is more or less the same for the two profiles. If the hard soil in the profile is strong enough to resist base penetration, the punch-through capacity will be the same for the two profiles, provided the two profiles have the same top layer thickness and identical soil type in the deep layer.

The most important geometric parameter for the punch-through capacity is the foundation radius. A 20% increase in radius improves the factor of safety against punch-through by 70%, while a 10% reduction in radius reduces the factor of safety by about 45% when the same total load is assumed. For all practical cases the maximum foundation radius is known and will not lead to any uncertainties in the calculations. The other involved geometric parameter is the side wall height of the spudcan. This parameter has, however, only got a small effect on the factor of safety. Even though it is a bit difficult to measure or estimate the effects of the side wall height of the spudcan, it can be stated that a reduction in penetrated side wall height generally will reduce the bearing capacity.

When it comes to the soil parameters, it has been found that the most important among these is the thickness of the top layer. In case soil profile A is considered with a top layer consisting of stiff clay, the undrained shear strength of this stiff clay has a large influence on the punch-through capacity for a spudcan that has penetrated into this layer. Other soil parameters of importance are the undrained shear strength in top of the soft clay layer, s_{u0}, and the rate of increase in undrained shear strength with depth in this layer, k. The larger the distance is from the spudcan base to the soft clay, the more important is the parameter k.

3.7 EVALUATION OF CONSOLIDATION EFFECTS ON SOIL STRENGTH

Two sets of analyses have been carried out, one for a spudcan embedded at 20 m depth in a soft clay and one for a mat founded at 2 m depth in a somewhat softer clay. For the spudcan as well as for the mat typical geometry properties and load data were assumed, and the analyses and their results are presented in the following.

The deeply penetrated spudcan that was analyzed first was for simplicity assumed to have a flat base with a diameter of 10 m, and the soil in which it was embedded was assumed to be a soft clay with undrained shear strength increasing linearly with depth,

$$s_u = 66.0 + 1.2 \cdot z\,(kPa\ ),$$

with z=depth in m.

Other soil properties assumed were:

- coefficient of lateral pressure at rest $K_o = 1.0$

- hyperbolic stress-strain relationship
- initial Young's Modulus $E_i = 1000 \cdot s_u$
- submerged weight of soil $\gamma' = 7\ kN/m^3$
- Poisson's ratio $\nu = 0.45$

An axisymmetric and stepwise finite element analysis of the spudcan-soil system was hence carried out by means of the AXIPLN program, covering typical levels of vertical loading. AXIPLN computed the initial stresses at various points in the soil deposit plus the principal stresses σ_1 and σ_3 after each considered load increment.

Two load cases were selected for further studies, namely selfweight loading of 18.8 and 31.4 MN corresponding to average contact stresses of 240 and 400 kPa, respectively. For evaluation of consolidation effects for these two load cases a pore pressure generation model had to be adopted. It was decided to neglect possible dilatancy effects and restrict the study to pore pressure changes caused by changes in the principal stresses only. It was assumed that the pore pressure change Δu due to a certain applied load was equal to the change in average normal stress which could be calculated as $(\Delta\sigma_1 + \Delta\sigma_3)/2$ for this axisymmetric problem. Hence, $\Delta u = (\Delta\sigma_1 + \Delta\sigma_3)/2$ was computed for an adequate number of points within the considered soil volume and was in turn given as input to the consolidation program OCEAN2 together with typical consolidation properties for the soil, namely
- coefficient of consolidation $c_v = 1.0\ m^2/yr = 3.17 \cdot 10^{-8}\ m^2/sec.$
- permeability $k = 1.0 \cdot 10^{-10}\ m/sec.$

For each of the two considered load cases a time step analysis of the corresponding consolidation, i.e. drainage of introduced excess pore pressures, was performed by means of OCEAN2. The results of this study are presented in Figs. 3.8 through 3.13 showing pore pressure distribution in the soil volume surrounding the spudcan immediately after load application plus after 25% and 50% reduction of the introduced pore pressure just underneath the spudcan. The analyses indicated that the 25% reduction would be achieved independently of load level after 2.5 years of consolidation while the 50% reduction would be achieved after 7.5 years, i.e. no significant consolidation takes place during the very few months a spudcan-supported jackup platform is located at a certain site. In connection with these two consolidation analyses it should be noted that it is maybe not appropriate to use a permeability as low as $1.0 \cdot 10^{-10}$ m/sec in a zone above the spudcan where remoulding effects from the installation may be present, but it is on the other hand doubtful whether a higher permeability in this

zone would have influenced the reported analysis results significantly.

When the excess pore pressures drain, the effective stresses in the soil increase and cause the soil to gain strength. A reasonable estimate for the increase in undrained shear strength s_u at a time t after a considered load application is

$$\Delta s_u = \begin{cases} tg\varphi \cdot (\Delta u_o - \Delta u_t) & \text{for } \Delta u_t \leq \Delta u_o \\ 0 & \text{for } \Delta u_t > \Delta u_o \end{cases}$$

where Δu_o is initial excess pore pressure after load application and Δu_t is the part of Δu_o that remains after the time t. A friction angle $\varphi = 30^o$ (i.e. $tg\varphi = 0.58$) is a reasonable choice for the considered soft clay.

Redistribution of pore pressures may cause $\Delta u_t > \Delta u_o$ to occur in certain points during consolidation, and the reason for not considering negative Δs_u values in such cases is that $\Delta u_t > \Delta u_o$ corresponds to a load history similar to that for an overconsolidated clay for which it is generally accepted that s_u does not decrease significantly from the value it had at the preconsolidation level.

Based on stress levels computed by AXIPLN for a load case close to failure a most likely critical slip surface was determined and selected as a reference for an approximate evaluation of the increase in the spudcan's bearing capacity during consolidation. It appeared that a squeezing type of failure locally around the spudcan would be most likely, see Fig. 3.14, and although its shape and location might be changed during the consolidation, it was assumed to be accurate enough to base bearing capacity considerations on this slip surface.

A reasonable estimate of the relative increase in the spudcan's bearing capacity will hence be

$$\Delta Q_{rel} = \frac{\sum_l \Delta s_u \cdot \Delta l \cdot r}{\sum_l s_u \cdot \Delta l \cdot r}$$

where $\Sigma \Delta l$ is the length of the intersection between the slip surface and a considered vertical plane and r is the distance from the axis of symmetry to a considered piece of curve Δl.

The results of this study are presented in the below table:

Contact stress	Increase in bearing capacity, ΔQ_{rel}	
	2.5 years	7.5 years
240 kPa	6.0%	9.4%
400 kPa	8.6%	14.1%

A consolidation coefficient of $c_v = 1\ m^2/yr$ was used in this study, but for potential clays where spudcans are likely to be used one may expect c_v to vary within say 0.5-10 m^2/yr. Utilizing the fact that the consolidation time t is inversely proportional to the coefficient of consolidation c_v at a certain degree of consolidation, the above results can be generalized and presented in diagrams as indicated in Fig. 3.15. It appears that for higher c_v-values within the mentioned interval one can maybe expect a minor increase in the spudcan's bearing capacity during the time a jackup platform is situated at a certain location, i.e. say a couple of months, but generally the conclusions of the performed analyses will be that consolidation of the soil volume surrounding a spudcan will have no practical effect on the soil strength and the corresponding bearing capacity during the jackup platform's operational phase.

For say 5 to 10 times softer clay than the one considered for the spudcan in the above-presented analyses, a mat foundation will be more adequate than a foundation consisting of spudcans for a considered jackup platform, and therefore a typical mat foundation was also analyzed. The mat structure was assumed to be A-shaped, and the analyses were limited to a 10 m wide section of one of its two sides at an assumed shallow embedment of 2 m. A typical linear shear strength profile was assumed for the soil,

$$s_u = 5.0 + 1.2 \cdot z \, (kPa)$$

with z=depth in m, while all other soil properties were assumed to be identical to those selected for the spudcan analyses.

A plane strain and stepwise finite element analysis of the mat-soil system was hence carried out by means of the AXIPLN program, leading to computation of principal stresses σ_1 and σ_3 for various cases of vertical loading of the mat. Two of these load

cases were selected for further studies, namely selfweight loading of 0.3 MN/m and 0.4 MN/m corresponding to average contact stresses of 30 and 40 kPa, respectively.

As for the spudcan analyses, the AXIPLN results were interpreted in terms of excess pore pressures caused by the applied mat loads, and a consolidation analysis was subsequently performed by means of the OCEAN2 program. The results of these analyses are presented in Figs. 3.16 through 3.21 showing pore pressure distributions in the soil supporting the considered mat immediately after load application as well as after 25% and 50% reduction of the introduced pore pressures just underneath the mat. The analyses indicated that the 25% reduction would be achieved independently of load level after 3 years of consolidation while the 50% reduction would be achieved after 11 years, i.e. nor for a mat-supported platform any significant consolidation takes place during the very few month such a platform is located at a certain site.

It is interesting to notice that even though the drainage path is considerably shorter for the shallow mat function than for the deeply penetrated spudcan, the consolidation times for these two different foundations have been computed to be approximately identical or at least of the same order of size. The only reasonable explanation of this observation seems to be that plane strain drainage is generally much slower than axisymmetric drainage due to the different boundary conditions that govern these two drainage processes, and thus the effects of axisymmetric contra plane strain drainage more or less counteract the embedment effects in the present comparison.

A probable critical slip surface was assessed through the soil underneath the considered mat, see Fig. 3.22, and was selected as a reference for an approximate evaluation of the increase in the mat's bearing capacity during consolidation. The model for evaluation of Δs_u in the soil volume surrounding the previously considered spudcan was applied also for the soil supporting the mat, and a reasonable estimate of the relative increase in the mat's bearing capacity was hence assessed to be

$$\Delta Q_{rel} = \frac{\sum_l \Delta s_u \cdot \Delta l}{\sum_l s_u \cdot \Delta l}$$

where $\Sigma \Delta l$ is the length of the plane slip surface that was considered.

Use of this formula led to the following interpretation of the OCEAN2 analyses for the mat foundation:

Contact stress	Increase in bearing capacity, ΔQ_{rel}	
	3.0 years	11.0 years
30 kPa	6.5%	14.0%
40 kPa	8.8%	19.3%

Sliding might be more critical for the stability of the mat foundation than conventional bearing capacity failure, and ΔQ_{rel} was therefore also evaluated by summing up $\Delta s_u \cdot \Delta l$ and $s_u \cdot \Delta l$ over the mat-soil interface. The results are summarized in the below table:

Contact stress	Increase in sliding capacity, ΔQ_{rel}	
	3.0 years	11.0 years
30 kPa	37.8%	76.6%
40 kPa	50.0%	103.0%

The increase in sliding capacity during consolidation is significantly larger than the corresponding increase in conventional bearing capacity. Thus, sliding becomes a less likely failure mode after some consolidation has taken place than immediately after placing the mat on the seabed.

As for the spudcan analyses, also the results of the mat analyses have been interpreted and presented in terms of varying coefficient of consolidation c_v, and corresponding diagrams are included in Figs. 3.23 and 3.24 for bearing capacity and sliding capacity, respectively.

Conclusively, it may be stated that also for mat foundations no significant consolidation effects can be counted on during the very few months a jackup platform is present at a considered site, except maybe for minor effects when it comes to sliding capacity which appears to benefit more from the consolidation than conventional bearing capacity.

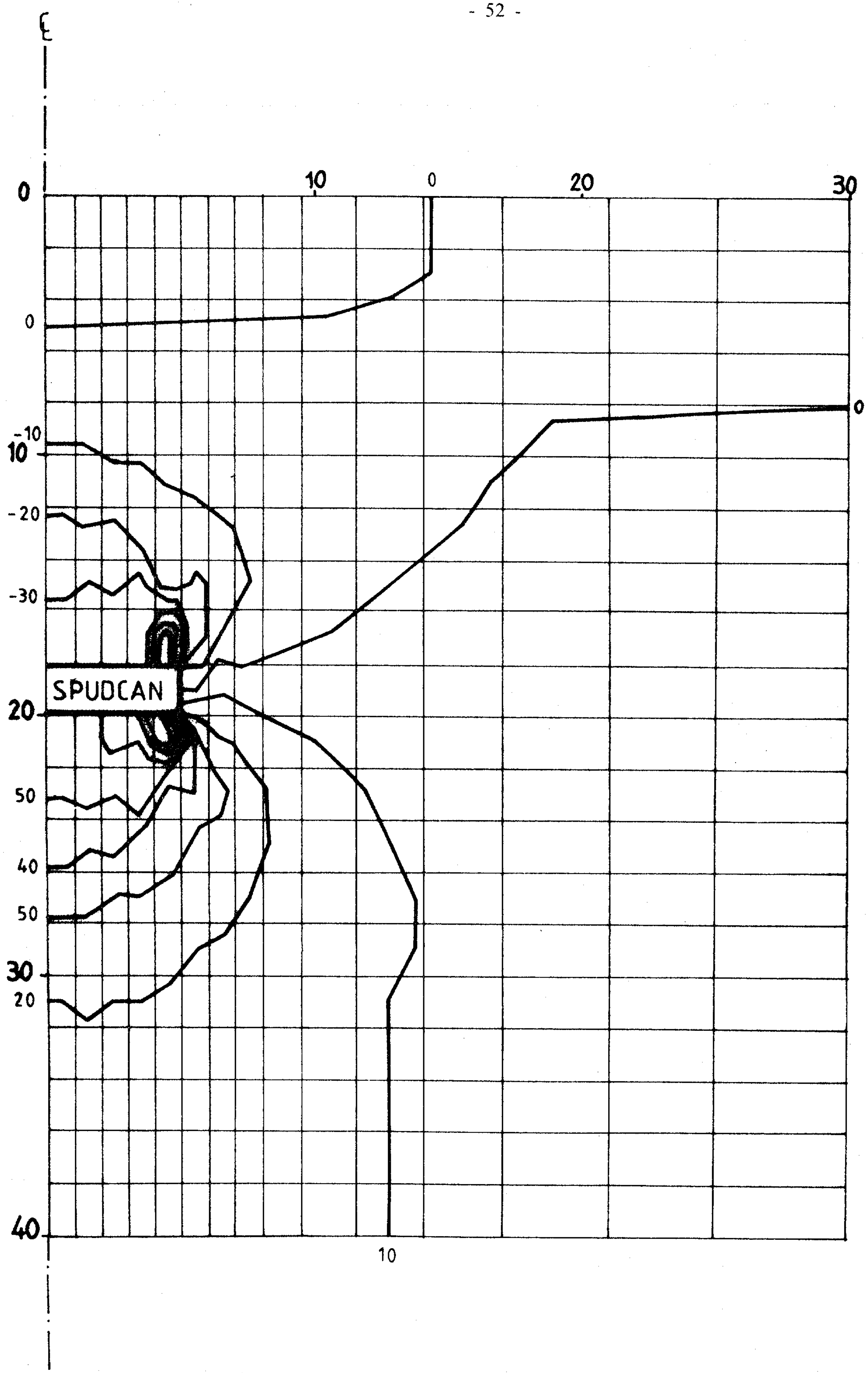


Fig. 3.8 Pore pressure distribution in soil around spudcan. Initially, after application of 18.8 MN (240 kPa) vertical load.

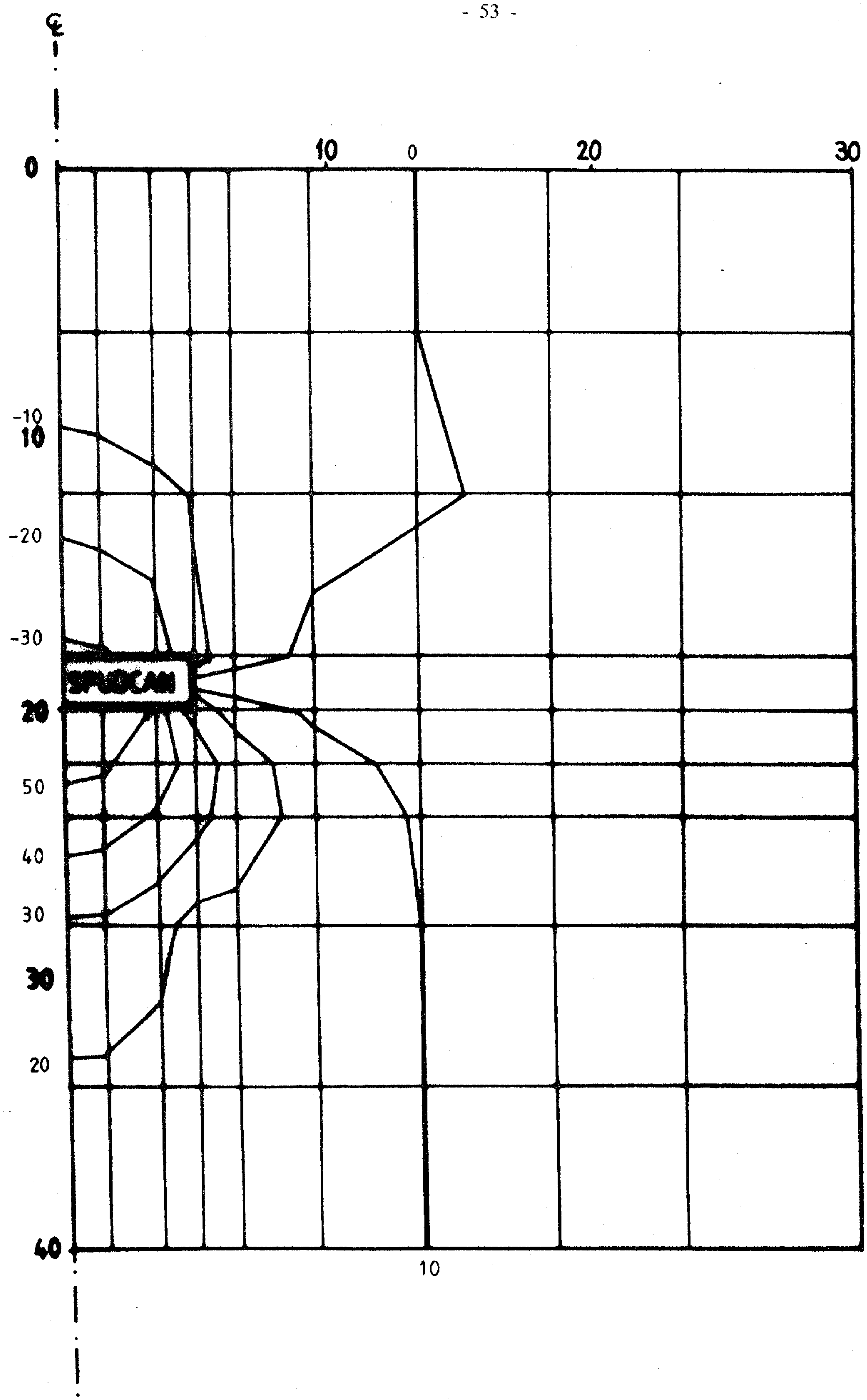


Fig. 3.9 Pore pressure distribution in soil around spudcan. After 25% reduction underneath spudcan, for 18.8 MN (240 kPa) vertical load.

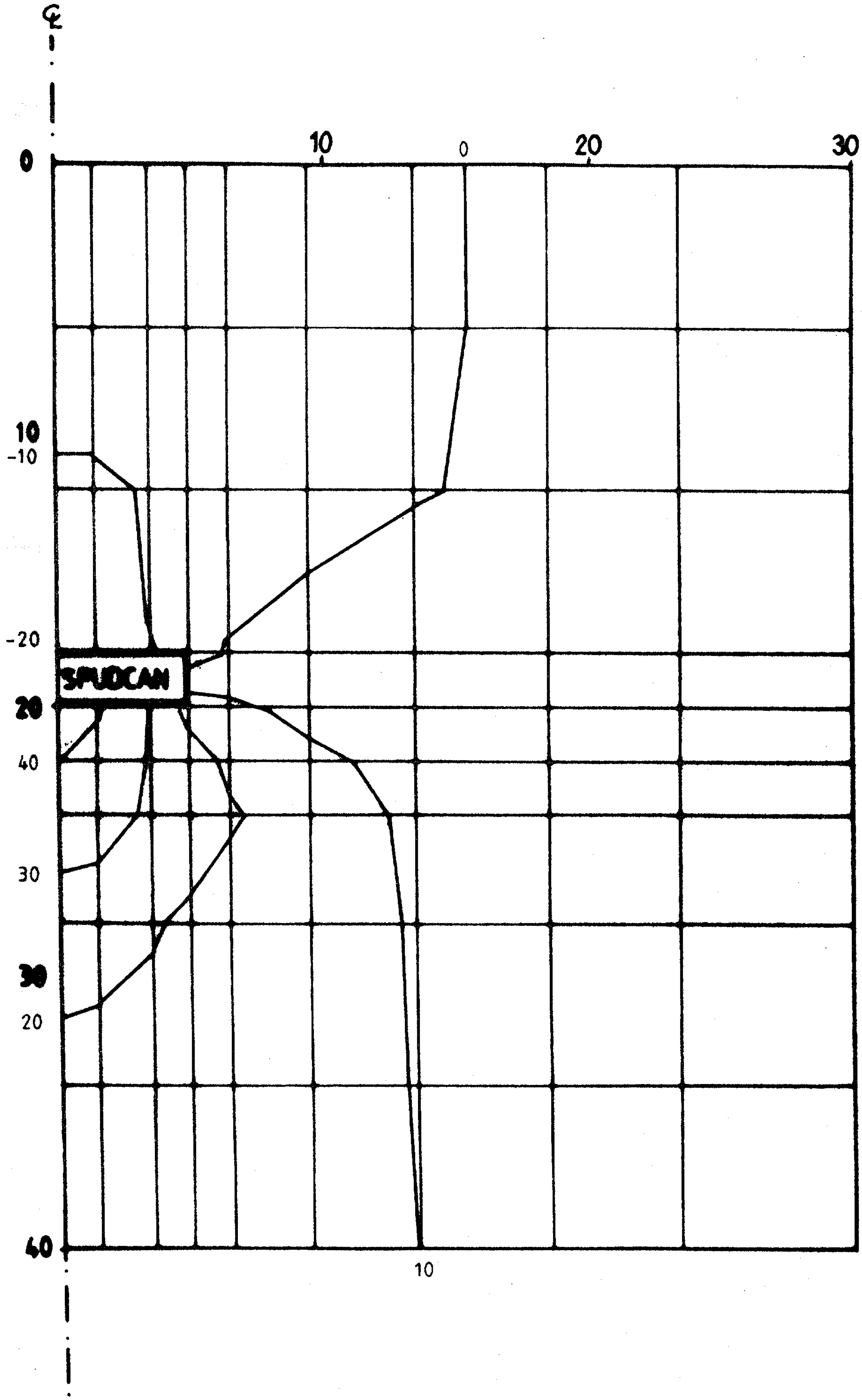


Fig. 3.10 Pore pressure distribution in soil around spudcan. After 50% reduction underneath spudcan, for 18.8 MN (240 kPa) vertical load.

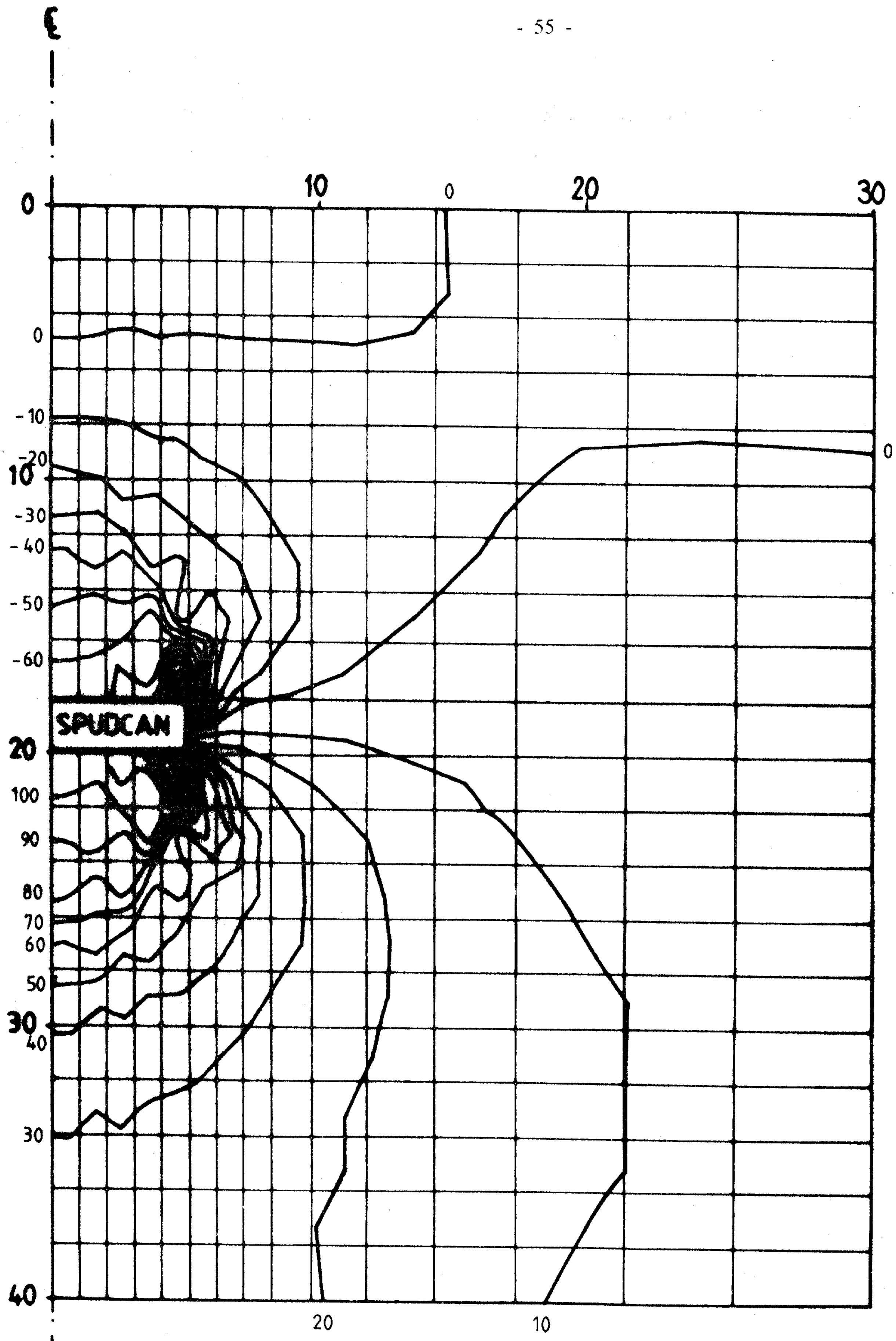


Fig. 3.11 Pore pressure distribution in soil around spudcan. Initially, after application of 31.4 MN (400 kPa) vertical load.

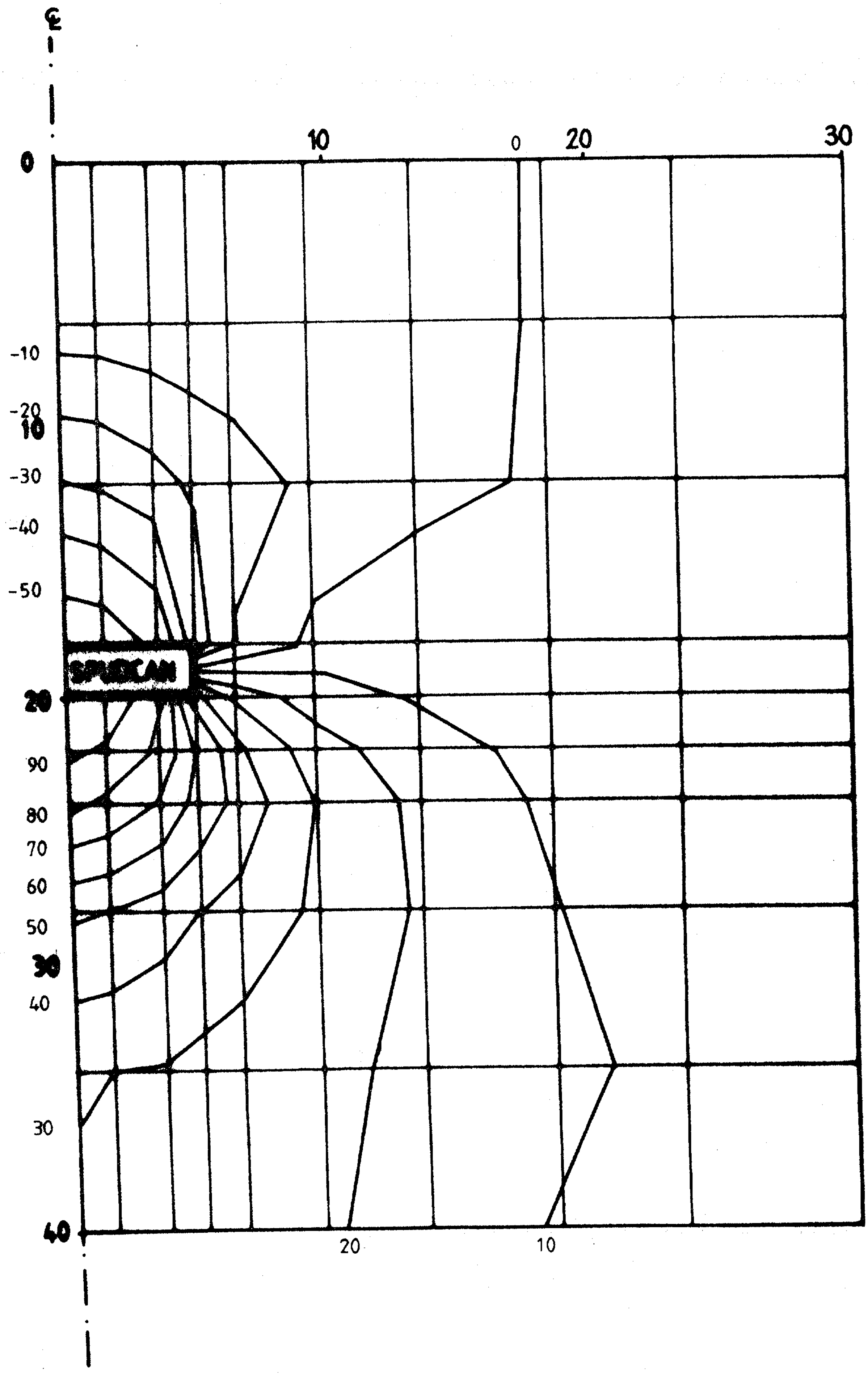


Fig. 3.12 Pore pressure distribution in soil around spudcan. After 25% reduction underneath spudcan, for 31.4 MN (400 kPa) vertical load.

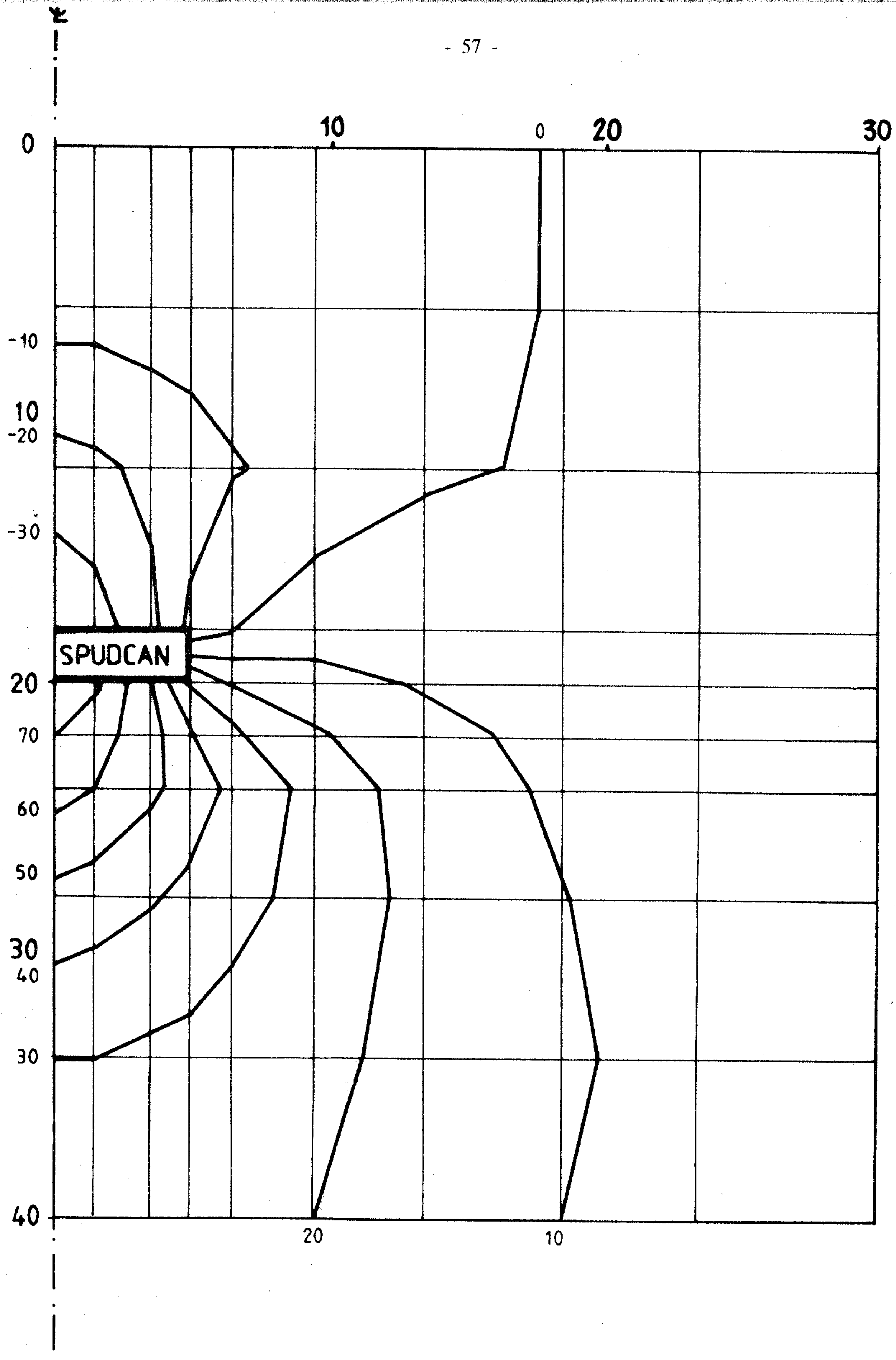


Fig. 3.13 Pore pressure distribution in soil around spudcan. After 50% reduction underneath spudcan, for 31.4 MN (400 kPa) vertical load.

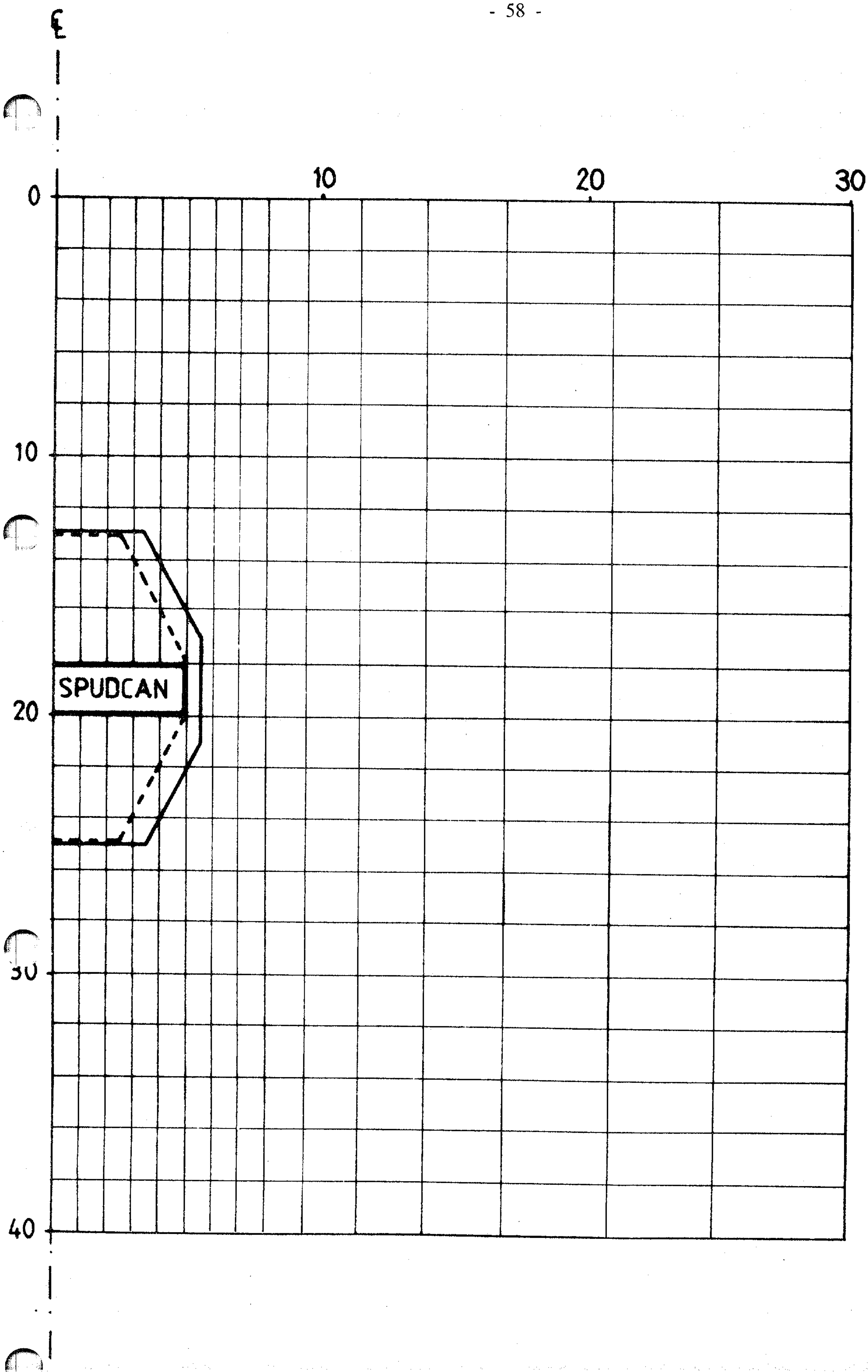
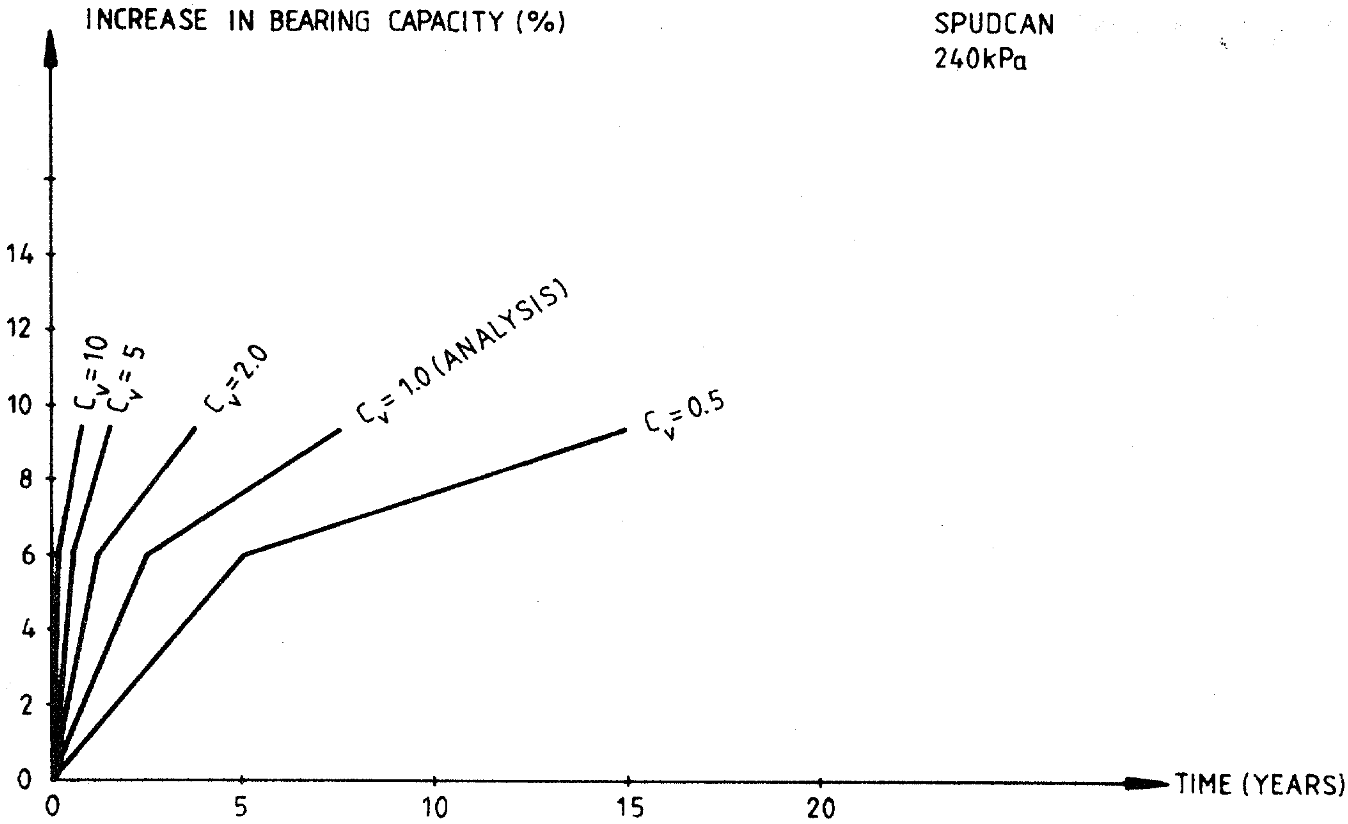


Fig. 3.14 Assumed critical slip surface for evaluation of increase in bearing capacity of spudcan.



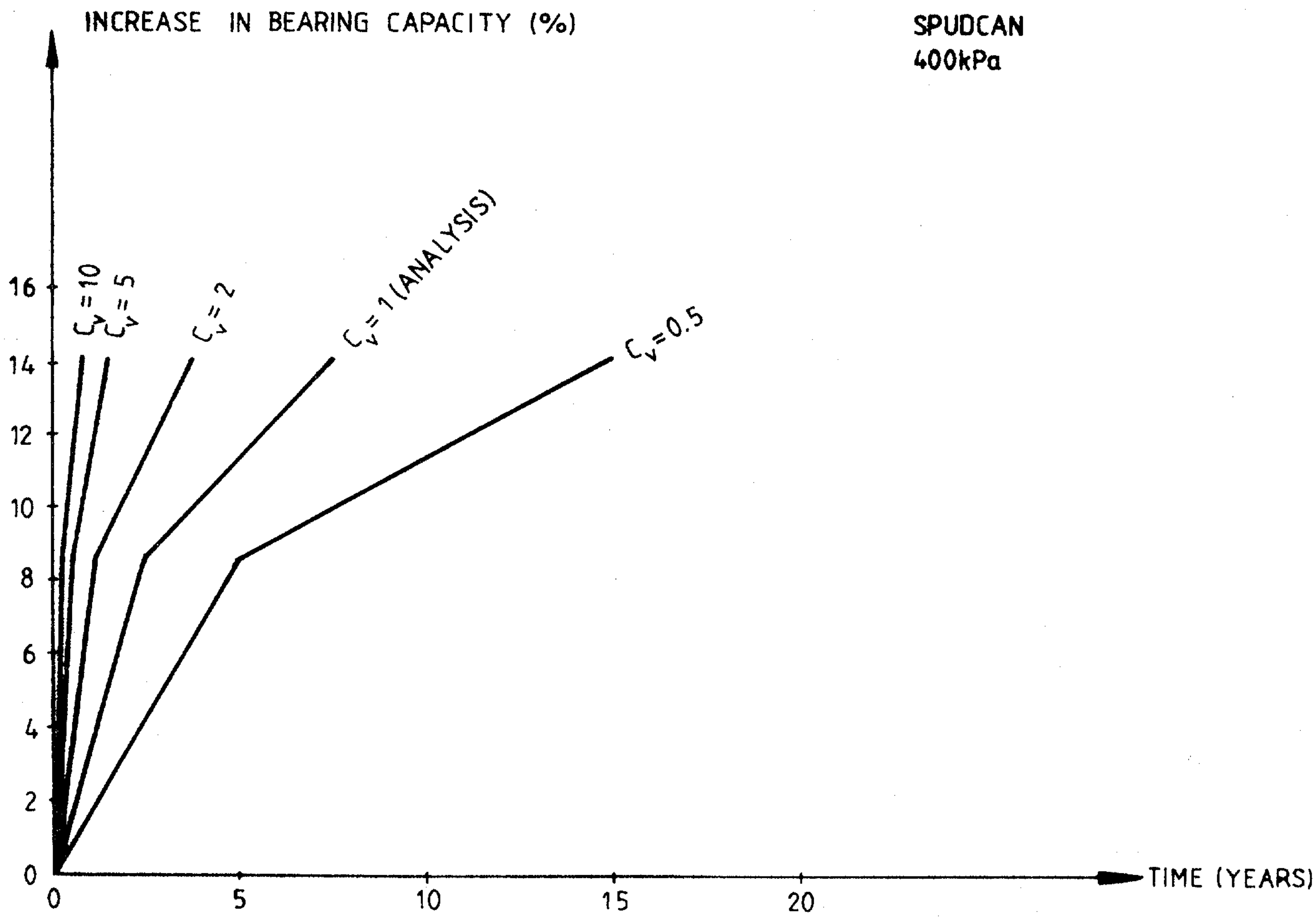


Fig. 3.15 Increase in bearing capacity of spudcan.

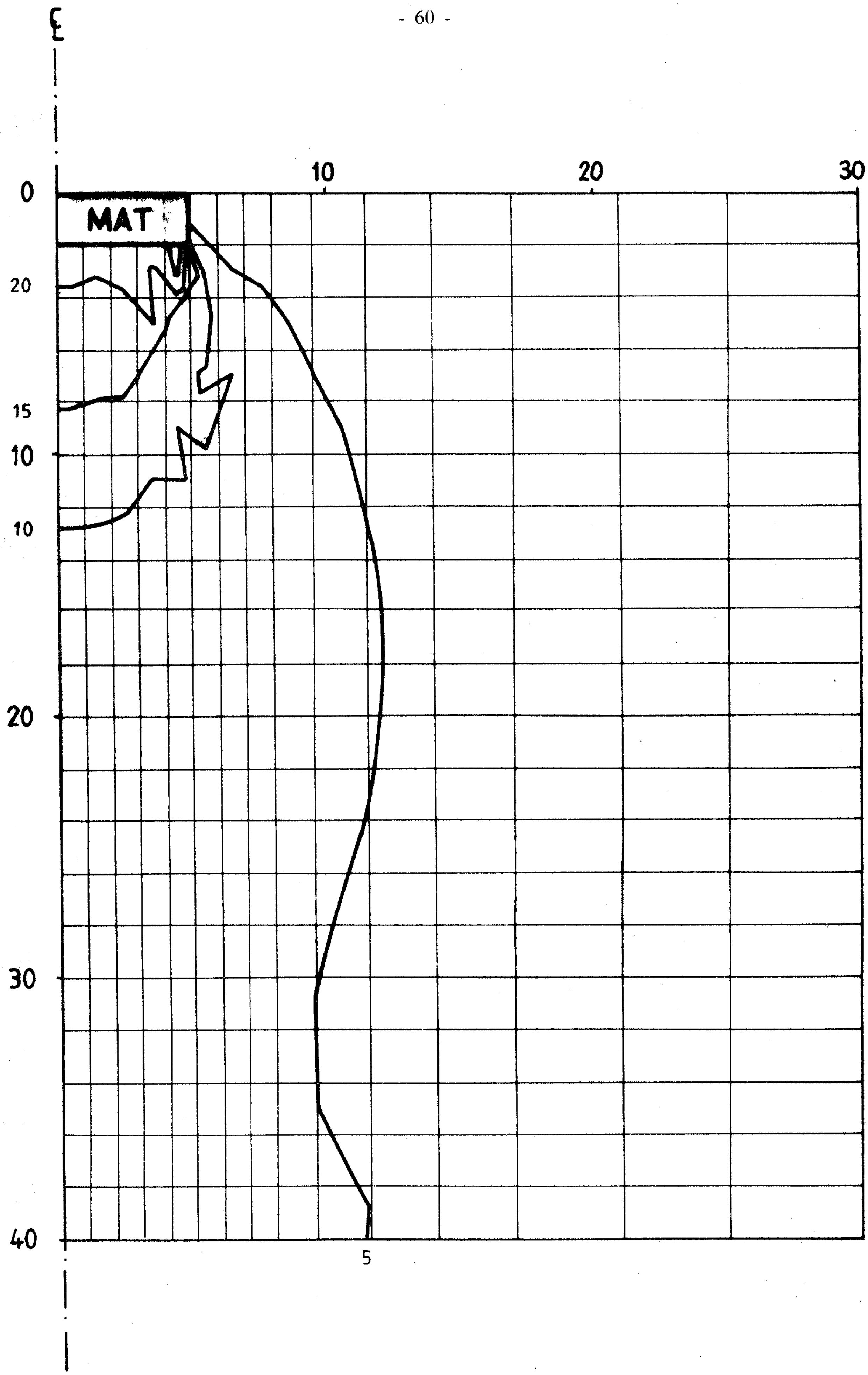


Fig. 3.16 Pore pressure distribution in soil beneath mat. Initially, after application of 0.3 MN/m (30 kPa) vertical load.

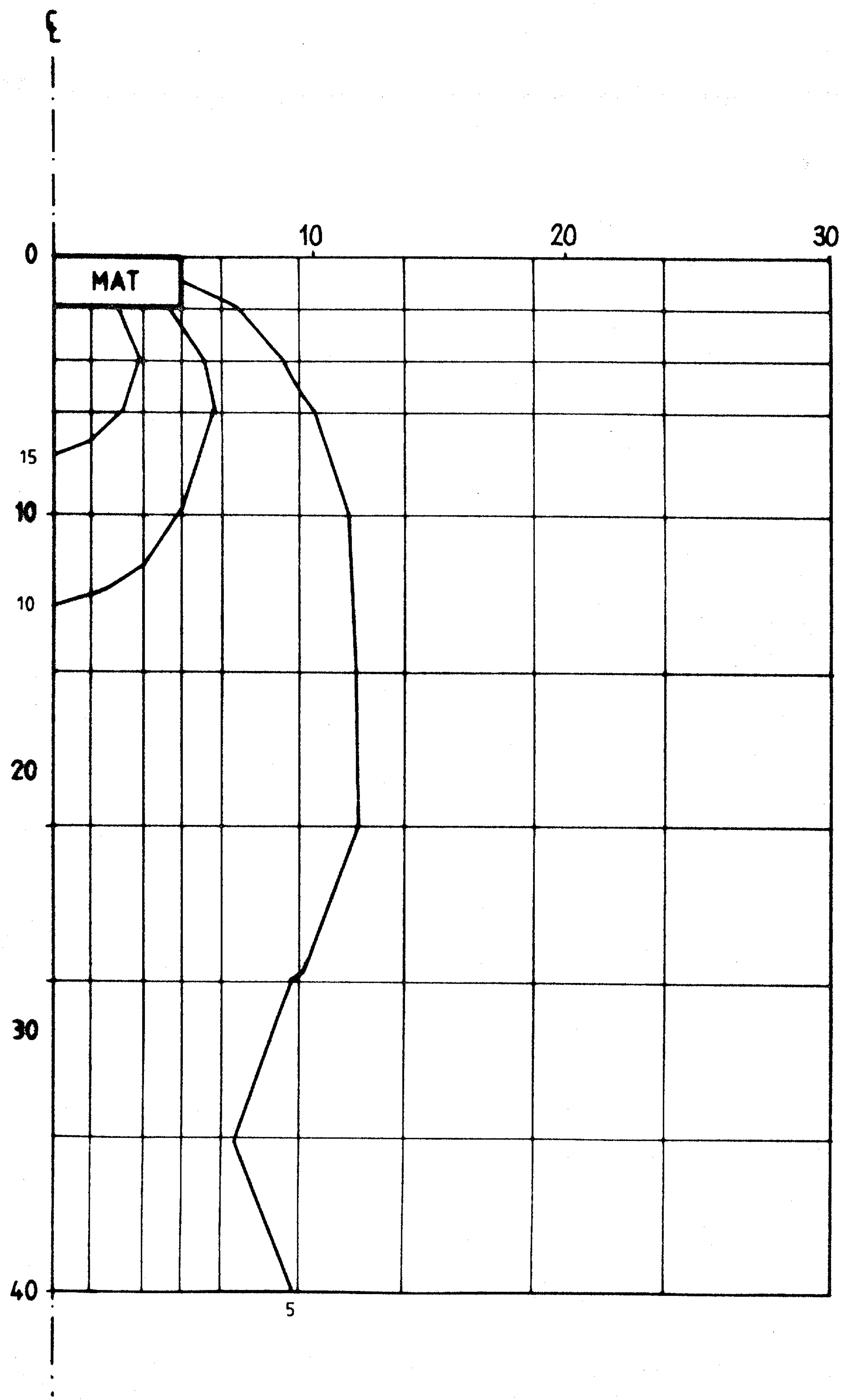


Fig. 3.17 Pore pressure distribution in soil beneath mat. After 25% reduction underneath mat, for 0.3 MN/m (30 kPa) vertical load.

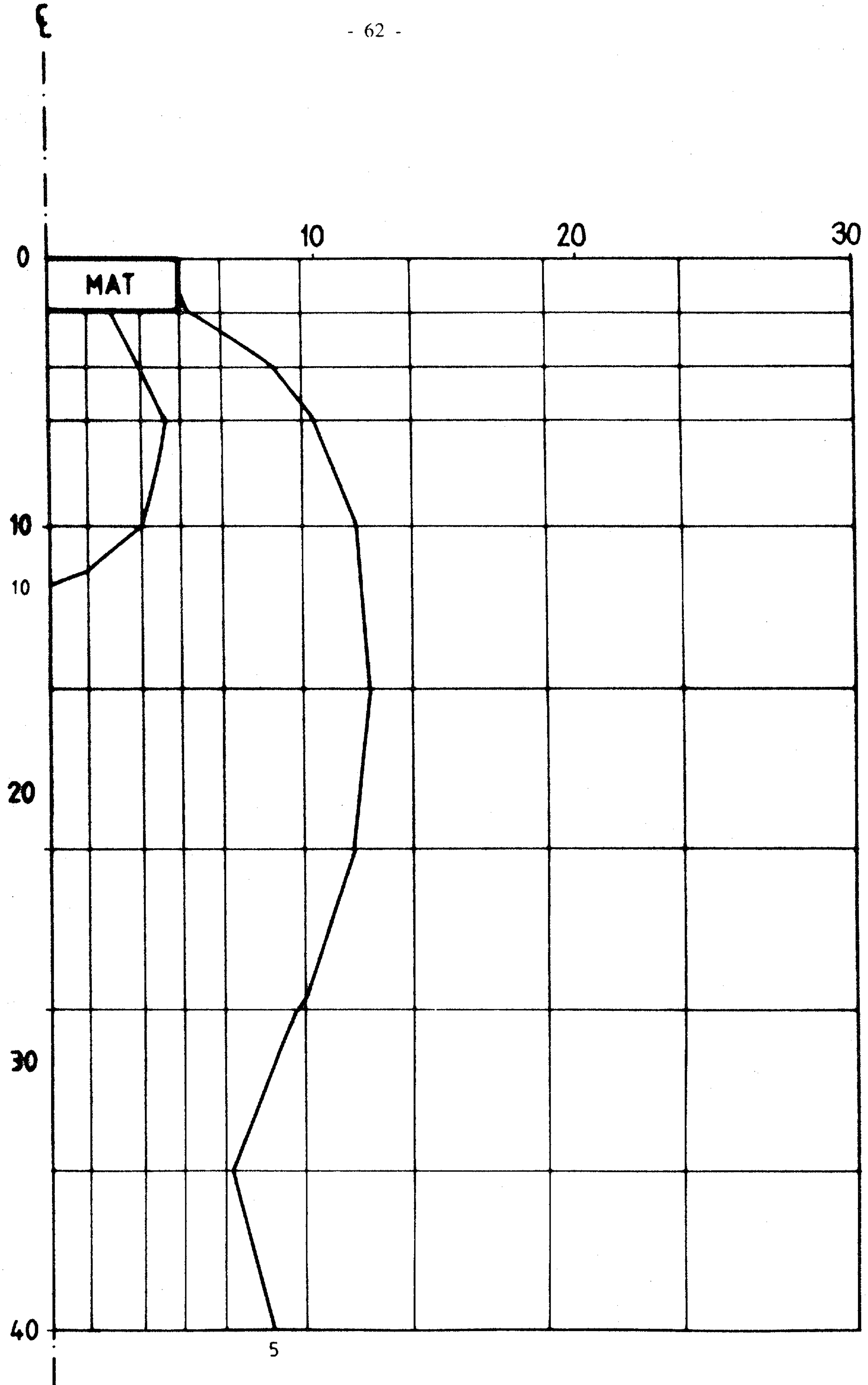


Fig. 3.18 Pore pressure distribution in soil beneath mat. After 50% reduction underneath mat, for 0.3 MN/m (30 kPa) vertical load.

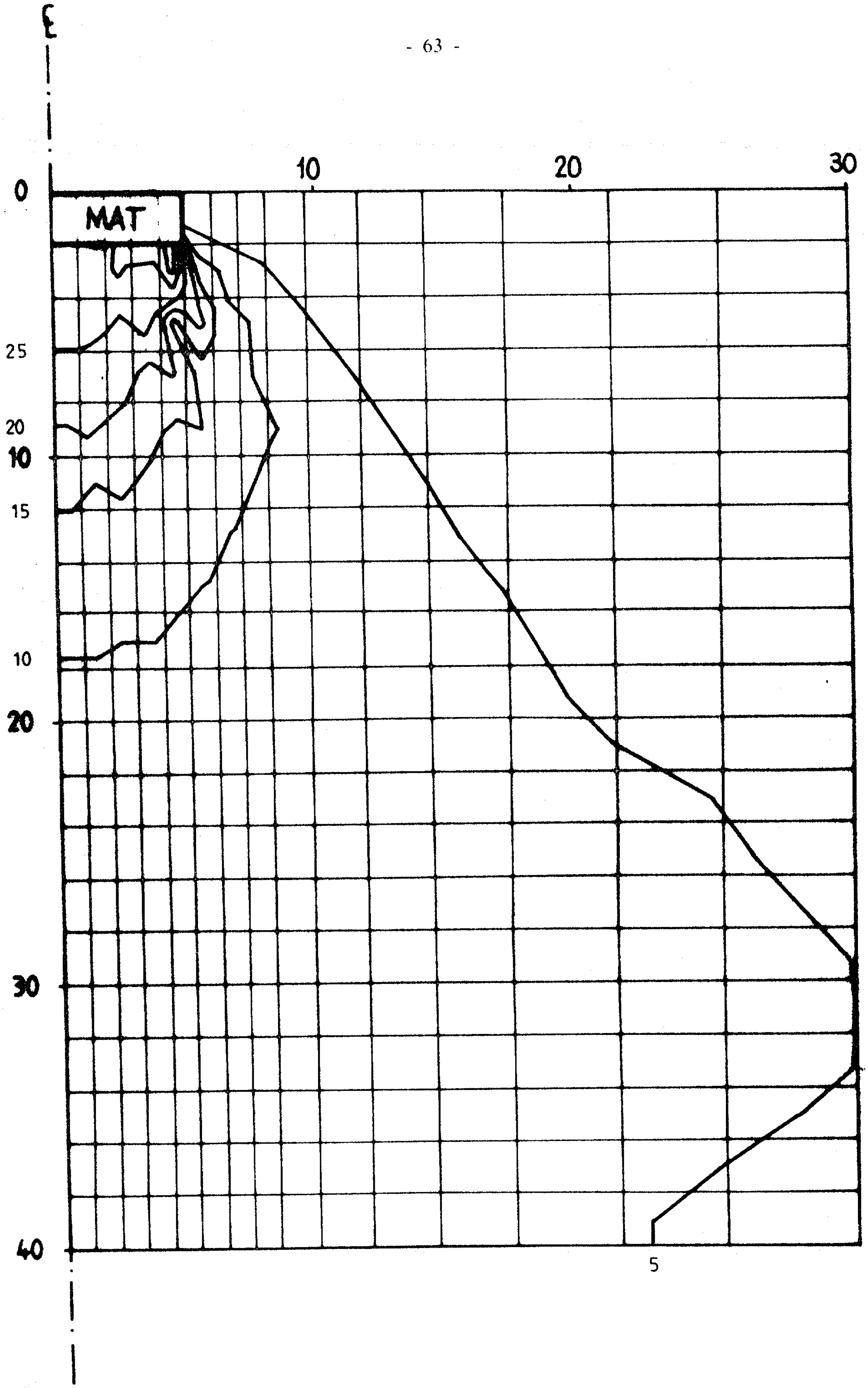


Fig. 3.19 Pore pressure distribution in soil beneath mat. Initially, after application of 0.4 MN/m (40 kPa) vertical load.

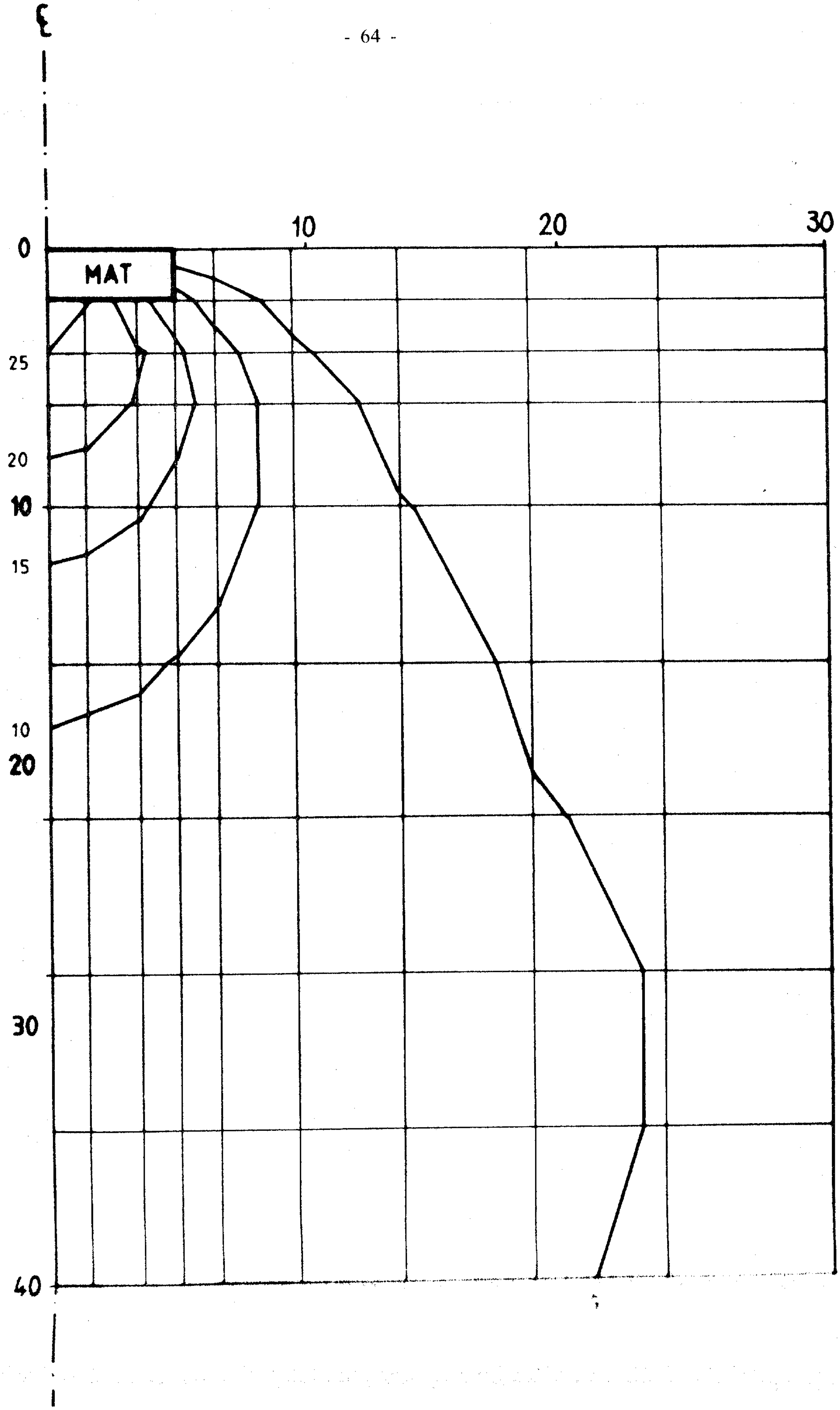


Fig. 3.20 Pore pressure distribution in soil beneath mat. After 25% reduction underneath mat, for 0.4 MN/m (40 kPa) vertical load.

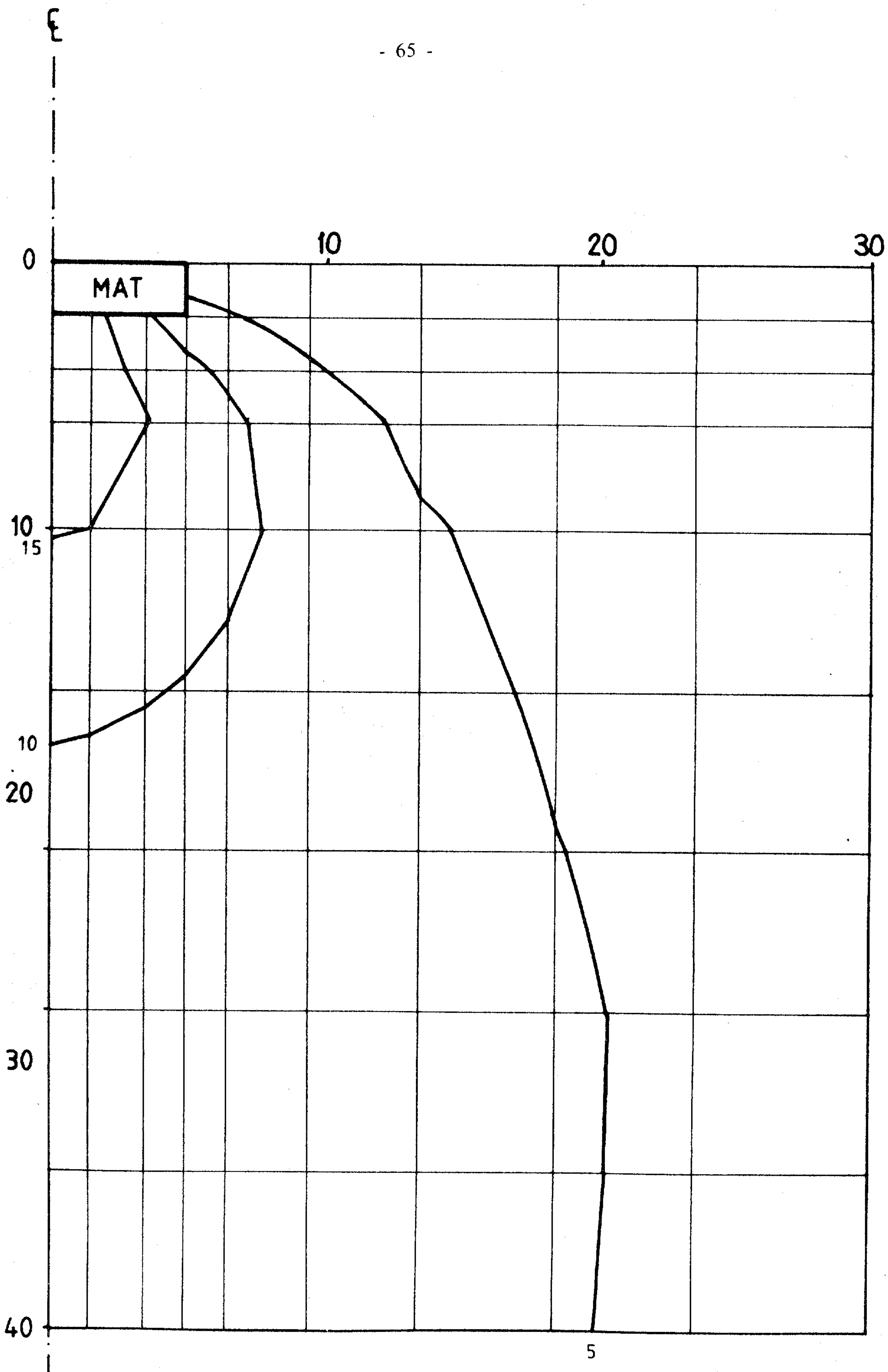


Fig. 3.21 Pore pressure distribution in soil beneath mat. After 50% reduction underneath mat, for 0.4 MN/m (40 kPa) vertical load.

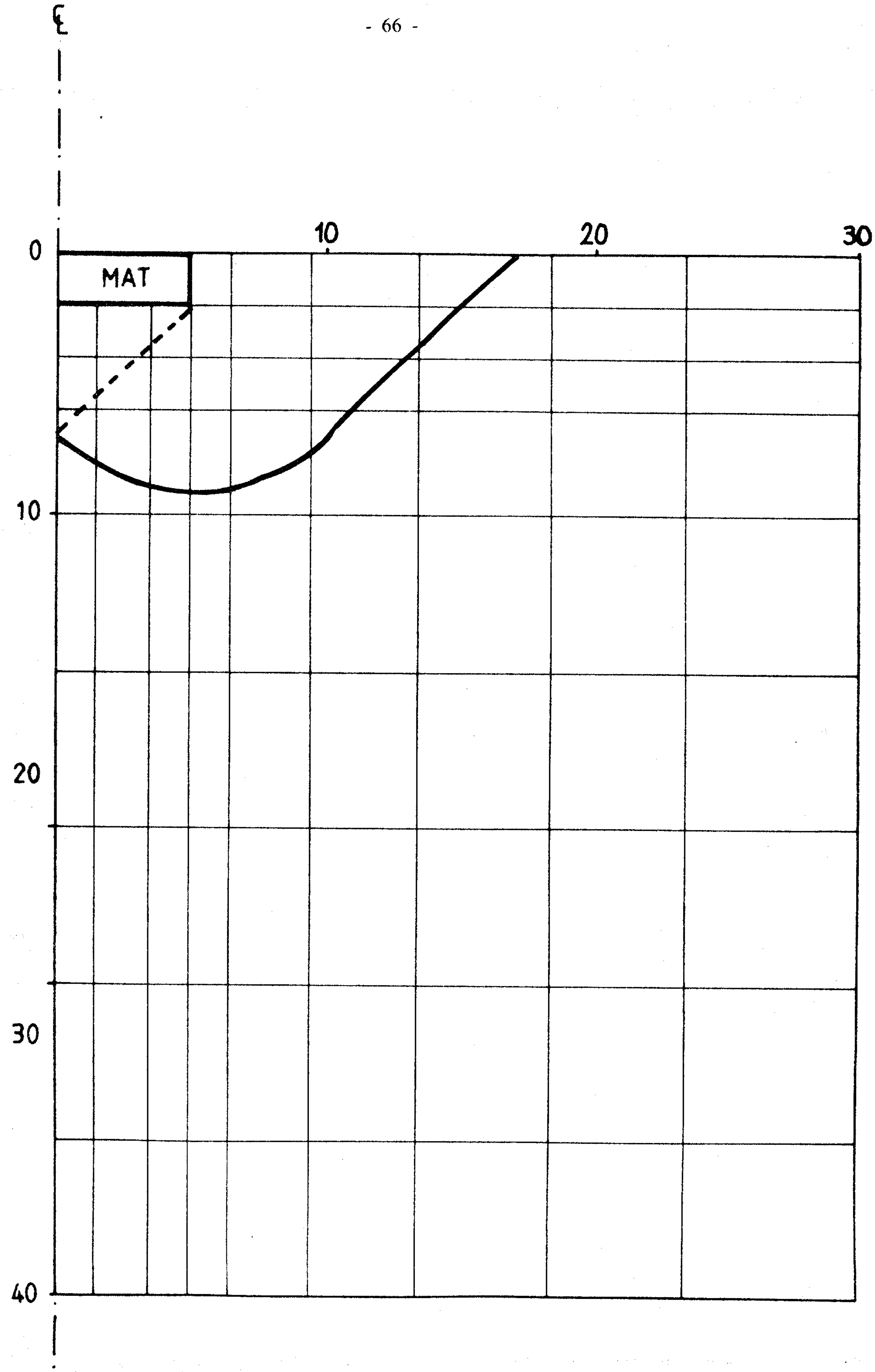
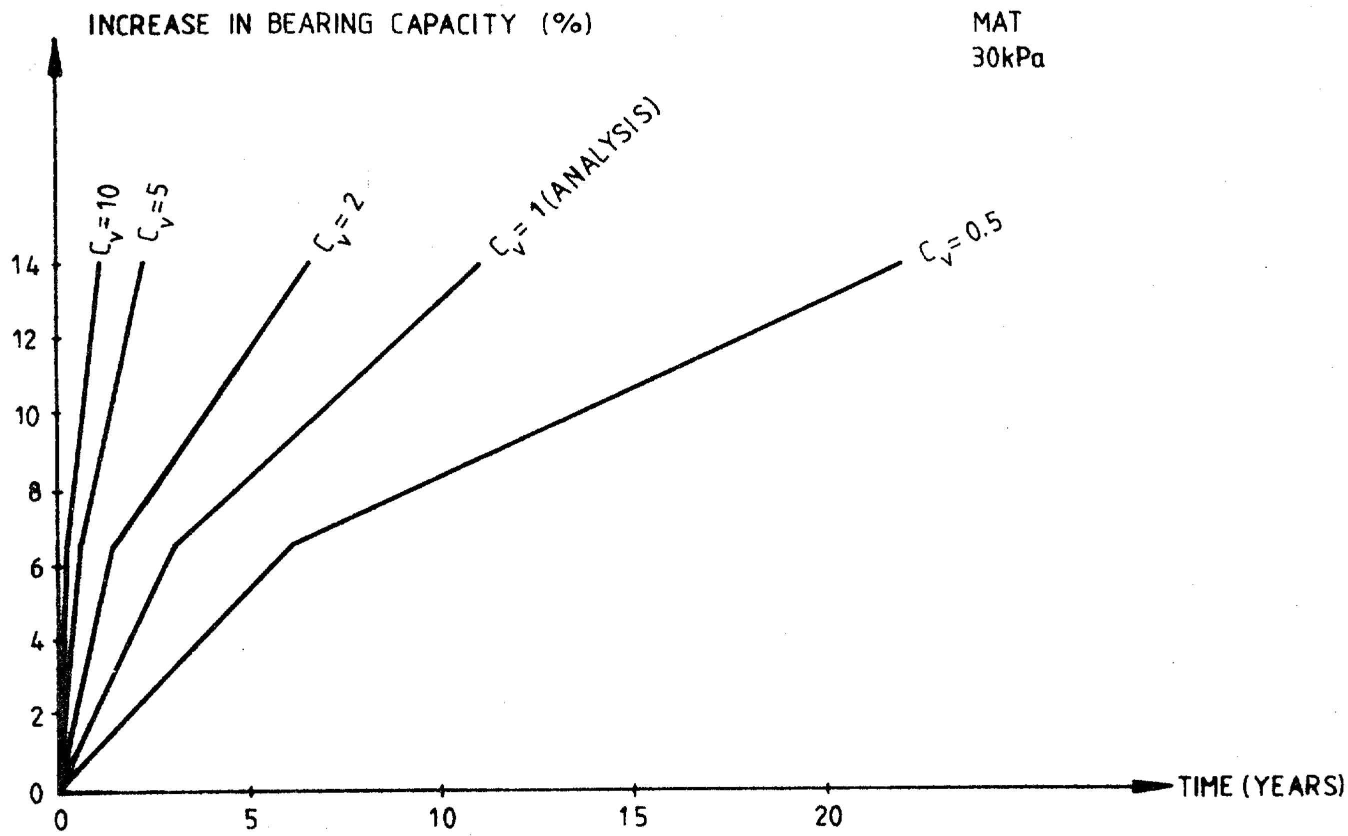


Fig. 3.22 Assumed critical slip surface for evaluation of increase in bearing capacity of mat.



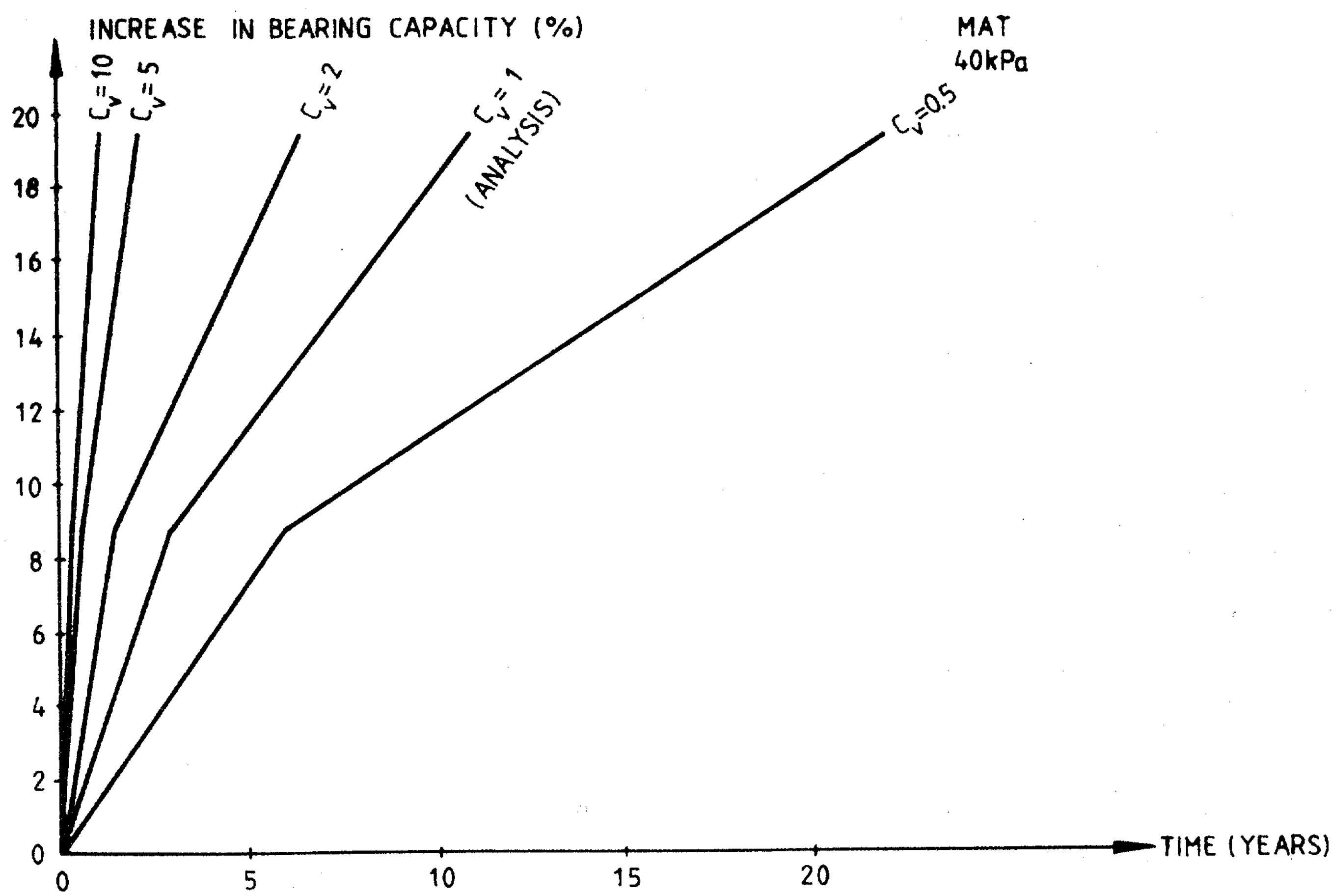
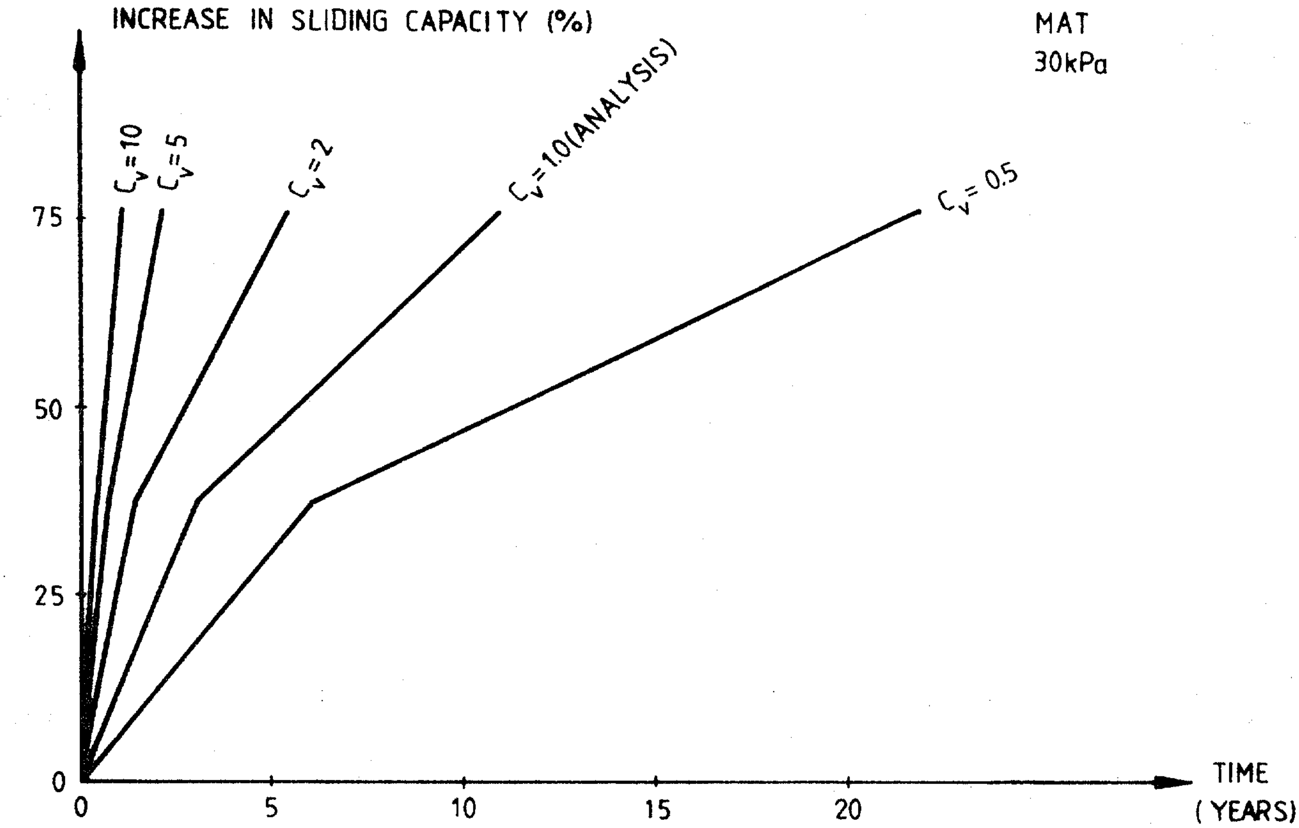


Fig. 3.23 Increase in bearing capacity of mat.



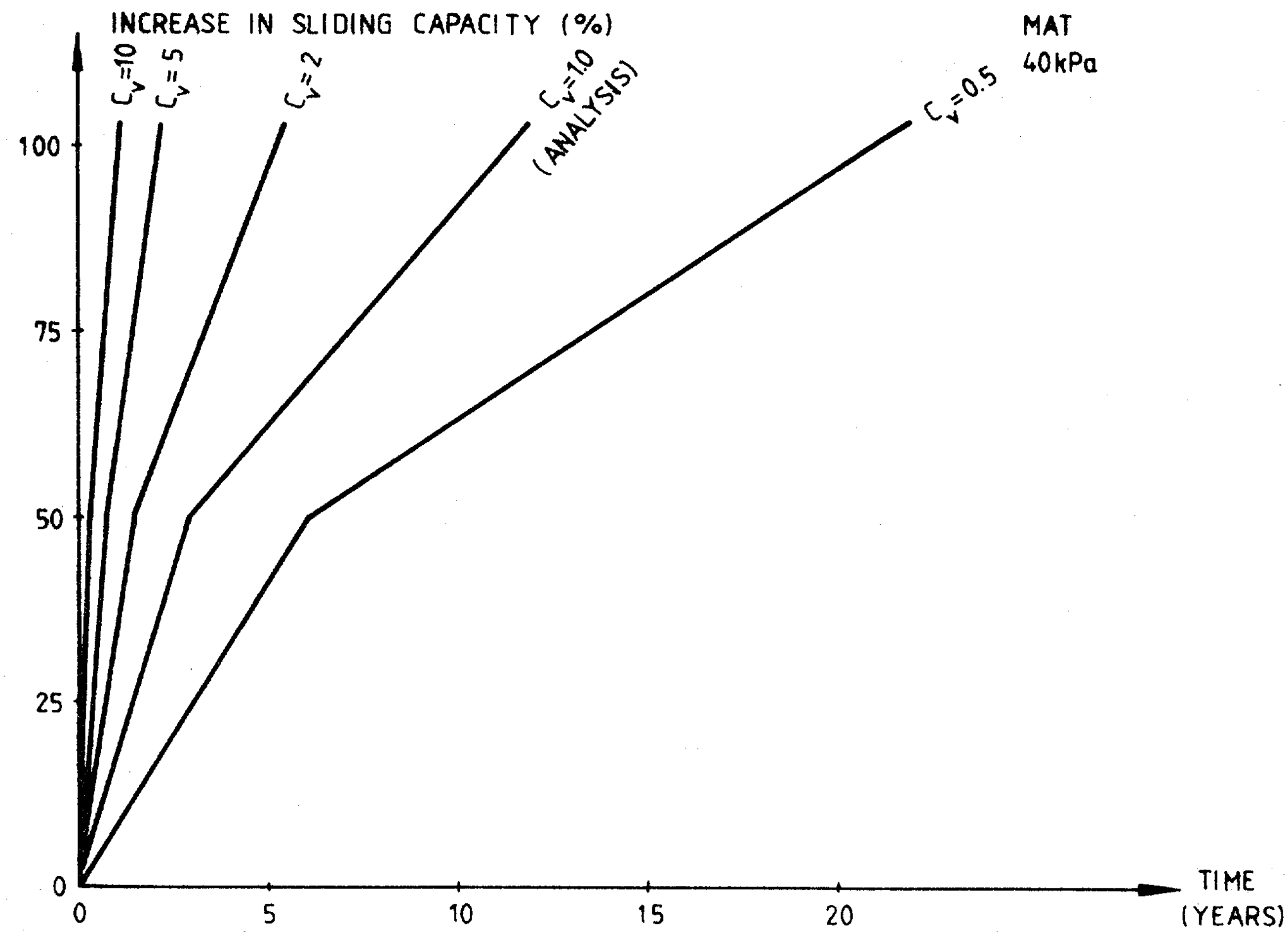


Fig. 3.24 Increase in sliding capacity of mat.

4. STABILITY OF MAT FOUNDATIONS UNDER GRAVITY AND ENVIRONMENTAL LOADS

4.1 GENERAL

As outlined in Chapter 2 the parametric study of the stability of mat supported jackup platforms was concentrated on mats on soft clay. In order to have a common basis for comparison of the three models evaluated, the same shear strength profile was selected for all models. Based on a series of trial tests with the program SAM and the three basic models described in Section 2.2 the undrained shear strength at mudline s_{uo} was chosen to 10 kPa and the shear strength gradient k was chosen to 1.2 kPa/m. With this shear strength profile the safety against foundation failure for the basic cases lies approximately in the range required for fixed gravity platforms.

With the selected basic parameters the safety factors for the three basic cases differ somewhat. For all models sliding turned out to be the critical failure mode with safety factors of 1.62, 1.52 and 1.34 for Model 1, 2 and 3, respectively.

The selection of a "basic" wave period was based on a series of runs where the wave period was varied over the range of interest. Periods of 13, 14 and 16 seconds give approximately the minimum safety factors for the three basic cases somewhat dependent on failure mode investigated. A variation of +/- 2 seconds does not influence the results too much.

As outlined in the PP2 report four different failure modes are considered by the program SAM:

- sliding failure at the tip of the skirts
- local failure under the fingers or cross beams of the mat
- a shallow failure under the most stressed edge of the mat
- deepseated overturning failure

The program evaluates the safety against failure for all of these four failure modes, and for the latter two failure modes a large number of failure surfaces are investigated. The program steps through a single wave in 20 steps in order to define the most critical combination of forces.

The forces considered are:

- wind on the deck structure
- wave and current forces on the legs
- wave forces on the mat
- wave pressure on the seabed

The time histories of the wave forces acting on the three models for the basic case are shown in Figures 4.1 to 4.3. The phase difference between the overturning moment and the horizontal force and the relative contribution of the different force components is seen to vary as a function of the water depth.

The large component of overturning moment and vertical force caused by wave pressure on top of the mat is counteracted by vertical wave pressure on the sea bed next to the mat. For the sliding, local and shallow failure modes the effect of this force component will be small, while for the deepseated failure mode this component will be the dominating force.

In the following the results of the parametric study will be presented and commented on. The main results have been summarized in Figure 4.4 where the factors of safety against failure in any of the four considered failure modes are shown as functions of the variation of individual parameters.

The safety factors shown are the minimum values occurring during a single wave. They do generally not occur at the same time, however the resistance against sliding, local failure and shallow failure is heavily dependent on the total horizontal force, and in all cases investigated the minimum safety for these failure modes occurred during time steps 16 to 17, i.e. with a phase lag of some 270 to 300 degrees relative to the passage of the wave crest.

The deepseated failure mode is totally dominated by the wave pressure acting on the mat and the sea bed. The overturning effect is at its maximum when the steepest part of the wave passes the platform and the horizontal force acts in negative direction rotating the platform and the soil about a point located above the sea bed. This load combination occurs during time step 6, i.e. with a phase lag of 90 degrees.

The safety factors presented are total safety factors. The middle value of the x-axis of

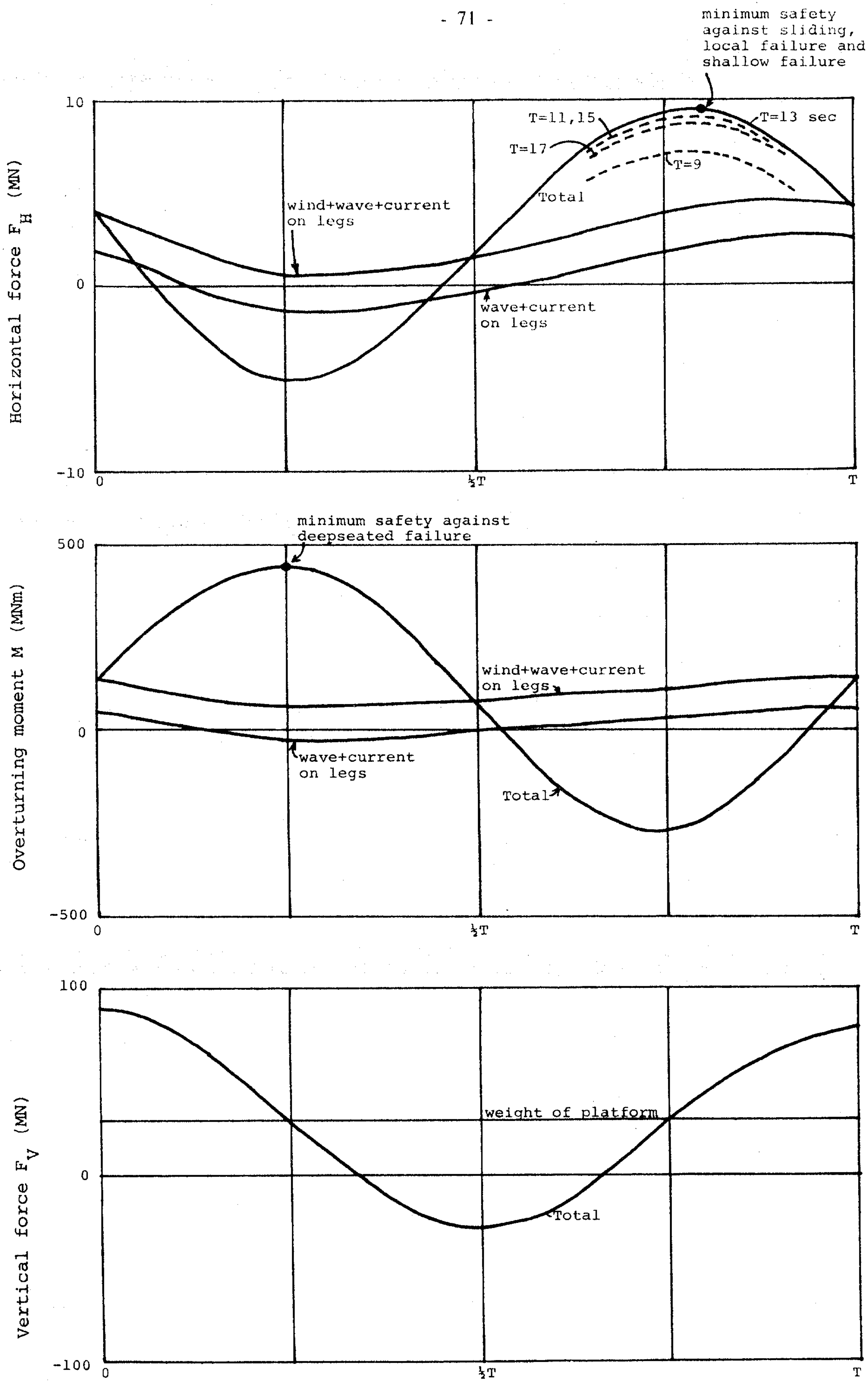


Fig. 4.1 Time history of foundation forces relative to mudline during passing of one wave, Model 1

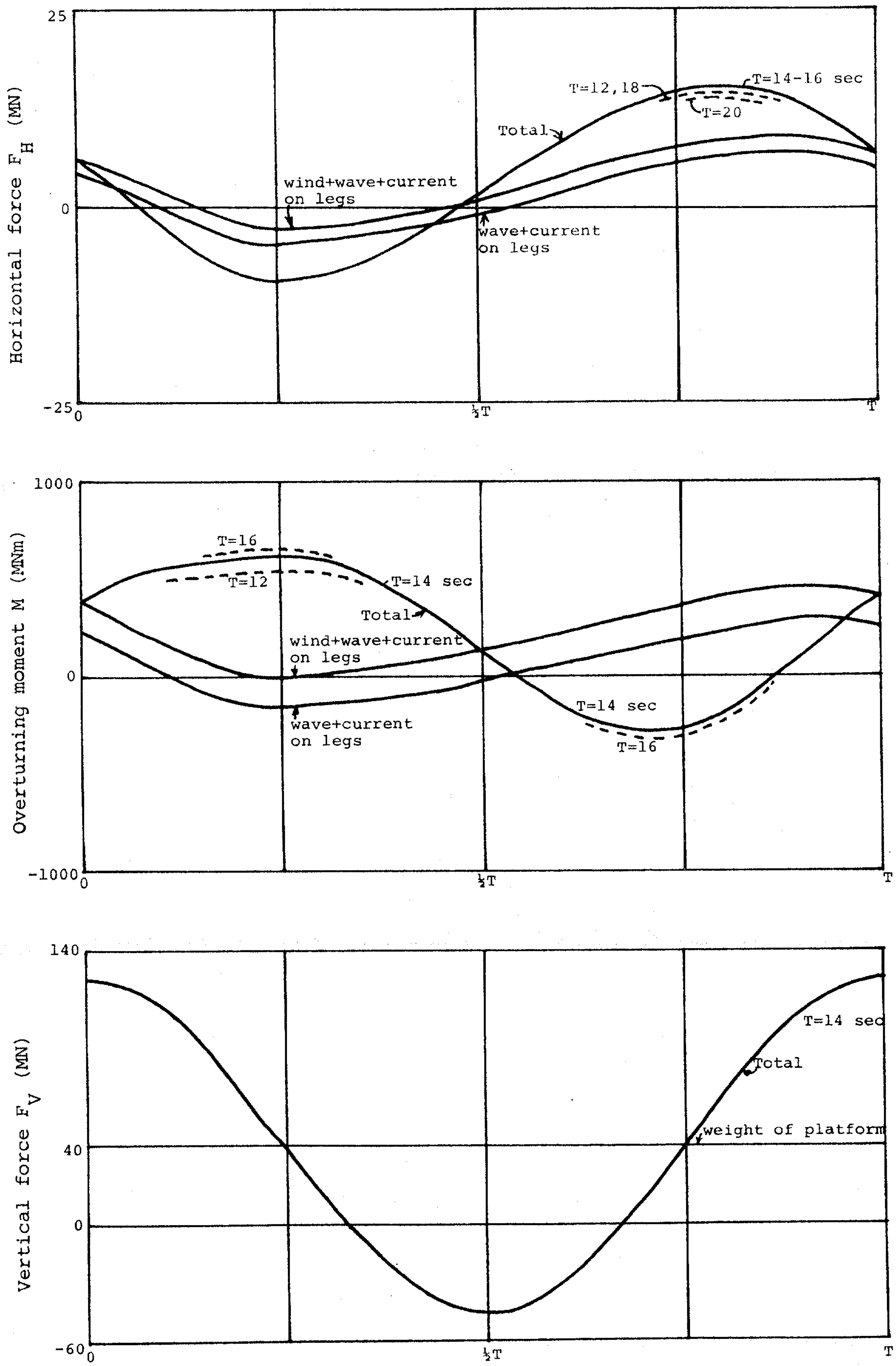


Fig. 4.2 Time history of foundation forces relative to mudline during passing of one wave, Model 2

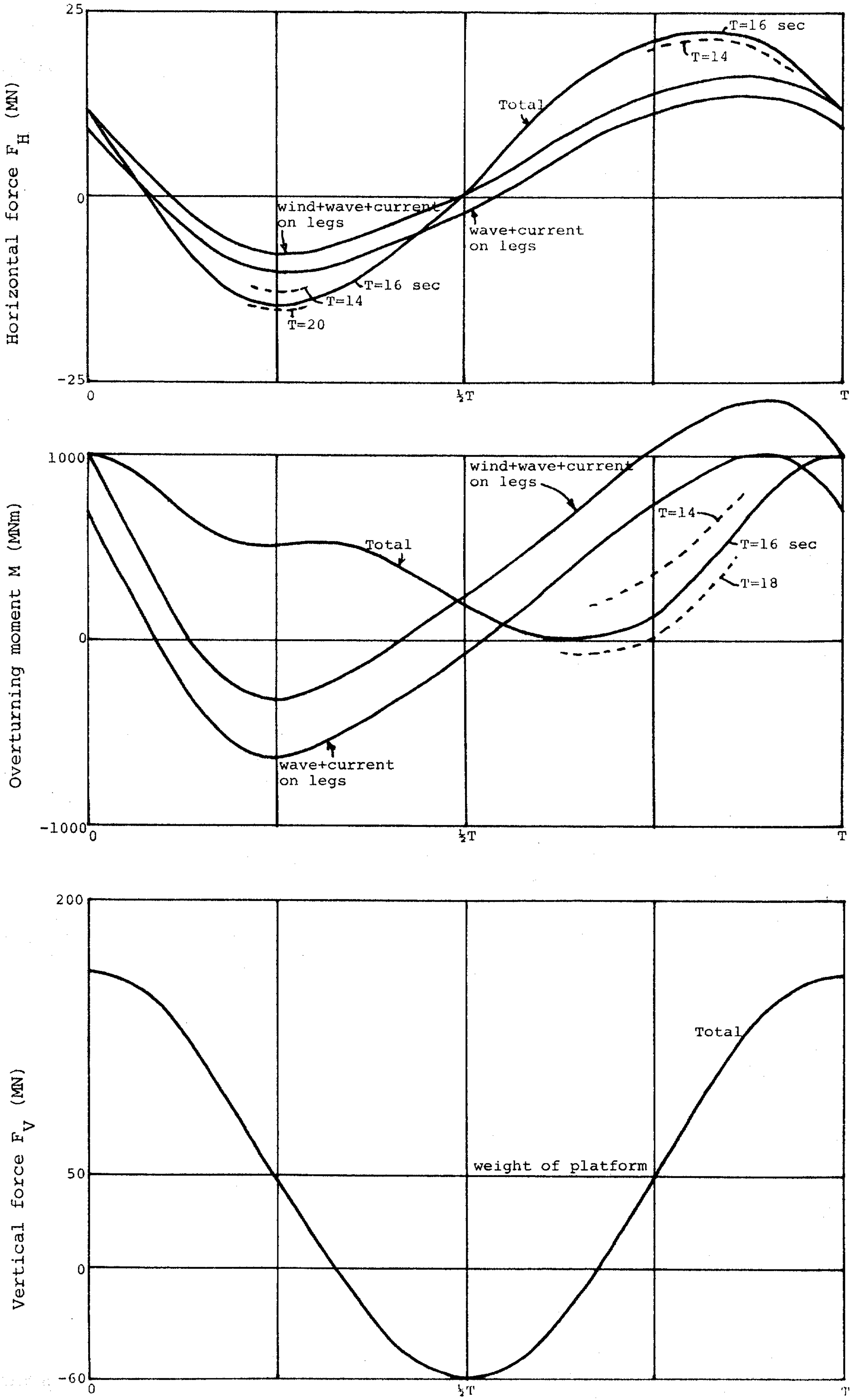
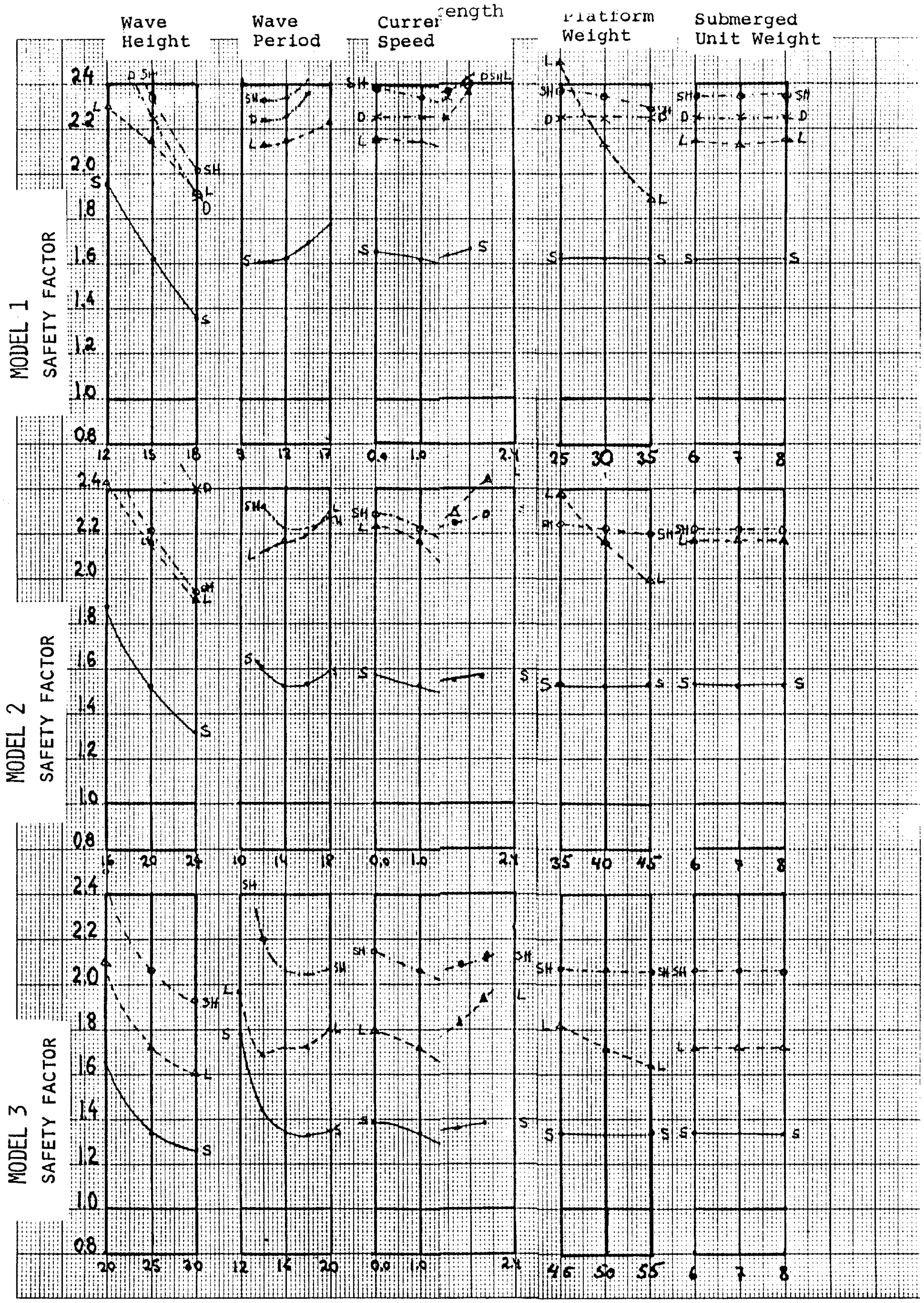


Fig. 4.3 Time history of foundation forces relative to mudline during passing of one wave. Model 3



Legend

———————— = S = sli

- - - - - - = L = loc

-·-·-·-·- = SH = sha

-··-··-··- = D = dee

...arameter study for mat foundation: ...ction of variation of individual parameters.

each diagram corresponds to the basic cases and will thus give the same safety factors in all diagrams.

It must be stressed that only one parameter has been varied at a time. Any coupling of parameter variations should be done with great care as nonlinear effects may be strong in some cases. It is also clear that other combinations of parameters for the basic cases may lead to changed results regarding the sensitivity of the different failure modes to variation of the different parameters. Nevertheless, the general findings are believed to be representative for mat supported platforms subjected to severe environmental conditions.

4.2 EVALUATION OF PARAMETER VARIATION ON THE FOUNDATION STABILITY

4.2.1 Some Comments to the Basic Cases

The critical failure mode for the basic cases was sliding. For Model 1 at 30 m water depth the three other failure modes are comparable. With increasing water depth the deepseated failure mode is decreasingly less important due to a strong reduction in the relative effect of wave pressure at the seabed and the mat, and in most diagrams the safety factors for this failure mode fall outside the range covered.

With increasing water depth the local failure mode is gradually getting more important due to the increase in overturning moment and thus in edge pressure.

The evaluations presented in the following sections are all referred to the results of the performed parameter study as presented in diagrams in Fig. 4.4.

4.2.2 Wave Height and Wave Period

The wave height was varied with +/- 20 percent and is seen to have a relatively strong effect on all models and on all failure modes.

The variation in wave period with +/- 4 seconds around the basic values does not seem to have a very large influence. It should be noted that the lower wave periods

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This shows that changes in foundation capacities during e.g. environmental loading cause changes in e.g. moment distributions, as well in the legs as between the footings. This behaviour has been verified by the parameter studies carried out and presented elsewhere in this report, and it is obvious that this is as much related to changes in foundation stiffnesses as it is to changes in foundation capacities during the various phases of loading, first of all due to the close relationship between these two foundation properties. Reference is therefore made to Section 6.1 where effects of foundation stiffnesses on moment and force distributions are treated in more detail.

utilized, the available moment capacity for that footing will become somewhat reduced relative to its original value as determined by the penetration depth and embedment of the footing.

This is a situation which typically occurs during environmental loading, and it is therefore interesting to watch the consequences for the force and moment distributions in the platform legs: Consider now that the vertical load on one of the footings has increased, and assume that the applied moment on that footing is unchanged. The degree of mobilization with respect to moment loading has then increased, and due to the nonlinearities of the soil this has led to a reduced rotational stiffness of the footing. Additional moment loading will therefore tend to be absorbed by other footings where rotational stiffnesses have not decreased, especially when the applied moment on the considered footing is approaching the current moment capacity of that footing. When the considered footing cannot take more moment when additional moment loading occurs, the corresponding top leg moment will become larger than if the footing could still have absorbed more moment. See Fig. 6.2.

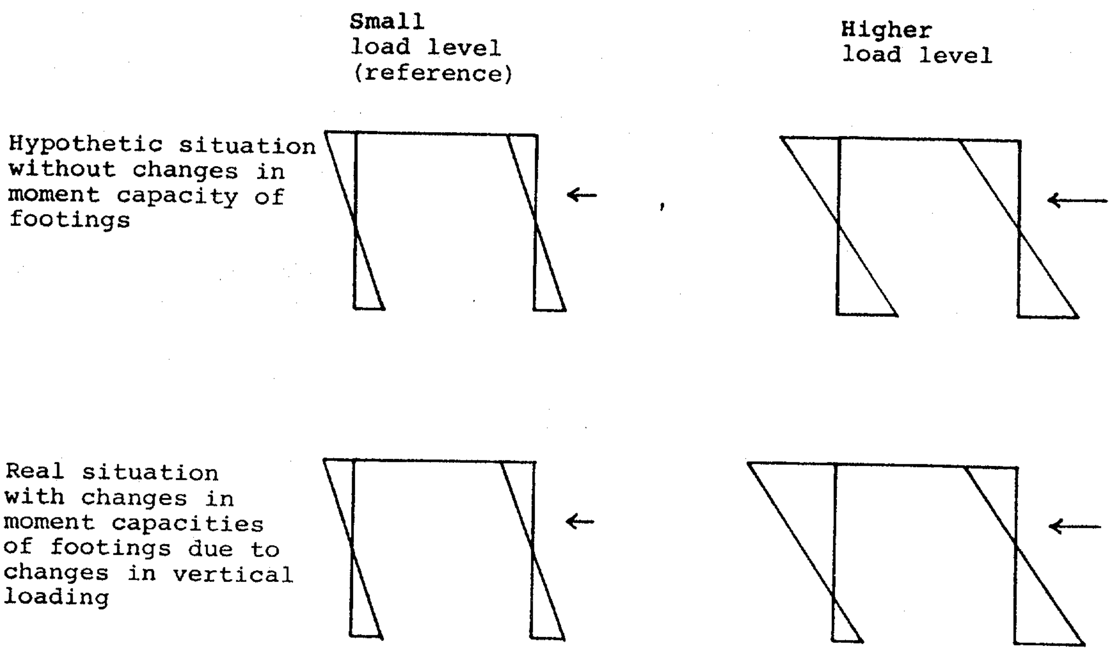


Fig. 6.2 Principle sketch for moment distributions in platform legs

ence is therefore made to Section 5.4 of this report where these topics are discussed.

The afore-mentioned observations with respect to force and moment distributions between the footings of a jackup platform have all been discussed with reference to foundation stiffness and soil-structure interaction at the footings. One should, however, not forget about the influence from the structural stiffness when the soil-structure interaction is considered. The leg stiffness governs more or less how much rotation is achieved at the corresponding footing and thereby - together with the rotational stiffness of the footing - how much of the available moment capacity of the footing is utilized. The stiffer the leg is, the less will the rotation of the footing be, and the higher will the top leg moment be. And the higher the rotational stiffness of the footing is, the less will the top leg moment be. This demonstrates an important feature of the soil-structure interaction, namely its dependency on soil stiffness in combination with structural stiffness.

Environmental loads are dynamic loads, and dynamic amplification of stresses and thereby of e.g. top leg moments may be critical by causing exceedance of static stress values if the exciting loads occur at frequencies close to the natural frequencies of a topical jackup platform. Such dynamic amplification is governed by the foundation stiffnesses in combination with the structural stiffness and thereby by the soil-structure interaction as the variation in the foundation stiffnesses, e.g. during the various phases of a load cycle, causes the natural frequencies of the structure to alter correspondingly. It is therefore important to assess the effects of the soil-structure interaction on the natural frequencies and thereby on the dynamic amplification factors for determination of e.g. critical stresses and top leg moments. This must be done properly by taking into account important parameters like soil strength, stress levels in the soil for actual loading, spudcan geometry, leg stiffnesses and leg geometry etc.

6.2 EVALUATION OF ULTIMATE FOUNDATION CAPACITY OF INDEPENDENT-LEG-SUPPORTED JACKUP PLATFORMS AND CORRESPONDING FORCE AND MOMENT DISTRIBUTION IN THE LEGS

Generally, the total foundation capacity of an independent-leg-supported jackup platform is composed of horizontal, vertical and moment capacities for each of its footings. Due to the soil behaviour, these capacities are coupled in such a way that if e.g. the vertical load on a footing is increased and thus more of the vertical capacity is

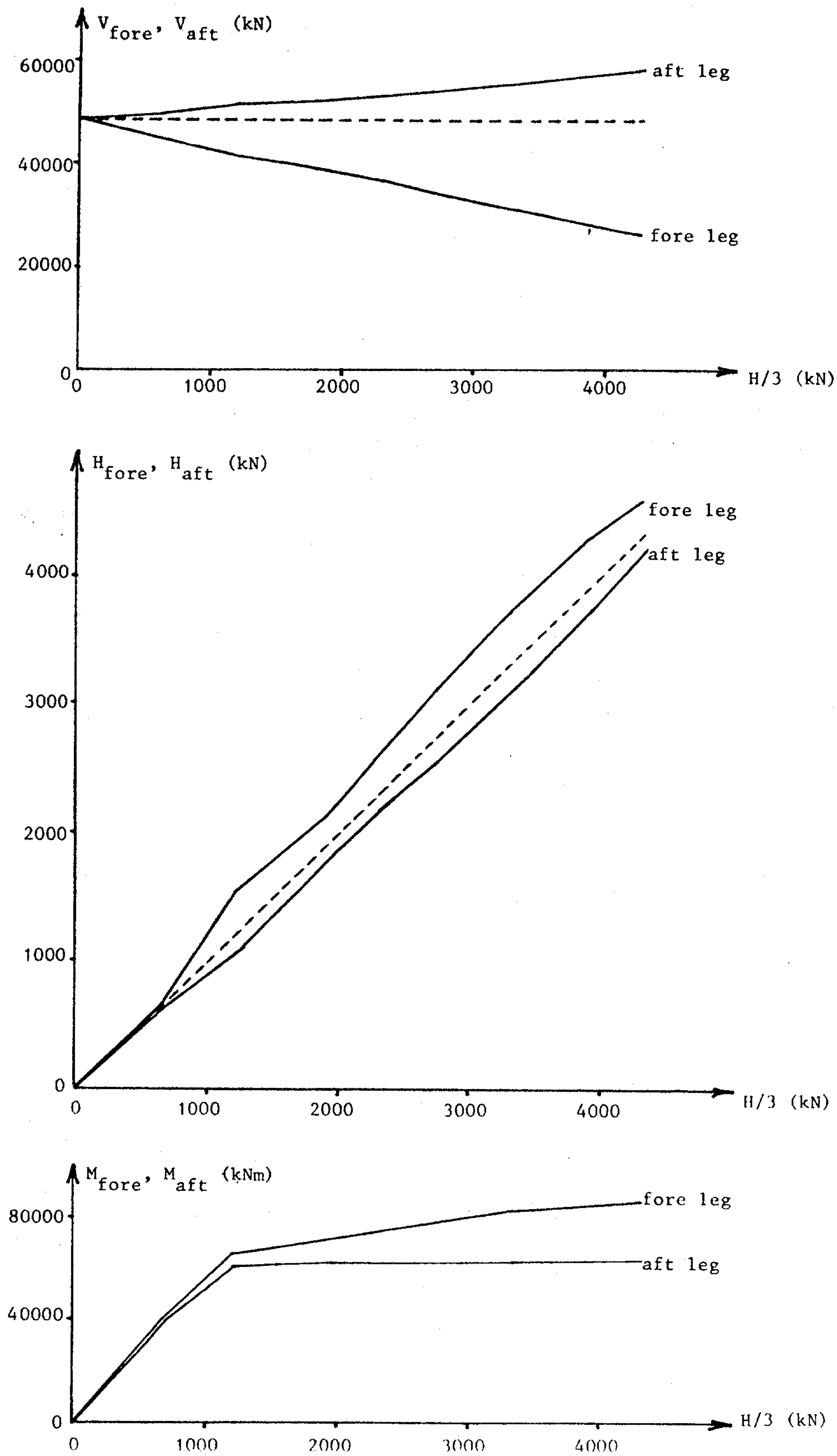


Fig. 6.1 Forces and moments transferred to the footings of a three-legged platform, presented as functions of one third of the applied horizontal environmental force heading in at the front.

cal.

The parameter study presented and discussed in Chapter 5 of this report may be interpreted with respect to effects of soil-structure interaction in a somewhat qualitative way:

Prior to initiation of environmental loading the platform weight is the only acting load, and all footings are subjected to the same vertical load and have identical sets of foundation stiffnesses. Consider now a situation where environmental loads are heading in at the front: At small load levels where stiffnesses are not much changed the horizontal forces due to the environmental loading are almost equally distributed between the legs. At higher load levels where the aft legs become the most stressed legs with respect to vertical loading the fore leg tends to attract an increasing part of the additional horizontal force while the aft legs do not get that much additional load. With respect to moments acting on the footings it is observed that the least vertically stressed legs (in this case the fore leg) absorb the largest footing moments. The unequal distribution between the footings at higher load levels seems to be more significant with respect to distribution of moments than with respect to distribution of horizontal forces, but this may be ascribed to the fact that only say 50-60 % of the available horizontal capacity is normally utilized, and a more distinct unequal distribution of horizontal forces between the footings may very well occur at higher degrees of horizontal mobilization. Generally, the above findings may be explained by the fact that the least vertically stressed footings have most capacity left for other types of loading, maybe especially for moment loading, and will thus naturally absorb e.g. larger moments than more heavily stressed footings which have lower moment capacities and lower corresponding rotational stiffnesses available.

The above-mentioned findings are illustrated in Fig. 6.1 which shows diagrams of forces and moments transferred to the footings of a three-legged jackup platform with environmental loading heading in at the front. It should be noted that the same conclusions as those referenced above with respect to force and moment distributions between the footings do also apply for the case that the environmental loading is heading in from the aft direction. Soil-structure interaction at the footings of independent-leg-supported jackup platforms is very much related to the stability of such platforms, and some reflections on the soil-structure interaction and its importance for force and moment distributions in and between the platform legs are therefore inevitably covered by the evaluations of capacity and stability for this platform type. Refer-

6. SOIL-STRUCTURE INTERACTION

6.1 EVALUATION OF EFFECTS OF FOUNDATION STIFFNESS ON MOMENT AND FORCE DISTRIBUTION IN AND BETWEEN THE LEGS

The total foundation stiffness of a footing consists of vertical, horizontal and rotational stiffnesses (as long as a two-dimensional modelling is considered with neglection of a possible stiffness in twisting). These stiffnesses are dependent on the vertical, horizontal and moment capacities, respectively, and as these capacities are coupled, the stiffnesses themselves will also be coupled.

The soil-structure interaction at the footings of a jackup platform influences the moment capacity and thereby also the rotational stiffness of the footings. This is dependent on how the total available soil capacity is utilized for mobilization of reaction against horizontal force, vertical force and moment acting on each footing. This implies e.g. that the less the vertical force on a footing is, the higher will the available moment capacity and the corresponding rotational stiffness be.

The importance of this may be illustrated by an example: Jackup platforms located in areas with deep waters are generally subjected to large overturning moments created by wind and waves, and this may lead to excessive top leg moments. An increase in moment capacity of the footings with corresponding increase in rotational stiffness will reduce the top leg moments and thereby improve the safety level against fatigue failure or allow the topical rig to operate at larger water depths than originally intended. Such an increase in moment capacity may be achieved by an improved embedment of the footings, but could for a certain considered footing also be a result of soil-structure interaction in terms of an adequate distribution of forces between the footings.

This indicates that it is important to study the effects of the soil-structure interaction at the footings, i.e. how the reaction forces are composed of horizontal force, vertical force and moment at each footing, and how this composition is changed from one stress level to another during environmental loading by redistribution of forces and moments between the legs due to changes in foundation stiffnesses. This in turn will e.g. allow for a determination of how much moment capacity one can count on at various stages during the environmental loading, and with reference to the above-mentioned example one can assess whether or not the corresponding top leg moments will be criti-

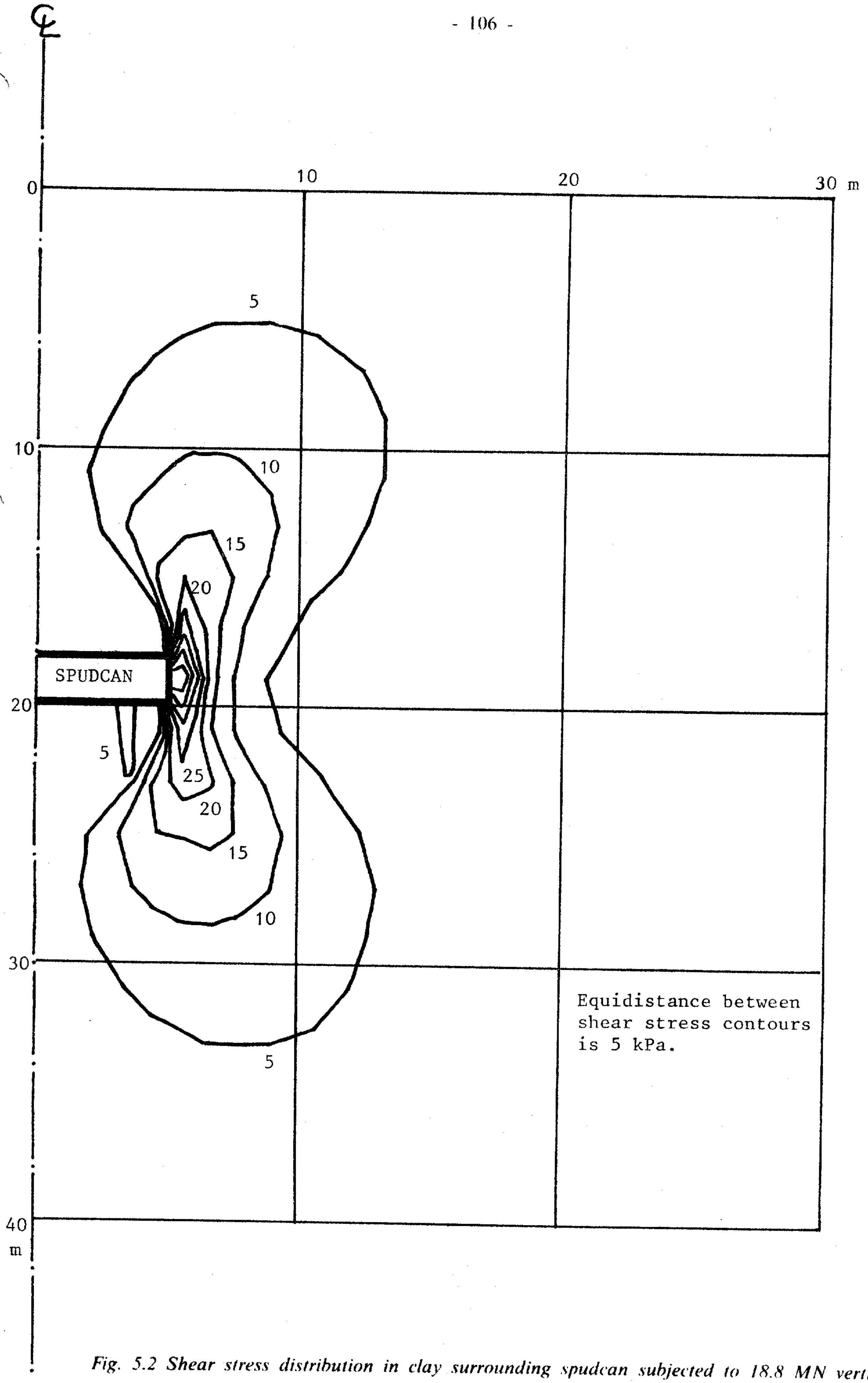


Fig. 5.2 Shear stress distribution in clay surrounding spudcan subjected to 18.8 MN vertical loading.

drainage in combination with rather short drainage paths, and a study by means of the OCEAN2 program revealed that generated excess pore pressures in a storm will be neutralized almost immediately by such dissipation and will thus not cause any problems for the spudcan.

Hence the conclusion will be that for a spudcan located in sand the effects of repeated loading due to ocean waves in a storm will be of no practical significance, but one should bear in mind that in the case of earthquake loading which occurs much faster than ocean wave loading the pore pressure dissipation is not likely to be fast enough to cancel out accumulated pore pressures right away, and some effects of repeated loading may thus be expected and should therefore be considered for this special case.

strength degradation of the soil will to some extent contribute to speed up this shakedown of the spudcan.

When the spudcan during a storm is shaked down to a deeper level, it will usually achieve a somewhat increased bearing capacity, at least due to the increased overburden and usually also with a contribution due to increasing shear strength with depth. This will tend to stabilize the spudcan, at least for moderate displacements where the loads can be considered unchanged. Large displacements may involve unfavourable increases in (for instance) the topical jackup platform's overturning moment and may thereby lead to critical conditions for the platform. A special case occurs of the spudcan is located in a stiff clay underlain by a soft clay: For this case the degradation and stress redistribution in the stiff clay together with shakedown of the spudcan may cause punch-through to become governing, so for this particular case shakedown to a deeper level may be somewhat critical and thus not have any stabilizing effect, even for small displacements.

Conclusively, it may be stated that for a spudcan embedded in clay the effects of repeated loading are governed by the shakedown phenomenon. Shakedown is a very difficult problem to handle quantitatively as no adequate theory exists as of today. However, as indicated above, the effects of shakedown are usually considered not to be critical for a spudcan, at least for small displacements, but may become critical if large displacements are involved. Practically, shakedown of spudcans in a storm will require a little extra leg length to be available for the topical jackup platform so that the additional spudcan displacements caused by shakedown can be counteracted by jacking up the platform correspondingly.

A spudcan may very well be located in sand, too, and for this case it will usually not penetrate the soil much but remain on top of the soil deposit or maybe be partly embedded in the soil.

Generally, the effects of repeated loading of a foundation on sand consist of accumulated excess pore pressures that will be generated gradually and lead to a somewhat reduced strength of the soil. This may be analyzed theoretically when storm duration, average wave period and occuring stress levels in the soil are known, and the resulting reduction in bearing capacity may be determined.

However, a spudcan located on top of a sand deposit will allow for a very fast dissipation of generated excess pore pressures due to three-dimensional axisymmetric

5.5 EVALUATION OF TENSILE CAPACITY

The factor of safety against overturning of a jackup platform is required to equal or exceed 1.5, and this implies that tension will practically never occur in any of the platform legs. Evaluation of tensile capacity of spudcans is therefore usually not of current interest, and hence no parameter study has been carried out for this particular topic. However, the problems and mechanisms involved with tension of spudcans are the same as those related to pull-out of spudcans, and they may therefore - if desireable - be treated the same way. Hence, reference is made to the evaluation of pull-out resistance in Section 3.5 of this report.

5.6 DISCUSSION OF EFFECTS OF REPEATED LOADING

A spudcan of 10 m diameter, embedded in clay at 20 m depth, was analyzed for pure vertical loading in connection with the evaluation of consolidation effects, see Section 3.7. From this analysis a shear stress distribution in the soil surrounding the spudcan can be interpreted as shown in Fig. 5.2 for the 18.8 MN vertical loading case. This figure indicates that the induced shear stresses in the soil are concentrated within a relatively limited zone near the spudcans side and edges, and it may be noted that similar shear stress distributions can be found for the other load cases analyzed.

When the spudcan becomes subjected to cyclic loading in a storm, the cyclic shear stresses that superimpose the static shear stresses in the soil will initially obtain a distribution similar to that of Fig. 5.2. Zones along the sides and under the edge of the spudcan will consequently become heavily stressed and maybe overloaded by the cyclic loading, and the shear strength within these zones will thus become degraded. According to practical experience this degradation will be completed as early as after a few number of cycles at a certain stress level and will cause a redistribution of shear stresses to less stressed zones to occur. The shear strength degradation will further cause the bearing capacity of the spudcan to become somewhat reduced.

The repeated cyclic loading at a relatively high stress level will cause cyclic shear deformations to accumulate in the soil, and these will in turn lead to cyclic penetration of the spudcan, i.e. socalled shakedown which will bring the spudcan down to a somewhat deeper level where the soil on an average has been less influenced by the cyclic loading and consequently is not yet that much degraded. The afore-mentioned shear

All models are studied for environmental loads heading in from aft direction. This condition causes the highest vertical stress level in a single leg to occur and is therefore the most critical with respect to bearing capacity failure. For a defined basic case with shallow penetration of the spudcans, the moment taken by one of the aft legs is found to be 13-27% higher than for the single fore leg. This percentage appears to increase to 25-47% for deeply penetrated spudcans. For shallowly as well as deeply penetrated spudcans the lowest percentage is obtained for a model No. 3 jackup platform, while the highest percentage is obtained by a model No. 2 jackup platform. These findings imply that the legs with the least vertical stress levels absorb the largest moments, and the same conclusion applies when the environmental forces are heading in from the opposite direction, i.e. the front direction.

For all jackup models an increased preload weight appears to improve the moment capacity. The improvement of the moment capacity for a certain considered leg is higher the lower the vertical stress level in that leg is.

Reductions in wave height, current velocity, drag diameter and preload reduce the moment taken by the footing. The highest reduction is achieved for legs with the lowest vertical stress level.

Conclusively, it may be stated that the critical parameters with respect to moment capacity and distribution of moments are the preload level and the spudcan diameter. An increase of the spudcan radius will increase the foundation area (if the soil is not too hard), reduce the penetration depth and the moment in the spudcan, and increase the initial rotational stiffness, which in turn will improve the moment capacity. More preload weight results in larger foundation area (for partly penetrated spudcans on sand or in firm clay) or improved vertical bearing capacity due to a deeper penetration (provided that punch-through can be let out of consideration). The effect of additional preload will be larger the more the soil strength increases with depth, because deeper penetration leads to increased overturning stability.

The other parameters mentioned above, i.e. wave height, current velocity and drag diameter, are all found to enlarge the spudcan moments when they are increased and do not affect the moment capacity of the spudcans directly.

$F_{v,cap}$ = bearing capacity for vertical load after 1 meter additional penetration of fore leg

$SF_{b,3}$ = factor of safety against bearing capacity failure calculated for the case that no horizontal load is acting on fore leg.

The tables show that the factor of safety against bearing capacity failure, SF_b will only be improved by additional displacements of the fore leg if the spudcan is only partly penetrated or the soil has a very strongly increasing strength with depth. This is because additional penetration is accompanied by increasing vertical load due to hull displacements.

For the second situation mentioned above, i.e. structure exposed to environmental forces heading in from front direction, the fore leg is found to take approximately 10% more of the horizontal force than each of the aft legs. An increase in drag diameter will transfer most of the additional loads to the aft legs. Increase in current velocity and wave height will increase the load level, but will hardly affect the distribution of forces between the footings. Increased preload does not increase the load level for horizontal force, but will transfer more of the horizontal force to the aft legs.

For both considered situations, the most critical parameter with respect to the horizontal stress level is the drag diameter, followed by wave height, while the least critical parameter among those investigated is the current velocity. The preload level does not affect the load level, but have a slight effect on the distribution of forces between the legs. However, if the factor of safety approaches 1.0, for instance for a flat-based spudcan in a thin soft clay layer overlaying a hard crust or sand, the preload level could be important for the distribution.

When a jackup rig is exposed to environmental forces, the horizontal force will initially be equally distributed between the legs. As the load level subsequently increases, the leg or legs with the lowest vertical stress level will attract still more of the additional load.

5.4.2 Moment capacity of spudcans - an overview of the most important parameters and their effects on the moment capacity

Below, the most important parameters for moment capacity are studied for the three jackup models described in Chapter 2, namely wave height, current velocity, drag diameter and preload level.

Model	F_h (kN)	F_v (kN)	Inclination(^{o})	$F_{v,pre}$ (kN)	Leg length (m)	WG + WGV (kN)	$SF_{b,1}$	$SF_{b,2}$
1, shallow	1890	28000	3.86	30000	43	72000	0.94	1.07
1, deep	1800	29100	3.54	30000	51	72000	1.00	1.03
2, shallow	3150	47500	3.79	48000	77	108000	0.89	1.01
2, deep	3070	48500	3.62	48000	83	108000	0.95	0.99
3, shallow	4460	69300	3.68	68000	122	144000	0.87	0.98
3, deep	4200	71300	3.37	68000	133	144000	0.91	0.95

Table 5.1 Forces acting on considered jackup platforms for selected basic cases, and corresponding computed safety factors for fore leg.

Notation:

F_h = maximum horizontal force from soil to spudcan

F_v = maximum vertical force from soil to spudcan

Inclination = angle between resulting force and vertical axis

$F_{v,pre}$ = load applied to soil for one leg during preloading

Leg length = length of leg from lower guide to tip of spudcan

$SF_{b,1}$ = factor of safety for bearing capacity failure calculated for actual horizontal load level

$SF_{b,2}$ = factor of safety for bearing capacity failure calculated when no horizontal load is present

Model	$\Delta F_{v,tilt}$ (kN)	$F_{v,tot}$ (kN)	$F_{v,cap}$ (kN)	$SF_{b,3}$
1, shallow	2843	30848	38000	1.23
1, deep	3339	32439	31667	0.92
2, shallow	6075	53575	53000	0.99
2, deep	6548	55048	50333	0.91
3, shallow	7953	77253	81667	1.06
3, deep	8670	79970	71667	0.90

Table 5.2 Forces acting on considered jackup platforms and corresponding safety factor for fore leg, when 1 m additional penetration of the fore leg has taken place relative to the basic cases of Table 5.1.

Notation:

$\Delta F_{v,tilt}$ = increased vertical load on fore leg due to 1 meter additional penetration

$F_{v,tot}$ = maximum total load on fore leg after 1 meter additional penetration

From the cases covered by Appendices I and J, four parameters were picked out for a further study, namely wave height, drag diameter, current velocity and preload weight. These parameters were chosen because of their large effect on the safety against sliding failure. Reference is made to Appendix K for details on the performed study on the effects of variation in these parameters on the distribution of horizontal forces between the jackup legs.

The findings of this study refer to two different considered load situations, namely
(1) Structure exposed to environmental forces heading in from aft direction.
(2) Structure exposed to environmental forces heading in from front direction.

For load situation No. 1, increase in drag diameter, preload or wave height is found to transfer more of the additional horizontal loads to the aft legs than to the fore leg. Increased current velocity will increase the load level, but tend to equal the distribution between the legs. Reductions in all the investigated parameters will decrease the total horizontal load level and the horizontal force will be equally shared between the legs.

If a jackup platform in a situation like this is exposed to storm loads the factor of safety against bearing capacity failure, SF_b will be close to or less than 1.0. This is because the bearing capacity will be reduced by the horizontal load. The horizontal load is at first equally shared between the legs, but as the stress level increases in one leg more and more of the horizontal load will be transferred to other legs with less vertical stress, in this case the aft legs. When the bearing capacity for the fore leg is reached, the footing will displace in the direction of the resulting force acting on the foundation. More and more of the the horizontal load will be transferred to the aft legs, and the bearing capacity will increase again.

Table 5.1 and 5.2 below show the forces acting on the considered jackup platforms for selected basic cases and for 1 meter additional penetration of the fore leg, respectively.

dissipation during this period, and the effects of this are further dealt with in Section 3.7. Unfortunately the increase in undrained shear strength after 1 or 2 months is so small that it will not have any significant influence on the bearing capacity.

Additional penetration of one leg in storm may be compensated by jacking up this leg as long as enough leg length is available and the jacking machinery is strong enough to perform the jacking during the storm.

5.4 ULTIMATE CAPACITY OF MULTI-LEG FOUNDATION WITH FLEXIBLE LEGS AND DISTRIBUTION OF HORIZONTAL FORCES BETWEEN THESE LEGS

A study has been performed on the three basic models for independent-leg-supported jackup platforms described in Chapter 2 of this report and has mainly dealt with platforms with three legs. However, rigs designed for water depths less than 60 meters have often got four legs. These rigs are normally preloaded by using two and two legs diagonally, thereby achieving a higher preload level than rigs equipped with only three legs. Higher preload level and four legs result in increased capacities for horizontal force and moment, and thereby a more equal distribution of forces and moments is achieved. For four-legged jackup platforms the stress level will in general be lower than for three-legged platforms, and the safety against failure vertically as well as horizontally will thus be higher. This justifies the selection of a three-legged platform for the study referenced in the following.

5.4.1 Distribution of horizontal forces between legs and evaluation of the most important parameters

The distribution of horizontal forces influences the stiffness and displacement of the spudcans and thereby also the load distribution in the entire structure. An important parameter for distribution of horizontal forces is the capacity of each footing.

In the cases checked for lateral capacity, the lowest value of the factor of safety against sliding was found to be $SF_{sl} = 1.5$. For most other cases SF_{sl} was found to be greater than 1.95. Reference is made to Appendices I and J.

As long as the applied load is only about 50-60% of the capacity, there will only be small differences between the lateral forces on the footings.

that sliding of fully penetrated spudcans is no problem for jackup platforms designed like our selected models and located on seabeds consisting of homogeneous clay with increasing undrained shear strength with depth. If the seabed consists of sand, the base of the footing will rest on the seabed surface or have a very shallow penetration, provided the sand layer is homogeneous, and there will be no sliding problem.

Conclusively, it may therefore be stated that that the only case where sliding may be a problem is when a shallowly penetrated spudcan is resting in a rather thin and very soft clay layer overlaying a much harder soil layer consisting of sand or stiff clay.

5.3 BEARING CAPACITY OF SPUDCANS

Methods for calculation of bearing capacity of spudcans are described in the PP2 report. Input to and results from the parametric study on this topic are described in Appendices I and J. Bearing capacity is studied for shallowly and deeply penetrated spudcans in homogeneous clay with increasing undrained shear strength with depth. For sand bearing capacity is only studied for a shallowly penetrated spudcan, since shallow penetration depths usually will allow for a sufficient bearing capacity in sand.

The preload level is definitely the most significant of the parameters influencing the safety against bearing capacity failure, represented by the factor of safety SF_b. An increase in ballast during preloading will improve SF_b. Other important parameters are wave height, current, drag diameter, and for fully penetrated spudcans the side wall height. For the largest considered jackup model the initial shear modulus for the soil also affects SF_b. Increases in shear modulus and side wall height improve SF_b, while increases in wave height, current and drag diameter reduce SF_b.

For most of the cases listed in Appendices I and J the factor of safety against bearing capacity failure is found to be below 1.0 which means that failure actually will occur. Hence the spudcan will displace vertically until SF_b becomes equal to 1.0. The magnitude of the vertical movement is given by the rate of increase in soil parameters and overburden pressure with depth. If the spudcan is only shallowly penetrated an additonal penetration will result in a rapid increase in bearing capacity.

Since jackup platforms are installed at their locations in calm weather, it usually takes some time after installation before storm or design loads occur. For clay there will be an increase in undrained shear strength due to consolidation and pore water

For larger jackup platforms than the considered model No. 1 platform the ratio H/BL and the preload level will both contribute to improve the safety against sliding.

A parameter study has been performed by means of the SAIL program in order to reveal the relative effects of variations in the involved parameters on the sliding resistance of shallowly penetrated spudcans. Input to and results from this parameter study can be found in Appendix I.

If a spudcan is penetrated so much that the entire spudcan base area is exposed to soil, or if the spudcan is V-shaped, the horizontal resistance will consist of skin friction as well as passive earth pressure on the vertical projection of the spudcan wall in addition to the pure sliding resistance.

The findings of the parameter study, referenced in Appendix I, indicate that for all of the three considered platform models, the most important parameters for sliding are wave height, current, platform weight, drag diameter of leg and preload weight. If wave height, current or drag diameter are increased, the safety against sliding will also increase. The minimum factor of safety against sliding was found to be $SF_{sl} = 1.63$ which confirms that sliding in clay will only be a problem for very weak clay overlaying a hard soil layer.

5.2 LATERAL RESISTANCE OF DEEPLY EMBEDDED SPUDCANS AND THEIR ATTACHED LEGS

The input and results for a parametric study on deeply embedded spudcans is presented in Appendix J. The applied method for calculation of lateral resistance of deeply embedded spudcans is described in Section 3.2.2 of the PP2 report.

120 cases have been considered in the study performed for deeply penetrated spudcans. The most important parameters appear to be wave height, current, platform weight, drag diameter, preload level and side wall height of the spudcan. Increases in wave height, current and drag diameter reduce the safety against sliding, while increases in platform weight, preload level and side wall height improve this safety.

The lowest obtained factor of safety against sliding was found to be $SF_{sl} = 1.51$ at a 20% increase of the maximum wave height from its selected basic value, thus implying considered led to wave steepnesses higher than the limit criterion implemented in SAM, and the wave height was reduced correspondingly giving a very rapid increase in safety with further decrease of the wave periods.

4.2.3 Current Speed

The basic current speed at still water level was selected to 1.0 m/s, with equal contributions from tidal and wind induced components. The assumed distributions with depth for the two components were presented in the PP2 report.

The total SWL current speed was varied with +/- 1.0 m/s and it can be seen from the third column of diagrams that the effect of current increases with increasing water depth, but the effect is limited to about +/- 5 percent on the safety factors.

4.2.4 Wind Speed

The effect of a wind speed variation of +/- 5 m/s is presented in the fourth column of diagrams showing that the local failure mode was most affected and that the effect decreased with increasing water depth. The wind force represents a limited percentage of the total horizontal force, and the effect on the safety factors is thus also limited.

4.2.5 Drag and Inertia Coefficients

The variation of the drag coefficient with +/- 0.1 about a basic value of 0.7 and the inertia coefficient with +/- 0.2 about a basic value of 2.0 had very limited effects on the foundation stability as can be seen from the fifth and sixth columns of diagrams.

4.2.6 Skirt Height

The effect of skirt height variations is shown in the seventh column of diagrams. The height was varied with +/- 0.2 m about a basic value of 0.5 m and it is seen that the sliding and local failure modes are affected mainly due to increased passive earth pressure with increasing skirt length and for the local failure mode a slight depth effect

will be present.

4.2.7 Mat Height

In the eighth column of diagrams the effect of mat height variations with +/- 0.5 m is presented. It is seen that the safety decreases with increasing mat height for all failure modes due to an increased horizontal force on the mat. It is further seen that the effect decreases with increasing water depth. In shallow water the mat height seems to have a considerable effect on the foundation stability.

4.2.8 Mat Shape and Area

The variation of the shape and dimensions of the mat as shown in Figure 2.13 has a very significant effect on the foundation stability, and it is seen from the ninth column of diagrams that the effect varies considerably amongst the different failure modes.

The sliding resistance is almost proportional to the mat area which varies as shown in Table 4.1 below for the three models:

Table 4.1. Variation in net mat areas.

Variation	Model 1	Model 2	Model 3
Small mat m**2	1050	1714	2325
Basic mat m**2	1300	1974	2600
Large mat m**2	1900	2730	3450

The local failure mode is most strongly affected by the variation in mat geometry. The most significant effect is considered to be the increase in sectional moment which reduces the edge stresses due to wave loading considerably, further the increase in mat area reduces the average contact stress and increases the sliding resistance thus reducing the effect of inclined loading on the bearing capacity of the soil, and last but not least the increase in the width of the mat beams and fingers forces the failure surface

to go deeper and gives thus an increase in shear strength.

The deepseated failure mode is seen to be negatively affected by the increase in mat dimension as this leads to a higher overturning moment caused by the wave pressure on the mat.

4.2.9 Undrained Shear Strength at Mudline, s_{uo}

The undrained shear strength at mudline is the dominating parameter. This is clearly seen from the tenth column of diagrams. All failure modes are strongly controlled by the undrained shear strength in the top meters.

The resistance against sliding and shallow failure is almost directly proportional to s_{uo} while the safety against deepseated failure is somewhat less sensitive due the increase in undrained shear strength with depth below mudline assumed for the basic cases.

The safety against local failure is most strongly affected. This is due to the fact that the reserve bearing capacity beyond what is needed to support the platform weight approaches zero as the undrained shear strength is reduced and the capacity to take inclined forces and overturning moments will thus be reduced faster than the sliding resistance. The effect seems to increase with increasing water depth due to correspondingly increasing overturning moment.

4.2.10 Shear Strength Gradient, k

The effect of variations of the shear strength gradient is shown in the eleventh column of diagrams. As can be seen, the sliding resistance is hardly affected. This will depend somewhat on the skirt height, but with the skirt heights commonly used for mat-supported jackup platforms the effect can be considered to be small.

The local failure mode is considerably more affected by reductions in the shear strength gradient and in shallow water the deepseated failure mode will gradually become the most critical as the gradient approaches zero, i.e constant undrained shear strength. This strongly underlines that in soft underconsolidated delta deposits sea bed instability might be critical and the presence of a mat-supported jackup platform will

not improve the situation.

4.2.11 Platform Weight

In the twelfth column of diagrams it is shown that a variation of the platform weights with +/- 5 MN only seems to have a significant effect on the local failure mode and a slight effect on the shallow failure mode.

4.2.12 Submerged Unit Weight of Soil

The submerged unit weight of soil is shown to be of insignificant importance for the basic cases evaluated. However, it is obvious that in cases where the mat penetration is significant, i.e. several feet, this parameter will be of a certain importance as outlined in Section 3.1.

4.3 CONSEQUENCES OF FOUNDATION FAILURE

4.3.1 Sliding Failure

If the sliding resistance of a mat-supported jackup platform is exceeded, it can be expected that uncontrollably large lateral displacements will occur. After a certain lateral movement, the passive earth pressure may be strongly reduced and the mat may start to float on the soil material enclosed between the skirts.

Dependent on the unevenness and inclination of the seabed the consequences may range from limited damage to capsize and total loss. The presence of pock marks (i.e. local crater-shaped depressions) will increase the probability of capsize, and gradually increasing water depth may reduce the air gap and allow waves to attack the hull structure.

Anyhow, sliding failure should be avoided, and it is obvious that a correct assessment of horizontal forces as well as the undrained shear strength of the top meters of the seabed are the essential parameters involved. The dominating horizontal force component in shallow water is the wave force acting against the mat structure (as can be

seen from Fig. 4.1). It is obvious that skirt height and mat dimensions (columns 7, 8 and 9 of Fig. 4.4) have a considerable influence on the sliding stability.

4.3.2 Local Failure

The effect of local failure under the most stressed finger of a mat will be a gradual sinking-in of the most heavily loaded part of the mat. This means that a platform tilt will develop as the platform is hit by large waves. The overturning moment will increase and tend to increase the tilt progressively. On the other hand, the increased penetration will reduce the horizontal wave force on the mat and the bearing capacity of the soil will increase under the penetrated part of the mat due to increased buoyancy and generally increasing shear strength with depth. To evaluate the consequences quantitatively the following somewhat simplified evaluation is carried out:

Assume a simplified mat geometry consisting of the two main fingers only with a center distance b, individual width B and length L, see Fig. 4.5. The center of gravity of the platform is assumed to be at a distance h above the bottom of the mat and halfway between the two fingers.

Assume further a platform tilt α. This leads to an average sinkage δ_v of the most heavily loaded mat finger relative to the center of the other finger:

$$\delta_v \approx b \cdot \alpha$$

The center of gravity will be displaced horizontally by a distance δ_h:

$$\delta_h = h \cdot \alpha$$

This leads to a corresponding increase in overturning moment ΔM_o:

$$\Delta M_o = W \cdot h \cdot \alpha$$

where W = weight of the platform.

The sinkage of the most loaded finger will on the other hand produce an increase in bearing capacity under the finger and thus also an increased resisting moment ΔM_R. The increase in bearing capacity has two components, one due to increased undrained shear strength Δs_u, and one due to increased penetration depth and correspondingly increased overburden pressure, Δq:

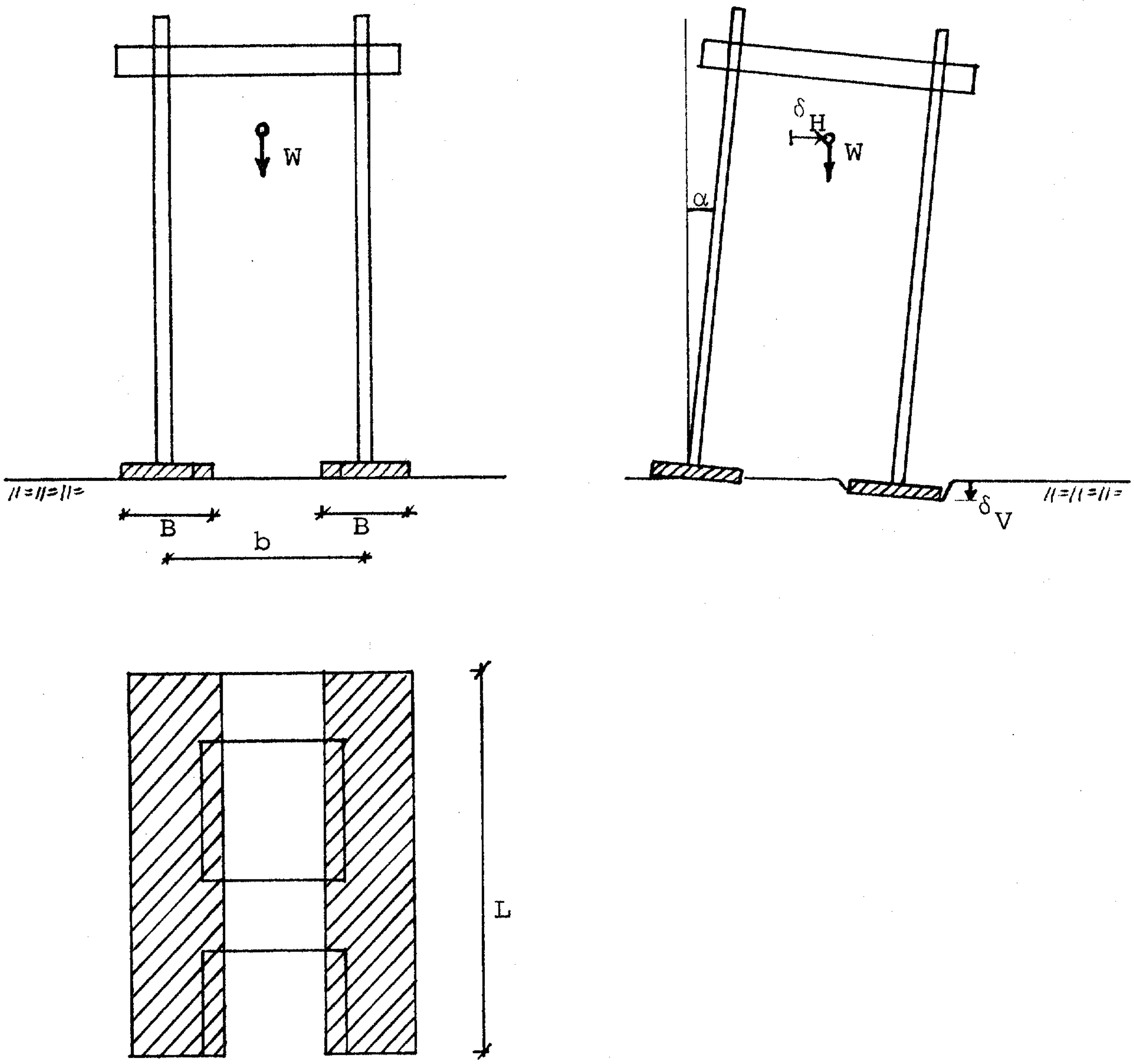


Fig. 4.5 Simplified mat geometry

$$\Delta q = N_c \cdot \Delta s_u + \gamma' \cdot \delta_v$$

$$\Delta s_u = k \cdot \delta_v$$

$$\delta_v = b \cdot \alpha$$

This gives

$$\Delta q = (N_c \cdot k + \gamma') \cdot b \cdot \alpha$$

Thus we get the following expression for the increase in resisting moment:

$$\Delta M_R = \Delta q \cdot B \cdot L \cdot b = B \cdot L \cdot b^2 \cdot \alpha \cdot (N_c \cdot k + \gamma')$$

where
B = width of finger
L = length of finger
b = distance between centerline of fingers
N_c = bearing capacity factor
γ' = submerged unit weight of soil
k = shear strength gradient

The expressions for the increase in overturning moment and resisting moment due to sinkage of the most heavily loaded mat finger have been evaluated for the three models considered in the parametric study. The mat geometry has been somewhat simplified to fit the two-finger model.

The submerged unit weight of soil was assumed to be 5 kN/m^3 and the shear strength gradient was varied from 0.0 to 1.5 kPa/m in steps of 0.3 kPa/m covering the range typical for soft clays.

The center of gravity was assumed to be at the SWL, i.e. 30, 60 and 90 m, respectively, above the bottom of the mats for the three models.

The simplified model parameters are summarized in Table 4.2 below.

Table 4.2. Summary of simplified model parameters.

Parameter	Model 1	Model 2	Model 3
B	≈ 10 m	12.5 m	≈ 15 m
L	55 m	65 m	75 m
b	≈ 30 m	37.5 m	45 m
h	≈ 30 m	≈ 60 m	≈ 100 m
W	≈ 30 MN	≈ 40 MN	≈ 50 MN

Based on these parameters the increases in overturning and resisting moments have been evaluated for tilt angles of 0° to 4°, and the results are shown as moment versus tilt for the three models in Fig. 4.6.

From this figure it is clearly seen that the increase in resisting moment is considerably greater than the increase in overturning moment. This means that local failure normally will be a self-stabilizing failure mode.

Comparing the net increase in resisting moment from Fig. 4.6 with the load histories in Figs. 4.1 to 4.3 it can be seen that a considerable part of the total overturning moments due to wind, waves and current can be taken by gradual shakedown of one or both fingers of a mat.

However, the self-stabilizing effect may be strongly reduced if a mat finger penetrates completely. As can be seen from the tenth column of diagrams in Fig. 4.4, local failure will be the critical failure mode for very soft clays. The trench stability of very soft clays is limited and it is to be expected that the material will gradually backfill the hole generated by a penetrating mat finger in these types of soil material. The overburden term of the bearing capacity will thus vanish and the beneficial effect of additional penetration will be strongly reduced.

This means that heavy penetration due to platform weight alone is an indication of a situation where additional sinkage and tilt of the platform will have to be expected during storm loading. If the shear strength gradient is low, unacceptable tilt and heavy burial of parts of the mat may be the result.

Although an additional sinkage of the mat will reduce the horizontal force acting on the mat and thus improve the sliding stability, the local failure mode may be considered critical for very soft clays and deeper waters. Proper dimensioning regarding the width of mat fingers and cross beams is vital for safe operation. Too slender design of fingers and cross beams should be avoided.

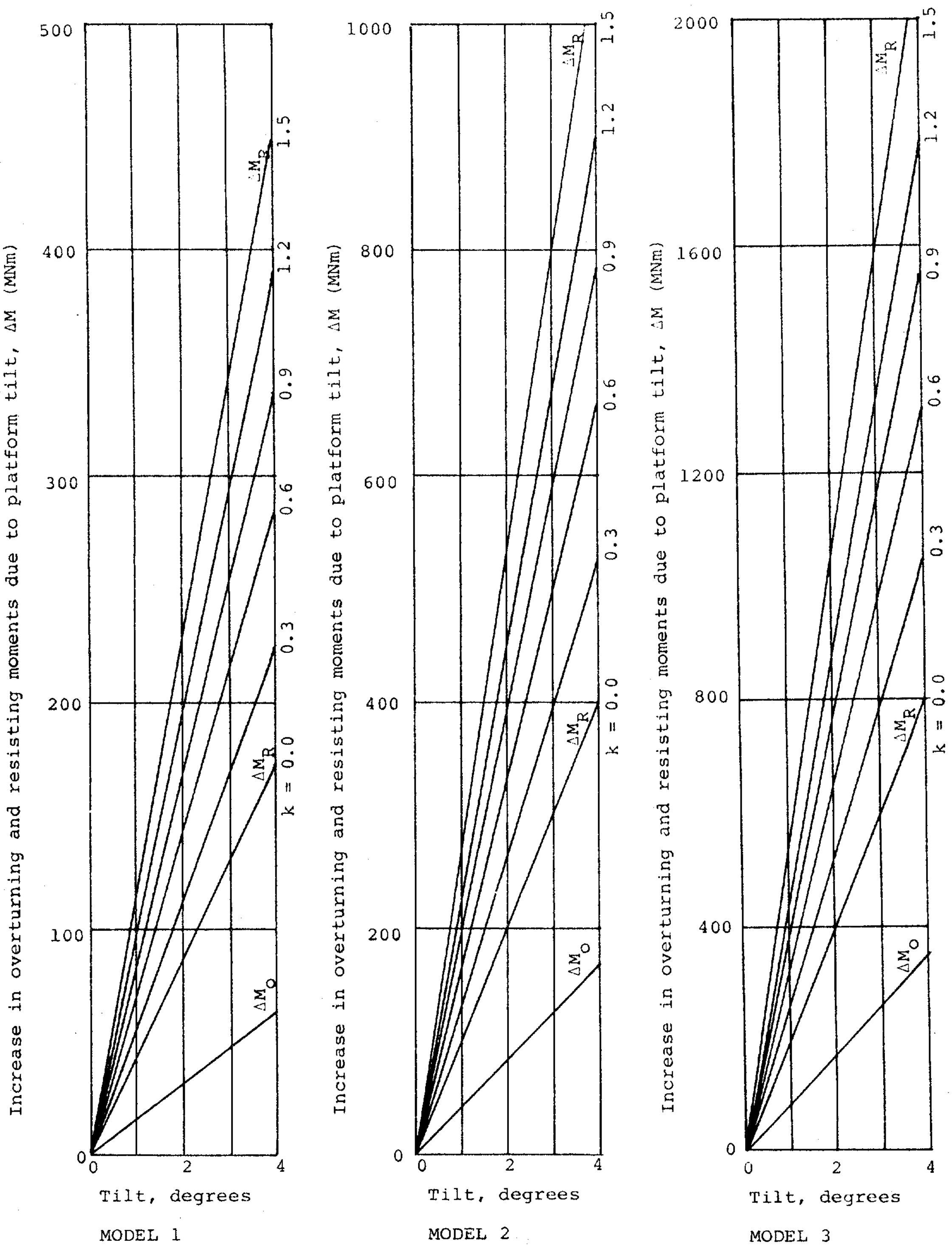


Fig. 4.6 Increase in overturning and resisting moments as function of tilt

4.3.3 Shallow failure

The evaluation revealed that shallow failure under the most heavily loaded edge will not be a critical failure mode for A-shaped mats. A review of the critical slip surfaces indicated that very shallow surfaces were the most critical. This is due to the very low bearing pressure of the mats investigated. The failure mode will in this case be very similar to the sliding failure mode. The difference in safety factors is mainly connected to the difference in assumptions for the failure surfaces, where the shallow failure analysis includes the strength of the soil under the hole and slot area, whereas the sliding failure analysis only takes skirt resistance (passive pressure and side friction) into account.

The consequences of shallow failure will thus be comparable to those of sliding failure, i.e. large, uncontrollable lateral displacements that will accumulate under the combined action of wind, current and waves.

4.3.4 Deepseated failure

As described previously, deepseated failure is mainly a result of the wave pressure acting on the mat and the surrounding seabed. The parametric study revealed that this failure mode may be critical in relatively shallow waters and in areas with very soft soils having a very low shear strength gradient with depth (see Model 1, tenth and eleventh row of diagrams in Fig. 4.4).

The consequences of deepseated failure is likely a very rapid rotation of the rig and the involved soil mass and most probably capsize and heavy damage to the structure.

It should be noted that the center of rotation of the critical failure surfaces was located well above mudline for all cases analyzed as mentioned under 4.1. The depth of the failure surface will thus be less than half the width of the mat. The dominating and critical parameter is the shear strength gradient of the soil.

4.4 EVALUATION OF EFFECTS OF REPEATED LOADING

Two sets of analyses have been carried out for a selected representative mat-supported jackup platform with given environmental conditions, one set for a clay deposit and

one set for a sand deposit.

The selected jackup platform was assumed to be a three-legged platform with a total weight of 40000 kN, supported by a 65 m long and 50 m wide mat with two 12 m wide strips and a net area of 2054 m^2. The mat was assumed to rest at seabed level, and it was further assumed to be equipped with 0.6 m long skirts penetrated into the soil.

The water depth was assumed to be 60 m, 1 m/sec current was assumed at still water level, and a wind speed of 50 m/sec. was selected. A design wave height of 20 m was applied and foundation loads were subsequently calculated by means of program SAM for various assumed wave periods.

The SAM analysis revealed that the most critical load conditions with respect to the foundation stability would occur at a wave period of 14 seconds in the case that the foundation soils consist of clay, and the loads found for this wave period were therefore applied in the subsequent analyses, as well for clay as for sand (for simplicity).

The analyses for clay were carried out first, and for this purpose a reasonable undrained shear strength profile $s_u = 7.0 + 1.2 \cdot depth$ (kPa) was assumed, corresponding to a soft clay which typically is present when mats are chosen for support of jackup platforms. The execution of SAM proved that sliding would be the most critical failure mode with a factor of safety equal to

F.S. = 1.30.

This factor of safety corresponds to static considerations only, so for evaluation of the effects of cyclic loading the program RELOC was applied.

RELOC computes corresponding values of maximum shear stresses and accumulated shear strains for various scalings of a certain storm with Rayleigh-distributed wave heights. This allows for a determination of the cyclic shear strength τ_{cf} that will be available after such a storm, expressed in terms of a percentage of the original undrained shear strength s_u. The input to such a RELOC analysis consists of a socalled strain-contour diagram for the soil and a storm history, and further information on the method can be found in Foss, Kvalstad and Dahlberg (1978).

In the present case, a design storm of 6 hours build-up, 6 hours full storm and 6 hours fade-out was modelled as sketched in Fig. 4.7 with an average wave period of 12 seconds, and a typical strain-contour diagram for a normally consolidated clay was selected as representative for the typical soft clay, see Fig. 4.8.

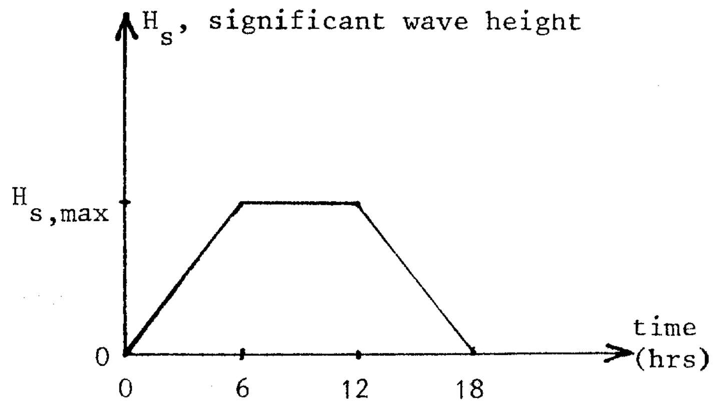


Fig. 4.7 Storm history

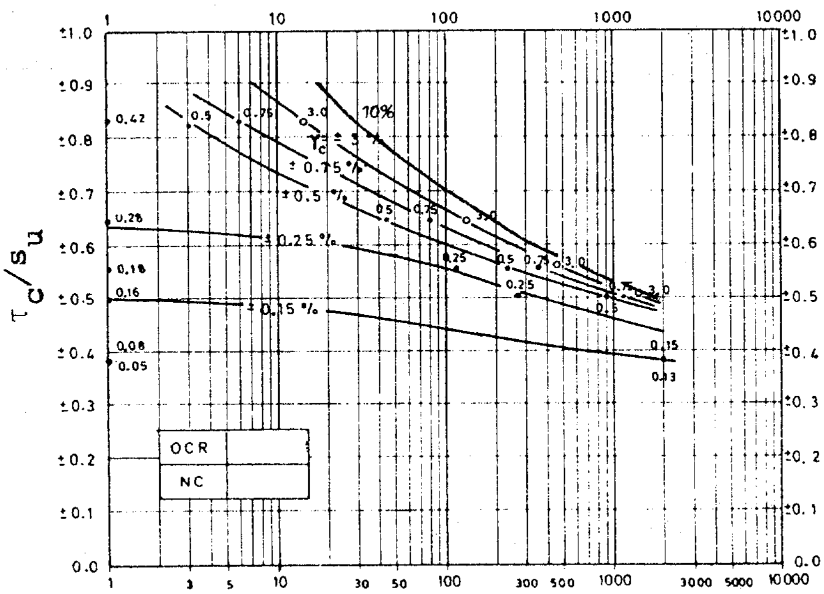


Fig. 4.8 Strain-contour diagram for cyclic loading of soft clay

The computations by means of RELOC gave for this input an available cyclic shear strength at the end of the applied storm of $\tau_{cf} = 0.91 \cdot s_u$. This would cause the factor of safety against sliding to become reduced to

F.S. = 1.18

which is fairly low, and a check run by program SAM revealed that sliding would still be the most critical failure mode.

The conclusion of this particular analysis for clay will be that effects of cyclic loading may very well have a significant influence on the available shear strength in the clay and may for a typical platform cause the stability against failure in cyclic loading to become critical.

For the selected undrained shear strength profile $s_u = 7.0 + 1.2 \cdot depth$, sliding was assessed to be the most critical failure mode, but it should be noted that in case a profile with a much reduced depth gradient, say 0.3 kPa/m instead of 1.2 kPa/m, one would obtain a much more complex picture where other failure modes become predominating, e.g. a shallow type failure becomes critical for long waves, while a local type failure seems to become critical for steeper waves. This observation justifies the rather thorough investigation for multiple modes of failure that program SAM covers.

The loads computed by SAM were also applied for evaluation of the stability of the considered mat foundation when supported by a sand. Resting on a sand, sliding will be the most critical failure mode for the platform no matter how the wave period varies, and due to the presence of skirts the critical sliding surface will be located at the skirt tip level and allow mobilization of full sand-sand friction. The sand was assumed to have a friction angle of $\varphi = 35^o$, and a simple expression for the factor of safety against sliding would hence be

$$F.S. = \frac{W_v \cdot tg\varphi}{W_{H,max}} = \frac{40000 \cdot 0.7}{12600} = 2.22$$

which indicates that the stability of the mat on a sand would cause no problems when assessed from a pure static point of view.

As a first stage for evaluation of the effects of cyclic loading, a stress analysis was conducted by means of the finite element program AXIPLN which computed initial static shear stresses τ_s in the soil corresponding to pure selfweight loading and cyclic shear stresses τ_c at various levels of environmental loading up to the design wave loading.

These results plus a strength curve diagram for the sand were subsequently applied as input to a pore pressure response analysis by means of the computer program OCEAN2. In the present case a strength curve diagram as determined in the laboratory for a dense sand was applied, see Fig. 4.9, and an equivalent storm of 6 hours duration was constructed and applied with an assumed average wave period of 10 seconds, see Fig. 4.10. The permeability of the sand was set to $k = 1.0 \cdot 10^{-4}$ m/sec and a typical compressibility of $m_v = 0.3 \cdot 10^{-4}$ m^2/kN was chosen. Reference is made to Rahman (1977) for more details about the employed methods.

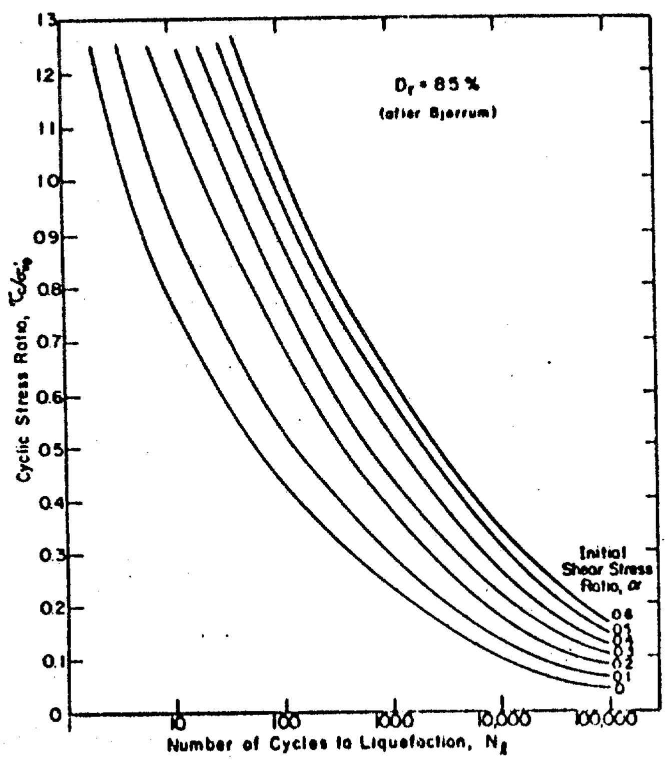


Fig. 4.9 Strength curves for dense sand with $D_R = 0.85$

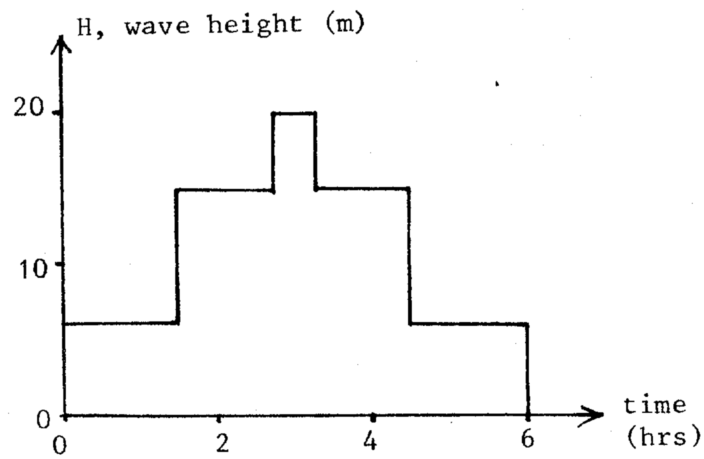


Fig. 4.10 Constructed equivalent design storm

Hence OCEAN2 performed a pore pressure response analysis covering the pore pressure generation due to the considered equivalent storm as well as the pore pressure dissipation due to consolidation. The most significant pore pressure response underneath the mat during the considered storm was found by this analysis to amount to

only 1.5% of the topical vertical stress, i.e. no significant effects of cyclic loading on the stability of the mat foundation on sand could be expected.

A lower assumed permeability (which would not be too likely) would have caused a somewhat more significant pore pressure response but yet not enough to influence the stability in a critical way. A less dense sand than the one analyzed would likely have developed larger pore pressures, but would at the same time have been more permeable and thereby allowed for a faster pore pressure dissipation. Hence, the net pore pressures would still have become somewhat limited, and the foundation stability would not have reached a critical level even for this case.

It appears that the overall stability of a mat foundation on sand is not influenced significantly by cyclic effects due to repeated wave loading, at least not for the case that has been considered here. What is probably of much more importance with respect to the stability of such a mat foundation in a storm is the scour in the sand along the sides of the mat, caused by induced pumping erosion in rocking, but it is beyond the scope of this analysis to go further into that topic. It is, however, still important to conduct cyclic stability analysis for mat foundations on sand as outlined for the present case above, especially for cases which involve more critical soil conditions, e.g. a mat on silty sand with low friction angle and low permeability.

4.5 EFFECTS OF SLOPING SEABED

The presence of a sloping seabed is only expected to cause problems for jackup platforms founded on soft clays.

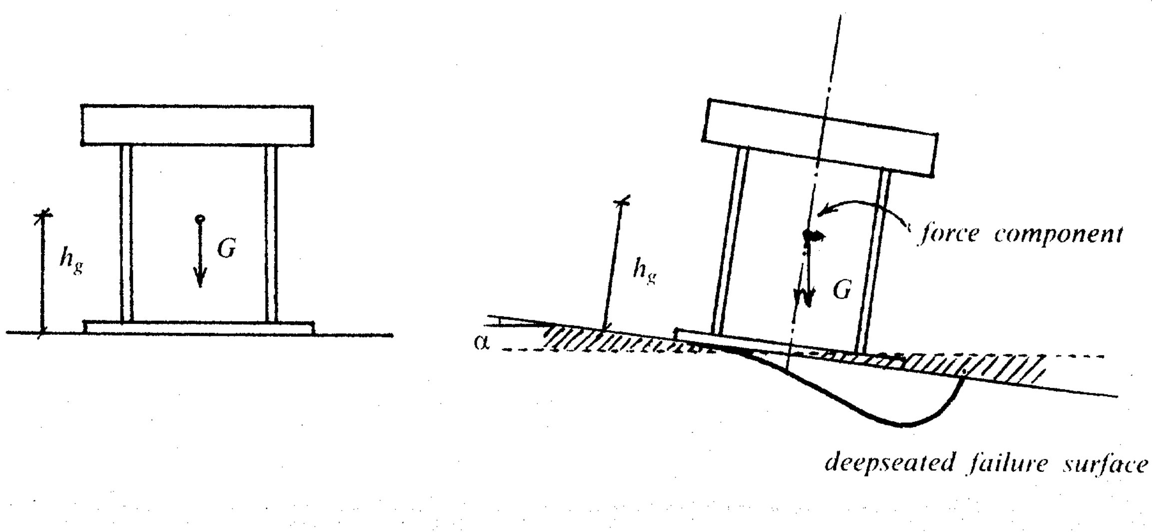


Fig. 4.11 Mat-supported jackup platform on sloping seabed

Reference is made to Fig. 4.11, and it appears that the driving lateral force will be increased by $\Delta Q_{lat} = G \cdot \sin\alpha$ and the driving moment about the seabed will be increased by $\Delta M = G \cdot \sin\alpha \cdot h_g$ when the inclination angle for the seabed is changed from zero to some value α, thus leading to a somewhat reduced factor of safety. G is the weight of the structure, and h_g is the distance from the seabed to the platform's center of gravity.

Sliding will usually be the critical failure mode, but in case deepseated failure is critical, the sloping seabed will also cause a reduction in the amount of stabilizing overburden soil and thereby contribute to the reduction in factor of safety.

However, the effects of a sloping seabed will generally be small within the relatively narrow range of variation that is likely for the seabed slope with the inclination angle α not exceeding a few degrees, and they have therefore not been further considered within the parameter studies. In addition, it should be noted that the effects of a sloping seabed may be compensated by appropriate jacking and levelling of a topical platform.

5. STABILITY OF INDEPENDENT-LEG-SUPPORTED JACKUP PLATFORMS SUBJECTED TO GRAVITY, WAVE, WIND AND CURRENT LOADS

5.1 SLIDING RESISTANCE OF SHALLOWLY EMBEDDED SPUDCANS

A spudcan is usually shallowly penetrated if the seabed consists of clay with undrained shear strength in excess of 50 to 60 kPa, or if the seabed consists of sand. Sliding is only of current interest for rather flat-based spudcans resting on the soil surface and for spudcans shallowly embedded in very soft clay overlaying a hard soil layer. For derivation of applied formulae reference is made to Appendix H.

The sliding capacity of a footing in sand is dependent on the applied vertical load, while for a footing in clay the sliding capacity is independent of the vertical load.

Maximum horizontal force $F_{h,max}$ on one leg for a three- or four-legged jackup platform equals

$$F_{h,max} = \frac{W_{g,tot}}{NL^2} \cdot \frac{BL}{H} \cdot \frac{1}{F_0}$$

where

$F_{h,max}$ = maximum horizontal force on one leg

$W_{g,tot}$ = total weight of entire structure including gravity and variable load, but not ballast for preloading

NL = number of legs (3 or 4)

BL = distance between fore and aft legs

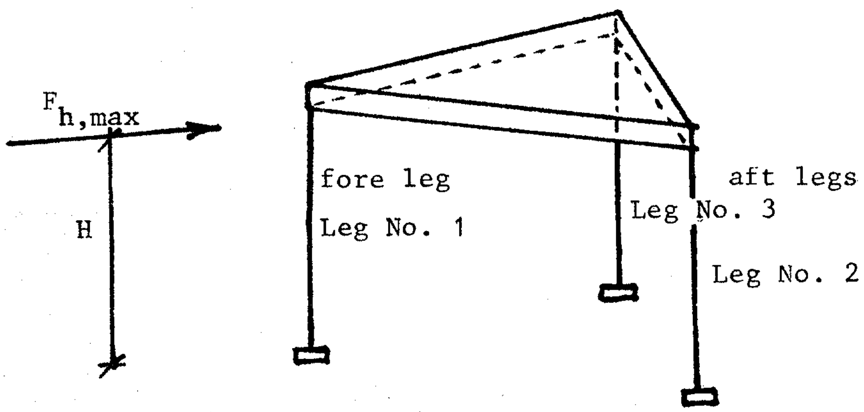
H = lever arm for both wind and wave forces, measured from seabed for a shallowly penetrated spudcan

F_0 = factor of safety against overturning of entire structure

This formula shows that the allowed maximum horizontal force may be increased by a reduction in lever arm H for the environmental forces or by an increase in the distance BL between fore leg and aft legs, provided the factor of safety against overturning is unchanged. This means that the allowed maximum force will increase for

decreasing water depth, but one should then keep in mind that the factor of safety against overturning will usually increase for decreasing water depth and not maintain constant.

Consider now a three-legged jackup platform on sand as indicated in Fig. 5.1. Leg No. 1 will be the most stressed leg when environmental forces are heading in from front direction (left hand side of figure). Jackup model No. 1 (see Section 2.2, Table 2.2) with a water depth of 30 m will be the most critical jackup model for visualization of this problem if one assumes a factor of safety against overturning $F_0 = 1.5$.



Leg No. 1: $$\frac{F_{h,max}}{F_{v,leg1}} = \frac{BL}{3H(F_o \mp 1)}$$

Legs No. 2 and 3: $$\frac{F_{h,max}}{F_{v,leg2or3}} = \frac{BL}{3H(F_o \pm 0.5)}$$

Environmental forces heading in from left hand side:
Use "-" for Leg No. 1 and "+" for Legs No. 2 and 3.

Environmental forces heading in from right hand side:
Use "+" for Leg No. 1 and "-" for Legs No. 2 and 3.

Fig. 5.1 Sketch of three-legged jackup platform

Applying the formulae derived in Appendix H, leg No. 1 is found to slide at this

overturning safety when the ratio between horizontal and vertical force on this leg exceeds 0.66, i.e. $F_h/F_v > 0.66$ which implies $tg\varphi < 0.66$, i.e. sliding of leg No. 1 will occur when $\varphi < 33^o$.

Prior to sliding of leg No. 1 the horizontal force acting on the two aft legs, legs No. 2 and 3, can be found to equal $F_h = 0.16F_v$ per leg. When leg no. 1 slides, the two aft legs will have to carry this leg's horizontal load in addition, and the horizontal aft leg loads will hence increase by 50% to $F_h = 0.24F_v$ per leg, provided that the total horizontal force were equally distributed between the three legs prior to the sliding of leg No. 1. This will further imply sliding of legs No. 2 and 3 if the friction angle should be as low as 13.5^o or less.

Cyclic effects could possibly reduce the effective friction angle for the sand, but this will not be a problem for normal spudcans with radii between 5 and 10 m for which the sand will be permeable enough to drain possible accumulated cyclic pore pressures right away.

Consider now a clay deposit instead of the sand: The sliding capacity in clay for a flatbased spudcan is dependent on the exposed area and the undrained shear strength of the soil beneath the spudcan. The factor of safety against sliding will be:

$$SF_{sl} = 0.49 \cdot \frac{H(F_o + K) \cdot s_{u,cyclic}}{BL \cdot s_{u,static}} \cdot \frac{N_c}{N_c^*}$$

where

SF_{sl} = factor of safety against sliding

H = lever arm for environmental forces, measured from seabed for shallowly penetrated spudcans

F_o = factor of safety against overturning

K = preloading factor = $\dfrac{\textit{Required ballast for preload}}{\textit{Used ballast for preload}}$

$s_{u,cyclic}$ = undrained cyclic shear strength of soil

$s_{u,static}$ = undrained static shear strength of soil

$N_c = 5.14$ = bearing capacity factor for clay

N_c^* = bearing capacity factor for clay, reflecting the effect on the bearing capacity from a hard soil layer underlaying the considered clay

The required ballast is the ballast which is necessary during preloading in order to achieve vertical loads on all legs equal to the maximum vertical load on the most stressed leg when the jackup platform is exposed to storm loads. The used ballast is the actual ballast that is used during preloading.

N_c^* is an equivalent bearing capacity factor which applied together with the undrained shear strength for the considered clay in top of a soil profile gives a correct bearing capacity of this top clay layer in combination with a deeper hard soil layer in that profile.

The formula shows that sliding will be critical for a jackup platform located in shallow water with no preload, little safety against overturning and long distance between the fore and aft legs. Some of the factors included in the formula are mutually dependent and effect each other, for instance BL, H and F_o. An increase in BL or a reduction in H (caused by a reduction in water depth) will both improve the safety against overturning.

An example is presented below for illustration of the use of the formula, and a model No. 1 jackup platform located on top of a uniform clay is considered for this purpose:

$$\frac{s_{u,cyclic}}{s_{u,static}} = 0.7, \quad H = BL = 33 \ m, \quad \frac{N_c}{N_c^*} = 1$$

Application of the formula for this case gives:

$$SF_{sl} \ = \ 0.49 \cdot 1.0 \cdot \frac{33 \cdot (F_o + k)}{33} \cdot 0.7 \ = \ 0.34 \cdot F_o$$

This means $SF_{sl} \leqslant 1.0$ for $F_o < 2.9$ assuming no preload (k=0) and $SF_{sl} \leqslant 1.0$ for $F_o < 1.9$ assuming full preload (k=1).

Jackup platforms used at water depths of 30 meters usually go through a preload procedure or have a safety against overturning greater than 3.0. In other words, sliding do not appear to be a problem for jackup platforms with normal design and preload routines, but for spesial designs with no preload, a low safety against overturning and flat-based spudcans, sliding may form a problem.

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TECHNICAL REPORT

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Project No.	233104
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Approved by	O. Furnes, Chief Research Engineer
Client/Sponsor of project	Several Companies, see Preface
Client/Sponsor ref.	
Work carried out by	Knut O. Ronold
Reporters sign.	Knut Olav Ronold

Summary

This technical report presents the findings of the overall work performed within the project "Foundation Stability of Jackup Platforms". Guidelines and recommendations have been worked out on the basis of the work performed in the three previous part projects. We have tried to avoid generalization of findings beyond the range of boundary conditions and assumptions made during the study. The guidelines and recommendations are thus restricted to the assumptions made, and extrapolations should be made with care.

4 Indexing terms

- JACKUP PLATFORMS
- FOUNDATION STABILITY
- GUIDELINES
- INSTALLATION, OPERATION

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PREFACE

This report presents the work carried out within Part Project 4 (PP4) of the joint industry project "Foundation Stability of Jackup Platforms". The project is at the time of writing sponsored by the following 14 companies:

Amoco Norway Oil Company
Arco Norway Inc.
Bethlehem Steel Corporation
Chevron Oil Field Research Company
Gusto Engineering B.V.
Getty Oil Company
Hitachi Zosen Corporation
Minerals Management Service
Mitsubishi Heavy Industries
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Det norske Veritas

The work presented herein is the result of a study where the main work was carried out during September to December 1983 and covers a technical summary of the previously conducted Part Projects 1, 2 and 3 together with recommendations and guidelines for design and operation of jackup platforms.

EXECUTIVE SUMMARY

The purpose of Part Project 4 (PP4) was to summarize the results of the entire project on foundation stability of jackup platforms and hence - based on these results - work out a project synthesis in terms of recommendations and guidelines for foundation of such platforms.

The previous part projects covered evaluation of state-of-the-art and current practice with respect to foundation stability, evaluation and partly development of theories and methods for analysis of foundation stability, and performance of a thorough parameter study of foundation stability, all with respect to jackup platforms. The review of these part projects resulted in a technical project summary with emphasis laid on the main findings of the parameter study. This was used as a base for derivation of recommendations and guidelines for design and operation of jackup platforms, first of all with respect to foundation problems.

Proper evaluation of the foundation stability of jackup platforms was found to be important for many reasons. Among these are first of all the severe consequences of foundation failure, e.g. in terms of structural damage or platform capsize during installation as well as in operation.

For mat-supported platforms sliding was generally found to be the most critical failure mode, while conventional bearing capacity failure was found to be the most likely failure mode for footings of platforms with independent legs. In special cases other failure modes were found to be governing for the foundation stability, and investigation of the stability for various relevant failure modes was therefore recommended in general. For platforms with independent legs it was further recommended especially to be aware of potential punch-through profiles which were found to be very risky for the foundation stability of this platform type.

Reliable evaluations of foundation stability of jackup platforms require a good knowledge of the soil strength profile, especially because the sensitivity of the stability to variations in the soil strength generally was found to be large, and execution of adequate soil investigations was therefore recommended as very important.

Other recommendations and guidelines were worked out for other topics with relation

to foundation stability of jackup platforms and are presented throughout this report - all, however, restricted to the various boundary conditions and assumptions made for the analyses on which they were based.

1. INTRODUCTION

The original scope of work for the entire project on foundation stability of jackup platforms has - broadly speaking - been followed throughout the execution of this project, while, however, the project participants have not been involved in this execution as much as intended. More emphasis than originally planned has been laid on the development of analysis tools under PP2, first of all by the development of the computer programs SAM and SAIL for rationalization and systematization of the parameter study under PP3.

On the other hand, some topics have been covered to a lesser extent in the parameter study than originally intended: Erosion has, for instance, not been considered, first of all due to lack of possibilities for a quantitative and computational treatment of this topic, and soil-structure interaction has only been considered very briefly because it is a very comprehensive topic, and the time and budget available did thus not allow for a thorough representation of this topic within the project.

It has been the aim of PP4 to summarize the findings of the entire project and synthesize these findings in terms of recommendations and guidelines for design and operation of jackup platforms, first of all with respect to foundation problems.

The work carried out during PP4 has essentially been based on the results of PP3 which show which parameters are most critical with respect to foundation stability, and recommendations and guidelines have been worked out based on this for various topics of interest in connection with design and operation of jackup platforms. Comparisons have been made with current practice for the different considered topics as far as current practice has been reported during PP1, and discrepancies have been pointed out and discussed. As PP2 dealt with evaluation and development of theories and methods for use in PP3, results of PP2 have not been directly involved in this synthesis.

The essential findings of the project are presented in a technical summary, and the recommendations and guidelines that have been worked out are presented subsequently. In this context it is important to notice that all given recommendations and guidelines are restricted to the boundary conditions and assumptions made during the various studies and analyses that they are based on, and generalization should therefore be made with great care.

2. TECHNICAL SUMMARY

2.1 INDEPENDENT-LEG-SUPPORTED JACKUP PLATFORMS

During installation of a jackup platform with independent legs, the platform legs are penetrated into the foundation soils to a certain depth. The penetration is a result of a continuous failure condition in the soil underneath the spudcan at the tip of each leg, and the final penetration depth is achieved when a stable condition is reached and supersedes the failure condition.

This is governed by the shear strength of the soil, the weight of the platform with preload, and the geometry of the spudcans. Of these parameters the undrained shear strength of the soil and its variation with depth is found to be the most important parameter with the final penetration depth being more sensitive to uncertainties in this very strength and its variation with depth than to uncertainties in any other parameters.

The stability of the hole generated around a deeply penetrating spudcan is found to be a parameter of concern as caving-in of the side wall may lead to a sudden additional load on top of the spudcan and thus cause a sudden additional penetration to occur. Punch-through failure is similarly found to be critical for the final penetration depth if an unfavourable soil profile in that respect is encountered, and the thickness of the top crust is found to be a very important parameter in this respect together with the undrained shear strength of the underlying softer clay, the presence of which is what makes the punch-through type of failure possible. For the study of punch-through failure it is, however, a limitation that the angle α of stress dispersion with depth through the top crust has not been varied, but this limitation is considered to be of minor importance, and the conclusions which have been drawn for this failure mode are expected to hold also for the case that a whole range of dispersion angles α should be possible.

During operation, the platform may be subjected to severe environmental loading, first of all caused by extreme ocean waves. Sliding is not found to be a critical failure mode for such conditions except for one very special case where a soft clay of limited thickness is overlying a harder stratum. Conventional bearing capacity failure is generally found to be the most likely failure mode, and the stability against such bearing capacity failure, which obviously also will be the stability against additional penetration, is found to be heavily dependent on the preload level in the penetration phase and on the soil strength and its variation with depth. The most critical soil conditions are thin strata with sand or stiff clay overlying a weaker zone of clay, i.e. the potential punch-through profile, but soft clays may as well be critical, as considerable additional penetrations may develop in storms as a result of instabilities in cases where insufficient preloading has been carried out in the penetration phase.

Other findings with respect to the importance of soil properties imply that strength gain and corresponding improvement of bearing capacity because of consolidation for platform weight are minor, and shakedown due to repeated loading in storm is usually not critical for the platform stability except if large displacements are resulting and the soil profile is a potential punch-through profile. Scour was not analyzed but may dominate the foundation stability for platforms which are set on sand, especially when considering that spudcans are usually not fully penetrated in sands.

Table 2.1 presents the ranges of parameters investigated for jackup platforms with independent legs. These ranges form the boundary conditions for the performed parameter studies, and conclusions drawn for this platform type in this report are therefore restricted to these parameter ranges only.

A significant part of the studies of the foundation stability of platforms with independent legs was carried out by means of the computer program SAIL (Stability Analysis of Independent Legs), which was developed during PP2 on the basis of available methods and theory.

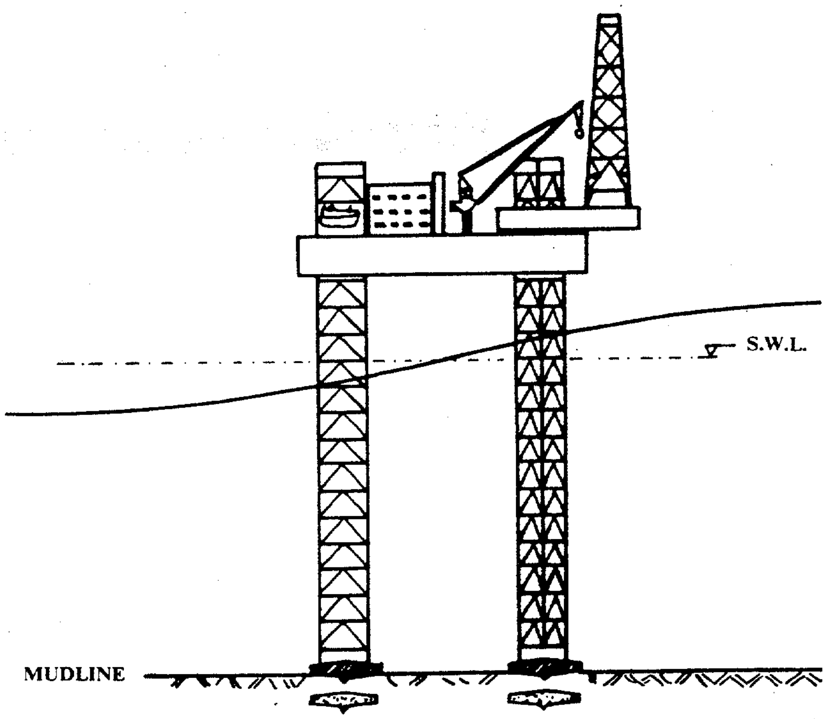
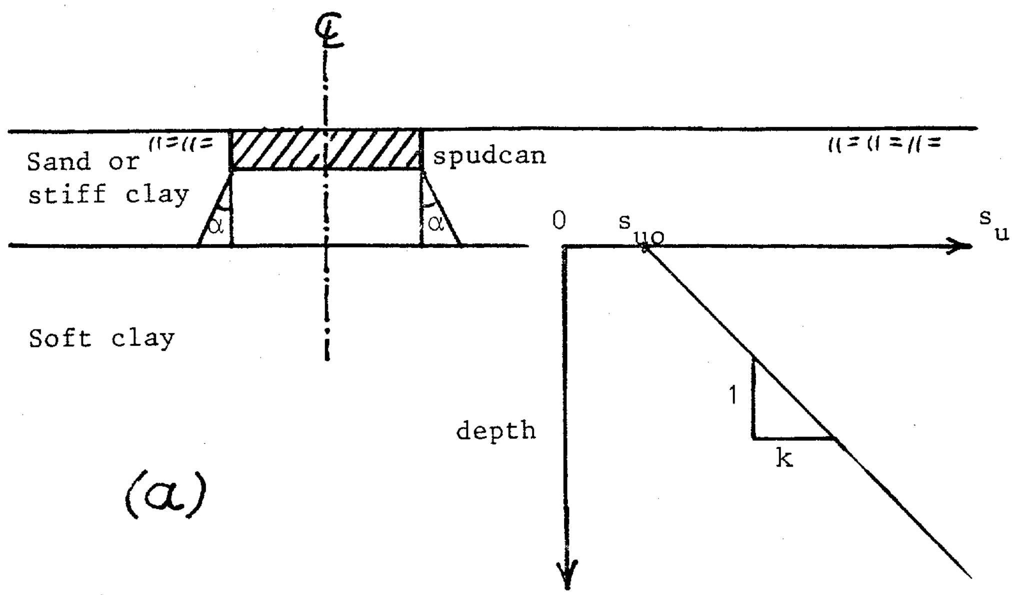


Fig. 2.1 Principle sketch of jackup platform with independent legs and spudcans.



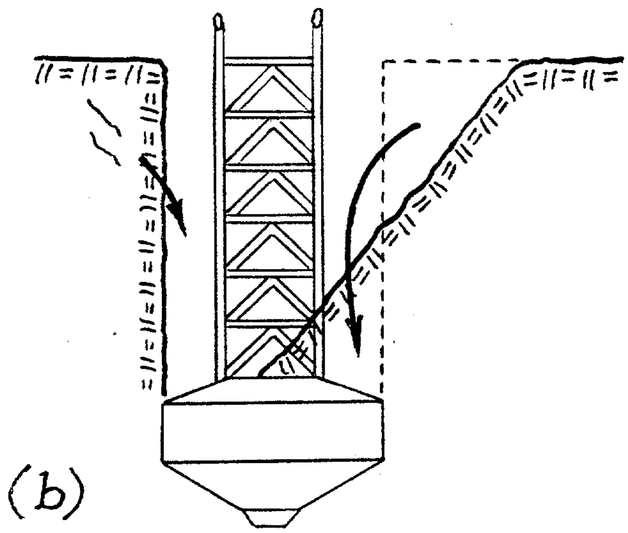


Fig. 2.2 Critical conditions for foundation stability under platform installation
a) Spudcan in typical punch-through profile
b) Caving-in of side wall above spudcan

Table 2.1 Jackup platforms with independent legs.
Ranges of parameters investigated.

Parameter	Range
Submerged platform weight, kN	48000-144000
Variable load, kN	12000-24000
Preload, kN	18000-60000
Water depth, m	30-100
Wave height, m	12-30
Wave period, sec	9-20
Current velocity, m/sec	0-2
Wind velocity, m/sec	45-55
Drag coefficient	0.7
Inertia coefficient	2.0
Effective leg diameter for drag, m	4-6
Effective leg diameter for inertia, m	1.6-2.4
Bending stiffness of leg EI, kNm^2	$0.6 \cdot 10^9 - 2.4 \cdot 10^9$
Axial stiffness of leg EA, kN	$0.8 \cdot 10^8 - 2.6 \cdot 10^8$
Shear stiffness of leg GA, kN	$1.6 \cdot 10^6 - 2.4 \cdot 10^6$
Distance between fore and aft legs, m	30-52
Spudcan diameter, m	10-16
Shear strength of stiff crust, kPa	25-90
Shear strength at top of soft clay under crust, kPa	2.5-7.5
Shear strength at mudline for uniform clay, kPa	20-60
Shear strength gradient with depth, kPa/m	0.6-1.8
Submerged unit weight of clay, kN/m^3	6-8
Submerged unit weight of sand, kN/m^3	10

2.2 MAT-SUPPORTED JACKUP PLATFORMS

Penetration of mats due to platform weight during installation of mat-supported jackup platforms is usually very limited, and additional displacements due to consolidation settlements are also very small during the short time such platforms are present at each location.

Sliding is generally found to be the critical failure mode for mat-supported jackup platforms subjected to environmental loading, and the shear strength of the soil immediately below mudline is found to be the most dominating parameter governing the stability against sliding, with wave force, skirt height and mat geometry being other parameters of importance.

However, local failure under the most stressed edge of the mat may become a more critical failure mode for very deep waters due to development of high soil stresses. Local failure is normally a self-stabilizing failure mode if the mat geometry is properly designed, except for very soft soils which allow for large penetrations and backfilling of soil and thereby counteract the self-stabilizing effects. For very shallow waters deep-seated failure may become the most critical failure mode.

Also for this platform type it was discovered that strength gains due to consolidation effects will be minor during the short time such a platform is present at a specific location. As sliding is the most critical failure mode, effects of repeated loading in terms of a reduced shear strength may tend to influence the stability of this platform type more than the stability of platforms with independent legs, first of all because of the two-way cyclic loading type involved with the sliding mode of failure.

Table 2.2 presents the ranges of parameters investigated for mat-supported jackup platforms. These ranges form the boundary conditions for the performed parameter studies, and conclusions drawn for this platform type in this report are therefore restricted to these parameter ranges only.

Most of the studies of foundation stability of mat-supported platforms were carried out by means of the computer program SAM (Stability Analysis of Mats), which was developed during PP2 and based on available methods and theory. Furthermore, most of the studies were carried out for clays only, thereby reflecting that mat foundations are usually only of interest for platforms on soft clayey soils.

Table 2.2 Mat-supported jackup platforms.
Ranges of parameters investigated.

Parameter	Range
Submerged platform weight, kN	25000-55000
Water depth, m	30-100
Wave height, m	12-30
Wave period, sec	9-20
Current velocity, m/sec	0-1
Wind velocity, m/sec	45-55
Drag coefficient	0.6-0.8
Inertia coefficient	1.8-2.2
Effective leg diameter, m	2.5-4.5
Skirt height, m	0.3-0.7
Mat thickness, m	2.5-4.5
Mat area, m^2	1050-3450
Finger width, m	7.5-20.0
Shear strength at mudline, kPa	5-15
Shear strength gradient with depth, kPa/m	0.3-1.8
Submerged unit weight of soil, kN/m^3	6-8

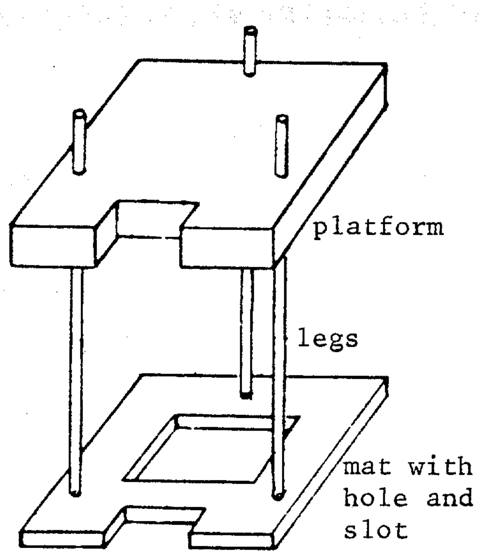


Fig. 2.3 Principle sketch of idealized mat-supported jackup platform.

3. RECOMMENDATIONS AND GUIDELINES FOR DESIGN OF JACKUP PLATFORMS WITH RESPECT TO FOUNDATION

3.1 MAGNITUDE AND DISTRIBUTION OF CONTACT STRESSES ON SPUDCANS AND MATS

No particular evaluation of the contact stress distribution on spudcans and mats has been performed during the parameter study, so the base for giving recommendations with respect to stress distributions is very limited. However, for design purposes commonly accepted bearing capacity formulae may be very relevant for evaluation of e.g. average spudcan pressures and local stress concentrations under mats.

State-of-the-art for design of jackup foundations has so far been limited to consider relatively conservative ranges for the contact stresses depending on whether spudcans or mats are used for transfer of the forces to the soil. The choice of foundation type is essentially a matter of which soil type a topical platform mainly is meant to be used for, and this soil type should always be kept in mind during a design procedure, cfr. the referenced importance of the soil strength in the technical summary. The conservatism in the applied stress ranges does first of all reflect the economical consequences of an insufficient design, i.e. difficult and expensive repairs. For mat foundations subjected to environmental loading it has been common to apply a linear stress distribution over the width which due to conservatism in the stress values usually will be sufficient, but especially for deep waters one should bear in mind the high edge stresses that may develop.

When it comes to the foundation stability and its dependence on the contact stresses, the stress distributions at spudcans have hardly any influence at all, because it is the total load level at each footing, not the stress distribution, which is of importance. With respect to mats, sliding is usually the critical failure mode which for clays will be independent of the contact stresses and their distribution, however, for deep waters high edge stresses may develop and cause local failure to be more critical, but this does usually not involve any serious risk because of its self-stabilizing characteristics.

3.2 FOUNDATION-INDUCED MOMENTS IN LEGS

For mat-supported platforms the legs can be considered as fixed to the mat, and foundation-induced moments in the legs are of no relevance. For platforms with independent legs, however, foundation-induced moments do exist in the legs, and the following considerations are therefore referring to this platform type:

Environmental loading causes moments in the platform legs to be induced, and these moments are transferred to the soil through the footings depending on the available moment capacities of the footings. These capacities are essentially dependent on the current horizontal and vertical load, i.e. they depend on the degree of mobilization of total available soil capacity. The larger the available moment capacity and rotational stiffness of a footing is, the larger moment will the footing attract, and the less will the top leg moment be. This implies that the footing foundations govern the moments and their distribution along the legs.

With respect to design of legs against bending it is therefore important to have a good control with the moment capacities of the footings, i.e. a good knowledge of the footings' degree of mobilization at given load combinations is required. Reasonably conservative assumptions have to be made, i.e. a low footing capacity will be unfavourable for design of upper parts of legs, while a high footing capacity will be unfavourable for design of lower parts of legs.

In this context one may realize that it sometimes may be unconservative to design legs according to the very common design assumption that the leg is "pinned at least 3 m below seabed". This assumption may not always be representative for the actual soil stiffness and footing capacity.

One must be very careful not to overestimate the soil stiffness and footing capacity when upper parts of legs are to be designed. As of today there is a tendency to "stretch" the soil stiffness and moment capacity of footings somewhat to the extreme in order to reduce the maximum top leg moments and thereby allow existing platforms to be used at greater water depths than they originally were designed for. The philosophy behind this tendency is i.a. the desire for utilization of consolidation effects on soil strength and stiffness, but during the present project these effects have appeared to be very insignificant. The consequence is that if existing platforms are to be used at greater water depths than originally intended an adequate preloading must be carried out, providing for the necessary foundation stiffness and thereby preventing the top leg moments from exceeding their original design values when the platforms are set in deep waters.

The effect of cyclic loading under extreme environmental conditions needs careful consideration in this context. High edge pressures and local softening of the soil under the spudcan edges may result in severe reductions in rotational stiffnesses.

3.3 PENETRATION DEPTH AND PULL-OUT RESISTANCE

The penetration depth is usually very limited for mat foundations due to their large areas and correspondingly low contact stresses. Very often not even full contact is obtained over the foundation area before a sufficient bearing capacity is mobilized. The penetration depth is therefore usually of very little importance for the design of mat-supported platforms, and no guidelines in that respect are thus necessary. The penetration is neither a parameter of importance for the platform stability, as sliding is the critical failure mode, and this failure mode is very little sensitive to variations in a penetration depth which is very small anyway. However, one must be aware of one exception from this generalization, namely the case where very soft, underconsolidated clay deposits are encountered, allowing for significant penetrations in spite of low contact stresses and large mat areas.

For platforms with independent legs the penetration depth is a much more significant parameter which generally plays an important role for the platform behaviour. A certain minimum penetration depth is always required to provide the necessary stability of the footings in the extreme load condition, while on the other hand punch-through of layered soils or caving-in of generated soil walls at large penetrations should be avoided. These conditions may both cause sudden additional penetrations to occur with risks for structural damage and may be avoided by making a deliberate choice of penetration depth, i.e. first of all by selection of proper spudcan diameter and preload level, provided a relevant soil strength profile is known. In this context, commonly accepted bearing capacity formulae are generally recommended as a good tool for prediction of this penetration depth. Rather than predicting and avoiding the risks involved with unfavourable soil profiles, current practice has so far been to design the structure so that it can generally resist the consequences of sudden additional penetrations if e.g. a typical punch-through profile should be encountered.

The pull-out resistance is of great importance for design of mat-supported platforms as well as platforms with independent legs. Especially for mat-supported platforms enormous suction forces may develop during pull-out due to the large areas that are involved, and suction releasers will be necessary in order to reduce the pull-out resistance to the skirt pull-out resistance only. This may still be a significant resistance due to setup effects, i.e. soil strength recovery after the remoulding that occurs during installation, and should be considered for the structural design. For platforms with independent legs the pull-out resistance may also be significant, especially for deeply penetrated legs, and suction releasers and jetting systems have proven to be beneficial tools to reduce the involved tensile forces. From a pure geotechnical point of view, suction releasers are considered most suitable for clay, while jetting systems may be most efficient for sand.

Prediction of pull-out resistance may be carried out by means of available procedures and formulae presented in the PP2 and PP3 reports, and the designer must hence ensure that his platform gets sufficient hull buoyancy, jetting capacity etc. to provide the necessary pull-out force to counteract this resistance and release the platform when it is to be removed from a site.

3.4 FOUNDATION CAPACITY UNDER VERTICAL, HORIZONTAL AND MOMENT LOADING

The foundation capacity is of importance for the stability of a jackup platform, and it is therefore important to ensure that a sufficient foundation capacity is present when the platform is installed in the environment it is meant for. The foundation capacity consists of vertical and horizontal capacities and moment capacity, and these are all coupled which makes the matter of foundation capacity rather complicated. Application of commonly accepted bearing capacity formulae is, however, a good approach for evaluation of foundation capacities: The horizontal capacity may be determined by simple shear considerations in the horizontal plane, hence the vertical capacity may be found depending on the acting vertical and horizontal forces and by application of a bearing capacity formula, and finally the moment capacity can be found by introducing the maximum possible eccentricity of the current vertical load for the determined vertical capacity. A very central parameter in this procedure is the soil strength and its variation with depth, and for design purposes one must therefore have the likely soil profiles for the topical platform in mind. Especially for independent-leg-supported platforms it is essential to ensure that the bearing capacities of the footings are large enough to leave sufficient moment capacities, thereby to prevent unnecessarily extreme top leg moments and of course also to ensure a sufficient overall platform stability. In this context the spudcan diameter is an important parameter which may be varied to meet the requirements.

With respect to the foundation capacity it is important to notice that according to the findings of the parameter study one cannot count on any strength gain from consolidation of the soil, but must in fact consider a possible strength degradation due to cyclic loading, at least for mat foundations on clay. For foundations on sand, e.g. spudcans which are not fully penetrated, erosion may reduce the effective contact area and thereby lead to a reduced foundation capacity. This may be of a significant importance, but has not been covered by the investigations conducted in the parameter study. Placing of sandbags around partly penetrated spudcans in sand may be a proper approach to avoid possible hazards in terms of a reduced stability due to erosion.

3.5 PRELOADING

During installation of platforms with independent legs the preload is governing the penetration depth of the legs together with the platform weight and the soil strength. The preload is meant to ensure that the legs penetrate deeply enough to provide a sufficient capacity to resist the loads occurring in the extreme load condition for the platform. It is therefore often common practice to preload each of the legs to a total load level corresponding to the highest expected vertical load in one leg during the extreme storm. Theoretically, this preload will just so provide the necessary capacity for support of the extreme vertical load. However, when this extreme vertical load occurs in a storm it is accompanied by a horizontal load and a moment, and the mentioned preload may therefore not be sufficient to provide the necessary capacity for support of the combined load, especially not if possible cyclic loading effects and lack of consolidation effects on the soil strength are considered. The conclusion from this must be that one should usually preload to a total load level somewhat higher than the highest expected vertical load on one footing during storm, thereby to account appropriately for effects of concurrent horizontal force and overturning moment and possible effects of cyclic loading. Bearing capacity theory for combined loading may be used in conjunction with similar theory for pure vertical loading to evaluate the necessary vertical preload force.

During design of a jackup platform it is according to the above very important to provide a sufficient preload capacity, e.g. in terms of available ballast chamber volume for spudcans, in order to meet the needs and secure the stability for the environmental range that the platform is being built for.

4. RECOMMENDATIONS AND GUIDELINES FOR EVALUATION OF A JACKUP PLATFORM'S SUITABILITY FOR A SPECIFIC LOCATION

4.1 SITE INVESTIGATION AND SOIL TESTING

According to the technical summary in Chapter 2 it is quite clear that for both considered types of jackup platforms the shear strength of the soil and its variation with depth are by far the most important parameters governing the foundation stability of these platforms. To be more specific, the strength of the upper meter or so is usually of interest for mat-supported platforms, while the shear strength variation to some depth becomes important for platforms with independent legs. However, also for mat-supported platforms the shear strength variation to some depth may be important, because deepseated failure in some cases may be critical for this platform type.

It is therefore essential that adequate soil investigations are carried out at a potential site for a jackup platform prior to location approval and platform installation. Based on our geotechnical judgment and the results of the parameter study, such investigations may consist of regular borings with soil sampling, but since the soil strength is found to be of more importance than the soil's deformation properties, vane tests and cone penetration tests will usually be sufficient, providing for derivation of desired soil strength profiles.

Cone penetration tests conducted to some depth will e.g. be adequate for detection of potential punch-through profiles and thereby allow for application of necessary measures to avoid punch-through, e.g. a proper selection of spudcan dimensions.

Interpretation of data from shallow seismic surveys will give valuable additional information for planning purposes, evaluation of soil layer thicknesses and their variations over a certain area etc.

The required extent of soil investigations to be carried out should reflect already existing information, e.g. results of previously performed soil investigations, and it is therefore not reasonable to recommend any fixed procedure, as long as adequate information about the soil strength and its variation with depth is provided.

Traditionally, the current practice with respect to soil investigations for jackup platforms varies from no investigation at all to full investigation with deep borings etc. Absence of soil investigations is often founded on the assumption that the preloading practice provides a sufficient capacity against foundation failure and that the preloading operation is carried out successfully without the benefit of prior soil investigations.

These assumptions are, however, not quite in accordance with the findings of the present project: Preloading to a total vertical load equal to the highest expected vertical load in one leg is not always sufficient to secure the stability when this vertical load occurs in a storm and is accompanied by a horizontal load as well as a moment. And among the soil profiles which are likely to be encountered, potential punch-through profiles do represent a hazard to the platform in terms of instability, sudden additional penetration and risk for structural damage during the preloading operation.

These findings do both emphasize the needs for information about the soil profile. In this context it is therefore positive to notice todays's trend of paying more attention to the soil investigations as a result of the increasing use of jackup platforms for still more rough environments.

4.2 PREDICTION OF PENETRATION DEPTH AND PULL-OUT RESISTANCE

Mat-supported jackup platforms penetrate very little, so prediction of penetration depth is of main importance for platforms with independent legs. For this purpose use of commonly accepted bearing capacity formulae is known to give reliable results, and use of such formulae is therefore recommended, e.g. Skempton's formula or Brinch-Hansen's formula. Good penetration predictions by use of these formulae are dependent on a good knowledge of the soil's strength profile, and good soil investigations are thus required. Use of bearing capacity formulae is also in accordance with current practice.

Pull-out resistance for mats as well as for spudcan footings depends on submerged weight of structure, friction forces on sides (e.g. on skirts under mats) and suction forces. The effect of the latter may be eliminated by means of suction releasers or jetting systems. Uncertainties may be involved with respect to effect of weight and strength of possible backfilled soil, so care should be taken in this respect.

For prediction of pull-out resistance reference is further made to formulae and procedures presented in Section 2.4 of the PP2 report and Sections 3.3 and 3.5 of the PP3 report, forming the only available tools in this respect as of today, as no written procedure has come to our knowledge during our review of current practice.

4.3 STABILITY EVALUATION, FOUNDATION CAPACITY

Current practice with respect to evaluation of the stability of a topical jackup platform is limited to an evaluation of the overturning stability and an evaluation of the vertical stability. Sliding is only rarely investigated. The evaluation of the overturning stability is the only stability evaluation required by the classification societies and is carried out as a pure moment equilibrium without any geotechnical considerations at all. The evaluation of the vertical stability is automatically included when the penetration depth is predicted.

These evaluations are very limited and are not considered to be sufficient with respect to evaluation of the foundation stability, because the findings of the parameter study show that other failure modes than those involved with the above evaluations are critical for the foundation stability. For mat-supported platforms sliding is usually the most critical failure mode for the foundation, but deepseated failure as well as local failure may in certain cases be more critical. For platforms with independent legs bearing capacity failure for combined load on each footing represents the most likely failure mode, but sliding may in special cases be more critical.

The consequences involved with foundation failure caused by these failure modes may be severe and cover structural damage or even loss of the platform. It is therefore recommended that evaluations of a platform's suitability for a specific location should always include evaluation of the foundation stability with respect to the following failure modes:
- sliding for horizontal load (for mats as well as for independent footings)
- deepseated failure (for mats)
- local failure (for mats)
- bearing capacity failure for combined load on footings

Formulae and methods presented in the PP2 report may be recommended for use in this respect, but the computer programs SAM and SAIL developed for the parameter study may also form very useful tools for stability evaluations if their limitations and simplifications are borne in mind.

Reliable stability evaluations are essentially very much dependent on a good knowledge of the design environmental forces and of the soil strength profile at the site. With respect to the latter it is important to be aware of potential punch-through profiles which may be critical for the platform stability, even in the installation phase before any environmental forces occur. Consolidation effects on the soil strength are usually minor and should not be counted on, while some reduction in soil strength due to repeated loading in storm may be more likely, at least for mats, and should be considered in the stability evaluations.

4.4 PRELOADING

Preloading is only of topical interest for jackup platforms with independent legs and is carried out to allow these platforms to meet the safety requirements safely in the extreme loading condition. The necessary preload should therefore be determined in dependence of the expected loads in the design storm, and it is recommended to preload each leg to a total vertical load which should be calculated based on the highest expected vertical load for one footing in conjunction with concurrent horizontal force and overturning moment for that footing. Bearing capacity theory may be used for determination of this necessary preload, which will be somewhat higher than the highest expected vertical load, because the combined loading in storm requires a somewhat larger foundation capacity than that required for the pure vertical load alone.

With respect to the platform's suitability it is important to verify that it has got a sufficient preload capacity to provide the preload that is necessary for achievement of the desired penetration and a satisfactory stability in the extreme loading condition.

Various procedures exist for execution of preloading: Four-legged platforms may be preloaded by so-called diagonal loading, i.e. two diagonal legs are preloaded at once by releasing the other two legs. Three-legged platforms may be preloaded by ballasting one leg at a time or all legs at once. In terms of stability, preloading of one leg at a time is considered to be more safe than simultaneous preloading of all legs, although it is more time-consuming. This should be borne in mind, especially when preloading of a jackup platform is to be carried out at a site where the soil profile is a potential punch-through profile or a hazardous profile by other means.

4.5 SOIL-STRUCTURE INTERACTION

Soil-structure interaction is an important topic for evaluation of foundation stability of jackup platforms. This may be illustrated by considering a platform with independent legs subjected to environmental loading.

The separate footings will get different vertical loads as a response to the overturning moment, and this will leave different available foundation capacities and stiffnesses for the horizontal forces and moments on the footings. This means that horizontal forces will not be quite equally distributed between the legs, and some footings will attract larger moments than other footings. The foundation stability may therefore vary from one footing to another, and it is important to bear this in mind when stability evaluations are to be performed. It is, however, difficult to give any recommendations in this respect, because the matter of soil-structure interaction has only been considered to a relatively limited extent within this project, and then mainly in a qualitative way which only indicates the trends.

The limited representation of soil-structure interaction for jackup platforms within this project gives thus rise to a suggestion for further and rather deeper studies of this matter in the future, first of all with a view to a better quantification of its influence on the foundation stability. This will require some improvement and refinement of the representation of the foundation stiffnesses which are very central parameters in this context. It will be important e.g. to consider embedment effects and coupling between horizontal, vertical and rotational degrees of freedom, thereby to make the foundation stiffnesses more accurate and reliable. Also a realistic representation of the structure and its flexibility is of great importance, maybe even in three dimensions, and finite element analyses may hence form a useful tool for such studies.

The effect of cyclic loading may be of special importance for the rotational stiffnesses of spudcans when reliable predictions of foundation fixity under design loading from wind and waves are required.

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TECHNICAL REPORT

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VERITAS Report No.	83 - 0904
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Project No.	23 31 02
Title of Report	FOUNDATION STABILITY OF JACK-UP PLATFORMS APPENDICES TO FINAL REPORT FOR PP2: ANALYSIS METHODS
Approved by	
Client/Sponsor of project	Several Companies, see Foreword
Client/Sponsor ref.	
Work carried out by	Tore J. Kvalstad and Lars A. Kristiansen
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Summary

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FOUNDATION STABILITY

of

JACK-UP PLATFORMS

APPENDICES TO FINAL REPORT FOR PP2:

ANALYSIS METHODS

FEBRUARY 1983/KVAL

Revised August/September 1983/KVAL

APPENDIX A:

PP2 Analysis Methods:
Proposal for analysis methods
and programs to be used in PP3

Draft version June 10,1982/KVAL
Revised September 1983/KVAL

The specific analysis methods to be used in the project will to a certain extent depend on previous experience from the participants as well as other companies involved in the design and operation of jack-up platforms.

We have at the moment of writing received very few of the distributed questionnaires. In order to give you a basis for a discussion at the 2nd Steering Committee meeting we would like to outline a first proposal that we suggest will be followed by necessary adjustments as PP2 progresses.

As a general tool for analysing most of the effects listed in the PP2 specification of deliverables and to allow a parametric study as outlined for PP3 we suggest to develop a computer program system as presented in Figure 1.

To minimize possible limitations for the participants regarding applicability of the program system all program modules shall be written in FORTRAN to allow a simple transfer to the participants computer system.

The program system will consist of input modules for soil data, a simplified structure, footing geometry and environmental data and structure weight distribution.

A set of load calculation modules for evaluation of wind, wave and current forces will be implemented.

The structural stiffness will be modelled with a high level of simplification and non linear soil springs will be connected to the lower end of the legs.

A stepwise incremental procedure will be used for application of loads from the penetration and preloading phase through the extreme loading conditions to leg pull-out.

The nonlinear support springs will be evaluated by a set of program mocules based on bearing capacity analysis, lateral resistance and elastic lumped parameter models.

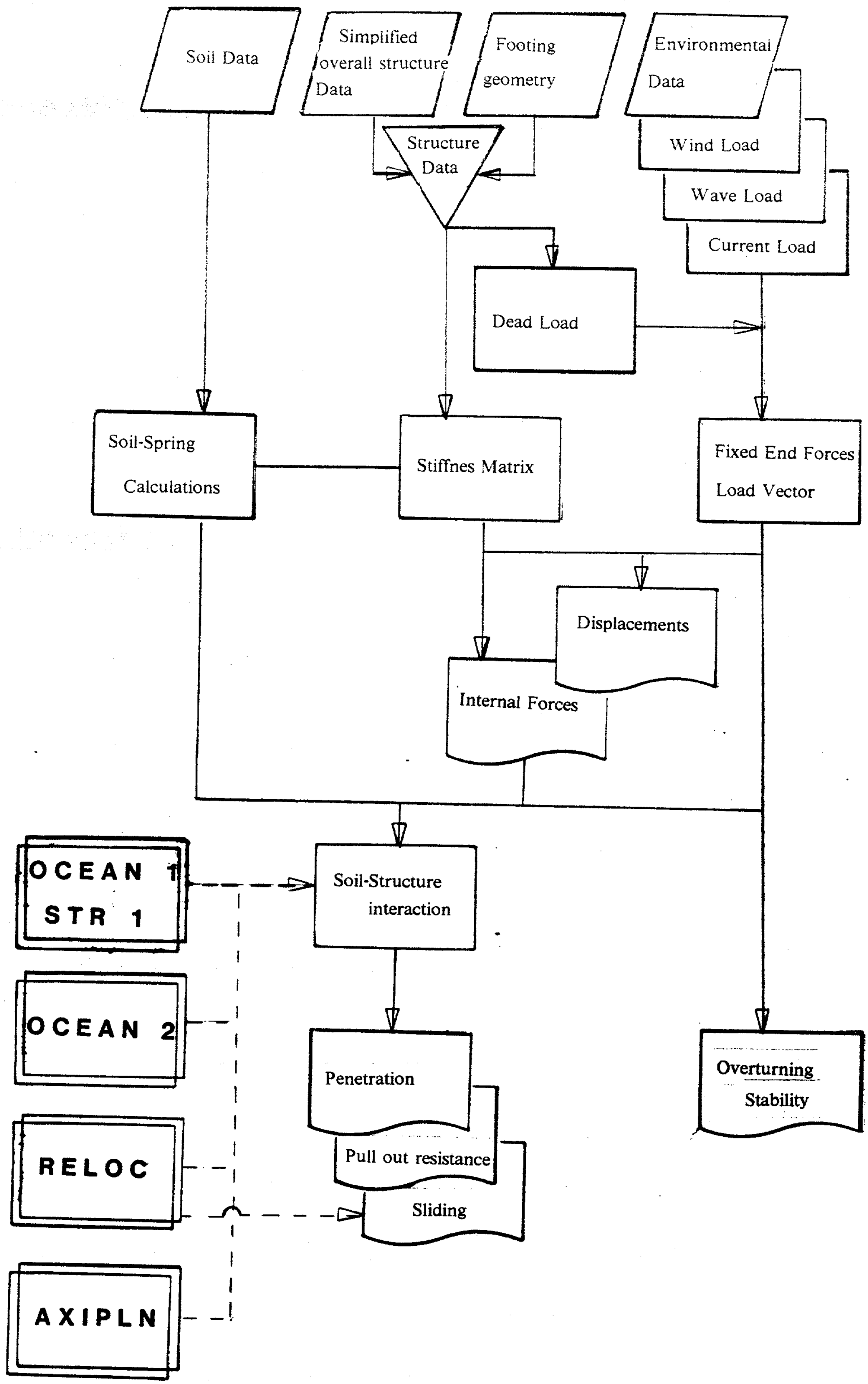
A set of programs have been defined for evaluation of special effects.

- Effect of pore pressure dissipation will be evaluated by a program OCEAN 2, an axisymmetric 3-dimensional program for consolidation.

- Validity and applicability of nonlinear spring models will be evaluated by program AXIPLN an axisymmetric and/or plain strain non linear FEM program for analysis of stresses and displacements under circular and strip footings.

- Effect of soil strength degradation on the sliding stability of mat-foundations will be evaluated by means of the program RELOC.

- Wave induced shear stresses in the soil mass and pore pressure increase will be evaluated by means of programs STR1 and OCEAN1.


Soil Data
Simplified overall structure Data
Footing geometry
Environmental Data
Structure Data
Wind Load
Wave Load
Current Load
Dead Load
Soil-Spring Calculations
Stiffnes Matrix
Fixed End Forces Load Vector
Displacements
Internal Forces
OCEAN 1 STR 1
Soil-Structure interaction
OCEAN 2
Penetration
Pull out resistance
Sliding
Overturning Stability
RELOC
AXIPLN

APPENDIX B:

PP3 Parametric Study:
A proposal for parameter
ranges to be analysed

Draft version June 10,1982/KVAL
Revised March 1983/KVAL

1 Introduction

The main objective of the parametric study is to determine the relative effect of the different parameters on the foundation stability and the overall behaviour of different types of jack-up platforms.

We are planning for a broad variation of the "most important parameters". The definition of the "most important parameters" is, however, dependent on the input to PP1 from the participants and other possible sources as well as the findings of this study and can thus at present not be completely defined.

To give a basis for a discussion we will in the following present a proposal for variations of the structural and foundation geometry, platform and environmental forces as well as the soil parameters. The proposed parameter ranges and the number of combinations will necessarily have to be limited in order to keep the possible number of combinations at a reasonable level.

2. ~meter variation

2.1 Variat... structural and foundation geometry

The variation in structural and foundation geometry should preferably allow simulation of most of the existing jack-up designs. A review of some of the the basic data from a large number of jack-up platforms has revealed that the jack-up platforms can be divided in a number of groups mainly based on their size. We propose to analyze three sets of what one could call "average jack-up platforms" regarding some of the basic features such as maximum water depth, weight, leg dimensions, wind area, etc. as outlined in Figure 1 and in Table 1.

The numbers may be subject to discussion and should at present merely be taken as an indication of the parameter ranges that will be covered.

The foundation geometry will comprise some simplified and idealized spud can shapes as well as mats as outlined in Figure 2.

2.2 Variation in environmental data and environmental forces.

The analyses to be carried out within the parametric study shall allow a simplified, but still realistic simulation of wind wave and current forces.

Wave, wind and current data may vary considerably from one offshore area to another and it is thus not possible to define one set of data that will reflect the environmental conditions world wide. However, world wide operation of mobile a drilling unit will require that the unit is designed to resist extreme conditions and we propose the values shown in Table 2.1 for wind, wave and current to be used in the parametric study.

The following comments should be noted.

Wind velocity:

The wind velocity 55 m/s represents an average wind speed to be applied to average wind area of the different models. It corresponds approximately to a 100 knot, 1 minute average velocity at 10 m height.

Wave height:

The wave heights of 15, 20 and 25 m for the three models at 30m, 60m and 100 m water depth respectively are not the most extreme wave heights for areas like the North Sea. It is, however, our impression that the given values are more representative for the majority of designs and offshore areas that extreme North Sea conditions where 3 to 5 m higher extreme waves would apply.

Current velocity:

The current velocity of 1 m/s is included to allow for evaluation of the relative effect of this parameter on the total environmental forces.

Wave periods: The wave lower values of wave periods have been chosen on the basis of a maximum steepness H/L of 1/7.

The wind forces F_w will be evaluated on the basis of the following rough approximation which is based on a review of a few available design calculations.

$$F_w = 0.2 \cdot 10^{-5} A_{hull} \cdot V_w^2 \quad (F_w \text{ in MN, } A_{hull} \text{ in m}^2 \text{ and } V_w \text{ in m/s})$$

where V^2 is the average wind velocity and A_{hull} the maxinum horizontal projection of the hull area.

The resultant wind force is assumed to act 12 m + airgap above the mean water level.

The hydrodynamic forces acting on the legs will be evaluated by means of Morison's ion and linear wave theory taking shallow water effects into accoun

$$dF = \tfrac{1}{2}\, C_D\, \rho\, D\, U\, |U|\, ds + C_m\, \rho\, A\, \dot{U} ds$$

A drag coefficient $C_D = 0.7$ and a mass coefficient $C_m = 2.0$ will be used.

Current and wave induced water particle velocities will be added vectorially.

2.3 Variation in soil parameters and stratification

The soil conditions on which a jack-up platform could operate vary within very wide limits. Within this study a selection of a few typical and idealized soil profiles will have to be made.

We propose the following three profiles which are shown schematically in Figure 3 .

Profile 1: A soft clay layer of thickness, T, underlain by a harder layer, clay or sand with a high bearing capacity. The undrained shear strength of the clay increases linearly from s_{uo} at mudline with a gradient k to $s_u(z) = s_{u,o} + k\,z$ at depth z.

Profile 2: A cohesive crust of thickness, T, with a constant undrained shear strength, S_{ucr}, underlain by a soft clay with the same characteristics as the clay of Profile 1.

Profile 3: A cohesionless layer of thickness, T, with a friction angel ϕ underlain by a soft clay with the same characteristics as the clay of Profile 1.

Profile 1 is representative for the soil conditions found on many parts of the world and the main problem for this type of soil seems to be excessive penetration of independent legs and the sliding stability of mats.

Profile 2 and 3 are representative for several areas where "punch-through" has turned out to be a problem. A relative thin crust of a material with higher strength overlying a soft stratum can be found in many offshore areas.

TABLE 1. Proposed basic parameters of 3 different jack-up structures

		MODEL 1	MODEL 2	MODEL 3	Variation
Water depth	d[m]	30.0	60.0	100.0	
Max. Wave height	H[m]	15.0	20.0	25.0	± 20%
Wave period range	T[s]	9-17	10-20	11-20	
Max. Wind speed (1 min. sustained)	v_w[m/s]	55.0	55.0	55.0	± 10%
Max current speed (tidal + wind induced)	v_c[m/s]	1.0	1.0	1.0	± 100%
Airgap (assumed 0.8H)	a[m]	12.0	16.0	20.0	
* Distance between fore and aft legs	b[m]	33.0	37.0	47.0	± 10%
* Platform weight	G[MN]	60	90	120	± 20%
* Projected area (hull) A hull	A_w[m^2]	250	400	530	
Lever arm for wind force (d + a + 12m)	l_w[m]	54.0	88.0	132.0	± 5%
Number of legs		3 or 4	3 or 4	3	
* Effective diameter of leg for drag	D_D[m]	5.0	5.0	5.0	± 20%
* Effective diameter of leg for inertia	D_i[m]	2.0	2.0	2.0	± 20%
* Leg stiffness, bending	EI [MNm^2]	$0.8 \cdot 10^6$	$1.3 \cdot 10^6$	$2.0 \cdot 10^6$	± 20%
* Leg stiffness, axial	EA [MN]	$1.0 \cdot 10^5$	$1.3 \cdot 10^5$	$2.2 \cdot 10^5$	± 20%
* Leg stiffness, shear	GA [MN]	$2.0 \cdot 10^3$	$2.0 \cdot 10^3$	$2.0 \cdot 10^3$	± 20%

* NOTE! The structure data presented are most representative for independent legs. Similar values will be suggested for different sizes of mat-supported rigs. We do however not have sufficient background material to estimate representative values before we received requested information from Baker Marine and Bethlehem Steel.

TABLE 2. Proposed basic parameters for 3 different mat supported jack-ups

Description of parameter	Dimension	MODEL 1	MODEL 2	MODEL 3	Variation
Water depth	d(m)	30.0	60.0	100.0	± 0 %
Max. wave height	H(m)	15.0	20.0	25.0	± 20 %
Wave period range	T(s)	9-17	10-20	10-20	
Max. wind speed (1.min.sustained)	v_w(m/s)	50	50	50	± 10 %
Max. current speed	v_c(m/s)	1.0	1.0	1.0	±100 %
Airgap (assumed 0.8 H	a(m)	12.0	16.0	20.0	
Distance between fore and aft legs	b(m)	25.0	30.0	35.0	
Platform weight	G(MN)	30	40	50	± 20 %
Projected area (hull)	$A_w(m^2)$	250	300	350	
Lever arm for wind force	l_w(m)	54	88	102	
Number of legs		3	3	3	
Total length of mat	TOTL(m)	55	65	75	
Total width of mat	TOTW(m)	40	50	60	
Net area/tot.area		.6	.6	.6	± 20 %
Mat thickness	TMAT	3.0	3.5	4.0	
Cylindrical legs	$D_i=D_d$ (m)	2.5	3.5	4.5	

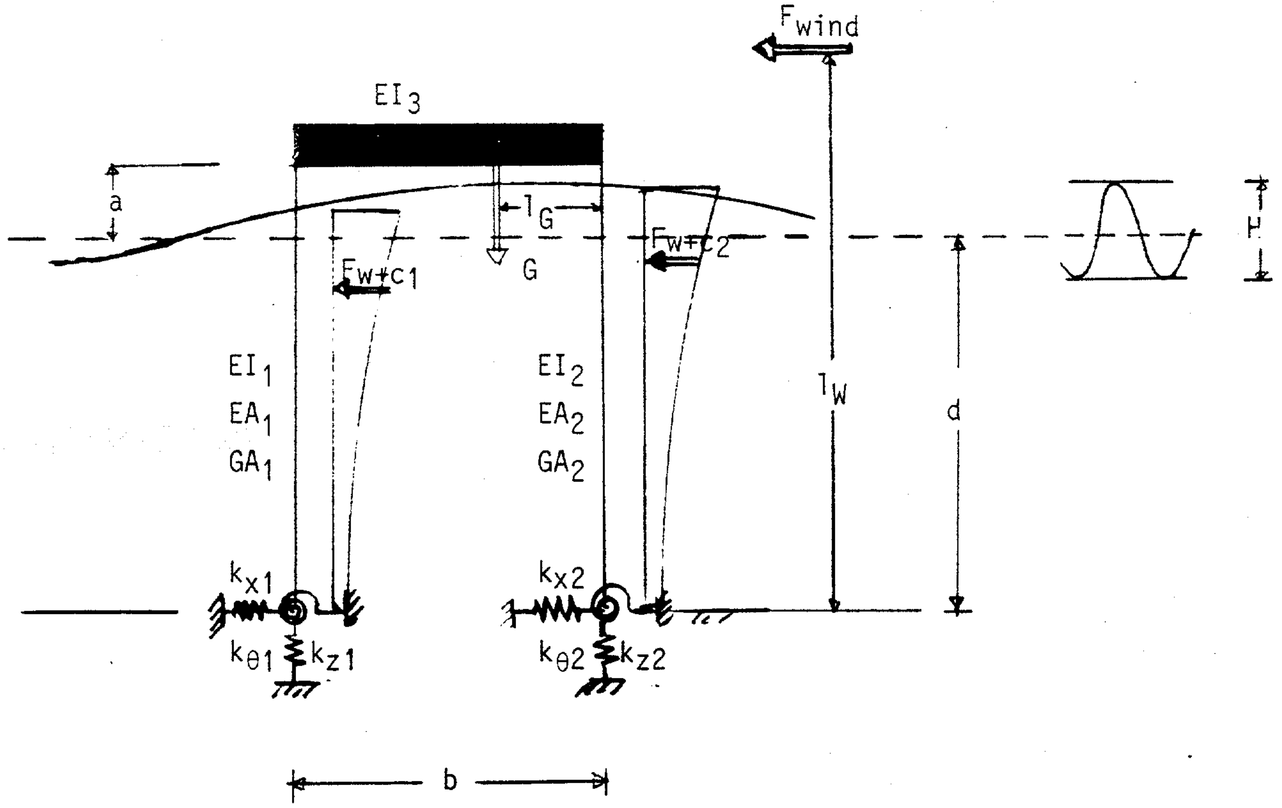


Figure 1. Proposed plane frame analysis of independent leg jack-up structure with basic dimensions, loads and stiffnesses.

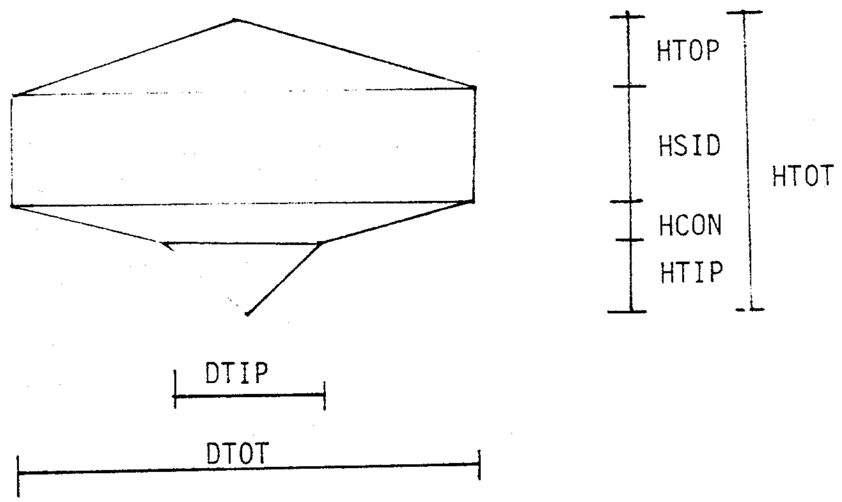


Figure 2a. Basic spud can shape to be studied

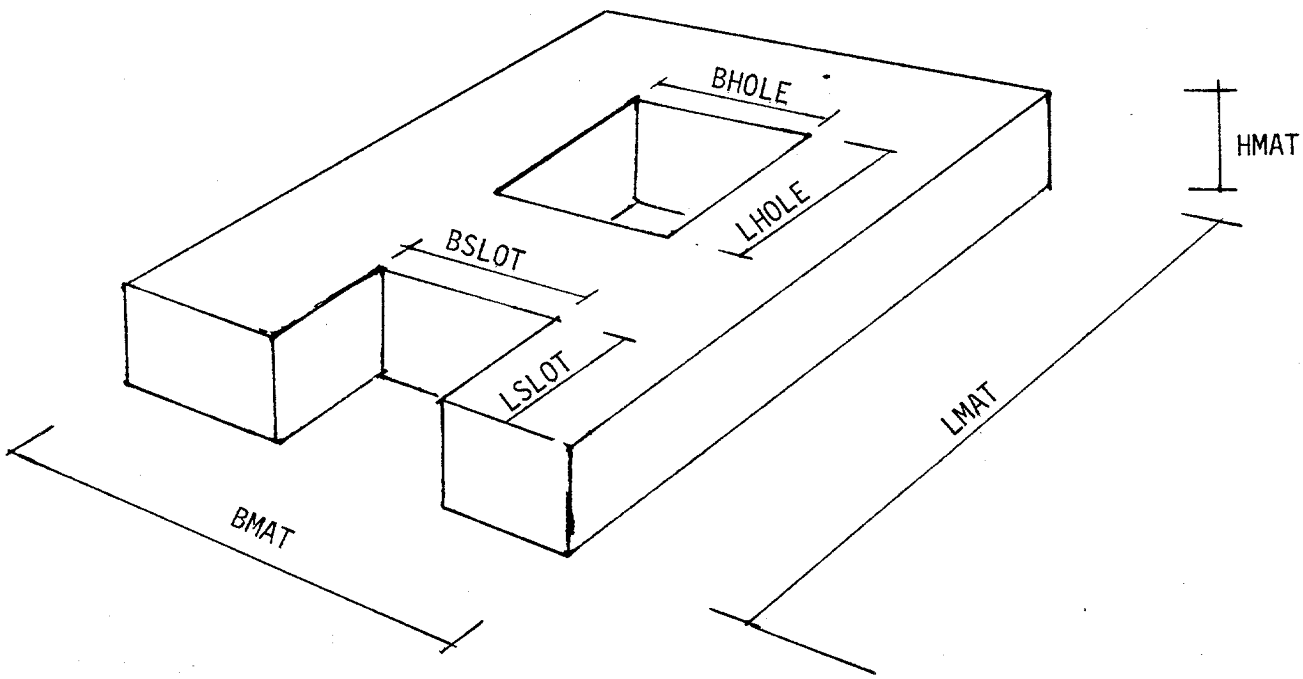


Figure 2b. Basic shape of mat foundations to be studied

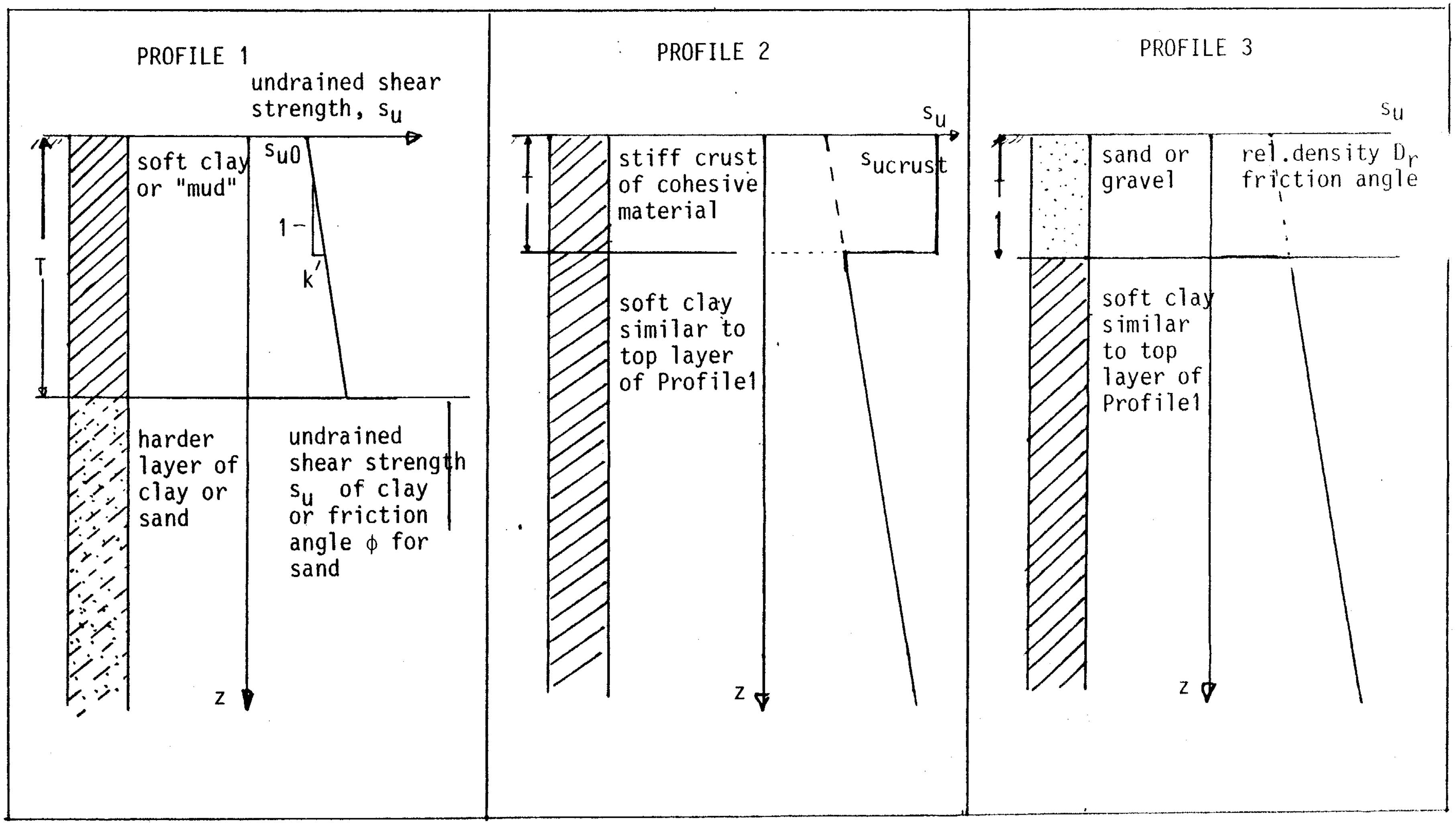


Figure 3. Idealized soil profiles to be investigated

APPENDIX C:

Bearing Capacity of Foundations
Appendix F: Foundations from
for
Design, Construction and Inspection
of Offshore Structures
(1977)

February 1983/KVAL

APPENDIX F
FOUNDATIONS

F1 BEARING CAPACITY OF FOUNDATIONS

F1.1 General

F1.1.1 Under certain conditions as explained in Section 9 (R) the stability of foundations and the point bearing capacity of piles may be analysed by means of bearing capacity formulae. In such cases, the procedure outlined below, in principle based on /1/, may be used.

F1.1.2 In the special case of a footing with horizontal base founded on a horizontal ground surface the design bearing capacity with respect to vertical load may be determined from the following formula.

$$q_d = \frac{1}{2}\gamma' b' N_\gamma s_\gamma d_\gamma i_\gamma + p_o' N_q s_q d_q i_q + c_d N_c s_c d_c i_c \quad \text{(F1-1)}$$

where

q_d	design bearing capacity
γ'	effective (submerged) unit weight of soil
b'	effective foundation width (see F1.2)
p_o'	effective overburden pressure at base level
c_d	design cohesion (c'/γ_{mc}) or design undrained shear strength (c_u/γ_{mc}) assessed on the basis of the actual shear strength profile, load configuration, and estimated depth of potential failure surface.
N_γ, N_q, N_c	bearing capacity factors, see F1.3
s_γ, s_q, s_c	shape factors, see F1.5
d_γ, d_q, d_c	depth factors, see F1.6
i_γ, i_q, i_c	load inclination factors, see F1.4

In the general case with inclined base and ground surface each term in Eq. F1–1 are multiplied with base and ground inclination factors, according to /1/.

The calculation is to be based on design shear strength parameters c_d and $\tan\phi_d$ defined as follows

$$c_d = \frac{c'}{\gamma_{mc}} \qquad \left(\text{or } c_d = \frac{c_u}{\gamma_{cm}}\right) \qquad \text{(F1-2)}$$

$$\tan\phi_d = \frac{\tan\phi'}{\gamma_{mf}} \qquad \text{(F1-3)}$$

where

c'	characteristic cohesion, see 9.6.4.2 (R).
c_u	characteristic undrained shear strength, see 9.6.4.3 (R).
ϕ'	characteristic angle of shearing resistance at the appropriate stress level, see 9.6.4.2 (R).
γ_{mc}, γ_{mf}	material coefficients associated with the actual type of analysis, see 9.5.5 (R) and 9.6.4 (R).

F1.1.3 Since the q- and c-factors in Eq. F1–1 are interrelated, see Eq. F1–8, the formula can be written in a somewhat simpler form. When the design angle of shearing resistance $\phi_d \neq 0$, the design bearing capacity is then:

$$q_d = \frac{1}{2}\gamma' b' N_\gamma s_\gamma d_\gamma i_\gamma + (p_o' + c_d \cot\phi_d) N_q s_q d_q i_q - c_d \cot\phi_d \qquad \text{(F1-4)}$$

For the special case that $\phi_d = 0$ (undrained failure in clay), additive constants are used instead of factors and we get:

$$q_d = c_d N_c (1 + s_{ca} + d_{ca} - i_{ca}) \qquad \text{(F1-5)}$$

F1.1.4 Mathematical expressions and numerical values for the various factors are given in F1.3 through F1.6. These are based on the assumption that an ideal plastic failure with full mobilization of the shear strength in the entire plastic zone governs the bearing capacity (general shear failure). For loose soils or at high stress levels, the shear strength may not be fully mobilized (local or punching shear failure). The bearing capacity may then be considerably smaller than that calculated for ideal plastic conditions. In calculations of bearing capacity, the effect of soil compressibility should be considered when appropriate.

F1.2 Effective foundation area

F1.2.1 In the calculation of bearing capacity according to /1/ the effective foundation area is used. This area is defined as follows:

The resultant of all horizontal and vertical forces acting from above upon the base of the foundation are combined into a resultant force. Each force is multiplied by a load coefficient, γ_f, according to 4.4.4.3 (R). The point where the resultant intersects the base, is called the load centre.

A rectangular "effective foundation area" is now determined. The geometrical centre of this area coincides with the load centre, and it follows as closely as possible the nearest contour of the actual base area. Two examples are shown in Figure F1.1. The width of the effective foundation area is b' and the length ℓ'.

C3

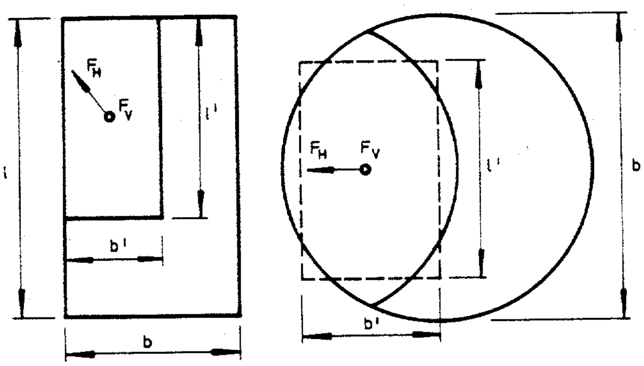


Figure F1.1 Effective foundation area A' = b'1'.

F1.3 Bearing capacity factors

F1.3.1 The expression for $N_c(\phi_d = 0)$ from /2/ is

$$N_c = \pi + 2 \simeq 5.14 \qquad (F1-6)$$

F1.3.2 The bearing capacity factor, N_q, from /3/ is

$$N_q = e^{\pi \tan\phi_d} \tan^2 (45 + \frac{\phi_d}{2}) \qquad (F1-7)$$

F1.3.3 The interrelationship between q- and c-factors is given by

$$N_c s_c d_c i_c = (N_q s_q d_q i_q - 1)\cot\phi_d \qquad (F1-8)$$

For an infinitely long surface footing subjected to a vertical, centric load this expression simplifies to

$$N_c = (N_q - 1)\cot\phi_d \qquad (F1-9)$$

F1.3.4 For N_γ, the following expression is recommended /1/

$$N_\gamma = 1.5 (N_q - 1)\tan\phi_d \qquad (F1-10)$$

while according to /4/

$$N_\gamma = 2(N_q + 1)\tan\phi_d$$

The latter relationship for N_γ is proposed for the calculation of local soil reaction stresses on the foundation structure as described in F5.

F1.3.5 Numerical values of N_c, N_q and N_γ are tabulated in Table F1.1.

Table F1.1 Bearing capacity factors N_c, N_q and N_γ

ϕ_d			N_γ	
Degrees	N_c	N_q	Caquot and Kerisel /4/	Brinch-Hansen /1/
0	5.14	1.00	0.00	0.00
1	5.38	1.09	0.07	0.00
2	5.63	1.20	0.15	0.01
3	5.90	1.31	0.24	0.02
4	6.19	1.43	0.34	0.05
5	6.49	1.57	0.45	0.07
6	6.81	1.72	0.57	0.11
7	7.16	1.88	0.71	0.16
8	7.53	2.06	0.86	0.22
9	7.92	2.25	1.03	0.30
10	8.35	2.47	1.22	0.39
11	8.80	2.71	1.44	0.50
12	9.28	2.97	1.69	0.63
13	9.81	3.26	1.97	0.78
14	10.37	3.59	2.29	0.97
15	10.98	3.94	2.65	1.18
16	11.63	4.34	3.06	1.43
17	12.34	4.77	3.53	1.73
18	13.10	5.26	4.07	2.08
19	13.93	5.80	4.68	2.48
20	14.83	6.40	5.39	2.95
21	15.82	7.07	6.20	3.50
22	16.88	7.82	7.13	4.13
23	18.05	8.66	8.20	4.88
24	19.32	9.60	9.44	5.75
25	20.72	10.66	10.88	6.76
26	22.25	11.85	12.54	7.94
27	23.94	13.20	14.47	9.32
28	25.80	14.72	16.72	10.94
29	27.86	16.44	19.34	12.84
30	30.14	18.40	22.40	15.07
31	32.67	20.63	25.99	17.96
32	35.49	23.18	30.22	20.79
33	38.64	26.09	36.19	24.44
34	42.16	29.44	41.06	28.77
35	46.12	33.30	48.03	33.92
36	50.59	37.75	56.31	40.05
37	55.63	42.92	66.19	47.38
38	61.35	48.93	78.03	56.17
39	67.87	55.96	92.25	66.75
40	75.31	64.20	109.41	79.54
41	83.86	73.90	130.22	95.05
42	93.71	85.37	155.54	113.95
43	105.11	99.01	186.54	137.10
44	118.37	115.31	224.64	165.58
45	133.88	134.87	271.76	200.81

C4

F1.4 Load inclination factors

F1.4.1 The expression for i_q and i_γ are:

$$i_q = 1 - \left[\frac{0.5F_{Hd}}{F_{Vd} + A'c_d \cot\phi_d}\right]^5 \qquad (F1-12)$$

$$i_\gamma = 1 - \left[\frac{0.7F_{Hd}}{F_{Vd} + A'c_d \cot\phi_d}\right]^5 \qquad (F1-13)$$

where

F_{Hd}	design horizontal load = $\gamma_f F_H$
F_{Vd}	design vertical load = $\gamma_b F_V$
F_H, F_V	characteristic horizontal and vertical load, respectively, compatible with the loading condition under consideration
γ_f	appropriate load coefficients according to 4.4.4 (R).
A'	effective foundation area

F1.4.2 The expression for i_{ca} in Eq. F1–5 is

$$i_{ca} = 0.5 - 0.5\sqrt{1 - \frac{F_{Hd}}{A'c_d}} \qquad (F1-14)$$

F1.4.3 The results of the calculations should be used with care when the ratio F_{Hd}/F_{Vd} approches or becomes less than 0.4.

F1.5 Shape factors

F1.5.1 The expression for s_q and s_γ are:

$$s_q = 1 + \frac{i_q b'}{l'} \sin\phi_d \qquad (F1-15)$$

$$s_\gamma = 1 - 0.4\,\frac{i_\gamma b'}{l'} \qquad (F1-16)$$

F1.5.2 For the case of $\phi_d = 0$, s_{ca} is:

$$s_{ca} = 0.2(1 - 2\,i_{ca})\,\frac{b'}{l'} \qquad (F1-17)$$

F1.6 Depth factors

F1.6.1 For shallow foundations, especially those of offshore gravity structures, the depth factor has almost negligible effect on the calculated bearing capacity. In this context we therefore use

$$d_q = d_c = 1.0 \qquad (F1-18)$$

which implies that $d_{ca} = 0$ in Eq. F1–5.
The depth factor d_γ is per definition equal to unity, thus

$$d_\gamma = 1.0 \qquad (F1-19)$$

F1.6.2 In special cases values $d_q > 1.0$ and $d_{ca} > 0$ may still be used, provided that the foundation installation procedure and other critical aspects allows for the mobilization of resisting shear stresses in the soil above the foundation level. In such cases the following expression for d_q, valid for $d < b'$, defines an upper limit for this contribution

$$d_q = 1 + 1.2\,\frac{d}{b'}\,\tan\phi_d\,(1 - \sin\phi_d)^2 \qquad (F1-20)$$

The corresponding expression for d_{ca} is

$$d_{ca} = 0.3 \arctan\left(\frac{d}{b'}\right) \qquad (F1-21)$$

which approaches a limit value $d_{ca} = 0.47$ for large depths.

F1.7 Simplified bearing capacity formulae for end resistance of piles

F1.7.1 The expressions given in F1.7.2 and F1.7.3 are valid for circular or square footings founded at depths $d > 4b'$. The load is assumed to be centric and vertical.

F1.7.2 For piles in mainly cohesionless soils, the following expression for the design unit end resistance q_{dp} may be used:

$$q_{dp} = (1 + \sin\phi_d)\,p_o'N_q \qquad (F1-22)$$

F1.7.3 For piles in mainly cohesive soils, the design unit end resistance may be expressed as:

$$q_{dp} = 9c_d \qquad (F1-23)$$

F1.7.4 For limitations in the use of the expressions given in F1.7.2 and F1.7.3 reference is made to F2.2.5, F2.2.7 and F2.2.9.

References

/1/ Brinch-Hansen, J.: "A Revised and Extended Formula for Bearing Capacity". The Danish Geotechnical Institute, Bulletin No. 28. Copenhagen, 1970.

/2/ Prandtl, L.: "Über die härte plastischer Körper". Nachrichten der Gesellschaft der Wissenschaften. Göttingen, 1920.

/3/ Reissner, H., "Zum Erddruckproblem". Proc. 1st Intern. Congr. Appl. Mech. Delft, 1924.

/4/ Caquot, A. and Kerisel, J.: "Sur la terme de surface dans le calcul des fondations en milieu pulvérult". Proc. 3rd Intern. Conf. Soil Mech. and Found. Engng., Vol. 1, Zürich, 1953.

APPENDIX D:

Squeezing of a thin soft layer overlaying a stiff layer

April 1983/KVAL
Revised August 1983/KVAL

Derivation of Expression for Evaluation of Squeezing Resistance

a) Circular Footing

Assumption: Flat based footing embedded at depth z in a soft clay layer of thickness T. Thickness of "squeezed" zone of material is $d = T - z$.

Full friction mobilized at top and bottom of squeezed zone, i.e. between spud-can base and soft clay and between soft layer and strong layer.

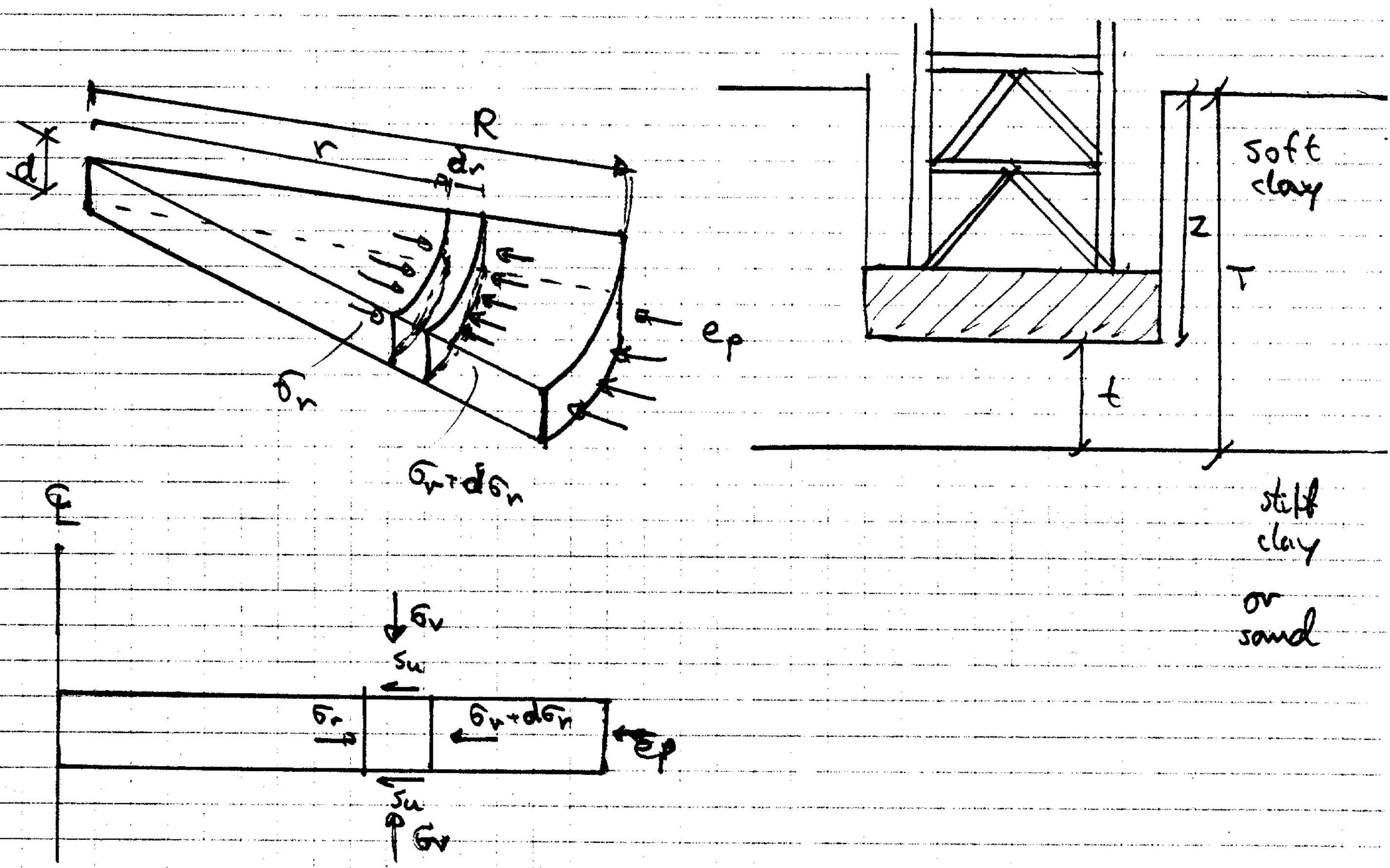


Figure D1. Geometry of squeezed zone, circular footing

The problem may be solved in the following way:

In the figure above the soil stresses on a sector of a circular "squeze" zone is shown.

Equilibrium in radial direction leads to the following expression:

Radial force on inner surface

$R_i = \sigma_r(r) \cdot r \cdot d\alpha \cdot d$

Radial force on outer surface

$F_o = (\sigma_r(r) + d\sigma_r) \cdot (r + dr) \cdot d\alpha \cdot d$ +

Top and bottom resistance

$F_{TB} = 2 \cdot r\, d\alpha \cdot dr \cdot s_u$

$$\Sigma F_r = 0 \Rightarrow \sigma_r(r) \cdot r\, d\alpha \cdot d = (\sigma_r(r) + d\sigma_r)(r + dr)\, d\alpha \cdot d + 2 r\, d\alpha \cdot dr \cdot s_u$$

Neglecting second order differentials with respect to r we get:

$$-\partial\sigma_r \cdot r = \sigma_r(r) \cdot dr + 2 r\, dr \frac{s_u}{d}$$

$$-\frac{\partial\sigma_r}{\sigma_r(r)} = \frac{\partial r}{r} + 2 \cdot \frac{s_u}{d} \cdot dr \qquad \text{and} \qquad \sigma_r(R) = e_p$$

The above differential equation has the following solution

$$\sigma_r = e_p \frac{R}{r} + \frac{c_u}{d}\left(\frac{R^2}{r} - r\right)$$

For undrained conditions the ultimate vertical stress cannot exceed the below expression

$$\sigma_{v\,ult} = \sigma_r + 2 s_u$$

By integrating σ_v over the footing area we get the squeezing resistance:

$$F = \int \sigma_v \cdot dA = \int_0^R \sigma_v \cdot 2\pi r\, dr = \int_0^R (\sigma_r + 2 s_u)\, 2\pi r\, dr$$

Inserting the expression for σ_r into the above equation gives

$$F = A \cdot \left(2 e_p + 2 s_u + \frac{4}{3} \frac{s_u \cdot R}{d}\right)$$

the average vertical pressure is thus

$$q_{av} = \frac{F}{A} = 2 e_p + 2 s_u + \frac{4}{3} \cdot \frac{s_u \cdot R}{d}$$

The passive pressure for a surface footing (or better close to surface)

$$e_p = 2 \cdot s_u + \gamma z$$

Thus,

$$q_{av} = 4 s_u + 2\gamma z + 2 s_u + \frac{4}{3} \frac{s_u D}{d} = 6 s_u + \frac{2}{3} \frac{s_u \cdot D}{d} + 2\gamma z$$

$$\frac{q}{s_{u\,av}} = N_{c\,\substack{squeeze \\ circle}} = 6 + \frac{2}{3} \frac{B}{d} + 2 \frac{\gamma z_{av}}{s_u}$$

$$s_{u\,av} = s_{u0} + k\left(z + \frac{t}{2}\right)$$

b) Strip Footing

For a strip footing a similar derivation of the squeezing capacity can be derived.

It can be shown that in this case the average ultimate squeezing pressure can be expressed as:

$$q_{ult} = 2 s_u + e_p + \frac{3}{4} \frac{s_u B}{d}$$

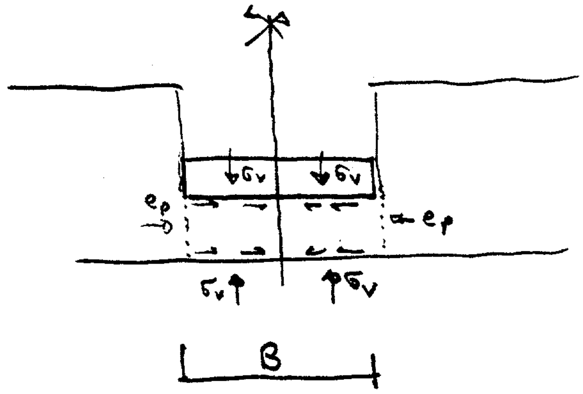


Figure D2. Geometry of squeezed zone, strip footing

again assuming

$$e_p = 2 s_u + \gamma' z$$

leads to

$$N_{c,\,squeeze\ strip} = \frac{q_{av}}{s_u} = 4 + \frac{3B}{4d} + \frac{\gamma' z}{s_u}$$

By comparing the derived expressions with bearing capacity formulae it can be shown that the expression for a circular footing tends to overestimate the capacity while the expression for strip footings tend to underestimate the capacity somewhat.

An alternative would be to use the expression for strip footing and adjust with Brinch-Hansen's shape and depth factors, as shown below:

$$N^*_{c,squeeze} = \left(4 + \frac{3}{4}\frac{D}{d}\right)(1 + s_{ca} + d_{ca}) + \frac{\gamma z_{av}}{s_u}$$

In Figure D.3 the expressions are compared with Brinch-Hansens bearing capacity formula for footings at the surface.

The conclusion to be drawn from this comparison is that the solution for a strip footing adjusted with Brinch-Hansen's shape and depth factors ~~p~~ seems to give reasonable and slightly conservative results. With this expression the squeezing effect start to be significant when the relative thickness of the squeezed zone (d/D) decreases below 0.6.

In practise this means that the calculated penetration depth of spud cans can be reduced with 0 to 3 m when

$$N_{c,squeeze,circle} = 6 + \frac{2}{3}\frac{D}{d} + 2\frac{\gamma z_{av}}{s_u}$$

$$N_{c,squeeze,strip} = 4 + \frac{3}{4}\frac{B}{d} + \frac{\gamma z_{av}}{s_u}$$

$$N^*_{c,squeeze} = \left(4 + \frac{3}{4}\frac{B}{d}\right)(1 + s_{ca} + d_{ca}) + \frac{\gamma z_{av}}{s_u}$$

$$N^*_{c\,Brinch\text{-}Hansen} = 5.14\,(1 + s_{ca} + d_{ca}) + \frac{\gamma z}{s_u}$$

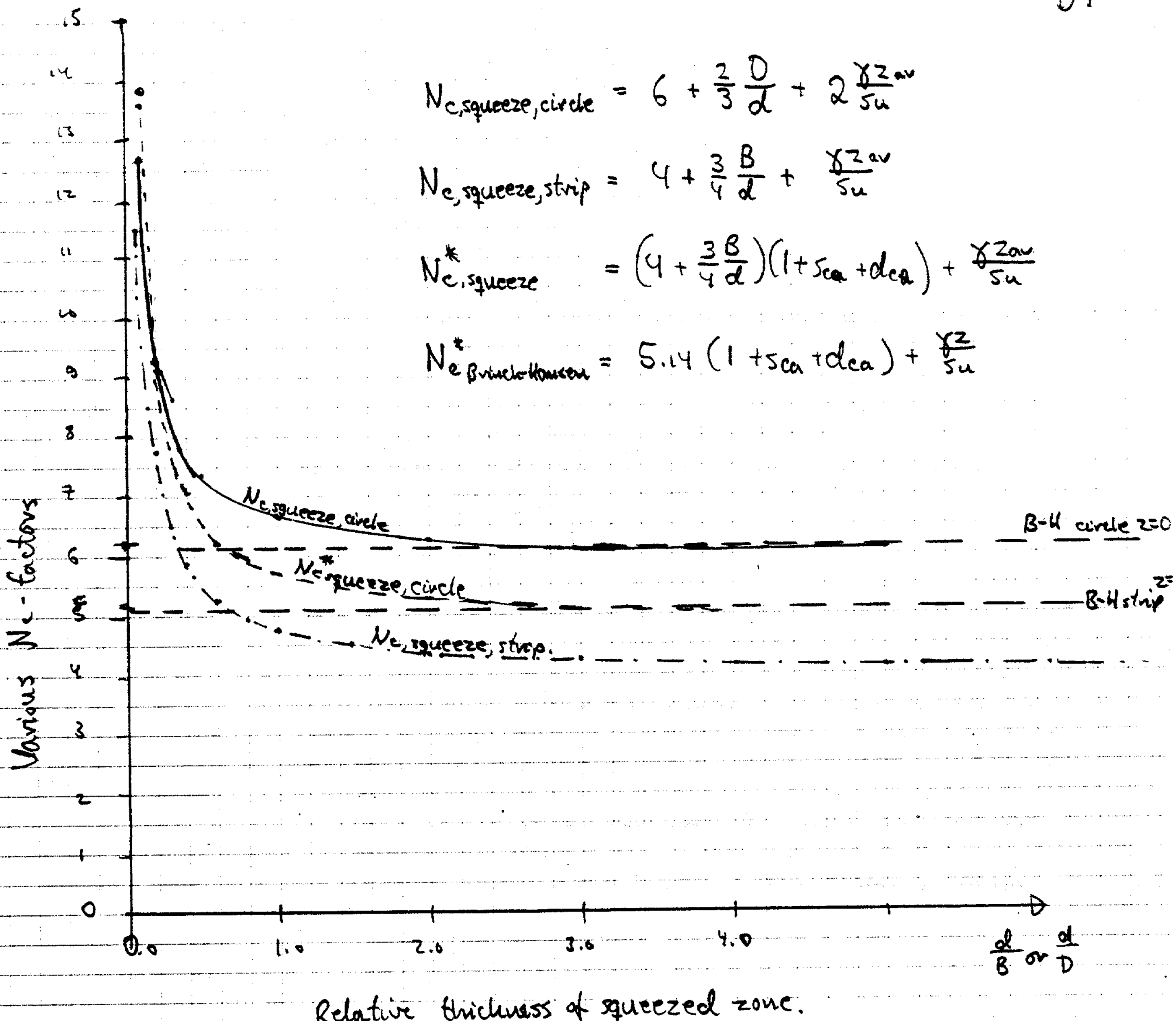


Figure D3. Comparison of solutions for squeezing capacity with Brinch-Hansen's formula for strip and circular footings. (Footing close to surface, i.e. $\frac{\gamma z}{s_u} = 0$)

the squeezing effect is taken into account, dependent on the actual situation regarding shear strength of the top layer, layer thickness, platform weight etc.

The expression for $N^*_{c,squeeze}$ has been adopted in the computer program SAIL.

APPENDIX E:

Derivation of expressions for shallow failure surface wedge method

February 1983/KVAL

Revised August-September 1983/KVAL

1. Center of cylinder below left edge of mat

1.a) Shear resistance on cylinder surface

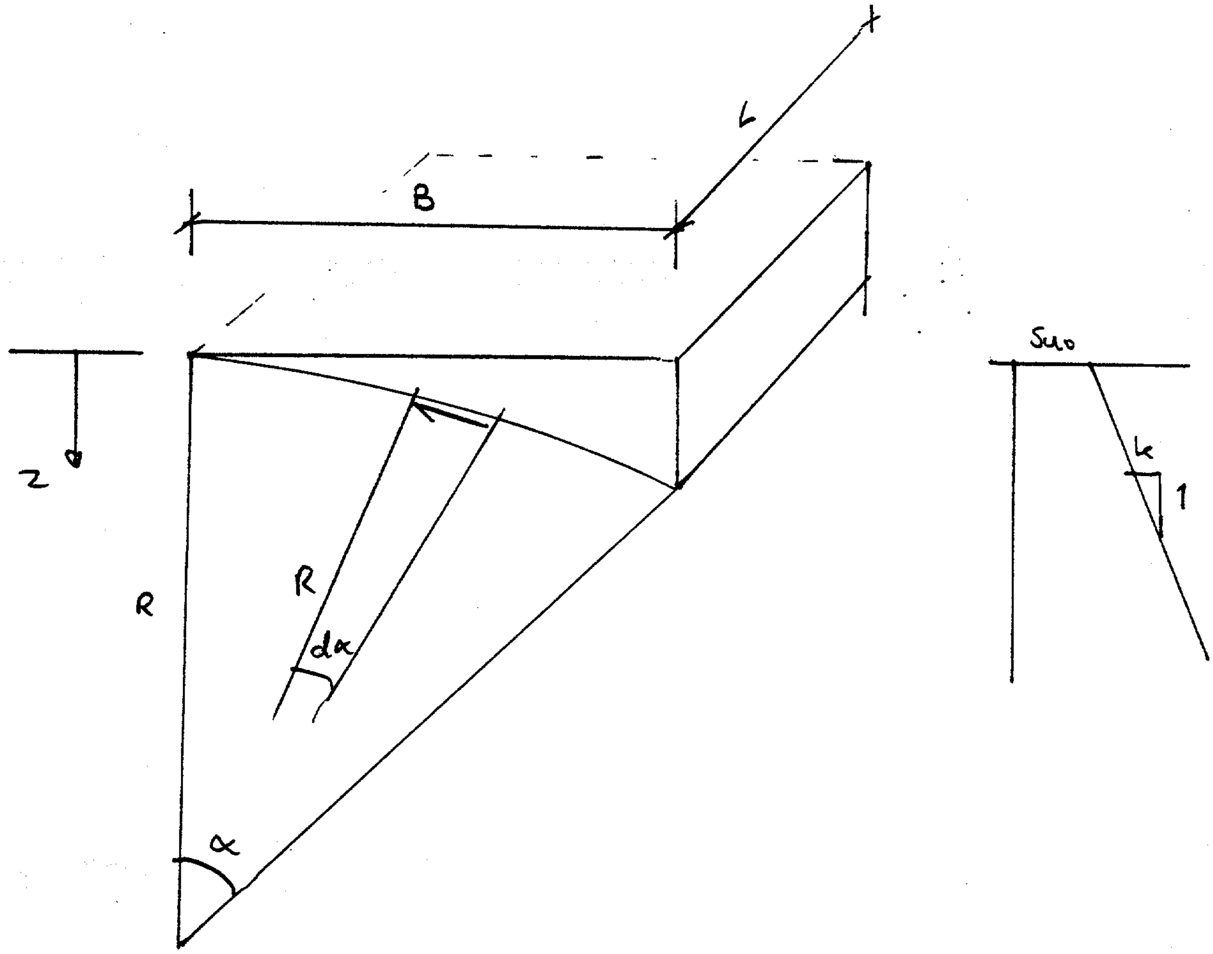
The moment resistance along the cylinder surface is found by integrating the ~~shear~~ undrained shear strength along the cylinder surface

$$M = \int_0^{\alpha} s_u(z) \cdot L \cdot R^2 d\alpha$$

$$s_u(z) = s_{u0} + kz$$

$$z = R \cdot (1 - \cos\alpha)$$

$$M = L \cdot R^2 \cdot s_{u0} \left[\alpha + \frac{kR}{s_{u0}} (\alpha - \sin\alpha)\right]$$

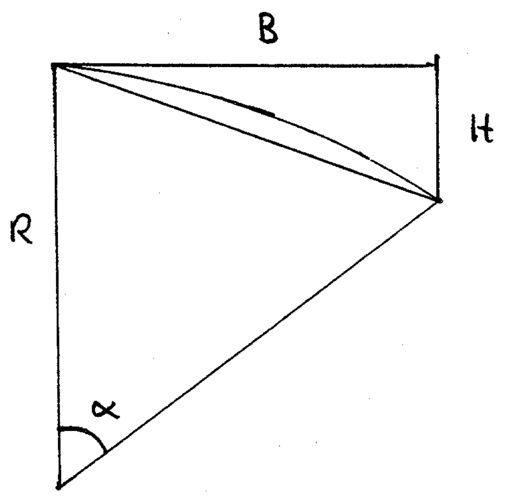


1.6 Shear resistance on sides of slipping soil mass,

The resisting moment due to the shear force on the sides of the slipping soil mass is calculated approximately by the following expression which take the resistance on both sides (front and back) into account

$$M \cong B \cdot H\left(s_{u0} + \tfrac{1}{3}kH\right)\left(R + \frac{H\cos\alpha}{3}\right) - R^2(\alpha - \sin\alpha)\cdot\left(s_{u0} + \tfrac{1}{2}kH\right)\cdot R\left(1 - \tfrac{2}{3}\left(1 - \cos\tfrac{\alpha}{2}\right)\right)$$

This expression can be explained as the resistance of a triangular area multiplied by the average ~~resistance~~ undrained shear strength ~~of~~ over the triangula multiplied by the approximate lever arm of the triangula. This is the top line and overestimates the resistance. The resistance of the excess section between the circular line and the straight side of the triangula is subtracted in the second line.



$H = R(1-\cos\alpha)$

Triangula: Area $= \frac{1}{2}BH$

$s_{uav} \hat{=} s_{u0} + \frac{1}{3}kH$

lever arm $\approx R + \dfrac{H\cos\alpha}{3}$

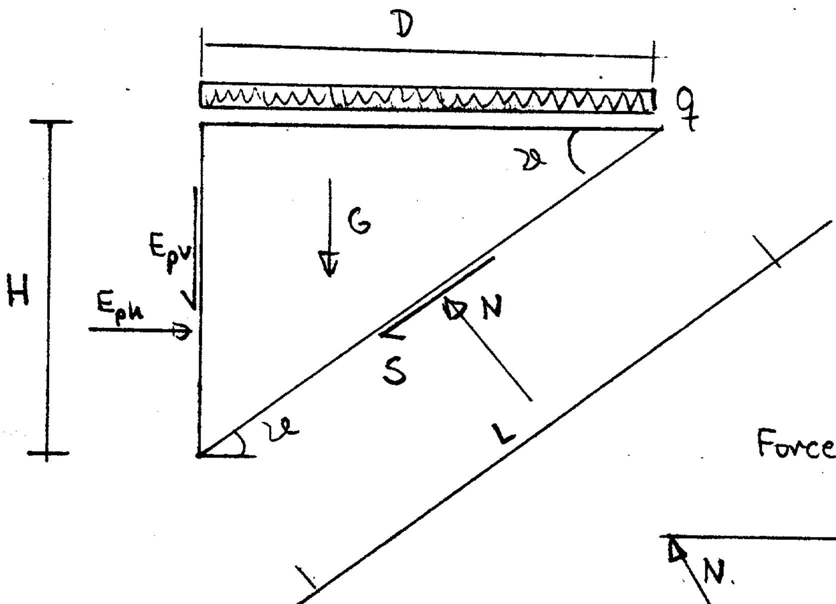
Sector: Area $= \frac{1}{2}R^2(\alpha - \sin\alpha)$

$s_{uav} \approx s_{u0} + \frac{1}{2}kH$

leverarm $\approx R\left(1 - \frac{2}{3}\left(1 - \cos\frac{\alpha}{2}\right)\right)$

1. c Passive earth pressure components

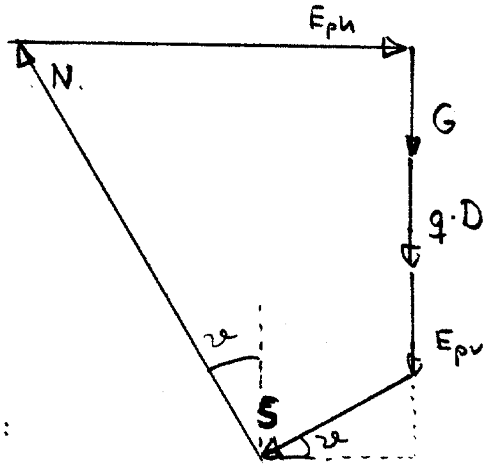
The passive earth pressure caused by a wedge shaped soil mass being pushed along a slip plane inclined ϑ to the horizontal is influenced by the vertical force component acting downwards at the front side of the wedge due to the downward moving cylinder wedge.



$$D = \frac{H}{tg\,\vartheta}$$

$$L = \frac{H}{\sin\vartheta}$$

Force vector diagram



Horizontal equilibrium requires:

$$E_{ph} = (G + q \cdot D + E_{pv} + S \cdot \sin\vartheta)\, tg\,\vartheta + S \cdot \cos\vartheta$$

The minimum value of E_{ph} is found by derivating the expression with respect to ϑ and ~~then~~ determining the zero value of the derivate.

Assuming a long wedge with no significant effect of the side forces on the wedge sides the previous expression ~~is valid~~ for E_{ph} is valid and gives

$$\vartheta_{min} = \arctan\sqrt{\tfrac{1}{2}}$$

and

$$E_{ph} = \left(2\sqrt{2}\, s_{u0} \cdot H \cdot \left(1 + \tfrac{1}{2}\frac{kH}{s_{u0}}\right) + \tfrac{1}{2}\gamma H^2 + qH\right) \cdot L$$

as can be seen the strength dependent part of this expression is $\sqrt{2}$ times greater than the normally used value of $2 s_{u0} \cdot H$ where no vertical component on the front side is accounted for.

The lever arms of the s_{u0} and q parts is half the $R - \frac{H}{2}$, while for the kH and γH part the lever arm will be $R - \frac{2}{3}H$

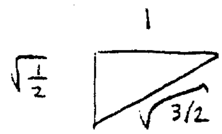
Thus,

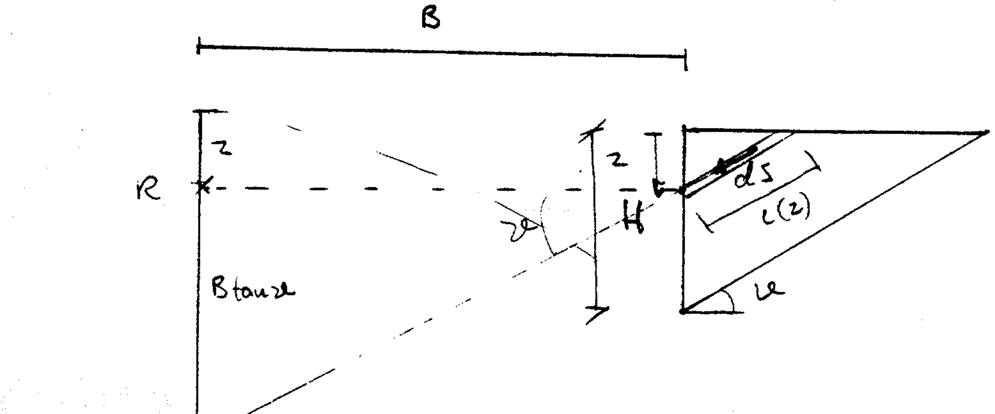
$$M = \left(R - \frac{H}{2}\right)\left(2\sqrt{2}\, s_{u0} H + qH\right) L + \left(R - \tfrac{2}{3}H\right)\left(\sqrt{2}\, kH^2 + \tfrac{1}{2}\gamma H^2\right) L$$

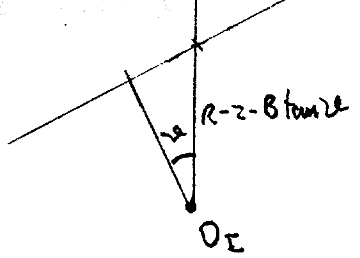
The vertical component gives a moment

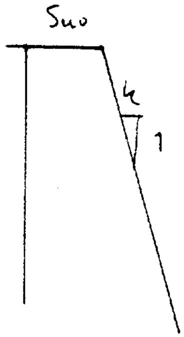
$$M = s_{u0} \cdot B \cdot H \left(1 + \tfrac{1}{2}\frac{kH}{s_{u0}}\right) \cdot L$$ where B is the lever arm

1d. Side force on passive wedge









$$ds = l(z) \cdot s_{uav}$$

$$dM = ds \cdot (R - z - B\tan\vartheta) \cdot \cos\vartheta$$ ← (corrected May, 1983 Unl)

$$s_{uav} = s_{uo} + \tfrac{1}{2}kz$$

$$l(z) = \frac{z}{\sin\vartheta}$$

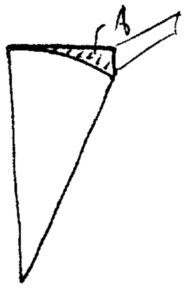
The side force is found by integrating the shear resistance along lines parallel to the slip angle ϑ of the passive wedge from the top corner to the bottom slip line. Front and back side must be included.

$$M = 2\int_0^H \left(s_{uo} + \tfrac{1}{2}kz\right)\frac{z}{\tan\vartheta}\cdot\left(R - z - B\tan\vartheta\right)dz$$

(corrected May 1983, Unl)

$$M = s_{uo}\cdot H^2\cdot R\cdot 2\sqrt{2}\left[\frac{1}{2} + \frac{1}{6}\frac{kH}{s_{uo}} - \frac{1}{3}\frac{H}{R} - \frac{1}{8}\cdot\frac{kH}{s_{uo}}\cdot\frac{H}{R} - \frac{1}{2\sqrt{2}}\cdot\frac{B}{R} - \frac{1}{6\sqrt{2}}\frac{kH}{s_{uo}}\cdot\frac{B}{R}\right]$$

+ e) Weight of slipping soil mass of sylinder wedge



Volume $= A \cdot L$

$A = BR - \frac{1}{2}B(R-H) - \frac{1}{2}R^2\alpha$

Unit submerged weight γ

$$M = \gamma' \cdot L \cdot \left[\frac{1}{6}B^2 \cdot R + \frac{1}{3}B^2 \cdot H - \frac{1}{3}R^3\alpha \cdot \sin\frac{\alpha}{2} \right]$$

1 f) Safety against failure along cylinder surface ~~below~~ with center below left edge of mat

$$M_{Res} = L \cdot R^2 \cdot s_{uo} \left[\alpha + \frac{kR}{s_{uo}} (\alpha - \sin\alpha) \right]$$

$$+ \left(R - \frac{H}{2}\right) \left[2\sqrt{2}\, s_{uo} \cdot H + qH \right] \cdot L$$

$$+ \left(R - \frac{2}{3}H\right) \left(\sqrt{2}\, kH^2 + \frac{1}{2} \gamma H^2 \right) L$$

$$+ B \cdot s_{uo}\, H \cdot L \left(1 + \frac{1}{2} \frac{kH}{s_{uo}} \right)$$

(corrected May, 1983 Ural)

$$+ 2\, s_{uo} \cdot H^2 \cdot R\sqrt{2} \left[\frac{1}{2} + \frac{1}{6} \frac{kH}{s_{uo}} - \frac{1}{3} \frac{H}{R} - \frac{1}{8} \frac{kH}{s_{uo}} \cdot \frac{H}{R} - \frac{1}{2\sqrt{2}} \frac{B}{R} - \frac{1}{6\sqrt{2}} \frac{kH}{s_{uo}} \cdot \frac{B}{R} \right]$$

$$+ BH \left(s_{uo} + \frac{1}{3} kH \right) \left(R + \frac{H\cos\alpha}{3} \right)$$

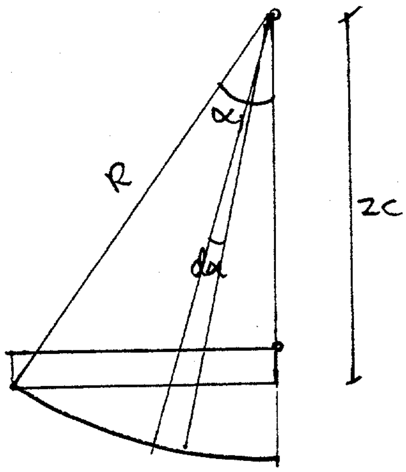
$$- R^2 (\alpha - \sin\alpha) \left(s_{uo} + \frac{1}{2} kH \right) R \left(1 - \frac{2}{3} \left(1 - \cos\frac{\alpha}{2} \right) \right)$$

$$M_o = F_V \left(\frac{B}{2} + \frac{M}{F_V} \right) + F_H \cdot R + \gamma' L \left[\frac{1}{6} B^2 R + \frac{1}{3} B^2 H - \frac{1}{3} R^3 \alpha \sin\frac{\alpha}{2} \right]$$

$$SF_I = M_{Res} / M_o$$

2. Center of cylinder above right edge of mat

2. a) Shear resistance on cylinder surface



$$M = \int_0^{\alpha} s_u(z) \cdot L \cdot R^2 d\alpha$$

$$s_u(z) = s_{u0} + kz$$

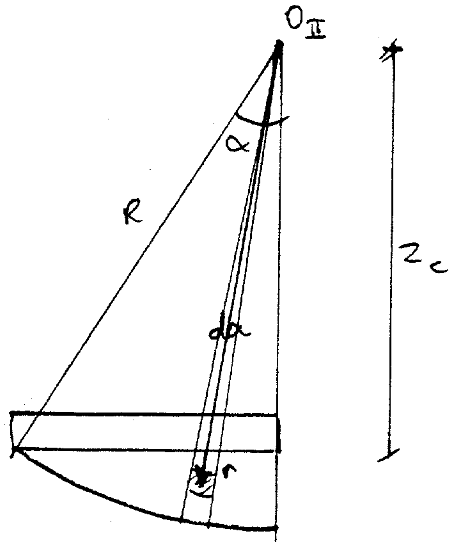
$$z = R\cos\alpha - z_c$$

$$z_c = R \cdot \cos\alpha_1$$

$$M = L \cdot s_u \cdot R^2 \left(\alpha_1 + \frac{kR}{s_{u0}} \cdot \sin\alpha_1 - \frac{kR}{s_{u0}} \cdot \cos\alpha_1 \cdot \alpha_1\right)$$

(corrected May 1983)
Uval

2.6) Shear resistance on sides of slipping soil mass



$dA = r\,d\alpha\,dr$

$dM = s_u(z) \cdot r^2\,d\alpha\,dr$

$s_u(z) = s_{u0} + k \cdot z$

$z = R\cos\alpha - z_c$

$$M = 2\int_0^{\alpha_1}\int_{z_c/\cos\alpha}^{R} \left(s_{u0} r^2 + k r^3 \cos\alpha - k r^2 \cdot z_c\right) dr\,d\alpha$$

$$= \frac{2}{3} s_{u0} \cdot R^3 \alpha_1 + \frac{1}{2} k R^4 \sin\alpha_1 - \frac{2}{3} k z_c R^3 \alpha - 2\left(\frac{1}{3} s_u z_c^3 - \frac{1}{12} k z_c^4\right) \int_0^{\alpha} \frac{1}{\cos^3\alpha}\,d\alpha$$

$$M = 2 s_{u0} R^3 \left[\frac{1}{3}\alpha + \frac{1}{4}\frac{kR}{s_{u0}} \cdot \sin\alpha_1 - \frac{1}{3}\frac{k z_c}{s_{u0}}\alpha_1\right]$$
$$-2 s_{u0} z_c^3 \left[\frac{1}{3} - \frac{1}{12}\frac{k z_c}{s_u}\right]\left(\frac{1}{2}\frac{\sin\alpha}{\cos^2\alpha} + \frac{1}{2}\ln\left|\frac{1+\tan\frac{\alpha}{2}}{1-\tan\frac{\alpha}{2}}\right|\right) \text{ for } \alpha \le \frac{\pi}{2}$$

if $\alpha = \frac{\pi}{2}$, then $M = 2 s_{u0} R^3 \left(\frac{\pi}{6} + \frac{1}{4}\frac{kR}{s_{u0}}\right)$

2.c Passive earth pressure components.

The passive earth pressure was derived in 1.c)
The lever arms changes slightly so that:

$$M = \left(R - \frac{H}{2}\right)\cdot\left(2\sqrt{2}\, s_{u0}\, H + q\cdot H\right)\cdot L$$
$$+\left(R\cos\alpha + \frac{2}{3}H\right)\left(\sqrt{2}\, k H^2 + \frac{1}{2}\gamma H^2\right)L$$

The lever arm of the vertical component of the passive earth pressure is for this case zero and no moment is thus developed.

2.d) Side force on passive wedge

The side force of the passive wedge is derived similarly as under 1d). The lever arm changes only.

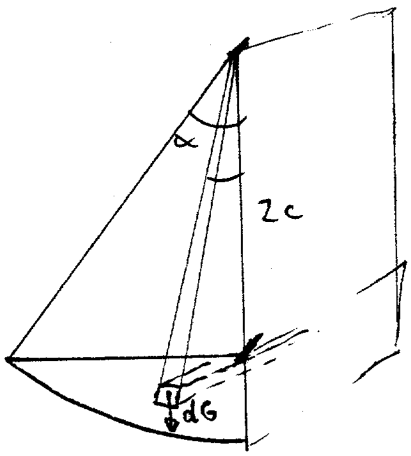
$$M = 2\int_0^H \cdot s_{uav}(z) \cdot l(z) \cdot \cos\vartheta \cdot (z_c + z)\,dz$$

$$l(z) = \frac{z}{\sin\vartheta}$$

$$s_{uav} = s_{u0} + \frac{1}{2}kz$$

$$M = 2\sqrt{2}\, s_{u0} H^3 \left(\frac{1}{2}\frac{z_c}{H} + \frac{1}{3} + \frac{k z_c}{6 s_u} + \frac{kH}{8 s_u}\right)$$

2 e) Weight of cylinder wedge



$dA = r\,d\alpha\,dr$

$dG = \gamma' \cdot L \cdot r\,d\alpha\,dr$

lever arm $= r \sin\alpha$

$z_c = R \cdot \cos\alpha$

$$M = \int_0^{\alpha} \int_{z_c/\cos\alpha}^{R} \gamma' \cdot L\, r^2 \sin\alpha\, dr\, d\alpha$$

$$M = \frac{1}{3} \gamma' \cdot L \cdot R^3 \left[1 - \frac{3}{2}\cos\alpha + \frac{1}{2}\cos^3\alpha \right]$$

2 f) Safety against failure along cylinder surface with center ~~below left~~ above right edge of mat

$$M_{Res} = LR^2 s_{uo}\left(\alpha_1 + \frac{kR}{s_u}\sin\alpha_1 - \frac{k z_c}{s_u}\alpha_1\right)$$
$$+\left(R - \frac{H}{2}\right)\left(2\sqrt{2}\cdot s_{uo}\cdot H + q\cdot H\right)\cdot L$$
$$+\left(R\cos\alpha + \frac{2}{3}H\right)\cdot\left(\sqrt{2}\,kH^2 + \frac{1}{2}\gamma H^2\right)L$$
$$+2\,s_{uo}R^3\left(\frac{1}{3}\alpha_1 + \frac{1}{4}\frac{kR}{s_{uo}}\cdot\sin\alpha - \frac{1}{3}\frac{kz_c}{s_{uo}}\cdot\alpha\right)$$
$$-\,s_{uo}\cdot z_c^3\left[\frac{1}{3} - \frac{1}{12}\frac{kz_c}{s_{uo}}\right]\left(\frac{\sin\alpha}{\cos^2\alpha} + \ln\left|\frac{1+\tan\frac{\alpha}{2}}{1-\tan\frac{\alpha}{2}}\right|\right)$$
$$+\;s_{uo}\cdot H^3\cdot 2\sqrt{2}\left(\frac{1}{2}\frac{z_c}{H} + \frac{1}{3} + \frac{1}{6}\frac{kz_c}{s_{uo}} + \frac{1}{8}\frac{kH}{s_{uo}}\right)$$

$$M_o = F_v\left(\frac{B}{2} - \frac{M}{F_v}\right) + F_H\cdot R\cos\alpha + \frac{1}{3}\gamma'\cdot L\cdot R^3\left[1 - \frac{3}{2}\cos\alpha + \frac{1}{2}\cos^3\alpha\right]$$

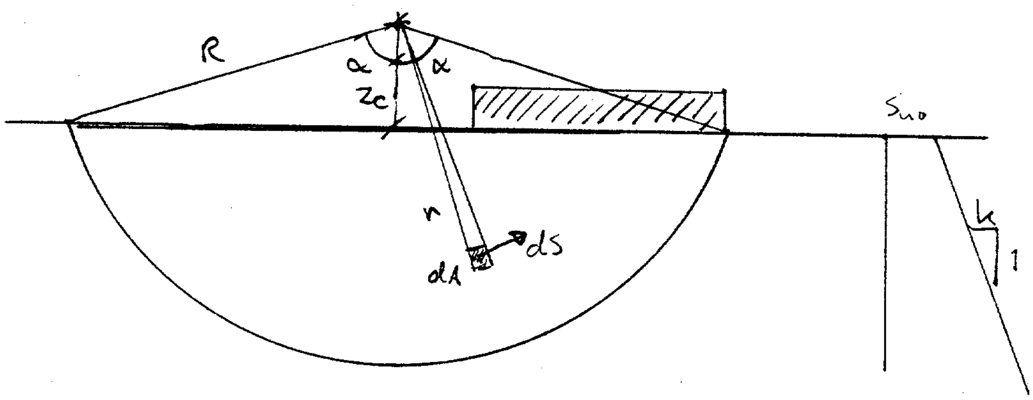
$$SF = M_{Res}/M_o$$

APPENDIX F:

Derivation of expressions for deepseated overturning failure method

December 1982/KVAL

a) Moment resistance of end surfaces (both ends)



$$dA = r\,d\alpha\,dr$$

$$dM = dS \cdot r = s_u r^2\,dr\,d\alpha$$

$$s_u(z) = s_{uo} + k \cdot z$$

$$z = r\cos\alpha - z_c$$

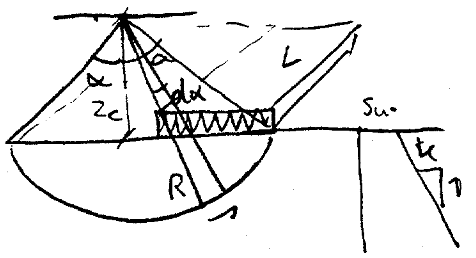
$$M = 2\int_{-\alpha}^{+\alpha}\int_{z_c/\cos\alpha}^{R} r^2\left[s_{uo} + k(r\cos\alpha - z_c)\right]dr\,d\alpha$$

$$M = s_{uo} R^3\left[\frac{4}{3}\alpha_1 + \frac{1}{2}\frac{kR}{s_{uo}} \cdot \sin\alpha - \frac{4}{3}\frac{kz_c}{s_{uo}}\alpha_1\right]$$

$$- s_u \cdot z_c^3\left[\frac{2}{3} - \frac{1}{6}\frac{kz_c}{s_{uo}}\right] \cdot \left[\frac{\sin\alpha}{\cos^2\alpha} + \ln\left|\frac{1+\mathrm{tg}\frac{\alpha}{2}}{1-\mathrm{tg}\frac{\alpha}{2}}\right|\right] \quad \text{for } \alpha < \frac{\pi}{2}$$

if $\alpha = \frac{\pi}{2}$ then $M = s_u \cdot R_0^3\left[\frac{2}{3}\pi + \frac{kR}{s_u}\right]$

b) Moment resistance of cylindersurface



$s_u(z) = s_{u0} + kz$

$z = R\cos\alpha - z_c$

$s_u(z) = s_{u0} + k(R\cos\alpha - z_c)$

$$dM = R^2 \cdot L \; s_u(z)\, d\alpha$$

$$M = \int_{-\alpha}^{+\alpha} R^2 \cdot L \cdot [s_{u0} + k(R\cos\alpha - z_c)]\, d\alpha$$

$$= 2LR^2 s_{u0}\left[\left(1 - \frac{kz_c}{s_u}\right)\alpha_1 + \frac{Rk}{s_u}\sin\alpha\right]$$

c) Safety against deepseated bearing capacity failure

$$M_{Res} = s_{u0} R^3 \left[\frac{4}{3} \alpha_1 + \frac{kR}{s_{u0}} \cdot \sin\alpha - \frac{4}{3} \frac{k z_c}{s_{u0}} \alpha_1 \right]$$
$$- s_u \cdot z_c^3 \left[\frac{2}{3} - \frac{1}{6} \frac{k z_c}{s_{u0}} \right] \left[\frac{\sin\alpha}{\cos^2\alpha} + \ln \left| \frac{1 + \tg\frac{\alpha}{2}}{1 - \tg\frac{\alpha}{2}} \right| \right]$$
$$+ 2 L R^2 s_{u0} \left[\left(1 - \frac{k z_c}{s_u}\right) \alpha_1 + \frac{R \cdot k}{s_u} \sin\alpha \right]$$

$$M_0 = M_{wavepr} + M_{wind+} + M_{wave+current} + F_v \cdot x - F_H \cdot z_c$$

$$FS_D = M_{Res} / M_0$$

APPENDIX G:

Description of program and users manual for program

SAIL

January 21,1983/LKri

Revised August-September 1983/LKri

APPENDIX G

G1

BRIEF DESCRIPTION OF THE PROGRAM SYSTEM S A I L

1 A): SIMPLIFIED FLOW DIAGRAM

1 B): BLOCK DIAGRAM

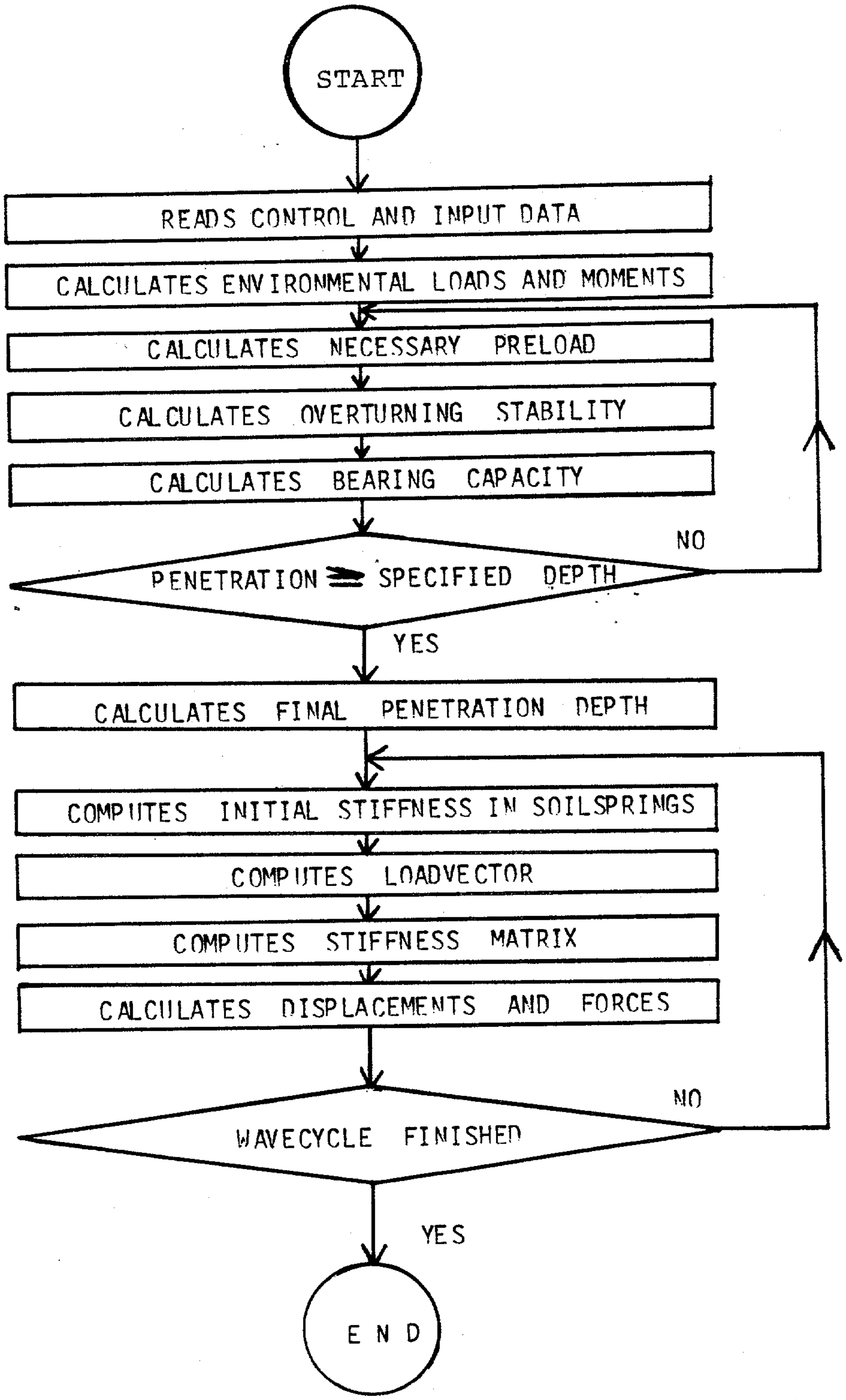
1 C): BRIEF DESCRIPTION OF ROUTINES

1 D): COPY OF OUTPUT - TEST. EXAMPLE

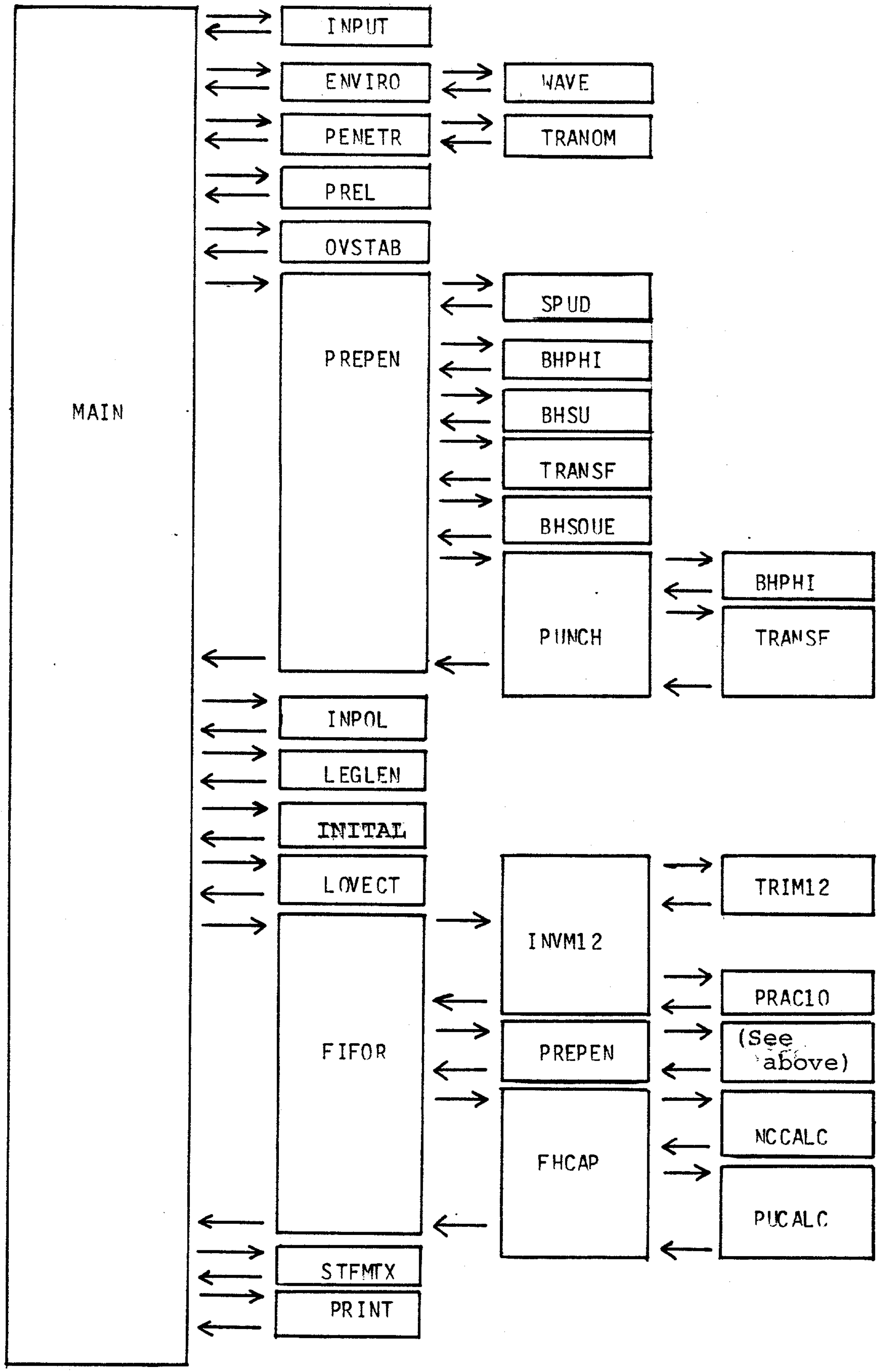
DATO: 21.1.83

Lars A. Kristiansen

SIMPLIFIED FLOW DIAGRAM FOR PROGRAM S A I L:



VIEW OF ROUTINES IN JACK-UP PROGRAM CALLED SAIL:



DESCRIPTION OF ROUTINES:

PART 1: STARTS PROGRAM AND READS NECESSARY DATA

MAIN: Administers the calls to several subroutines to compute penetration and stability of jack-ups supported by independent legs.

INPUT: Reads input data and print them with supplementing comments.

PART 2: COMPUTES LOADS FROM ENVIRONMENT

ENVIRO: Calulates total horizontal force and overturning moment due to wave, current and wind.

WAVE: Calculates wave forces on specified leg for specified timestep.

PART 3: PRELOADING AND PENETRATION PART

PENETR: Computes penetration step for further calculation.

TRANOM: Transforms overturning moments from seabed to penetration depth.

PREL: Computes maximum vertical load on one leg and find necessary preload for actual penetration step.

OVSTAB: Computes and check of overturning stability for actual penetration step.

PREPEN: Administers and compares various bearing capacity calculations.

SPUD: Computes exposed foundationarea for actual spudcan, soil and penetration step.

BHSU:	Computes bearing capacity by Brinch Hansen's formula for cohesive soils.
BPHI:	Computes bearing capacity by Brinch Hansen's formula for soils with internal friction.
TRANSF:	Transforms foundation area to fit Brinch Hansen's formula.
BHSOUE:	Computes bearing capacity for a soft soil layer overlaying a harder one. The so-called "squeezing" - effect.
PUNCH:	Comptes bearing capacity assuming a "punch-through" failure, when a harder layer is overlaying a softer one.
INPOL:	Computes final pentration depth.
LEGLEN:	Computes and checks necessary leglength.

PART 4: OPERATION PART

INITAL:	Computes initial stiffness in soilsprings.
LOVECT:	Computes loadvector from environmental loads.
STFMTX:	Computes stiffness matrix for soil/structure system.
FIFOR:	Administers calculation of final forces and displacements.

INVM12: Inverts the stiffness matrix.

TRIM12: Decomposes the stiffness matrix into a lower triangular matrix L and an upper matrix U. Stiffness matrix equals L * U by method of Crout.

PREPEN: Described previously.

FHCAP: Computes lateralcapacity.

NCCALC: Computes value for bearing capacity factor N_c, cohesive soil.

PUCALC: Computes value for earth pressure factor for soils with internal friction.

PRINT: Prints results.

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FOUNDATION STABILITY OF JACK-UP PLATFORMS

PART PROJECT NO. 2

ANALYSIS METHODS

User's Manual for Program

SAIL

Stability Analysis of Jack-ups supported by Independent Legs.

INPUT DATA PROCEDURE:

1. HEADING CARDS FORMAT(20A4)

Columns	1-80	TEXTI1	Two cards for identification
Columns	1-80	TEXTI2	of problem to be analysed.

2. CONTROL DATA CARD FORMAT (5I10)

Columns	1-10	IOP1	If IOP1=0 Calculate environmental forces only. If IOP1=1 Calculate environmental forces and execute rest of program.
	11-20	IOP2	If IOP2=0 Structure is exposed to waves propagating in positive X-direction. If IOP2=1 Structure is exposed to waves propagating in negative X-direction.
	21-30	IOP3	If IOP3=0 No print of input data. If IOP3=1 Print input data.
	31-40	IOP4	If IOP4=0 No print of wave forces. If IOP4=1 Print wave forces.
	41-50	IOP5	If IOP5=0 No print of penetration. If IOP5=1 Print penetration data.
	51-60	IOP6	If IOP6=0 No print of incremental values within each load step; only a tabulated overview of total forces, total displacements and safety against bearing failure and sliding is printed. If IOP6=1 Print of incremental values within each load step and a tabulated overview of total forces, total displacements and safety against bearing failure and sliding.

3. CRITICAL WAVE CARD FORMAT (F10.0)

This card is used only if IOP=1

Columns	1-10	TSET	Chosen wave period for stability analysis of foundation.

4. STRUCTURE GEOMETRY CARD NO. 1 FORMAT (3F10.0,I10,4E10.3)

Columns	1-10	BL	Breadth between legs.
	11-20	DW	Depth of water.
	21-30	AG	Air gap.
	31-40	NL	Number of legs.
	41-50	WG	Weight of structure (including legs and footing) and fixed equipment, but not including preload.
	51-60	GL	Difference between real and theoretical center of gravity. Real center on right hand side = +.
	61-70	DI	Average effective diameter for inertia force calculation.
	71-80	DD	Average effective diameter for drag force calculation.

5. STRUCTURE GEOMETRY CARD NO. 2 FORMAT (6E10.3)

Columns	1-10	AW	Hull area exposed to wind, do not include derrick and deck modules.
	11-20	PRECAP	Preloading capacity.
	21-30	SPEPRE	Specified preload. (If specified eq. 0, default value is precap)
	31-40	DISPL	Total available leg length.
	41-50	WGLEG	Leg weight for one leg.

51-60	ZPDEF	Depth at which bearing capacityis to bee calculated.

6. BARGE STIFFNESS CARD FORMAT (5E10.3)

Columns	1-10	EIB	Bending stiffness of barge.
	11-20	EAB	Axial stiffness of barge.
	21-30	GAB	Shear stiffness of barge.
	31-40	E	E-modulus for steel.
	41-50	PSTEEL	Poisson's ratio for steel.

7. LEG STIFFNESS CARD FORMAT (4E10.3)

One card should be included for each leg, i.e. NL cards totally. I indicates leg number.

Columns	1-10	EI(I)	Bending stiffness of leg.
	11-20	EA(I)	Axial stiffness of leg.
	21-30	GA(I)	Shear stiffness of leg.

8. FOOTING GEOMETRY CARD FORMAT (7F10.0)

Columns	1-10	DTIP	Tip diameter of spudcan.
	11-20	DTOT	Total diameter of spudcan.
	21-30	HTOP	Top height of spudcan.
	31-40	HSID	Side height of spudcan.
	41-50	HCON	Cone height of spudcan.
	51-60	HTIP	Tip height of spudcan.
	61-70	HTOT	Total height of spudcan.

9. ENVIRONMENTAL CARD NO. 1 — FORMAT (3F10.0)

Columns	1-10	VWI	Wind velocity.
	11-20	DARMW	Lever arm wind, measured from bottom of barge and upwards. Recommended value 12 meters.
	21-30	G	Acceleration of gravity.

10. ENVIRONMENTAL CARD NO. 2 — FORMAT (7F10.0)

Columns	1-10	HWM	Maximum wave height.
	11-20	TMAX	Maximum wave period.
	21-30	TMIN	Minimum wave period.
	31-40	TSTEP	Period stepping interval; investigated period increases with TSTEP from TMIN to TMAX.
	41-50	GAMW	Unit weight of seawater.
	51-60	CI	Inertia coefficient.
	61-70	CD	Drag coefficient.

TMAX, TMIN and TSTEP is only used if IOP1 = 0.

11. CURRENT CARD — FORMAT (2F10.0)

Columns	1-10	WH1	Resulting velocity at still water level (SWL) of current not generated at the surface or in the upper layers of water (e.g. tidal current).
	11-20	WH2	Resulting velocity at SWL of current generated at the surface or in the upper layers of water (e.g. wind generated current).

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Program assumes exponential (1/7) distribution of current created by WH1 between SWL and sea bed.

Program assumes linear distribution of current created by WH2, between 0 at sea bed and WH2 at SWL.

12. VARIABLE LOAD CARD — FORMAT (E10.3,F10.0)

Columns	1-10	WGV	Weight of variable load.
	11-20	VL	Lever arm variable load = Distance from theoretical center of gravity for structure to center of gravity for WGV. VL = + if the latter is on the right hand side of theoretical gravity center of structure.

13. SOIL PROFILE DATA CARD — FORMAT (I10,7F10.0)

Columns	1-10	NPRO	Soil profile type. NPRO=1; Soft clay with linearly increasing shear strength overlaying a stiff clay or sand layer. NPRO=2; Stiff clay crust overlaying a soft clay. NPRO=3; Sand layer overlaying a soft clay.

The terms soft and stiff clay should bee considered as relative description, i.e. the soft clay may very well have high su-values. The 'soft' clay must bee spesified with increasing shear strength with depth.

11-20	SU0	Undrained shear strength of soft clay layer at top of layer.
21-30	RK	Shear strength gradient for 'soft' clay, i.e. increase in undrained shear strength per vertical length unit below top of layer.
31-40	SUH	For clay: NPRO= 1 or 2, SUH= Undrained shear strength of 'hard' clay layer. For sand: NPRO= 1 or 3, SUH=Cohesion of sand layer.
41-50	PHI	Internal friction angle of sand layer.
51-60	TH	Thickness of top layer.
61-70	RR	Roughness ratio between steel and soil.
71-80	ALFA	Pressure influence angle - angel for load distribution with depth. Recommended value = 26,6 degrees.

14. SOIL STIFFNESS CARD FORMAT (4E10.3)

Columns	1-10	POIRA1	Poisson's ratio for upper layer.
	11- 20	POIRA2	Poisson's ratio for deep layer.
	21-30	GMOD1	Shear modulus for upper layer.
	31-40	GMOD2	Shear modulus for deep layer.
	41-50	GAMMSO	Submerged unit weight of soil.

TEST EXAMPLE: EXTRACT OF OUTPUT

```
        PROGRAM "SAIL"
VERSION 8
SEPTEMBER 1983
PROGRAMMED BY L. A. KRISTIANSEN, DET NORSKE VERITAS


*************************************************************************
"PROGRAM SAIL-Stability Analysis for jackups with Independent Legs"
TESTRUN to show extract of output for users manual.
*************************************************************************


INPUT DATA PRINTOUT:
********************

CONTROL DATA
************
OPTION NR.1,IOP1                                   =          1
(IOP1=0, Calculates enviromental forces only.
 IOP1=1, Calculates enviromental forces and
 rest of program.)

OPTION NR.2,IOP2                                   =          1
(IOP2=0, Structure is exposed to waves moving
 in positive X-direction.
 IOP2=1, Structure is exposed to waves moving
 in negative X-direction.)

OPTION NR.3,IOP3                                   =          1
(IOP3=0, No print of input data.
 IOP3=1, Prints input data.)

OPTION NR.4,IOP4                                   =          1
(IOP4=0, No print of wave forces.
 IOP4=1, Prints wave forces.)

OPTION NR.5,IOP5                                   =          1
(IOP5=0, No print of penetration.
 IOP5=1, Prints penetration data.)

OPTION NR.6,IOP6                                   =          1
(IOP6=0, Tabulated overview of total forces
 IOP6=1, Prints each loadstep plus overview.)


CHOSEN CRITICAL WAVE PERIOD                        = 0.160E+02


STRUCTURE GEOMETRY
******************
BREADTH BETWEEN LEGS, BL                           = 0.470E+02
DEPTH OF WATER, DW                                 = 0.100E+03
AIR GAP, AG                                        = 0.200E+02
NUMBER OF LEGS, NL                                 =         3
TOTAL STRUCTURE WEIGHT, WG                         = 0.120E+06
GRAVITY LEVERARM, GL                               = 0.000E+00
EFFECTIVE DIAMETER INERTIA, DI                     = 0.200E+01
EFFECTIVE DIAMETER DRAG, DD                        = 0.500E+01
WIND AREA, AW                                      = 0.530E+03
PRELOADING CAPACITY, PRECAP                        = 0.600E+05
SPECEFIED PRELOAD, SPEPRE                          = 0.000E+00
TOTAL AVAILABLE LEGLENGTH, DISPL                   = 0.140E+03
LEG WEIGHT, WGLEG                                  = 0.980E+04
CALC. BEAR.CAP. DOWN TO, ZPDEF                     = 0.250E+02
```

BARGE STIFFNESS:
****

AXIAL STIFFNESS, EAB	= 0.220E+10
SHEAR STIFFNESS, GAB	= 0.200E+08
E-MODULUS FOR STEEL, E	= 0.210E+09
POISSONS RATIO, PSTEEL	= 0.300E+00

LEG STIFFNESS:
****

LEG NR. 1

BENDING STIFFNESS, EI	= 0.200E+10
AXIAL STIFNESS, EA	= 0.220E+09
SHEAR STIFFNESS, GA	= 0.200E+07

LEG NR. 2

BENDING STIFFNESS, EI	= 0.200E+10
AXIAL STIFNESS, EA	= 0.220E+09
SHEAR STIFFNESS, GA	= 0.200E+07

LEG NR. 3

BENDING STIFFNESS, EI	= 0.200E+10
AXIAL STIFNESS, EA	= 0.220E+09
SHEAR STIFFNESS, GA	= 0.200E+07

FOOTING GEOMETRI:
****

TIP DIAMETER, DTIP	= 0.175E+01
TOTAL DIAMETER, DTOT	= 0.150E+02
TOP HEIGHT, HTOP	= 0.750E+00
SIDE HEIGHT, HSID	= 0.200E+01
CONE HEIGHT, HCON	= 0.750E+00
TIP HEIGHT, HTIP	= 0.100E+01
TOTAL HEIGHT, HTOT	= 0.450E+01

ENVIROMENTAL DATA:
****

WIND VELOCITY, VW	=-0.450E+02
LEVERARM WIND, DARMW	= 0.120E+02
GRAVITY CONSTANT, G	= 0.981E+01

WAVE DATA:
****

MAXIMUM WAVE HEIGHT, HWM	= 0.230E+02
MAXIMUM WAVE PERIOD, TMAX	= 0.200E+02
MINIMUM WAVE PERIOD, TMIN	= 0.110E+02
PERIOD STEP	= 0.100E+01
UNIT WEIGHT OF SEA WATER,GAMW	= 0.101E+02
INERTIA COEFFICIENT, CI	= 0.200E+01
DRAG COEFFICIENT, CD	= 0.700E+00

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WARNING! CALCULATED MAXIMUM POSSIBLE WAVEHEIGHT EXCEED WAVEHEIGHT FROM INPUT DATA.

"PROGRAM SAIL-Stability Analysis for jackups with Independent Legs"

WATER DEPTH	= 0.100E+03
WAVE HEIGHT	= 0.230E+02
WAVE STEEPNESS	= 0.616E-01
WAVE PERIOD	= 0.160E+02
WAVE LENGTH	= 0.373E+03

SUMMARY OF WAVE FORCES

TIME STEP	HORIZONTAL FORCES					OVERTURNING MOMENTS				
NO.	LEG 1	LEG 2	LEG 3	LEG 4	TOTAL	LEG 1	LEG 2	LEG 3	LEG 4	TOTAL
1	-0.296E+04	-0.181E+04	-0.181E+04	0.000E+00	-0.658E+04	-0.233E+06	-0.147E+06	-0.147E+06	0.000E+00	-0.527E+06
2	-0.289E+04	-0.807E+03	-0.807E+03	0.000E+00	-0.451E+04	-0.231E+06	-0.690E+05	-0.690E+05	0.000E+00	-0.369E+06
3	-0.228E+04	0.459E+02	0.459E+02	0.000E+00	-0.219E+04	-0.184E+06	-0.492E+04	-0.492E+04	0.000E+00	-0.194E+06
4	-0.133E+04	0.538E+03	0.538E+03	0.000E+00	-0.254E+03	-0.109E+06	0.301E+05	0.301E+05	0.000E+00	-0.491E+05
5	-0.362E+03	0.635E+03	0.635E+03	0.000E+00	0.909E+03	-0.352E+05	0.363E+05	0.363E+05	0.000E+00	0.374E+05
6	0.334E+03	0.633E+03	0.633E+03	0.000E+00	0.160E+04	0.159E+05	0.346E+05	0.346E+05	0.000E+00	0.851E+05
7	0.628E+03	0.709E+03	0.709E+03	0.000E+00	0.205E+04	0.362E+05	0.382E+05	0.382E+05	0.000E+00	0.113E+06
8	0.617E+03	0.755E+03	0.755E+03	0.000E+00	0.213E+04	0.343E+05	0.405E+05	0.405E+05	0.000E+00	0.115E+06
9	0.667E+03	0.695E+03	0.695E+03	0.000E+00	0.206E+04	0.362E+05	0.374E+05	0.374E+05	0.000E+00	0.111E+06
10	0.741E+03	0.498E+03	0.498E+03	0.000E+00	0.174E+04	0.398E+05	0.273E+05	0.273E+05	0.000E+00	0.945E+05
11	0.742E+03	0.179E+03	0.179E+03	0.000E+00	0.110E+04	0.398E+05	0.106E+05	0.106E+05	0.000E+00	0.610E+05
12	0.617E+03	-0.198E+03	-0.198E+03	0.000E+00	0.221E+03	0.335E+05	-0.103E+05	-0.103E+05	0.000E+00	0.129E+05
13	0.357E+03	-0.521E+03	-0.521E+03	0.000E+00	-0.684E+03	0.200E+05	-0.293E+05	-0.293E+05	0.000E+00	-0.386E+05
14	-0.800E+00	-0.680E+03	-0.680E+03	0.000E+00	-0.136E+04	0.808E+03	-0.400E+05	-0.400E+05	0.000E+00	-0.793E+05
15	-0.369E+03	-0.973E+03	-0.973E+03	0.000E+00	-0.232E+04	-0.202E+05	-0.630E+05	-0.630E+05	0.000E+00	-0.146E+06
16	-0.612E+03	-0.152E+04	-0.152E+04	0.000E+00	-0.366E+04	-0.351E+05	-0.107E+06	-0.107E+06	0.000E+00	-0.250E+06
17	-0.788E+03	-0.222E+04	-0.222E+04	0.000E+00	-0.522E+04	-0.485E+05	-0.166E+06	-0.166E+06	0.000E+00	-0.380E+06
18	-0.121E+04	-0.280E+04	-0.280E+04	0.000E+00	-0.682E+04	-0.818E+05	-0.218E+06	-0.218E+06	0.000E+00	-0.517E+06
19	-0.185E+04	-0.300E+04	-0.300E+04	0.000E+00	-0.785E+04	-0.135E+06	-0.238E+06	-0.238E+06	0.000E+00	-0.611E+06
20	-0.253E+04	-0.264E+04	-0.264E+04	0.000E+00	-0.781E+04	-0.193E+06	-0.212E+06	-0.212E+06	0.000E+00	-0.617E+06

MAX POSITIV HORIZONTAL FORCE= 0.213E+04 AT STEP NO. 8
WAVE PERIOD FOR MAX POSITIV HORIZONTAL FORCE =16.0
CORRESPONDING OVERTURNING MOMENT= 0.115E+06

MAX NEGATIVE HORIZONTAL FORCE= -0.785E+04 AT STEP NO. 19
WAVE PERIOD FOR MAX NEGATIVE HORIZONTAL FORCE =16.0
CORRESPONDING OVERTURNING MOMENT= -0.611E+06

MAX POSITIVE OVERTURNING MOMENT= 0.115E+06 AT STEP NO. 8
WAVE PERIOD FOR MAX POSITIVE OVERTURNING MOMENT =16.0
CORRESPONDING HORIZONTAL FORCE= 0.213E+04

MAX NEGATIVE OVERTURNING MOMENT= -0.617E+06 AT STEP NO. 20
WAVE PERIOD FOR MAX NEGATIVE OVERTURNING MOMENT =16.0
CORRESPONDING HORIZONTAL FORCE= -0.781E+04

WIND FORCE:

MAX HORIZONTAL WINDFORCE	=-0.322E+04
LEVERARM	= 0.132E+03
CORRESPONDING OVERTURNING MOMENT RELATED TO MUDLINE	=-0.425E+06

FINAL PENETRATION AFTER PRELOADING = 0.124E+02

PRELOADWEIGHT IS ASSUMED EQUAL TO:
SPECIFIED PRELOAD(IF GIVEN)/

STRUCTUREWEIGHT + VARIABEL LOAD
AS GIVEN IN INPUT DATA.)

VERTICAL LOAD ON ONE LEG DURING PRELOADING = 0.680E+05
BEARING CAPASITY OF SOIL FOR ONE LEG = 0.680E+05

SAFETY AGAINST OVERTURNING,INCLUDE VARIABLE
AS SPECIFIED IN INPUT DATA, FOSAFE = 0.285E+01
NESSARY LEGLENGTH = 0.132E+03

SOIL REACTION FORCES ON LEFT SPUDCAN:

LOADCASE	FHTOT	FVTOT	MTOT	DHTOT	DVTOT	TOTROT	SFS	SFB
1	0.550E+03	0.506E+05	0.363E+05	-0.134E-02	-0.493E-01	-0.160E-02	0.157E+02	0.134E+01
2	0.107E+04	0.538E+05	0.598E+05	-0.280E-02	-0.131E+00	-0.468E-02	0.806E+01	0.126E+01
3	0.810E+03	0.536E+05	0.592E+05	-0.196E-02	-0.121E+00	-0.412E-02	0.107E+02	0.127E+01
4	0.108E+04	0.546E+05	0.610E+05	-0.278E-02	-0.158E+00	-0.581E-02	0.798E+01	0.125E+01
5	0.135E+04	0.554E+05	0.618E+05	-0.362E-02	-0.194E+00	-0.733E-02	0.642E+01	0.123E+01
6	0.168E+04	0.568E+05	0.623E+05	-0.477E-02	-0.262E+00	-0.997E-02	0.514E+01	0.120E+01
7	0.208E+04	0.590E+05	0.624E+05	-0.625E-02	-0.396E+00	-0.146E-01	0.417E+01	0.115E+01
8	0.251E+04	0.618E+05	0.626E+05	-0.808E-02	-0.660E+00	-0.223E-01	0.345E+01	0.110E+01
9	0.300E+04	0.648E+05	0.629E+05	-0.105E-01	-0.125E+01	-0.371E-01	0.288E+01	0.105E+01
10	0.336E+04	0.666E+05	0.644E+05	-0.125E-01	-0.425E+01	-0.102E+00	0.257E+01	0.101E+01
11	0.347E+04	0.667E+05	0.645E+05	-0.132E-01	-0.446E+01	-0.107E+00	0.250E+01	0.100E+01
12	0.318E+04	0.646E+05	0.628E+05	-0.113E-01	-0.102E+01	-0.322E-01	0.272E+01	0.104E+01
13	0.258E+04	0.610E+05	0.598E+05	-0.757E-02	0.498E+01	0.981E-01	0.336E+01	0.111E+01
14	0.182E+04	0.568E+05	0.595E+05	-0.383E-02	0.563E+01	0.115E+00	0.475E+01	0.120E+01
15	0.111E+04	0.533E+05	0.593E+05	-0.107E-02	0.585E+01	0.123E+00	0.776E+01	0.128E+01
16	0.582E+03	0.512E+05	0.555E+05	0.647E-03	0.592E+01	0.126E+00	0.149E+02	0.133E+01
17	0.225E+03	0.502E+05	0.513E+05	0.165E-02	0.595E+01	0.128E+00	0.385E+02	0.136E+01
18	0.228E+02	0.495E+05	0.480E+05	0.217E-02	0.596E+01	0.129E+00	0.380E+03	0.137E+01
19	0.326E+00	0.495E+05	0.476E+05	0.223E-02	0.596E+01	0.129E+00	0.265E+05	0.137E+01
20	-0.318E+01	0.496E+05	0.481E+05	0.223E-02	0.596E+01	0.129E+00	0.272E+04	0.137E+01
21	0.535E+02	0.499E+05	0.503E+05	0.210E-02	0.595E+01	0.128E+00	0.162E+03	0.136E+01
22	0.222E+03	0.507E+05	0.545E+05	0.168E-02	0.593E+01	0.127E+00	0.390E+02	0.134E+01

SOIL REACTION FORCES ON RIGHT SPUDCAN:

LOADCASE	FHTOT	FVTOT	MTOT	DHTOT	DVTOT	TOTROT	SFS	SFB
1	0.106E+04	0.934E+05	0.720E+05	-0.130E-02	0.246E-01	-0.158E-02	0.163E+02	0.146E+01
2	0.215E+04	0.902E+05	0.126E+06	-0.280E-02	0.519E-01	-0.458E-02	0.805E+01	0.151E+01
3	0.219E+04	0.904E+05	0.124E+06	-0.287E-02	0.500E-01	-0.421E-02	0.790E+01	0.150E+01
4	0.282E+04	0.894E+05	0.135E+06	-0.388E-02	0.576E-01	-0.583E-02	0.613E+01	0.152E+01
5	0.323E+04	0.886E+05	0.142E+06	-0.460E-02	0.635E-01	-0.725E-02	0.535E+01	0.154E+01
6	0.385E+04	0.872E+05	0.153E+06	-0.574E-02	0.730E-01	-0.975E-02	0.449E+01	0.156E+01
7	0.481E+04	0.850E+05	0.167E+06	-0.767E-02	0.871E-01	-0.143E-01	0.360E+01	0.160E+01
8	0.594E+04	0.822E+05	0.183E+06	-0.103E-01	0.103E+00	-0.219E-01	0.291E+01	0.165E+01
9	0.704E+04	0.792E+05	0.198E+06	-0.134E-01	0.119E+00	-0.365E-01	0.246E+01	0.169E+01
10	0.770E+04	0.774E+05	0.215E+06	-0.157E-01	0.128E+00	-0.102E+00	0.225E+01	0.171E+01
11	0.756E+04	0.773E+05	0.215E+06	-0.152E-01	0.129E+00	-0.106E+00	0.229E+01	0.172E+01
12	0.662E+04	0.794E+05	0.212E+06	-0.116E-01	0.119E+00	-0.311E-01	0.261E+01	0.169E+01
13	0.516E+04	0.830E+05	0.206E+06	-0.686E-02	0.101E+00	0.992E-01	0.336E+01	0.164E+01
14	0.359E+04	0.872E+05	0.205E+06	-0.299E-02	0.780E-01	0.117E+00	0.482E+01	0.156E+01
15	0.236E+04	0.907E+05	0.205E+06	-0.590E-03	0.554E-01	0.124E+00	0.733E+01	0.150E+01
16	0.173E+04	0.928E+05	0.205E+06	0.442E-03	0.404E-01	0.127E+00	0.100E+02	0.147E+01
17	0.140E+04	0.938E+05	0.205E+06	0.946E-03	0.316E-01	0.129E+00	0.124E+02	0.145E+01
18	0.115E+04	0.945E+05	0.205E+06	0.130E-02	0.263E-01	0.130E+00	0.150E+02	0.144E+01
19	0.109E+04	0.945E+05	0.205E+06	0.138E-02	0.257E-01	0.130E+00	0.158E+02	0.144E+01
20	0.117E+04	0.944E+05	0.205E+06	0.128E-02	0.266E-01	0.129E+00	0.148E+02	0.144E+01
21	0.143E+04	0.941E+05	0.205E+06	0.907E-03	0.299E-01	0.129E+00	0.121E+02	0.145E+01
22	0.190E+04	0.933E+05	0.205E+06	0.228E-03	0.365E-01	0.128E+00	0.911E+01	0.146E+01

LOADCASE NR. 6 STRUCTURE WEIGHT,FULL WINDLOAD AND WAVELOAD (STEP NUMBER M=15)

LOADS ON LEFT SPUDCAN:	STEP VALUES	TOTAL VALUES
HORIZONTAL FORCE	FH = 0.335E+03	TOTFH = 0.168E+04
VERTICAL FORCE	FV = 0.140E+04	TOTFV = 0.568E+05
MOMENT FORCE	M = 0.512E+03	TOTMOM = 0.623E+05
DISPLACEMENT ON LEFT SPUDCAN:		
HORIZONTAL DISPLACEMENT	DH =-0.115E-02	TOTDH =-0.477E-02
VERTICAL DISPLACEMENT	DV =-0.676E-01	TOTDV =-0.262E+00
ROTATION	R =-0.263E-02	TOTR =-0.997E-02
SAFETY AGAINST SLIDING		SFS = 0.514E+01
SAFETY AGAINST VERTICAL FAILURE		SFB = 0.120E+01
TOP LEG MOMENT	TLM = 0.201E+05	TOTTLM = 0.146E+06

LOADS ON RIGHT SPUDCAN:	STEP VALUES	TOTAL VALUES
HORIZONTAL FORCE	FH = 0.619E+03	TOTFH = 0.385E+04
VERTICAL FORCE	FV =-0.140E+04	TOTFV = 0.872E+05
MOMENT FORCE	M = 0.108E+05	TOTMOM = 0.153E+06
DISPLACEMENT ON RIGHT SPUDCAN:		
HORIZONTAL DISPLACEMENT	DH =-0.114E-02	TOTDH =-0.574E-02
VERTICAL DISPLACEMENT	DV = 0.949E-02	TOTDV = 0.730E-01
ROTATION	R =-0.250E-02	TOTR =-0.975E-02
SAFETY AGAINST SLIDING		SFS = 0.449E+01
SAFETY AGAINST VERTICAL FAILURE		SFB = 0.156E+01
TOP LEG MOMENT	TLM = 0.457E+05	TOTTLM = 0.269E+06

APPENDIX H:

Description of program and users manual for program

SAM

Draft version printed January 24, 1982/KVAL
Revised September 1983/KVAL

FOUNDATION STABILITY OF JACK-UP PLATFORMS

PART PROJECT 2

ANALYSIS METHODS

DESCRIPTION OF PROGRAM

and

USERS MANUAL

for

SAM

Stability Analysis of Mats

Revised version, September 20, 1983

CONTENTS Page

1. INTRODUCTION

The computer program SAM (Stability Analysis of Mats) was written as a part of the work performed in Part Project 2 of the Joint Industry supported Project 'Foundation Stability of Jack-Up Platforms' which is carried out by Det norske Veritas on behalf of 14 participating companies.

The purpose of the program is to evaluate the safety against foundation failure of mat supported jack-up platforms for a broad variation of the basic parameters describing the environment, the platform and the soil conditions.

It has not been the intention of the investigators to develop a commercial program as this lies far beyond the scope of work and the budget of this research project. There will thus be several limitations and simplifications included with respect to boundary conditions, flexibility and efficiency. Nevertheless it is felt that the development of the program fulfils its purpose for this research project and that it also may be of value for designers, operators and oil companies in their daily work.

2. GENERAL DESCRIPTION OF PROGRAM

The computer program SAM consists of a main program for data input and control of the flow scheme. The main program calls a number of subroutines that calculates forces due to wave pressure on sea bed and mat, wave and current induced forces on the legs and evaluates the stability of the mat foundation.

Linear wave theory is used for calculation of water particle velocities and accelerations as well as wave pressure on the mat and the sea bed.

Current velocity is computed as the sum of tidal and wind induced current velocities with an exponential and linear distribution with depth below still water level (SWL) respectively.

The wave pressure on the mat is integrated over the surface of the mat to arrive at resulting forces and overturning moments. The effect of the presence of the mat on the wave pressure has not been included (i.e. no reflection or diffraction effects have been included)

The penetration of the mat into the sea bed is evaluated by means of bearing capacity theory.

Four different types of foundation failure have been included:

- sliding along a shallow failure surface directly under the mat with eventual mobilization of passive earth pressure and side friction on embedded parts of the mat structure.

- local bearing capacity failure under the most loaded beam of an A-shaped mat

- shallow bearing capacity failure under the most loaded edge of the mat as a result of combined horizontal and vertical forces

- deepseated overturning foundation failure due to wave pressure on sea bed combined with the overturning moment acting on the platform

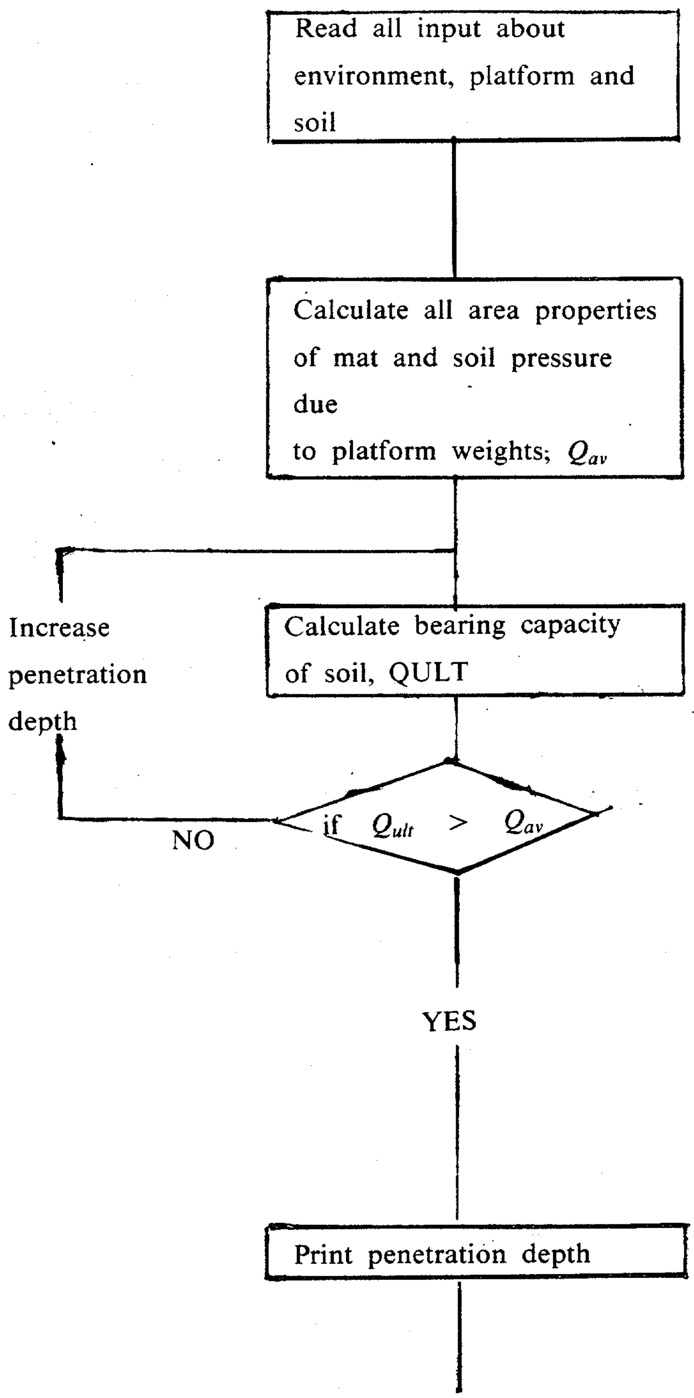
A range of different failure surfaces have been included for the two latter failure modes described above. The failure surfaces are varied automaticly by the program. The shape of these surfaces have been restricted to circles and straight lines or a combination of these two shapes.

Only clay has been considered in SAM. The foundation stability of mat supported jack-ups on sandy sea beds is assumed to be sufficient. Scour will probably be a more significant problem. Punch-through of stiff crusts is not considered to represent a problem due to the very low average contact pressure developing under a mat.

The soil profile considered corresponds to profile no. 1, i.e. a clay layer of thickness TH over a stronger layer. The shear strength of the top layer increases from s_{u0} at mudline with a gradient k with depth below mudline. This profile is considered to be representative for soil conditions where foundation stability could represent a problem for a mat-supported jack-up, however, the profile does not allow a general description of multiple layers and complex variations in strength with depth.

3. FLOW CHART

The flow chart below gives a brief description of the main operations included in the program.



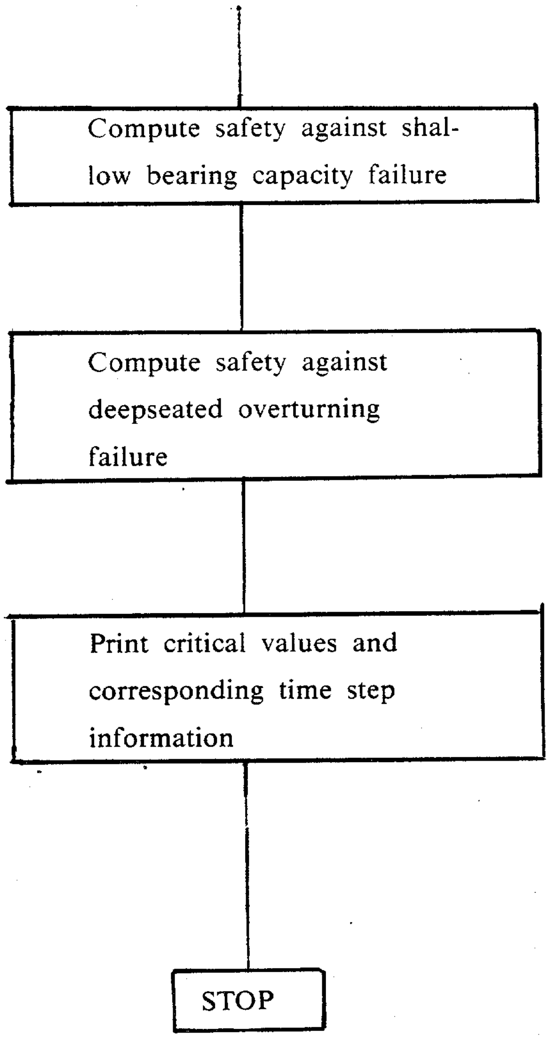
Compute wave forces on mat Print wave forces

Compute wave and current forces on legs and add wind forces

Set up load vectors for all wave periods and time steps investigated. Print total load vectors

Compute safety against sliding for all load cases

Compute safety against local bearing capacity failure under most loaded beam of an A-shaped mat


Compute safety against shal-
low bearing capacity failure
Compute safety against
deepseated overturning
failure
Print critical values and
corresponding time step
information
STOP

4. DESCRIPTION OF SUBROUTINES

The following list gives a brief outline of the different subroutines called by the main program

SUBROUTINE NAME	DESCRIPTION	DETAILS
MATPEN	Calculates all area properties of mat and average soil pressure Checks with bearing capacity from subroutine **MATBRH** and performs incremental stepping of penetration depth	
MATBRH	Calculates the bearing capacity of the soil	Based on Brinch-Hansen bearing capacity formula
WAVEPR	Calculates the wave pressure induced forces on the mat structure	Integrates pressure over total area and subtracts forces from hole and slot Forces and overturning moment are computed for waves coming in parallel to x-axis and y-axis
MATENV	Calculates wind forces on hull structure and combined wave and current forces on legs Loops on wave periods and steps through 21 timesteps	Based on linear wave theory. Checks steepness of wave and reduces wave height to corresponding max allowable steepness. Calls subroutine MATWAV for

	for each wave period	calculation of drag and inertia forces on the indi- vidual legs.
MATWAV	Calculates leg forces on individual legs from combined wave and current using linear wave theory and Morisons's equation	Leg is divided in 10 segments from current level of water surface down to sea bed. Forces for all segments added to give total force on single leg. Overturning moment relative to sea bed is computed as well.
TOTFOR	Adds all forces from weights, wind, wave and current as well as wave pressure induced forces on mat and sets up total load vector for mat for all wave periods and time steps (21 steps per period) in x- and in y-direction	
MATOTM	Computes safety against overturning about mat edges without concidering bearing capacity of soil.	
MATSLI	Computes safety against sliding along a shallow failure surface directly under the mat.	Includes passive earth pressure and side friction on embedded parts of mat
MATLOC	Computes safety against local bearing capacity failure under most stresses beam of an A-shaped mat	Part of area required to resist sliding is assumed to have a reduced bearing capacity while the rest

	subjected to combined vertical and horizontal forces and overturning moment.	of the area can mobilize full bearing capacity. Moment equlibrium about the least stressed edge is calculated.
SHBEAR	Computes safety against bearing capacity failure under edge of mat due to combined effect of horizontal and vertical forces	Simplified failure surface is varied to establish critical geometry and determine minimum value of safety
DEBEAR	Computes safety against deepseated bearing capacity failure due to wave pressure against mat and sea bed and overturning moment on legs and hull due to waves, current and wind.	Circular failure surface assumed. Center and radius is varied within reasonable limits to establish the critical surface.

5. INPUT DATA PROCEDURE

1.	HEADING CARDS			FORMAT (20A4)
	Columns	1-80	TEXT(I)	Two (2) cards for identification of problem to be analysed
2.	PHYSICAL CONSTANTS CARD			FORMAT (4F10.3)
	Columns	1-10	GAMW	Unit weight of sea water
		11-20	G	Gravity acceleration (9.81m/s**2)
		21-30	CI	Inertia coefficient
		31-40	CD	Drag coefficient
3.	WATER DEPTH AND AIR GAP			FORMAT (2F10.3)
	Columns	1-10	DW	Depth of water
		11-20	AG	Air gap
4.	WAVE DATA CARD			FORMAT (4F10.3)
	Columns	1-10	WH	Wave heigth
		11-20	TMIN	Minimum wave period to be analysed
		21-30	TMAX	Maximum wave period to be analysed
		31-40	TSTEP	Period stepping interval, i.e periods TMIN, TMIN+TSTEP, TMIN+2*TSTEP, and so on until TMAX will be analysed.
5.	CURRENT DATA CARD			FORMAT (2F10.3)
	Columns	1-10	VC1	Tidal current speed at SWL

11-20	VC2	Wind induced current speed at SWL
		Program assumes exponential (1/7) distribution of tidal induced current between SWL and sea bed.
		Program assumes linear distribution of wind induced current between VC2 at SWL and 0 at sea bed.

6. WIND DATA CARD FORMAT (4F10.3)

Column	1-10	VW	Wind speed
	11-20	WAREAX	Wind area in x-direction
	21-30	WAREAY	Wind area in y-direction
	31-40	WARM	Wind lever arm above air gap

7. MAT GEOMETRY CARD No. 1 FORMAT (6F10.3)

Column	1-10	TOTL	Total length of mat (y-direction)
	11-20	TOTW	Total width of mat (x-direction)
	21-30	SLOTL	Length of slot (y-direction)
	31-40	SLOTW	Width of slot (x-direction)
	41-50	HOLEL	Length of hole (y-direction)
	51-60	HOLEW	Width of hole (x-direction)

8. MAT GEOMETRY CARD No. 2 FORMAT (4F10.3)

Column	1-10	TMAT	Thickness of mat (z-direction)
	11-20	ALE	Distance from hole to edge in y-direction (see figure)
	21-30	BLE	Distance between end of slot and hole in y-direction (see figure)
	31-40	YLE	Distance between edge of mat and

x-axis (see figure)

NOTE! YLE defines position of x-axis relative to the mat as well as the rest of the platform i.e. legs, weights and their lever arms, etc.

The y-axis is always assumed to be in the middle of the mat, i.e. the mat is always assumed to be symmetric about the y-axis.

41-50	SKH	Height of skirt
51-60	SKTH	Thickness of skirt tip
61-70	ALFASK	Side friction ratio i.e. ratio between side friction and undrained shear strenght of soil

NOTE! It is assumed that the skirt is present along the total periphery of the mat, including the slot and hole.

9. LEG CARD NO. 1 FORMAT (I5)

Column	1-5	NL	Number of legs (maximum 4)

10. LEG CARD NO. 2 FORMAT (8F10.3)

Column	1-10	X(I)	X-coordinate of leg no. I
	11-20	Y(I)	Y-coordinate of leg no. I
	21-30	HLEG(I)	Total length of leg no. I
	31-40	DI(I)	Average effective diameter

		for inertia force calculation
41-50	DD(I)	Average effective diameter for drag force calculation
51-60	EI(I)	Bending stiffness of leg no. I
61-70	EA(I)	Axial stiffness of leg no. I
71-80	GA(I)	Shear stiffness of leg no. I

One card required for each leg (i.e. 3 or four cards)

11. PLATFORM WEIGHT CARD FORMAT (8F10.3)

Columns	1-10	GW	Weight of platform hull and fixed equipment
	11-20	GLX	Lever arm of GW with respect to x-axis
	21-30	GLY	Lever arm of GW with respect to y-axis
	31-40	GVAR	Variable load
	41-50	GVARLX	Lever arm of GVAR with repect to x-axis
	51-60	GVARLY	Lever arm of GVAR with repect to y-axis
	61-70	GLEG	Weight per leg submerged
	71-80	GMAT	Submerged weight of mat

12. SOIL DATA CARD NO. 1 FORMAT (I5)

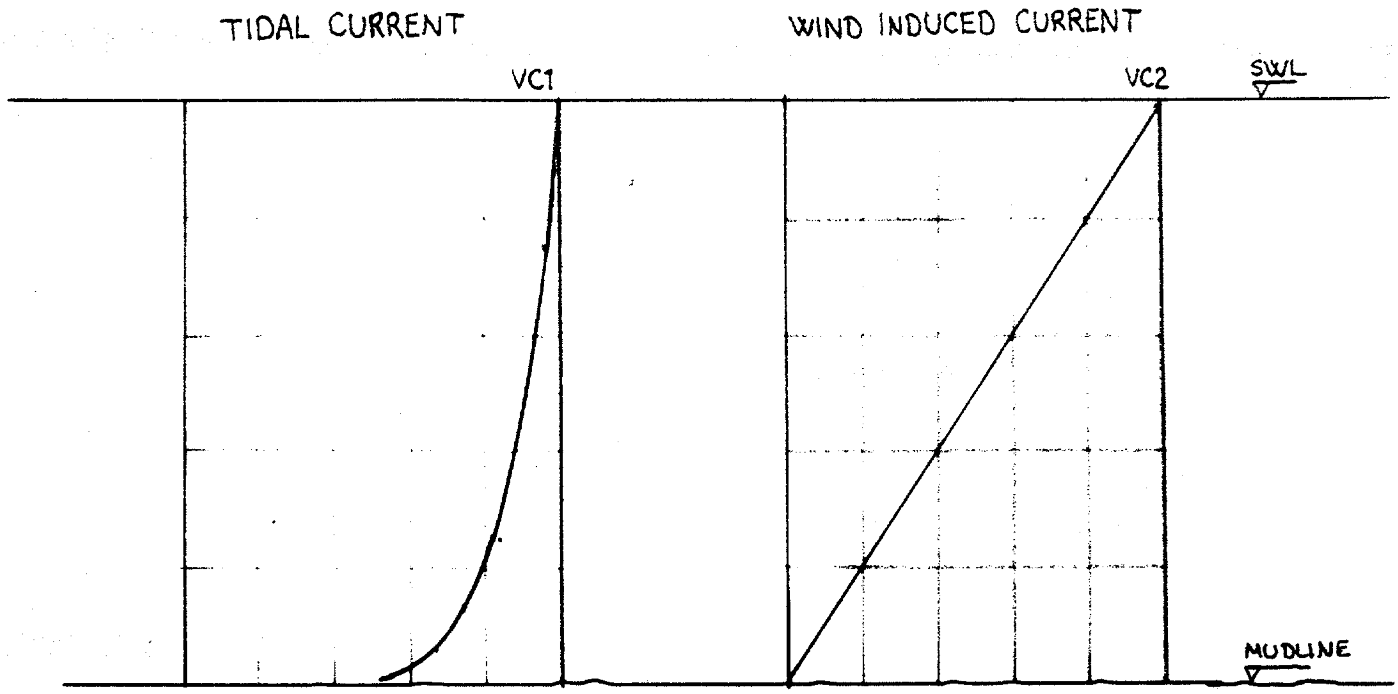
Column	1-5	NPRO	Soil profile type NPRO=1 : Soft clay with linearly increasing shear strength overlaying a stiff clay or sand layer NPRO=2 : Stiff clay crust overlaying

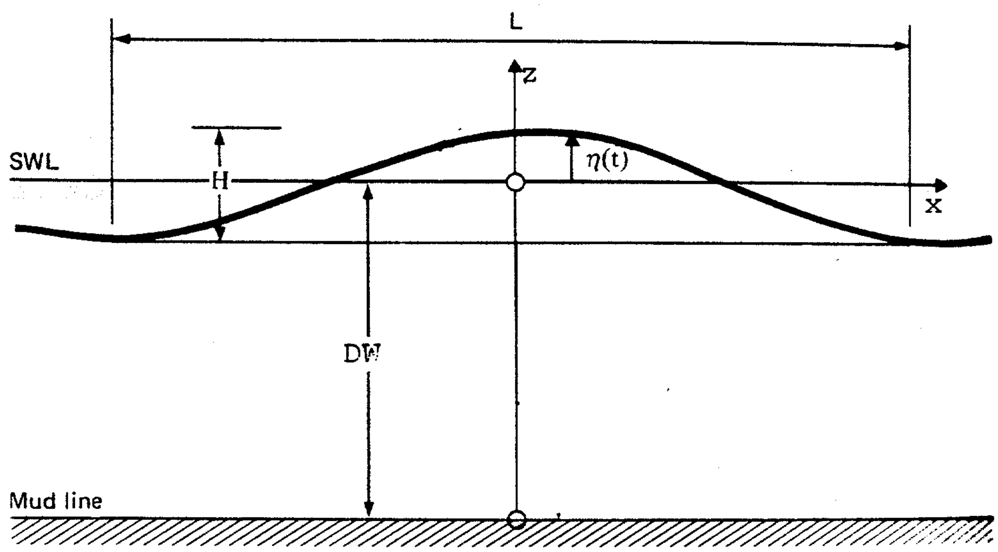
a soft clay

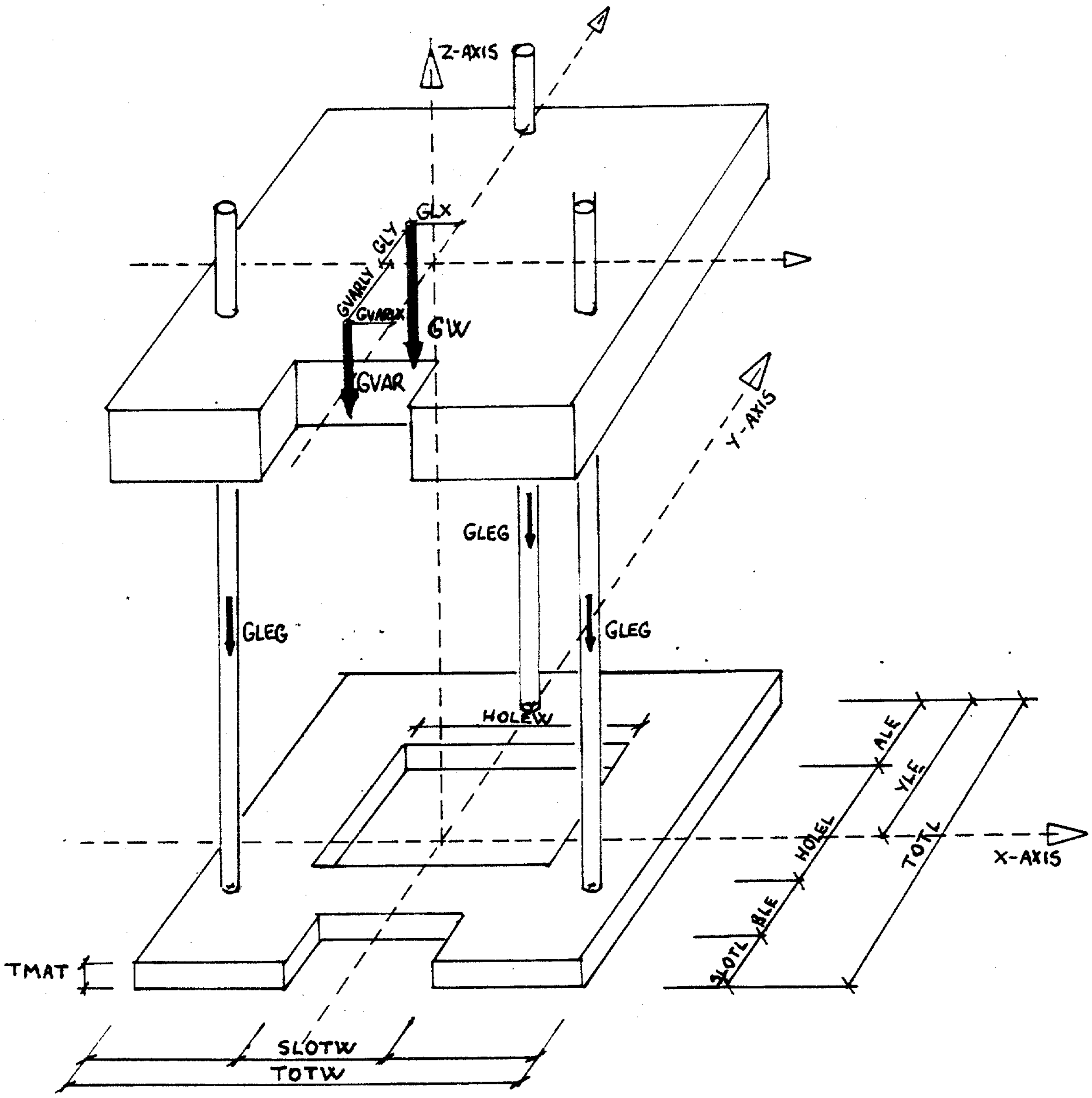
NPRO=3 : Sand layer overlaying a soft clay layer

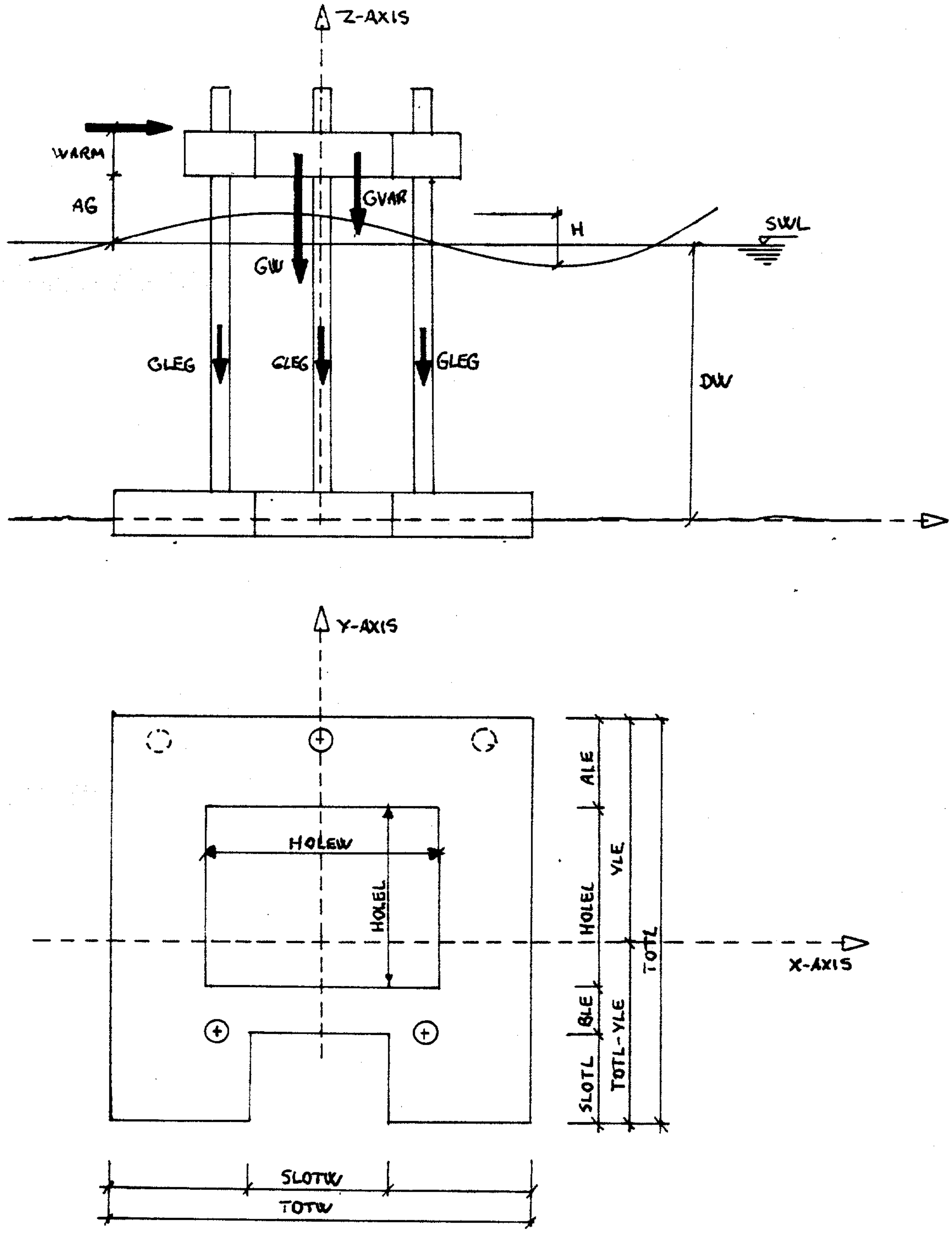
13. SOIL DATA CARD NO. 2 FORMAT (6F10.3)

Column	1-10	SUO	Undrained shear strength of soft clay layer at sea bed level
	11-20	RK	Shear strength gradient, i.e. increase in undrained shear strength per unit distance below sea bed, of soft clay layer
	21-30	SUH	Undrained shear strength of hard clay layer or cohesion of sand layer
	31-40	PHI	Friction angle of sand layer
	41-50	TH	Thickness of top layer
	51-60	RR	Roughness ratio between mat surface and soil
	61-70	GAMS	Unit submerged weight of soil


TIDAL CURRENT
WIND INDUCED CURRENT
VC1
VC2
SWL
MUDLINE


L
z
SWL
H
η(t)
x
DW
Mud line


Z-AXIS
GLX
GLY
GVARLY
GVARX
GW
GVAR
Y-AXIS
GLEG
GLEG
GLEG
HOLEW
ALE
YLE
HOLEL
TOTL
X-AXIS
BLE
SLOTL
TMAT
SLOTW
TOTW



Z-AXIS
WARM
AG
GVAR
GW
H
SWL
GLEG
GLEG
GLEG
DW
Y-AXIS
HOLEW
HOLEL
ALE
YLE
HOLEL
TOTL
X-AXIS
BLE
TOTL-YLE
SLOTL
SLOTW
TOTW

6. DESCRIPTION OF OUTPUT

The program gives a complete output of the input parameters before proceeding with the calculation of the penetration of the mat.
A list of the incremental penetration procedure is printed showing how the bearing capacity of the soil increases with increasing penetration until equilibrium is achieved.

For each wave period analyzed a table showing the forces and overturning moments due to wave pressure against the mat in x- and in y-direction is printed

The wind forces and the corresponding overturning moments in both directions are printed.

The combined wave and current forces are printed for all wave periods and timesteps giving horizontal forces and overturning moments in both directions.

The total weight of the platform due to hull, variables, legs and mat and the corresponding moments due to eccentricities of hull weight and variables are printed.

The total load vectors (vertical force, horizontal force and overturning moment) are printed for each wave period and each time step.

The safety against overturning is printed.

The safety against sliding is printed.

The safety against local bearing capacity failure is printed giving information about the critical geometry of the failure surface.

The safety against deepseated bearing capacity failure due to overturning moments from wave pressure on sea bed and mat as well as wind and combined wave and current forces on hull and legs respectively is printed with information about critical position of slip circle.

The printout of a sample problem is added.

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PROGRAM SAM
VERSION 4/FEBRUARY 1983
PROGRAMMED BY T.J. KVALSTAD
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1)

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*******************************************************************************
* MODEL 2, MAT ON SOFT CLAY SUO=7KPA, K=0.3KN/M**3, I.E. K-VARIATION
* 60 M WATER DEPTH, 20 M WAVEHEIGHT, PERIOD 10 TO 20 SEC.
*******************************************************************************


INPUT DATA PRINTOUT
******************

UNIT WEIGHT OF SEA WATER =     10.000
GRAVITY CONSTANT, G      =      9.810
INERTIA COEFFICIENT, CI  =      2.000
DRAG COEFFICIENT, CD     =      0.700


WATER DEPTH              =     60.000
AIR GAP                  =     16.000


WAVE HEIGHT              =     20.000
MINIMUM WAVE PERIOD      =     10.000
MAXIMUM WAVE PERIOD      =     20.000
PERIOD STEP              =      2.000


TIDAL INDUCED CURRENT VC1=      0.500
WIND INDUCED CURRENT VC2 =      0.500


WIND VELOCITY "AVERAGE"  =     50.000
WIND AREA X-DIR,WAREAX   =    300.000
WIND AREA Y-DIR,WAREAY   =    200.000
WIND LEVER ARM           =     12.000


TOTAL LENGTH OF MAT      =     65.000
TOTAL WIDTH OF MAT       =     50.000
SLOTLENGTH               =     26.000
SLOTWIDTH                =     26.000
HOLELENGTH               =     20.000
HOLEWIDTH                =     26.000
THICKNESS OF MAT         =      3.000
WIDTH OF END STRIP, ALE  =     10.000
WIDTH OF MIDDLE STRIP,BLE=      9.000
DISTANCE TO CG, YLE      =     28.000

SKIRT HEIGHT, SKH        =      0.500
SKIRT TIP THICKN. SKTH   =      0.020
STEEL-SOIL FRICTION RATIO=      1.000


NUMBER OF LEGS, NL       =      3
                            LEG NO.1     LEG NO.2     LEG NO.3     LEG NO.4
X-COORDINATE OF LEGS     =   -15.000       15.000        0.000        0.000
Y-COORDINATE OF LEGS     =   -11.000      -11.000       25.000        0.000
TOTAL LEG LENGTH         =    80.000       80.000       90.000        0.000
EFFECTIVE DIAMETER INERT.=     3.000        3.000        3.000        0.000
EFFECTIVE DIAMETER DRAG  =     3.000        3.000        3.000        0.000
BENDING STIFFNESS, EI    = 0.130E+10    0.130E+10    0.130E+10    0.000E+00
AXIAL STIFFNESS,    EA   = 0.130E+09    0.130E+09    0.130E+09    0.000E+00
SHEAR STIFFNESS.    GA   = 0.200E+07    0.200E+07    0.200E+07    0.000E+00
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PLATFORM WEIGHT            = 40000.000
LEVER ARM OF G IN X-DIR. =       0.000
LEVER ARM OF G IN Y-DIR. =       0.000
VARIABEL LOAD, GVAR        =     0.000
LEVER ARM OF GVAR, X-DIR.=       0.000
LEVER ARM OF GVAR, Y-DIR.=       0.000
WEIGHT PER LEG, GLEG       =     0.000
WEIGHT OF MAT, GMAT        =     0.000

SOIL PROFILE TYPE          =     1
UNDR. SHEAR STRENGTH OF
SOFT CLAY LAYER ON TOP     =     7.000
SHEAR STRENGTH GRADIENT    =     0.300
UNDR. SHEAR STRENGTH OR
COHESION OF HARD LAYER     =   300.000
FRICTION ANGLE,HARD LAYER=       0.000
THICKNESS OF TOP LAYER     =    75.000
ROUGHN. RATIO SOIL/STEEL =       1.000
SUBM. UNIT WEIGHT OF SOIL=       7.000
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H22

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SUBROUTINE MATPEN

 TOTAL AREA OF MAT          . =   3250.00
-SLOT AREA                    =    676.00
-HOLE AREA                    =    520.00
=NET AREA OF MAT              =   2054.00

 INERTIA MOMENT ABOUT Y-AXIS =  0.610E+06
 INERTIA MOMENT ABOUT X-AXIS =  0.732E+06

 X-COORD. OF CENTER OF GRAVIT=      0.00
 Y-COORD. OF CENTER OF GRAVIT=     -1.25

 AVERAGE VERTICAL STRESS      =     19.47
 CORNER STRESS +X,+Y          =     19.47
 CORNER STRESS -X,+Y          =     19.47
 CORNER STRESS -X,-Y          =     19.47
 CORNER STRESS +X,-Y          =     19.47

PENETRATION   BEARING   SQUEEZING
  OF MAT      CAPACITY  CAPACITY


PENETRATION AFTER SET DOWN ESTIMATED TO BE, ZPEN=        0.0
```

3)

H23

4)

PROGRAM *SAM*   : STABILITY ANALYSIS OF MATS

SUBROUTINE WAVEPR: CALCULATION OF VERTICAL FORCES
HORIZONTAL FORCES AND
OVERTURNING MOMENTS DUE TO
WAVE PRESSURE ON THE SEABED

WAVEHEIGHT	=	20.0
WATERDEPTH	=	60.0
WAVE PERIOD	=	10.0
WAVE LENGTH	=	153.8
WAVE STEEPN.	=	0.130

**WARNING! GIVEN WAVEHEIGHT WH = 20.000 EXCEEDS
MAX WAVEHEIGHT HWMAX = 15.383 FOR
WAVE PERIOD TI(IT) = 10.000
ANALYSIS WILL BE
PERFORMED WITH HW(IT) = 15.383 GIVING
WAVE STEEPNESS WS(IT) = 0.100

TIME STEP NO	TIME IN SECONDS	WAVES IN X-DIRECTION VERTICAL FORCE	HORIZONTAL FORCE	OVERTURNING MOMENT	WAVES IN Y-DIRECTION VERTICAL FORCE	HORIZONTAL FORCE	OVERTURNIN MOMENT
1	0.00	20870.06	0.00	0.00	19874.19	-98.74	-148.11
2	0.50	19848.61	-785.85	89831.93	18651.07	-842.26	88568.54
3	1.00	16884.23	-1494.78	170870.66	15602.26	-1503.34	168615.66
4	1.50	12267.11	-2057.39	235183.33	11026.18	-2017.25	232157.45
5	2.00	6449.20	-2418.61	276474.41	5370.79	-2333.71	272973.84
6	2.50	0.00	-2543.07	290703.50	-810.33	-2421.72	287069.91
7	3.00	-6449.21	-2418.61	276474.47	-6912.14	-2272.68	273065.44
8	3.50	-12267.12	-2057.39	235183.27	-12337.33	-1901.18	232331.50
9	4.00	-16884.24	-1494.78	170870.45	-16554.87	-1343.57	168855.09
10	4.50	-19848.61	-785.85	89831.98	-19151.89	-654.45	88850.31
11	5.00	-20870.06	0.00	-0.01	-19874.19	98.74	148.10
12	5.50	-19848.61	785.85	-89831.95	-18651.07	842.26	-88568.55
13	6.00	-16884.23	1494.78	-170870.56	-15602.26	1503.34	-168615.56
14	6.50	-12267.12	2057.39	-235183.06	-11026.19	2017.25	-232157.19
15	7.00	-6449.20	2418.61	-276474.22	-5370.79	2333.71	-272973.66
16	7.50	0.00	2543.07	-290702.28	810.34	2421.72	-287069.69
17	8.00	6449.20	2418.61	-276474.47	6912.14	2272.68	-273065.44
18	8.50	12267.11	2057.39	-235183.02	12337.34	1901.18	-232331.25
19	9.00	16884.23	1494.78	-170870.66	16554.87	1343.57	-168855.30
20	9.50	19848.61	785.85	-89832.01	19151.89	654.45	-88850.34
21	10.00	20870.06	0.00	0.04	19874.19	-98.74	-148.07

PROGRAM *SAM* : STABILITY ANALYSIS OF MATS

5)

SUBROUTINE WAVEPR: CALCULATION OF VERTICAL FORCES
HORIZONTAL FORCES AND
OVERTURNING MOMENTS DUE TO
WAVE PRESSURE ON THE SEABED

WAVEHEIGHT = 20.0
WATERDEPTH = 60.0
WAVE PERIOD = 12.0
WAVE LENGTH = 212.3
WAVE STEEPN. = 0.094

TIME STEP NO	TIME IN SECONDS	WAVES IN X-DIRECTION VERTICAL FORCE	HORIZONTAL FORCE	OVERTURNING MOMENT	WAVES IN Y-DIRECTION VERTICAL FORCE	HORIZONTAL FORCE	OVERTURNING MOMENT
1	0.00	59337.62	0.00	0.00	57820.67	-170.40	-255.59
2	0.60	56433.43	-1623.12	171215.59	54395.87	-1743.68	168600.06
3	1.20	48005.15	-3087.36	325671.60	45646.45	-3146.29	320952.19
4	1.80	34877.78	-4249.39	448248.91	32428.80	-4240.91	441887.53
5	2.40	18336.33	-4995.46	526947.80	16036.82	-4920.40	519567.28
6	3.00	-0.01	-5252.53	554065.63	-1924.97	-5118.25	546388.25
7	3.60	-18336.34	-4995.45	526947.63	-19698.32	-4815.09	519725.00
8	4.20	-34877.79	-4249.39	448248.31	-35543.48	-4040.60	442187.41
9	4.80	-48005.15	-3087.36	325670.94	-47509.38	-2870.58	321365.06
10	5.40	-56433.43	-1623.12	171215.63	-55585.59	-1419.57	159086.27
11	6.00	-59337.60	0.00	-0.06	-57820.66	170.40	255.54
12	6.60	-56433.43	1623.12	-171215.07	-54395.88	1743.68	-168600.36
13	7.20	-48005.11	3087.36	-325670.84	-45646.41	3146.29	-320951.41
14	7.80	-34877.75	4249.39	-448247.53	32428.78	4240.91	-441886.16
15	8.40	-18336.31	4995.45	-526946.75	-16036.79	4920.40	-519566.16
16	9.00	0.00	5252.53	-554064.56	1924.98	5118.25	-546387.19
17	9.60	18336.33	4995.45	-526946.63	19698.33	4815.09	-519724.00
18	10.20	34877.80	4249.38	-448247.22	35543.48	4040.59	-442186.31
19	10.80	48005.14	3087.36	-325670.63	47909.37	2870.58	-321364.75
20	11.40	56433.47	1623.12	-171215.72	55585.57	1419.57	-169086.38
21	12.00	59337.67	-0.01	0.75	57820.68	-170.40	-254.85

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PROGRAM *SAM*     : STABILITY ANALYSIS OF MATS

SUBROUTINE MATENV: CALCULATION OF WIND FORCES
WINDSPEED    =      50.0
WINDAREAX    =     300.0
WINDAREAY    =     200.0
LEVERARM     =      72.0 RELATIVE TO SEA BED
```

WIND IN X-DIRECTION HORIZONTAL FORCE	WIND IN X-DIRECTION OVERTURNING MOMENT	WIND IN Y-DIRECTION HORIZONTAL FORCE	WIND IN Y-DIRECTION OVERTURNING MOMENT
2270.64	163486.23	1513.76	108990.82

10)

H26

4)

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PROGRAM *SAM*    : STABILITY ANALYSIS OF MATS

SUBROUTINE MATENV: CALCULATION OF HORIZONTAL FORCES
                   ON LEGS DUE TO COMBINED WAVE AND
                   CURRENT BY MORISONS EQUATION
WAVEHEIGHT   =      20.0
WATERDEPTH   =      60.0
WAVE PERIOD  =      14.0
WAVE LENGTH  =     270.6
WAVE STEEPN. =    0.0739
```

TIME STEP NO	TIME IN SECONDS	WAVES IN X-DIRECTION VERTICAL FORCE	HORIZONTAL FORCE	OVERTURNING MOMENT	WAVES IN Y-DIRECTION VERTICAL FORCE	HORIZONTAL FORCE	OVERTURNING MOMENT
1	0.00		4075.69	194692.28		3825.75	194692.28
2	0.70		2343.56	121959.13		2202.37	121959.13
3	1.40		270.06	32621.77		340.68	32621.77
4	2.10		-1622.82	-47088.92		-1341.98	-47088.92
5	2.80		-2890.93	-97153.93		-2504.46	-97153.93
6	3.50		-3375.38	-112736.22		-3003.95	-112736.22
7	4.20		-3279.98	-105103.23		-3114.05	-105103.23
8	4.90		-2873.23	-87376.47		-2892.65	-87376.47
9	5.60		-2361.29	-68731.96		-2375.12	-68731.96
10	6.30		-1731.35	-49107.02		-1725.60	-49107.02
11	7.00		-931.71	-26186.14		-923.30	-26186.14
12	7.70		42.53	1453.92		37.02	1453.92
13	8.40		1133.51	33835.22		1095.93	33835.22
14	9.10		2203.24	68301.91		2112.68	68301.91
15	9.80		3098.36	101106.93		2914.66	101106.93
16	10.50		3832.99	133618.09		3698.23	133618.09
17	11.20		4466.34	167894.31		4452.32	167894.31
18	11.90		5039.49	204048.95		4998.57	204048.95
19	12.60		5353.02	230966.66		5221.66	230966.66
20	13.30		5100.15	231873.00		4872.45	231873.00
21	14.00		4075.69	194692.23		3825.75	194692.23

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15)

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PROGRAM *SAM*     : STABILITY ANALYSIS OF MATS
SUBROUTINE TOTFOR: CALCULATION OF TOTAL FORCE
                   VECTORS AND OVERTURNING MOM-
                   MENTS FOR WAVE+CURRENT,WIND
                   WAVE PRESSURE ON MAT AND
                   WEIGHT OF HULL,VARIABLE,LEGS
                   AND MAT

TOTAL WEIGHT OF PLATFORM,         GTOT  = 40000.000
TOTAL OVERTURNING MOMENT ABOUT
Y-AXIS DUE TO WEIGHTS,            GMY   =     0.000
TOTAL OVERTURNING MOMENT ABOUT
X-AXIS DUE TO WEIGHTS,            GMX   =     0.000
```

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PROGRAM *SAM*- : STABILITY ANALYSIS OF MATS
SUBROUTINE TOTFOR: CALCULATION OF TOTAL FORCE VECTORS AND OVERTURNING MOMMENTS FOR WAVE+CURRENT,WIND WAVE PRESSURE ON MAT AND WEIGHT OF HULL,VARIABLE,LEGS AND MAT

(2)

PERIOD NO. IT = 1
WAVE PERIOD IN SECONDS TI = 10.000

TIME STEP NO.	TIME IN SECONDS	WAVES IN X-DIRECTION VERTICAL FORCE	HORIZONTAL FORCE	OVERTURNING MOMENT	WAVES IN Y-DIRECTION VERTICAL FORCE	HORIZONTAL FORCE	OVERTURNING MOMENT
1	0.00	0.609E+05	0.441E+04	0.276E+06	0.599E+05	0.301E+04	0.222E+06
2	0.50	0.598E+05	0.244E+04	0.314E+06	0.587E+05	0.118E+04	0.258E+06
3	1.00	0.569E+05	0.474E+03	0.338E+06	0.556E+05	-0.384E+03	0.281E+06
4	1.50	0.523E+05	-0.119E+04	0.352E+06	0.510E+05	-0.155E+04	0.294E+06
5	2.00	0.464E+05	-0.232E+04	0.358E+06	0.454E+05	-0.231E+04	0.300E+06
6	2.50	0.400E+05	-0.279E+04	0.357E+06	0.392E+05	-0.267E+04	0.299E+06
7	3.00	0.336E+05	-0.262E+04	0.346E+06	0.331E+05	-0.267E+04	0.288E+06
8	3.50	0.277E+05	-0.190E+04	0.320E+06	0.277E+05	-0.223E+04	0.263E+06
9	4.00	0.231E+05	-0.782E+03	0.278E+06	0.234E+05	-0.134E+04	0.221E+06
10	4.50	0.202E+05	0.578E+03	0.221E+06	0.208E+05	-0.757E+02	0.165E+06
11	5.00	0.191E+05	0.206E+04	0.155E+06	0.201E+05	0.138E+04	0.101E+06
12	5.50	0.202E+05	0.358E+04	0.917E+05	0.213E+05	0.285E+04	0.385E+05
13	6.00	0.231E+05	0.503E+04	0.377E+05	0.244E+05	0.423E+04	-0.145E+05
14	6.50	0.277E+05	0.628E+04	0.862E+03	0.290E+05	0.537E+04	-0.506E+05
15	7.00	0.336E+05	0.730E+04	-0.118E+05	0.346E+05	0.627E+04	-0.628E+05
16	7.50	0.400E+05	0.798E+04	0.220E+04	0.408E+05	0.692E+04	-0.487E+05
17	8.00	0.464E+05	0.828E+04	0.415E+05	0.469E+05	0.722E+04	-0.960E+04
18	8.50	0.523E+05	0.809E+04	0.984E+05	0.523E+05	0.703E+04	0.468E+05
19	9.00	0.569E+05	0.737E+04	0.163E+06	0.566E+05	0.621E+04	0.111E+06
20	9.50	0.598E+05	0.612E+04	0.225E+06	0.592E+05	0.479E+04	0.172E+06
21	10.00	0.609E+05	0.441E+04	0.276E+06	0.599E+05	0.301E+04	0.222E+06

H29

17)

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PROGRAM *SAM*    : STABILITY ANALYSIS OF MATS
SUBROUTINE TOTFOR: CALCULATION OF TOTAL FORCE
                   VECTORS AND OVERTURNING MOM-
                   MENTS FOR WAVE+CURRENT,WIND
                   WAVE PRESSURE ON MAT AND
                   WEIGHT OF HULL,VARIABLE,LEGS
                   AND MAT

PERIOD NO.                     IT   =       2
WAVE PERIOD IN SECONDS         TI   =   12.000
```

TIME STEP NO.	TIME IN SECONDS	WAVES IN X-DIRECTION VERTICAL FORCE	HORIZONTAL FORCE	OVERTURNING MOMENT	WAVES IN Y-DIRECTION VERTICAL FORCE	HORIZONTAL FORCE	OVERTURNING MOMENT
1	0.00	0.993E+05	0.620E+04	0.364E+06	0.978E+05	0.482E+04	0.309E+06
2	0.60	0.964E+05	0.279E+04	0.457E+06	0.944E+05	0.159E+04	0.400E+06
3	1.20	0.880E+05	-0.730E+03	0.519E+06	0.856E+05	-0.153E+04	0.459E+06
4	1.80	0.749E+05	-0.373E+04	0.559E+06	0.724E+05	-0.408E+04	0.498E+06
5	2.40	0.583E+05	-0.570E+04	0.586E+06	0.560E+05	-0.576E+04	0.524E+06
6	3.00	0.400E+05	-0.644E+04	0.596E+06	0.381E+05	-0.645E+04	0.534E+06
7	3.60	0.217E+05	-0.608E+04	0.576E+06	0.203E+05	-0.632E+04	0.515E+06
8	4.20	0.512E+04	-0.486E+04	0.518E+06	0.446E+04	-0.535E+04	0.457E+06
9	4.80	-0.801E+04	-0.305E+04	0.419E+06	-0.791E+04	-0.363E+04	0.360E+06
10	5.40	-0.164E+05	-0.859E+03	0.288E+06	-0.156E+05	-0.142E+04	0.232E+06
11	6.00	-0.193E+05	0.161E+04	0.143E+06	-0.178E+05	0.102E+04	0.888E+05
12	6.60	-0.164E+05	0.420E+04	0.108E+04	-0.144E+05	0.356E+04	-0.508E+05
13	7.20	-0.801E+04	0.670E+04	-0.120E+06	-0.565E+04	0.597E+04	-0.170E+06
14	7.80	0.512E+04	0.887E+04	-0.208E+06	0.757E+04	0.801E+04	-0.256E+06
15	8.40	0.217E+05	0.105E+05	-0.250E+06	0.240E+05	0.948E+04	-0.298E+06
16	9.00	0.400E+05	0.116E+05	-0.240E+06	0.419E+05	0.105E+05	-0.287E+06
17	9.60	0.583E+05	0.120E+05	-0.173E+06	0.597E+05	0.110E+05	-0.221E+06
18	10.20	0.749E+05	0.118E+05	-0.578E+05	0.755E+05	0.108E+05	-0.106E+06
19	10.80	0.880E+05	0.108E+05	0.884E+05	0.879E+05	0.968E+04	0.382E+05
20	11.40	0.964E+05	0.897E+04	0.237E+06	0.956E+05	0.765E+04	0.185E+06
21	12.00	0.993E+05	0.620E+04	0.364E+06	0.978E+05	0.482E+04	0.309E+06

H30

18)

```
PROGRAM *SAM*    : STABILITY ANALYSIS OF MATS
SUBROUTINE TOTFOR: CALCULATION OF TOTAL FORCE
                   VECTORS AND OVERTURNING MOM-
                   MENTS FOR WAVE+CURRENT,WIND
                   WAVE PRESSURE ON MAT AND
                   WEIGHT OF HULL,VARIABLE,LEGS
                   AND MAT

PERIOD NO.                        IT    =      3
WAVE PERIOD IN SECONDS            TI    =    14.000
```

TIME STEP NO.	TIME IN SECONDS	WAVES IN X-DIRECTION VERTICAL FORCE	HORIZONTAL FORCE	OVERTURNING MOMENT	WAVES IN Y-DIRECTION VERTICAL FORCE	HORIZONTAL FORCE	OVERTURNING MOMENT
1	0.00	0.129E+06	0.635E+04	0.358E+06	0.127E+06	0.517E+04	0.303E+06
2	0.70	0.124E+06	0.271E+04	0.480E+06	0.122E+06	0.168E+04	0.423E+06
3	1.40	0.112E+06	-0.109E+04	0.567E+06	0.109E+06	-0.185E+04	0.507E+06
4	2.10	0.922E+05	-0.434E+04	0.627E+06	0.894E+05	-0.484E+04	0.565E+06
5	2.80	0.674E+05	-0.649E+04	0.666E+06	0.647E+05	-0.682E+04	0.603E+06
6	3.50	0.400E+05	-0.727E+04	0.681E+06	0.376E+05	-0.756E+04	0.618E+06
7	4.20	0.126E+05	-0.688E+04	0.658E+06	0.108E+05	-0.732E+04	0.595E+06
8	4.90	-0.122E+05	-0.559E+04	0.586E+06	-0.132E+05	-0.619E+04	0.525E+06
9	5.60	-0.318E+05	-0.372E+04	0.465E+06	-0.320E+05	-0.430E+04	0.406E+06
10	6.30	-0.444E+05	-0.137E+04	0.309E+06	-0.438E+05	-0.193E+04	0.252E+06
11	7.00	-0.487E+05	0.134E+04	0.137E+06	-0.473E+05	0.756E+03	0.831E+05
12	7.70	-0.444E+05	0.422E+04	-0.299E+05	-0.423E+05	0.358E+04	-0.814E+05
13	8.40	-0.318E+05	0.703E+04	-0.173E+06	-0.292E+05	0.631E+04	-0.222E+06
14	9.10	-0.122E+05	0.947E+04	-0.278E+06	-0.940E+04	0.864E+04	-0.325E+06
15	9.80	0.126E+05	0.112E+05	-0.335E+06	0.153E+05	0.103E+05	-0.381E+06
16	10.50	0.400E+05	0.123E+05	-0.334E+06	0.424E+05	0.113E+05	-0.379E+06
17	11.20	0.674E+05	0.126E+05	-0.268E+06	0.692E+05	0.117E+05	-0.314E+06
18	11.90	0.922E+05	0.123E+05	-0.143E+06	0.932E+05	0.113E+05	-0.190E+06
19	12.60	0.112E+06	0.113E+05	0.238E+05	0.112E+06	0.102E+05	-0.256E+05
20	13.30	0.124E+06	0.928E+04	0.200E+06	0.124E+06	0.811E+04	0.149E+06
21	14.00	0.129E+06	0.635E+04	0.358E+06	0.127E+06	0.517E+04	0.303E+06

W31

23)

PROGRAM *SAM*   :STABILITY ANALYSIS OF MATS

SUBROUTINE MATSLI:CALCULATION OF SAFETY AGAINST
SLIDING FAILURE AT SKIRT TIP

WAVEHEIGHT = 20.0
WATERDEPTH = 60.0
WAVE PERIOD = 12.0
WAVE LENGTH = 212.3
WAVE STEEPN. = 0.0942

RESULTS OF SLIDING RESISTANCE ANALYSIS

TIME STEP NO.	X-DIRECTION RESIST-ANCE	HORIZ. FORCE	SAFETY FACTOR	Y-DIRECTION RESIST-ANCE	HORIZ. FORCE	SAFETY FACTOR
1	16794.5	6203.3	2.707	16572.9	4818.3	3.440
2	17071.5	2793.7	6.111	16669.6	1587.8	10.498
3	16267.4	-730.0	-22.283	16016.4	-1529.6	-10.471
4	15292.1	-3733.8	-4.096	15830.5	-4082.9	-3.877
5	15085.5	-5699.1	-2.647	15323.1	-5763.5	-2.659
6	15014.3	-6441.4	-2.331	15259.5	-6448.9	-2.366
7	16009.1	-6078.0	-2.634	15303.9	-6320.1	-2.421
8	16009.1	-4860.7	-3.294	16080.8	-5353.7	-3.004
9	16009.1	-3053.2	-5.243	15980.5	-3627.5	-4.405
10	16009.1	-858.7	-18.644	15883.0	-1423.3	-11.159
11	16009.1	1611.4	9.935	15983.1	1021.9	15.641
12	15732.1	4200.3	3.746	15716.6	3555.1	4.421
13	15482.1	6704.1	2.309	15478.7	5972.5	2.592
14	15283.7	8867.6	1.724	15462.6	8013.0	1.930
15	15156.4	10524.0	1.440	15346.8	9476.5	1.619
16	15572.4	11585.2	1.344	15312.4	10506.4	1.457
17	15616.3	12019.0	1.299	15546.8	11016.9	1.411
18	15743.6	11802.1	1.334	15677.2	10784.0	1.454
19	16267.4	10836.9	1.501	15874.6	9680.5	1.640
20	16517.4	8972.2	1.841	16303.9	7648.3	2.132
21	16794.5	6203.2	2.707	16572.9	4818.3	3.440

MINIMUM SAFETY FACTORS FOR THIS PERIOD WERE:
IN X-DIRECTION IN TIME STEP NO. 17 FSX = 1.30
IN Y-DIRECTION IN TIME STEP NO. 17 FSY = 1.41

H32

24)

PROGRAM *SAM* :STABILITY ANALYSIS OF MATS

SUBROUTINE MATSLI:CALCULATION OF SAFETY AGAINST
SLIDING FAILURE AT SKIRT TIP

```
WAVEHEIGHT   =    20.0
WATERDEPTH   =    60.0
WAVE PERIOD  =    14.0
WAVE LENGTH  =   270.6
WAVE STEEPN. =  0.0739
```

RESULTS OF SLIDING RESISTANCE ANALYSIS

TIME STEP NO.	X-DIRECTION RESIST-ANCE	HORIZ. FORCE	SAFETY FACTOR	Y-DIRECTION RESIST-ANCE	HORIZ. FORCE	SAFETY FACTOR
1	16794.5	6346.3	2.646	16572.4	5174.0	3.203
2	17120.3	2707.6	6.323	16889.1	1682.3	10.039
3	16174.7	-1085.9	-14.895	15921.3	-1848.5	-8.613
4	15941.4	-4343.8	-3.670	15699.3	-4837.9	-3.245
5	14869.3	-6488.3	-2.292	15136.7	-6816.7	-2.221
6	14786.9	-7274.7	-2.033	15065.5	-7562.3	-1.992
7	16009.1	-6877.3	-2.328	15121.3	-7324.0	-2.065
8	16009.1	-5594.2	-2.862	16143.0	-6194.0	-2.606
9	16009.1	-3717.3	-4.307	16032.7	-4296.5	-3.732
10	16009.1	-1367.3	-11.708	15920.0	-1930.8	-8.245
11	16009.1	1338.9	11.957	15983.6	756.0	21.143
12	15683.3	4219.8	3.717	15666.8	3584.6	4.371
13	15389.4	7030.8	2.189	15383.6	6312.7	2.437
14	15156.1	9465.5	1.601	15161.6	8636.1	1.756
15	15006.4	11237.0	1.335	15192.3	10254.5	1.482
16	15414.6	12273.6	1.256	15149.8	11284.1	1.343
17	15466.2	12605.0	1.227	15392.1	11689.8	1.317
18	15941.4	12301.7	1.296	15545.4	11327.5	1.372
19	16174.7	11250.3	1.438	15962.6	10170.6	1.569
20	16468.6	9277.4	1.775	16253.1	8105.2	2.005
21	16794.5	6346.3	2.646	16572.4	5174.0	3.203

MINIMUM SAFETY FACTORS FOR THIS PERIOD WERE:
IN X-DIRECTION IN TIME STEP NO. 17 FSX = 1.23
IN Y-DIRECTION IN TIME STEP NO. 17 FSY = 1.32

29)

PROGRAM *SAM* :STABILITY ANALYSIS OF MATS

SUBROUTINE MATLOC:CALCULATION OF SAFETY AGAINST
LOCAL BEARING CAPACITY FAILURE
UNDER MOST STRESSED BEAM OF
AN A-SHAPED MAT

WAVEHEIGHT	=	20.0
WATERDEPTH	=	60.0
WAVE PERIOD	=	12.0
WAVE LENGTH	=	212.3
WAVE STEEPN.	=	0.0942

DETAILS ABOUT PART AREAS

AREA NO.	AREA M**2.	CENTER COORDINATES X-COORD.	Y-COORD.	RATIO B/L	AVERAGE DEPTH
1	312.00	-19.00	-24.00	0.23	2.40
2	312.00	19.00	-24.00	0.23	2.40
3	108.00	-19.00	-6.50	0.00	3.60
4	117.00	-13.00	-6.50	0.00	1.80
5	117.00	13.00	-6.50	0.00	1.80
6	108.00	19.00	-6.50	0.00	3.60
7	240.00	-19.00	8.00	0.00	2.40
8	240.00	19.00	8.00	0.00	2.40
9	120.00	-19.00	23.00	0.83	2.20
10	130.00	-13.00	23.00	0.00	5.00
11	130.00	13.00	23.00	0.00	5.00
12	120.00	19.00	23.00	0.83	2.20

TIME STEP NO.	SAFETY FACTOR X-DIRECTION	SAFETY FACTOR Y-DIRECTION
1	1.52	1.68
2	1.66	1.80
3	1.79	1.91
4	1.87	1.95
5	1.94	1.96
6	1.95	-2.04
7	1.96	-2.05
8	1.94	2.01
9	1.95	1.97
10	1.91	2.00
11	1.87	1.97
12	1.77	1.88
13	1.69	1.78
14	1.52	1.62
15	1.39	1.55
16	1.35	1.39
17	1.31	1.36
18	1.28	1.33
19	1.26	1.43
20	1.42	1.55
21	1.52	1.68

MINIMUM SAFETY FACTORS FOR THIS PERIOD WERE:
IN X-DIRECTION IN TIME STEP NO. 19 FSX = 1.26
IN Y-DIRECTION IN TIME STEP NO. 18 FSY = 1.33

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30)

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PROGRAM *SAM*    :STABILITY ANALYSIS OF MATS

SUBROUTINE MATLOC:CALCULATION OF SAFETY AGAINST
                  LOCAL BEARING CAPACITY FAILURE
                  UNDER MOST STRESSED BEAM OF
                  AN A-SHAPED MAT
WAVEHEIGHT   =      20.0
WATERDEPTH   =      60.0
WAVE PERIOD  =      14.0
WAVE LENGTH  =     270.6
WAVE STEEPN. =    0.0739
```

DETAILS ABOUT PART AREAS

AREA NO.	AREA M**2.	CENTER COORDINATES X-COORD.	Y-COORD.	RATIO B/L	AVERAGE DEPTH
1	312.00	-19.00	-24.00	0.23	2.40
2	312.00	19.00	-24.00	0.23	2.40
3	108.00	-19.00	-6.50	0.00	3.60
4	117.00	-13.00	-6.50	0.00	1.80
5	117.00	13.00	-6.50	0.00	1.80
6	108.00	19.00	-6.50	0.00	3.60
7	240.00	-19.00	8.00	0.00	2.40
8	240.00	19.00	8.00	0.00	2.40
9	120.00	-19.00	23.00	0.83	2.20
10	130.00	-13.00	23.00	0.00	5.00
11	130.00	13.00	23.00	0.00	5.00
12	120.00	19.00	23.00	0.83	2.20

TIME STEP NO.	SAFETY FACTOR X-DIRECTION	SAFETY FACTOR Y-DIRECTION
1	1.53	1.68
2	1.66	1.80
3	1.78	1.90
4	1.86	1.95
5	1.91	1.86
6	1.91	-1.97
7	1.94	1.87
8	1.93	1.94
9	1.91	1.97
10	1.92	2.00
11	1.88	1.98
12	1.79	1.89
13	1.70	1.75
14	1.53	1.59
15	1.41	1.43
16	1.14	1.41
17	1.11	1.38
18	1.30	1.35
19	1.27	1.45
20	1.41	1.52
21	1.53	1.68

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MINIMUM SAFETY FACTORS FOR THIS PERIOD WERE:
IN X-DIRECTION IN TIME STEP NO.   17 FSX =      1.11
IN Y-DIRECTION IN TIME STEP NO.   18 FSY =      1.35
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N35

31

H 36

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PROGRAM *SAM*     :STABILITY ANALYSIS OF MATS

SUBROUTINE MATLOC:CALCULATION OF SAFETY AGAINST
                  LOCAL BEARING CAPACITY FAILURE
                  UNDER MOST STRESSED BEAM OF
                  AN A-SHAPED MAT
WAVEHEIGHT   =     20.0
WATERDEPTH   =     60.0
WAVE PERIOD  =     16.0
WAVE LENGTH  =    327.1
WAVE STEEPN. =   0.0611
```

DETAILS ABOUT PART AREAS

AREA NO.	AREA M**2.	CENTER COORDINATES X-COORD.	Y-COORD.	RATIO B/L	AVERAGE DEPTH
1	312.00	-19.00	-24.00	0.23	2.40
2	312.00	19.00	-24.00	0.23	2.40
3	108.00	-19.00	-6.50	0.00	3.60
4	117.00	-13.00	-6.50	0.00	1.80
5	117.00	13.00	-6.50	0.00	1.80
6	108.00	19.00	-6.50	0.00	3.60
7	240.00	-19.00	8.00	0.00	2.40
8	240.00	19.00	8.00	0.00	2.40
9	120.00	-19.00	23.00	0.83	2.20
10	130.00	-13.00	23.00	0.00	5.00
11	130.00	13.00	23.00	0.00	5.00
12	120.00	19.00	23.00	0.83	2.20

TIME STEP NO.	SAFETY FACTOR X-DIRECTION	SAFETY FACTOR Y-DIRECTION
1	1.54	1.68
2	1.66	1.80
3	1.77	1.90
4	1.85	1.93
5	1.89	1.85
6	1.89	1.82
7	1.90	1.90
8	1.92	1.93
9	1.91	1.97
10	1.92	2.01
11	1.88	1.98
12	1.79	1.89
13	1.71	1.76
14	1.54	1.60
15	1.42	1.44
16	1.15	1.42
17	1.13	1.39
18	1.32	1.36
19	1.29	1.46
20	1.42	1.53
21	1.54	1.68

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MINIMUM SAFETY FACTORS FOR THIS PERIOD WERE:
IN X-DIRECTION IN TIME STEP NO.   17 FSX =      1.13
IN Y-DIRECTION IN TIME STEP NO.   18 FSY =      1.36
```

PROGRAM *SAM* :STABILITY ANALYSIS OF MATS

36)

SUBROUTINE SHBEAR:CALCULATION OF SAFETY AGAINST

SHALLOW FOUNDATION FAILURE

WAVEHEIGHT = 20.0
WATERDEPTH = 60.0
WAVE PERIOD = 14.0
WAVE LENGTH = 270.6
WAVE STEEPN. = 0.0739

RESULTS OF SHALLOW BEARING CAPACITY ANALYSIS

POSITIV RADIUS MEANS CENTER OF SLIP CYLINDER ABOVE FOOTING
NEGATIV RADIUS MEANS CENTER OF SLIP CYLINDER BELOW FOOTING
POSITIV SAFETY FACTOR MEANS SLIP IN POSITIV DIRECTION (RIGHT)
NEGATIV SAFETY FACTOR MEANS SLIP IN NEGATIV DIRECTION (LEFT)

TIME	X-DIRECTION		Y-DIRECTION	
STEP NO.	RADIUS	SAFETY FACTOR	RADIUS	SAFETY FACTOR
1	-92.11	1.99	-119.74	2.12
2	-92.11	2.35	-95.79	2.58
3	-73.68	2.86	-95.79	3.28
4	-144.74	-3.46	-150.53	-3.29
5	-428.95	-3.48	-735.53	-3.32
6	-1000.00	-3.30	-1300.00	-3.17
7	-1000.00	-3.67	-1300.00	-3.41
8	-1000.00	-4.89	-1300.00	-4.29
9	-115.79	-8.70	-1300.00	-7.13
10	-73.68	-7.88	-119.74	-6.62
11	-60.53	-8.15	-78.68	-7.54
12	-1000.00	6.98	-1300.00	8.98
13	-1000.00	3.52	-1300.00	4.02
14	-1000.00	2.43	-1300.00	2.67
15	-218.42	1.89	-283.95	2.02
16	-144.74	1.58	-188.16	1.61
17	-115.79	1.49	-150.53	1.47
18	-115.79	1.51	-150.53	1.49
19	-115.79	1.60	-150.53	1.61
20	-92.11	1.76	-119.74	1.81
21	-92.11	1.99	-119.74	2.12

37)

PROGRAM *SAM*   :STABILITY ANALYSIS OF MATS

SUBROUTINE SHBEAR:CALCULATION OF SAFETY AGAINST
SHALLOW FOUNDATION FAILURE

```
WAVEHEIGHT   =    20.0
WATERDEPTH   =    60.0
WAVE PERIOD  =    16.0
WAVE LENGTH  =   327.1
WAVE STEEPN. =  0.0611
```

RESULTS OF SHALLOW BEARING CAPACITY ANALYSIS

POSITIV RADIUS MEANS CENTER OF SLIP CYLINDER ABOVE FOOTING
NEGATIV RADIUS MEANS CENTER OF SLIP CYLINDER BELOW FOOTING
POSITIV SAFETY FACTOR MEANS SLIP IN POSITIV DIRECTION (RIGHT)
NEGATIV SAFETY FACTOR MEANS SLIP IN NEGATIV DIRECTION (LEFT)

TIME STEP NO.	X-DIRECTION RADIUS	X-DIRECTION SAFETY FACTOR	Y-DIRECTION RADIUS	Y-DIRECTION SAFETY FACTOR
1	-92.11	1.97	-119.74	2.08
2	-92.11	2.28	-119.74	2.51
3	-73.68	2.75	-95.79	3.15
4	-178.95	-3.34	-150.53	-3.08
5	-218.42	-3.39	-478.95	-3.20
6	-1000.00	-3.28	-1300.00	-3.09
7	-1000.00	-3.70	-1300.00	-3.38
8	-1000.00	-5.02	-1300.00	-4.34
9	-115.79	-5.94	-78.68	-5.33
10	-73.68	-5.51	-95.79	-4.92
11	-60.53	-5.81	-65.00	-5.51
12	-60.53	-6.99	-65.00	-6.95
13	-1000.00	3.73	-1300.00	4.29
14	-1000.00	2.45	-1300.00	2.69
15	-218.42	1.87	-233.95	1.95
16	-144.74	1.55	-188.16	1.53
17	-115.79	1.49	-150.53	1.43
18	-115.79	1.52	-150.53	1.48
19	-115.79	1.61	-119.74	1.61
20	-92.11	1.76	-119.74	1.80
21	-92.11	1.97	-119.74	2.08

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PROGRAM *SAM*   : STABILITY ANALYSIS OF MATS

SUBROUTINE DEBEAR: CALCULATION OF SAFETY AGAINST
DEEPSEATED FOUNDATION FAILURE

WAVEHEIGHT = 20.0
WATERDEPTH = 60.0
WAVE PERIOD = 14.0
WAVE LENGTH = 270.6
WAVE STEEPN. = 0.0739

WAVES IN X-DIRECTION

RESULTS OF STABILITY ANALYSIS FOR DEEP SEATED FAILURE SURFACE
INCLUDING WAVE PRESSURE EFFECTS OVER TOTAL MAT AREA INCLUDIN HOLE AND SLOT AREA

TIME STEP NO	CENTER COORDINATES XC --	YC --	RADIUS OF SLIP SURFACE	SAFETY FACTOR SFD
1	-50.00	30.00	80.78	1.81
2	-25.00	30.00	58.31	1.64
3	-25.00	30.00	58.31	1.42
4	-25.00	30.00	58.31	1.34
5	-25.00	30.00	58.31	1.38
6	-25.00	30.00	58.31	1.54
7	0.00	30.00	52.24	1.65
8	25.00	30.00	67.65	1.89
9	25.00	30.00	67.65	2.04
10	50.00	30.00	80.78	2.23
11	50.00	30.00	80.78	2.46
12	-50.00	30.00	80.78	2.27
13	-25.00	30.00	67.65	2.10
14	-25.00	30.00	67.65	1.94
15	0.00	30.00	52.24	1.74
16	25.00	30.00	58.31	1.59
17	25.00	30.00	58.31	1.42
18	25.00	30.00	58.31	1.39
19	25.00	30.00	58.31	1.49
20	50.00	30.00	80.78	1.72
21	-50.00	30.00	80.78	1.81

MINIMUM SAFETY AGAINST DEEPSEATED FAILURE
FOR THIS WAVE PERIOD AND WAVE DIRECTION = 1.34

WAVES IN Y-DIRECTION

RESULTS OF STABILITY ANALYSIS FOR DEEP SEATED FAILURE SURFACE
INCLUDING WAVE PRESSURE EFFECTS OVER TOTAL MAT AREA INCLUDIN HOLE AND SLOT AREA

TIME STEP NO	CENTER COORDINATES XC --	YC --	RADIUS OF SLIP SURFACE	SAFETY FACTOR SFD
1	-32.50	39.00	75.80	2.19
2	-32.50	39.00	75.80	1.78
3	-32.50	39.00	75.80	1.61
4	-32.50	39.00	75.80	1.59
5	-32.50	39.00	75.80	1.71
6	0.00	39.00	63.41	1.70
7	0.00	39.00	63.41	1.78
8	0.00	39.00	63.41	2.08

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13	-32.50	39.00	75.80	2.28
14	0.00	39.00	63.41	2.15
15	0.00	39.00	63.41	1.83
16	0.00	39.00	63.41	1.74
17	32.50	39.00	75.80	1.73
18	32.50	39.00	75.80	1.61
19	32.50	39.00	75.80	1.63
20	32.50	39.00	75.80	1.82
21	-32.50	39.00	75.80	2.19

MINIMUM SAFETY AGAINST DEEPSEATED FAILURE
FOR THIS WAVE PERIOD AND WAVE DIRECTION = 1.59

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PROGRAM *SAM* : STABILITY ANALYSIS OF MATS

SUBROUTINE DEBEAR: CALCULATION OF SAFETY AGAINST
DEEPSEATED FOUNDATION FAILURE

WAVEHEIGHT = 20.0
WATERDEPTH = 60.0
WAVE PERIOD = 16.0
WAVE LENGTH = 327.1
WAVE STEEPN. = 0.0611

WAVES IN X-DIRECTION

RESULTS OF STABILITY ANALYSIS FOR DEEP SEATED FAILURE SURFACE
INCLUDING WAVE PRESSURE EFFECTS OVER TOTAL MAT AREA INCLUDIN HOLE AND SLOT AREA

TIME STEP NO	CENTER COORDINATES XC --	YC --	RADIUS OF SLIP SURFACE	SAFETY FACTOR SFD
1	-50.00	30.00	80.78	1.80
2	-50.00	30.00	80.78	1.57
3	-25.00	30.00	58.31	1.40
4	-25.00	30.00	58.31	1.29
5	-25.00	30.00	58.31	1.29
6	-25.00	30.00	58.31	1.40
7	0.00	30.00	52.24	1.57
8	25.00	30.00	67.65	1.75
9	50.00	30.00	80.78	1.93
10	50.00	30.00	80.78	2.04
11	50.00	30.00	80.78	2.44
12	-50.00	30.00	80.78	2.07
13	-50.00	30.00	80.78	1.96
14	-25.00	30.00	67.65	1.79
15	0.00	30.00	65.43	1.64
16	25.00	30.00	58.31	1.44
17	25.00	30.00	58.31	1.33
18	25.00	30.00	58.31	1.33
19	25.00	30.00	58.31	1.46
20	50.00	30.00	80.78	1.60
21	-50.00	30.00	80.78	1.80

MINIMUM SAFETY AGAINST DEEPSEATED FAILURE
FOR THIS WAVE PERIOD AND WAVE DIRECTION = 1.29

WAVES IN Y-DIRECTION

RESULTS OF STABILITY ANALYSIS FOR DEEP SEATED FAILURE SURFACE
INCLUDING WAVE PRESSURE EFFECTS OVER TOTAL MAT AREA INCLUDIN HOLE AND SLOT AREA

TIME STEP NO	CENTER COORDINATES XC --	YC --	RADIUS OF SLIP SURFACE	SAFETY FACTOR SFD
1	-65.00	39.00	105.01	2.11
2	-32.50	39.00	75.80	1.76
3	-32.50	39.00	75.80	1.53
4	-32.50	39.00	75.80	1.46
5	-32.50	39.00	75.80	1.51
6	0.00	39.00	63.41	1.60
7	0.00	39.00	63.41	1.68
8	32.50	39.00	75.80	1.96

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13	-32.50	39.00	83.44	2.12
14	-32.50	39.00	75.80	1.98
15	0.00	39.00	63.41	1.72
16	0.00	39.00	63.41	1.64
17	32.50	39.00	75.80	1.53
18	32.50	39.00	75.80	1.47
19	32.50	39.00	75.80	1.55
20	32.50	39.00	75.80	1.80
21	-65.00	39.00	105.01	2.11

MINIMUM SAFETY AGAINST DEEPSEATED FAILURE
FOR THIS WAVE PERIOD AND WAVE DIRECTION = 1.46

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PROGRAM *SAM* : STABILITY ANALYSIS OF MATS

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MAIN PROGRAM SAM :

SUMMARY TABLE OF SAFETY FACTORS FOR THE FOUR DIFFERENT FOUNDATION FAILURE MODES ANALYZED

WAVE PERIOD SEC.	SLIDING UNDER SKIRT TIP		LOCAL BEARING FAILURE UNDER MOST LOADED BEAM OF A-MAT		SHALLOW FAILURE UNDER EDGE (TOTAL FAILURE)		DEEPSEATED OVERTURNING FAILURE	
	X-DIR.	Y-DIR.	X-DIR.	Y-DIR.	X-DIR.	Y-DIR.	X-DIR.	Y-DIR.
10.0	1.88	2.16	1.50	1.59	2.23	2.54	2.83	4.00
12.0	1.30	1.41	1.26	1.33	1.58	1.63	1.58	1.97
14.0	1.23	1.32	1.11	1.35	1.49	1.47	1.34	1.59
16.0	1.23	1.31	1.13	1.36	1.49	1.43	1.29	1.46
18.0	1.26	1.35	1.14	1.37	1.52	1.46	1.28	1.44
20.0	1.31	1.40	1.31	1.38	1.58	1.52	1.30	1.47

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COMPUTER LISTING OF PROGRAM SAIL

The listing comprises the main program SAIL and the following subroutines:

BHPHI
BHSQUE
BHSU
ENVIRO
FHZCAP
FIFOR
INITAL
INPOL
INPUT
INVM12
LEGLEN
LOVECT
OVSTAB
PENETR
PRAC10
PREL
PREPEN
PRINT
PUNCH
SPUD
STFMTX
TRANSF
TRIM12
WAVE
ZPCORR

```
C*************************************************************************************
C     ROUTINE: SAIL = MAIN ROUTINE                         DET NORSKE VERITAS
C
C
C     PURPOSE:
C     JASTAB IS A COMPUTER PROGRAM FOR CALCULATION OF PENETRATION AND
C     STABILITY OF JACKUP PLATFORMS.  IT CONSISTS OF VARIOUS ROUTINES FOR
C     CALCULATION OF THE INVOLVED PARAMETERS.  THE MAIN ROUTINE IS CALLED
C     JASTAB.  THIS MAIN ROUTINE ADMINISTER THE CALLS TO SEVERAL SUBROUTINES.
C
C*************************************************************************************
C
C     METHOD:NONE
C
C     INPUT ARGUMENT: NONE
C
C     INTERNAL PARAMETERS:
C
C     OUTPUT ARGUMENTS: NONE
C*************************************************************************************
C
C
      IMPLICIT DOUBLE PRECISION (A-H,O-Z), INTEGER (I-N)
      CHARACTER*4 ITEXT
      COMMON/JU1/IOP1,IOP2,IOP3,IOP4,IOP5,IOP6,TSET,BL,DW,AG,NL,WG,
     *GL,DI,DO,AW,PRECAP,DISPL,SPEPRE,WGLEG,ZPDEF,DTIP,DTOT,HTOP,HSID,
     *HCON,HTIP,HTOT,DMET,VWI,DARMW,HWM,TMIN,TMAX,TSTEP,GAMW,GAMMSO,
     *CI,CD,
     *VH1,VH2,WGV,VL,NPRO,SUO,RK,SUH,PHI,TH,RR,ALFA,
     *ZPD(50),ZPEN,ATOP(50),ADEP(50),ASIDO(50),ASIDT(50),
     *AO,ITR,COST,COSD,CO,COH,
     *FVULT(50),FVSU,FVPHI,FVSQUE,FVPUNC,BTOT,BEFF,FH,
     *FTOT(20,4,10),OMTOT(20,4,10),RLAR(20,4,10),FPRE(20,10),
     *FIVECT(12,22),OVTMP(20,10),RLARP(20,10),SPRNR,
     *PI,G,MSTART,ITSTRT,FHWIND
C
      COMMON/JU4/POIRA1,POIRA2,GMOD1,GMOD2,FVACT,STFKZG(4),
     *STFKZI(4),STFKXI(4),STFKRI(4)
C
      COMMON/JU5/PSTEEL,E,EIB,EAB,EI(4),EA(4),GA(4),STFKZ(4),
     *STFKX(4),STFKR(4),STF(12,12),STF1(6,6),STF2(6,6),STF3(6,6)
C
      COMMON/JU6/FVULTI
C
      COMMON/JU7/ZPFIN
C
      COMMON/JU8/ZP(50),IPEN
C
      SAVE   /JU1/,/JU2/,/JU3/,/JU4/,/JU5/,/JU6/,/JU7/
      DIMENSION FIX(3,12,22),DISP(12,22),FORCE(12,22),
     *SFS(2,22),SFB(2,22),FZ(4),TOTFOR(12,22),TDISP(12,22)
C     DEFINISION OF PI
      PI=4.0*ATAN(1.0000)
C
C     READ AND PRINT INPUT DATA
C
      CALL INPUT
C
C     OPTION: IF IOP1 EQ.0 CALCULATE ENVIROMENTAL FORCES ONLY.
C             IF IOP1 EQ.1 CALCULATE ENVIROMENTAL FORCES AND THEN
C             REST OF PROGRAM - TSET=TIME FOR WAVE PERIOD MUST BEE
```

```
C               SPECIFIED.
C
      IF(IOP1.EQ.1) GO TO 1000
C     CALCULATE ENVIROMENTAL LOADS.
      CALL ENVIRO(OMMAX,OMMIN,FHWAVE,OMWIND)
      STOP
 1000 CONTINUE
      TMAX=TSET
      TMIN=TSET
      TSTEP=1.0
C
C     CALCULATE ENVIROMENTAL LOADS
C
      CALL ENVIRO(OMMAX,OMMIN,FHWAVE,OMWIND)
C
C     CALCULATE RESULTING ENVIROMENTAL FORCES
C
      IF(IOP2.EQ.0)OMPREI=OMMAX+OMWIND
      IF(IOP2.EQ.1)OMPREI=OMMIN+OMWIND
      IPEN=1
      ZP(IPEN)=0.0
 1020 CONTINUE
C
C     CALCULATE PENETRATION DEPTH AND TRANSFORM OVERTURNING
C     MOMENTS TO CORRECT SPUDCANDEPTH.
C
      IF(ZP(IPEN).GT.ZPDEF) GO TO 1040
      CALL PENETR(OMPREI,OMPPEN)
 1040 CONTINUE
C
C     CALCULATE NECESSARY PRELOAD.
C
      CALL PREL(WG,WGV,NL,BL,PRECAP,IOP5,OMPPEN,FVMAX)
C
C     CHECK OVERTURNING STABILITY AND NECESSARY MIN VARIABLE LOAD
C
      CALL OVSTAB(WG,GL,BL,DTOT,WGV,VL,OMPPEN,IOP5)
C
C     CALCULATE BEARING CAPASITY FOR ACTUAL PENETRATION.
C
      CALL PREPEN
C
C     PRINT:
C            PENETRATION DEPTH,
C            NECESSARY PRELOAD,
C            TOTAL WEIGHT,
C            AND BEARING CAPASITY.
C
      IF(ZP(IPEN).GT.ZPDEF) GO TO 1025
      GO TO 1020
C
C     IF PRELOAD IS SPECIFIED SET PRELOADING WEIGHT = SPECIFIED PRELOAD
C     + STRUCTURE WEIGHT.
C     IF PRELOAD IS NOT SPECIFIED SET PRELOAD WEIGHT = PRELOAD CAPASITY
C     + STRUCTURE WEIGHT.
C
 1025 CONTINUE
      IF(SPEPRE.GT.0.0) GO TO 1060
      GO TO 1080
 1060 CONTINUE
      ACTPRE=(SPEPRE+WG+WGV)
```

```
      GO TO 1100
 1080 CONTINUE
      ACTPRE=(PRECAP+WG+WGV)
 1100 CONTINUE
      FVACT=ACTPRE/NL
C
C     CALCULATE ACTUAL PENETRATION BY CALLING AN "INTERPOLASJONS"-ROUTINE
C
      CALL INPOL(FVACT,FVULT,ZPDEF,INPO,ZP)
C
C     SET OPTION FOR PRINT FROM PENETRATION ROUTINES EQUAL TO CIRO.
C
      IOP5=0
C
C     CALL ITERATIVE ROUTINE TO CORRECT THE PENETRATION
C     DEPTH SO PRELOAD EQUAL BEARING CAPACITY WITH AN ACCURANCY
C     OF 2 PERCENT.
C
      CALL ZPCORR(FVACT,FVULT,INPO)
C
C     WHEN PENETRATION ROUTINE IS FINISHED SET PENETRATION DEPTH
C
C      IPEN=IPEN+1
C
C     SET IOP5=1 AGAIN
C
      IOP5=1
C
C     TRANSFORM OVERTURNING MOMENT TO CORRECT PENETRATION DEPTH.
C
      OMPPEN=OMPREI*(ZPFIN+DW)/DW
C
C     CHECK OVERTURNING STABILITY AND NECESSARY MIN VARIABLE LOAD
C
      CALL OVSTAB(WG,GL,BL,DTOT,WGV,VL,OMPPEN,IOP5)
C
C     CALCULATION OF NECESSARY LEGLENGTH AND CHECK AGAINST GIVEN LEGLENGTH.
C
      CALL LEGLEN(DW,AG,ZPFIN,DISPL)
C
C     OPERATION ROUTINE
C
C
C     CALCULATE INITIAL VALUES FOR SOILSPRINGS
C
      CALL INITAL(FZ)
C
C
C     CALCULATE LOADVECTOR FOR STRUCTURE
C
      CALL LOVECT(FIX)
C
C     SET OPTION FOR PRINT FROM PENETRATION ROUTINES EQUAL TO CIRO.
C
      IOP5=0
C
C     CALCULATE STIFFNESSMATRIX FOR STRUCTURE AT ACTUAL PENETRATION
C     AND LOAD.
C
C
C     CALCULATE DISPLACEMENTS IN STRUCTURE, FINAL FORCE AND STRESSLEVEL
```

```
C       AND PRINT RESULTS.
C
C       LOADINGROUTINE:GRAVITY LOAD IS INCLUDED IN INITAL.
C       FIRST THE FOUNDATION IS EXPOSED TO FULL WINDLOAD (INCLUDED GRAVITY)
C       IN TWO STEPS.
C       THEN THE STRUCTURE IS LOADED WITH SPECIFIED WAVE LOADS,
C       STARTING WITH WAVESTEP CREATING LOWEST ABS(OVERTURNING MOMENT.)
C
C       MLOAD COUNTS NUMBER OF LOADINGS
C
        MLOAD=1
 1170   CONTINUE
        IF(MLOAD.LE.2) GO TO 1200
        IF(MLOAD.GE.4) GO TO 1185
        M=MSTART+2
        IF(M.GT.22) GO TO 1190
        GO TO 1210
 1185   CONTINUE
        M=M+1
        IF(M.GT.22) GO TO 1190
        GO TO 1210
 1190   CONTINUE
        M=M-20
        GO TO 1210
 1200   CONTINUE
        M=MLOAD
 1210   CONTINUE
        CALL STFMTX(MLOAD,STFKZG,STFKZI,STFKXI,STFKRI)
        CALL FIFOR(M,MLOAD,MSTART,GAMMSO,STF,STF1,STF2,STF3,STFKZG,
       .STFKZI,STFKXI,STFKRI,PI,HTIP,HCON,HSID,DTIP,DTOT,NPRO,SUO,
       .RK,SUH,PHI,ZP,IPEN,TH,RR,NL,AO,BTOT,FIX,FZ,FH,DISP,FORCE,
       .SFS,SFB,FIVECT,TOTFOR,TDISP,STFKZ,STFKX,STFKR)
        CALL PRINT(IOP6,M,MLOAD,DISP,FORCE,SFS,SFB,
       .TOTFOR,TDISP,IOP5)
        MLOAD=MLOAD+1
        IF(MLOAD.LE.22) GO TO 1170
C        EVALUATION
C        PRINT
        END
```

```
      SUBROUTINE BHPHI
C
C     PURPOSE:
C     THIS ROUTINE CALCULATE THE BEARING CAPASITY OF A SOIL CONSISTING OF
C     FRICTION-MATERIAL.
C     METHOD:
C     BRINCH-HANSENS FORMULA
C
      IMPLICIT DOUBLE PRECISION (A-H,O-Z), INTEGER (I-N)
      COMMON/JU1/IOP1,IOP2,IOP3,IOP4,IOP5,IOP6,TSET,BL,DW,AG,NL,WG,
     .GL,OI,OD,AW,PRECAP,DISPL,SPEPRE,WGLEG,ZPDEF,DTIP,DTOT,HTOP,HSID,
     .HCON,HTIP,HTOT,DMET,VWI,OARMW,HWM,TMIN,TMAX,TSTEP,GAMW,GAMMSO,
     .CI,CO,
     .VH1,VH2,WGV,VL,NPRO,SUO,RK,SUH,PHI,TH,RR,ALFA,
     .ZPD(50),ZPEN,ATOP(50),ADEP(50),ASIDD(50),ASIDT(50),
     .AC,ITR,COST,COSD,CO,COH,
     .FVULT(50),FVSU,FVPHI,FVSGUE,FVPUNC,BTOT,BEFF,FH,
     .FTOT(20,4,10),OMTOT(20,4,10),RLAR(20,4,10),FPRE(20,10),
     .FIVECT(12,22),OVTMP(20,10),RLARP(20,10),SPRNR,
     .PI,G,MSTART,ITSTRT,FHWIND
      COMMON/JU8/ZP(50),IPEN
      IF(NPRO.EQ.1) GO TO 10
      IF(NPRO.EQ.3) GO TO 20
      IF(NPRO.LT.1) GO TO 99
      IF(NPRO.GT.3) GO TO 99
      IF(PHI.LE.0.0)GO TO 200
   10 CONTINUE
      GAMS=GAMMSO
      GO TO 90
   20 CONTINUE
      GAMS=GAMMSO
   90 CONTINUE
  100 CONTINUE
      FV=FVPHI
C     CALCULATION OF TRANSF-FACT FROM DEG TO RAD
      RAKO=PI/180.0
      CNQ=TAN((45.0+PHI/2.0)*RAKO)**2.0*(EXP(PI*TAN(PHI*RAKO)))
      CNGAMM=2.0*(CNQ+1.0)*TAN(PHI*RAKO)
      IF((FV+AO*COH).EQ.0.0) GO TO 1000
      CIGAMM=(1.0-(0.7*FH)/(FV+AO*COH*1.0/TAN(PHI*RAKO)))**5.0
      CIQ=(1.0-(0.5*FH)/(FV+AO*COH*1.0/TAN(PHI*RAKO)))**5.0
      GO TO 1100
 1000 CONTINUE
      CIGAMM=1.0
      CIQ=1.0
 1100 CONTINUE
      SGAMMA=1.0-0.4*(CIGAMM*BEFF/BTOT)
      SQ=(1.0+CIQ*(BEFF/BTOT)*SIN(PHI*RAKO))
      QD=(1.0/2.0)*(GAMS)*BEFF*CNGAMM*SGAMMA*CIGAMM
     .+(COH*1.0/TAN(PHI*RAKO)+ZP(IPEN)*GAMS)*CNQ*SQ*CIQ
     .-COH*1.0/TAN(PHI*RAKO)
      FVPHI=QD*AO
      DIFF=ABS((FV-FVPHI)/FVPHI)
      IF(DIFF.GT.0.05) GO TO 100
      RETURN
  200 CONTINUE
      WRITE(6,9000)
   99 WRITE(6,9100)
 9100 FORMAT(5X,' * * * ERROR IN SOIL PROFILE NUMBER * * * ')
 9000 FORMAT(5X,'TAN PHI LESS THAN CIRO=ERROR IN INPUT DATA')
      RETURN
      END
```

```
      SUBROUTINE BHSQUE(NPRO,BTOT,ZPEN,SUO,RK,TH,FH,GAMMSO,
     *AO,ASIDT,HTIP,HCON,HSID,IOP5,FVSQUE)
C
C     PURPOSE:
C     TO CALCULATE THE INCREASE IN BEARING CAPACITY IN A
C     SOFT LAYER OVERLAYING A HARD LAYER DUE TO THE SQUEEZING
C     EFFECT THAT OCCUR WHEN THE FOOTING APPROACHES THE HARD LAYER.
C
C     METHOD:
C     DESCRIBED IN THE PP.2 REPORT IN THE PROJECT CALLED:
C     "FOUNDATION STABILITY OF JACKUP PLATFORMS."
C
C     INPUT ARGUMENTS:
C     NPRO   = SOIL PROFILE NUMBER.
C     BTOT   = TOTAL BREADTH.
C     ZPEN   = PENETRATION DEPTH.
C     SUO    = UNDRAINED SHEAR STRENGTH IN TOP OF LAYER.
C     RK     = FACTOR GIVING THE RATE OF INCREASE IN
C              UNDRAINED SHEAR STRENGTH WITH DEPTH.
C     TH     = THICKNESS OF TOP LAYER.
C     FH     = RESULTING HORIZONTAL FORCE ON FOOTING.
C     GAMMSO = SUBMERGED UNIT WEIGHT OF SOIL.
C     AO     = EXPOSED FOUNDATION AREA.
C     ASIDT  = EXPOSED AREA FOR SIDE FRICTION IN SOIL.
C     HTIP   = HEIGHT OF SPUD-CAN TIP.
C     HCON   = HEIGHT OF SPUD-CAN CONE.
C     HSID   = HEIGHT OF SPUD-CAN SIDE.
C     IOP5   = PRINT SWITCH FOR PENETRATION DATA.
C              IF IOP5=0 ; NO PRINT OF PENETRATION DATA.
C              IF IOP5=1 ; PRINTS PENETRATION DATA.
C
C     INTERNAL VARIABLES:
C     ATOT   = TOTAL FOUNDATION AREA.
C     COH    = UNDRAINED SHEAR STRENGTH USED IN BEARING CAPACITY
C              CALCULATION.
C     SU     = UNDRAINED SHEAR STRENGTH USED IN CALCULATION OF
C              SIDEFRICTION.
C     THICK  = THICKNESS OF LAYER BETWEEN SPUD-CAN TIP AND DEEP
C              LAYER.
C     CISA   = LOAD INCLINATION FACTOR USED IN BRINCH-HANSENS
C              BEARING CAPACITY FORMULA (B.H.)
C     SCA    = SHAPE FACTOR USED IN B.H.
C     DCA    = DEPTH FACTOR USED IN B.H.
C     CONC   = NC FACTOR USED IN B.H. CORRECTED FOR SQUEEZING.
C     ZSFRIC = DEPTH FOR CALCULATING AVERAGE DEPTH FOR SIDE FRICTION
C     FVBASE = RESULTING BEARING CAPACITY OF BASE.
C     FVSIDE = RESULTING BEARING CAPACITY OF SIDE FRICTION.
C
C     OUTPUT ARGUMENTS:
C     FVSQUE = RESULTING VERTICAL BEARING CAPACITY CORRECTED FOR
C              SQUEEZING EFFECT.
C
      IMPLICIT DOUBLE PRECISION (A-H,O-Z),INTEGER (I-N)
C
C     THIS ROUTINE SHOULD ONLY BEE USED FOR SOIL PROFILE NR.1
C
      IF(NPRO.NE.1) GO TO 900
      ATOT=BTOT*BTOT
      COH=(SUO+ZPEN*RK)
      THICK=TH-ZPEN
      IF(THICK.LT.0.0) GO TO 960
```

```
      IF(THICK.LT.0.001) THICK=0.01
C
C     TO AVOID NEGATIVE VALUE UNDER SQUARE ROOT.
C
      IF(ABS(FH/(ATOT*COH)).GE.1.0) GO TO 6000
      CICA=0.5-0.5*SQRT(1.0-ABS(FH/(ATOT*COH)))
      GO TO 6100
 6000 CONTINUE
      CICA=0.5
 6100 CONTINUE
C
C     NO AREA REDUCTION.
C
      IF(ZPEN.GT.(HTIP+HCON+HSID)) GO TO 6200
      ZSFRIC=ZPEN-HTIP-HCON-0.5*(ZPEN-HTIP-HCON)
      GO TO 6300
 6200 CONTINUE
      ZSFRIC=ZPEN-HTIP-HCON-HSID/2.0
 6300 CONTINUE
      SU=SUC+RK*ZSFRIC
      FVSIDE=ASIDT*SU
      SCA=0.2*(1.0-2.0*CICA)
      DCA=0.3*ATAN(ZPEN/BTOT)
      CONC=4.0+2.0/3.0*BTOT/(THICK)
      FVBASE=A0*CONC*COH*(1.0+SCA+DCA-CICA)+A0*GAMMSO*ZPEN
      FVSQUE=FVBASE+FVSIDE
      IF(IOP5.EQ.0) GO TO 6500
      WRITE(6,1000)FVSQUE
 6500 CONTINUE
 1000 FORMAT(' BEARING CAPACITY VERTICAL, FVSQUE            =',E10.3)
      GO TO 950
  900 CONTINUE
      WRITE(6,9000)
 9000 FORMAT(' WARNING! THIS ROUTINE SHOULD ONLY BEE USED WHEN A '
     1      ' SOFT SOILLAYER IS OVERLAYING A HARDER ONE')
  950 CONTINUE
      RETURN
  960 CONTINUE
      FVSQUE=0.0
      RETURN
      END
```

```
      SUBROUTINE BHSU
C
C     PURPOSE:
C     THIS ROUTINE CALCULATE THE BEARING CAPACITY OF SOIL
C     CONSISTENT OF SU-MATERIAL
C
C     METHOD:BRINCH-HANSENS FORMULA
C
C     INPUT ARGUMENTS:
C     INTERNAL PARAMETERS:
C     OUTPUT ARGUMENTS:
C     PRINT SWITCH:
C
      IMPLICIT DOUBLE PRECISION(A-H,O-Z),INTEGER(I-N)
      COMMON/JU1/IOP1,IOP2,IOP3,IOP4,IOP5,IOP6,TSIT,BL,DW,AG,NL,WG,
     *GL,DI,DO,AW,PRECAP,DISPL,SPEPRE,WGLEG,ZPDEF,DTIP,DTOT,HTOP,HSID,
     *HCON,HTIP,HTOT,DMET,VWI,DARMW,HWM,TMIN,TMAX,TSTEP,GAMW,GAMMSO,
     *CI,CD,
     *VH1,VH2,WGV,VL,NPRO,SUO,RK,SUH,PHI,TH,RR,ALFA,
     *ZPO(50),ZPEN,ATOP(50),ADEP(50),ASIDD(50),ASIDT(50),
     *AO,ITR,COST,COSD,CC,COH,
     *FVULT(50),FVSU,FVPHI,FVSQUE,FVPUNC,BTOT,EEFF,FH,
     *FTOT(20,4,10),OMTOT(20,4,10),RLAR(20,4,10),FPRE(20,10),
     *FIVECT(12,22),OVTMP(20,10),RLARP(20,10),SPRNR,
     *PI,G,MSTART,ITSTRT,FHWIND
      COMMON/JU8/ZP(50),IPEN
      CALL TRANSF
      BEFF=BTOT
      IF(FH.GE.(AO*CO)) GO TO 10
      CICA=0.5-0.5*SQRT(1.0-FH/(AO*CO))
      GO TO 20
   10 CONTINUE
      CICA=0.5
   20 CONTINUE
      SCA=0.2*(1.0-2.0*CICA)*BEFF/BTOT
      ZPDCA=ZP(IPEN)
      IF(ZP(IPEN).GE.(HTIP+HCON))ZPDCA=ZP(IPEN)-HTIP-HCON
      DCA=0.3*ATAN(ZPDCA/BEFF)
      RNC=5.14
C
C     BRINCH-HANSENS FORMULA FOR SUMATERIAL
C
      QSU=RNC*CO*(1.0+SCA+DCA-CICA)+GAMMSO*ZPDCA
      FVSU=QSU*AO+ASIDT(IPEN)*COST*RR+ASIDD(IPEN)*COSD*RR
      IF (FH.LT.0.001) GO TO 1000
C      WRITE(6,8000)ZP(IPEN),FVSU
      RETURN
 1000 CONTINUE
C      WRITE(6,8010)ZP(IPEN),FVSU
 8000 FORMAT(' PENETRATION, ZP                                  =',E10.3/
    1       ' BEARING CAPACITY VERTICAL, FVULT                  =',E10.3)
 8010 FORMAT(' PENETRATION, ZP                                  =',E10.3/
    1       ' BEARING CAPAC. OF PLAIN VERT. LOADING, FVULT=',E10.3)
      RETURN
      END
```

```
      SUBROUTINE ENVIRO (OMMAX,OMMIN,FHWAVE,OMWIND)
C***********************************************************************
C*                          ROUTINE ENVIRO                             *
C***********************************************************************
C
C     PURPOSE:CALCULATE TOTAL HORIZONTAL FORCE AND OVERTURNING MOMENT
C     RELATIVE TO SEA BED, DUE TO WAVE CURRENT AND WIND
C     INPUT ARGUMENTS:
C     BL= BREATH BETWEEN LEGS.
C     CD= DRAG COEFFECIENT FOR FLOW IN THE DIRECTION OF FD AS DEFINED IN
C         DNV-RULES.
C     CI= INERTIA COEFFIC FOR ACCELERATION IN THE DIRECTION OF FI AS
C         DEFINED IN DNV-RULES.
C     DI= DIAMETER OF INERTIA.
C     DD= DIAMETER OF DRAG.
C     DW= DEPTH OF WATER.
C     NL= NUMBER OF LEGS. NL= 3 OR NL= 4.
C     TMAX= MAXIMUM VALUE OF WAVE PERIOD TO BE EXAMINED.
C     TMIN= MINIMUM VALUE OF WAVE PERIOD TO BE EXAMINED.
C     TSTEP= INCREMENT IN VALUE OF WAVE PERIOD FROM TMIN TO TMAX.
C     VH1= THE RESULTING VELOCITY, AT THE STILL WATER LEVEL, OF CURRENTS
C          NOT GENERATED AT THE SURFACE OR IN THE UPPER LAYERS OF WATER
C          (E.G. TIDAL CURRENT)
C     VH2= THE RESULTING VELOCITY, AT THE STILL WATER LEVEL, OF CURRENT
C          GENERATED AT THE SURFACE OR IN THE UPPER LAYERS OF WATER
C          (E.G. WIND GENERATED CURRENT)
C
C
C

      IMPLICIT DOUBLE PRECISION (A-H,O-Z),INTEGER(I-N)
      CHARACTER*4 ITEXT
      DIMENSION X(4),TCRIT(4),T(15),TI(15),WS(15),WL(15),HW(15),
     *FHSUM(20,10),OMSUM(20,10)
      COMMON/JU1/IOP1,IOP2,IOP3,IOP4,IOP5,IOP6,TSET,BL,DW,AG,NL,WG,
     *GL,DI,DD,AW,PRECAP,DISPL,SPEPRE,WGLEG,ZPCEF,DTIP,DTOT,HTOP,HSID,
     *HCON,HTIP,HTOT,DMET,VWI,DARMW,HWM,TMIN,TMAX,TSTEP,GAMW,GAMMSO,
     *CI,CD,
     *VH1,VH2,WGV,VL,NPRO,SUD,RK,SUH,PHI,TH,RR,ALFA,
     *ZPD(50),ZPEN,ATOP(50),ADEP(50),ASIDD(50),ASIDT(50),
     *AO,ITR,COST,COSD,CO,COH,
     *FVULT(50),FVSU,FVPHI,FVSQUE,FVPUNC,BTOT,BEFF,FH,
     *FTOT(20,4,10),OMTOT(20,4,10),RLAR(20,4,10),FPRE(20,10),
     *FIVECT(12,22),OVTMP(20,10),RLARP(20,10),SPRNR,
     *PI,G,MSTART,ITSTRT,FHWIND
      COMMON/JU2/ITEXT(20)
      COMMON/JU8/ZP(50),IPEN
C
C     PRINTSWITCH
C
      IF(IOP4.EQ.0) GO TO 900
C
C     NEW PAGE
C
      WRITE(6,8550)
  900 CONTINUE
C
C     CALCULATE WAVE PERIOD
C
      TN=(TMAX-TMIN)/TSTEP
      ITM=INT(TN)+1
```

```
      T(1)=TMIN
C
C     CALCULATE WAVELENGHT
C
      DO 1110 IT=1,ITM
      IF(IT.EQ.1) GO TO 1000
      T(IT)=T(IT-1)+TSTEP
 1000 CONTINUE
      WL1=G*T(IT)**2.0/(2.0*PI)
 1010 CONTINUE
      YHYP=2.0*PI*DW/WL1
      HY=TANH(YHYP)
      WL2=(G*T(IT)**2.0/(2.0*PI))*HY
      IF(ABS(WL2-WL1).LE.0.01) GO TO 1020
      WL1=WL2
      GOTO 1010
 1020 CONTINUE
      WL(IT)=WL2
C
C     CALCULATE WAVEHEIGHT
C
      IF(T(IT).LT.6.0) GO TO 1024
      HW(IT)=(T(IT)**2.0/(4.5+0.02*(T(IT)**2.0-36.0)))
      GO TO 1026
 1024 CONTINUE
      HW(IT)=(0.22*T(IT)**2.0)
 1026 CONTINUE
      IF(HW(IT).GT.HWM) GO TO 1030
      GO TO 1040
 1030 CONTINUE
C
C     IF CALCULATED WAVEHEIGHT EXCEED GIVEN MAX. WAVEHEIGHT; WARNING IS
C    . GIVEN AND WAVE HEIGHT IS SET TO GIVEN WAVE HEIGHT.
C
      WRITE(6,9000)
      HW(IT)=HWM
 9000 FORMAT(5X,'WARNING! CALCULATED MAXIMUM POSSIBLE WAVEHEIGHT EXCEED
     1 WAVEHEIGHT FROM INPUT DATA.')
 1040 CONTINUE
C
C     CALCULATE STEEPNES OF WAVE.
C
      WS(IT)=HW(IT)/WL(IT)
C
C     DEVIDE THE WAVE IN 20 STEPS. CALCULATE REAL TIME FOR WAVE
C     MOVEMENT.
      DO 1100 M=1,20
      TI(IT)=(M-1)*T(IT)/20
      IF(NL.EQ.3) GOTO  1050
      IF(NL.EQ.4) GOTO  1060
      WRITE(6,9100)
 9100 FORMAT(5X,'THIS ROUTINE IS MADE FOR JACKUPS WITH 3 OR 4 LEGS')
 1050 CONTINUE
C
C     IF IOP2=0 STRUCTURE EXPOSED TO WAVES PROPAGATING IN POSITIVE
C     X-DIRECTION.
C     IF IOP2=1 STRUCTURE EXPOSED TO WAVES PROPAGATING IN NEGATIVE
C     X-DIRECTION.
C
      IF(IOP2.EQ.1) GO TO 1055
      X(1)=-2./3.*BL
```

```
      X(2)= 1./3.*BL
      X(3)=X(2)
      GO TO 1070
 1055 CONTINUE
      X(1)=1./3.*BL
      X(2)=-2./3.*BL
      X(3)=X(2)
      GO TO 1070
 1060 CONTINUE
      X(1)=-1./2.*BL
      X(2)= 1./2.*BL
      X(3)=X(1)
      X(4)=X(2)
 1070 CONTINUE
C
C     CALL SUBROUTINE WAVE FOR EACH LEG.
C
      LN=1
      CALL WAVE (IOP1,IOP2,X(1),DW,M,LN,HW(IT),T(IT),TI(IT),IT,CD,CI,
     .DD,DI,WL(IT),VH1,VH2,PI,G,GAMW,FTOT(M,LN,IT),OMTOT(M,LN,IT),
     .RLAR(M,LN,IT))
      LN=2
      CALL WAVE (IOP1,IOP2,X(2),DW,M,LN,HW(IT),T(IT),TI(IT),IT,CD,CI,
     .DD,DI,WL(IT),VH1,VH2,PI,G,GAMW,FTOT(M,LN,IT),OMTOT(M,LN,IT),
     .RLAR(M,LN,IT))
      LN=3
      CALL WAVE (IOP1,IOP2,X(3),DW,M,LN,HW(IT),T(IT),TI(IT),IT,CD,CI,
     .DD,DI,WL(IT),VH1,VH2,PI,G,GAMW,FTOT(M,LN,IT),OMTOT(M,LN,IT),
     .RLAR(M,LN,IT))
      IF(NL.EQ.3) GO TO 1080
      LN=4
      CALL WAVE (IOP1,IOP2,X(4),DW,M,LN,HW(IT),T(IT),TI(IT),IT,CD,CI,
     .DD,DI,WL(IT),VH1,VH2,PI,G,GAMW,FTOT(M,LN,IT),OMTOT(M,LN,IT),
     .RLAR(M,LN,IT))
      GO TO 1090
 1080 CONTINUE
      FTOT(M,4,IT)=0.0
      OMTOT(M,4,IT)=0.0
      RLAR(M,4,IT)=0.0
 1090 CONTINUE
 1100 CONTINUE
 1110 CONTINUE
C
C     PRINT TABLE WITH RESULTS.
C
      DO 1190 IT=1,ITM
C
C     PRINTSWITCH:
C
      IF(IOP4.EQ.0) GO TO 1115
      WRITE(6,8560)ITEXT
      WRITE(6,8570)DW,HW(IT),WS(IT),T(IT),WL(IT)
      WRITE(6,8580)
 1115 CONTINUE
      DO 1120 M=1,20
      FHSUM(M,IT)=FTOT(M,1,IT)+FTOT(M,2,IT)+FTOT(M,3,IT)+FTOT(M,4,IT)
      OMSUM(M,IT)=OMTOT(M,1,IT)+OMTOT(M,2,IT)+OMTOT(M,3,IT)+
     .OMTOT(M,4,IT)
      IF(IOP4.EQ.0) GO TO 1120
      WRITE(6,8590)M,(FTOT(M,LN,IT),LN=1,4),FHSUM(M,IT),
     .(OMTOT(M,LN,IT),LN=1,4),OMSUM(M,IT)
```

```
 1  CONTINUE
C
C     CHOOSE MAX,MIN HORIZONTAL FORCE/ OVERTURNING MOMENTS WITH
C     CORRESPONING VALUE FOR OVERTURNING MOMENT/ HORIZONTAL FORCE.
C
      DO 1170 M=1,20
      IF(FHMAX.GE.FHSUM(M,IT))GO TO 1130
      FHMAX=FHSUM(M,IT)
      MAX=M
      ITMAX=IT
 1130 CONTINUE
      IF(FHMIN.LE.FHSUM(M,IT))GO TO 1140
      FHMIN=FHSUM(M,IT)
      MIN=M
      ITMIN=IT
 1140 CONTINUE
      IF(OMMAX.GE.OMSUM(M,IT))GO TO 1150
      OMMAX=OMSUM(M,IT)
      MAXO=M
      ITMAXO=IT
 1150 CONTINUE
      IF(OMMIN.LE.OMSUM(M,IT)) GO TO 1155
      OMMIN=OMSUM(M,IT)
      MINO=M
      ITMINO=IT
 1155 CONTINUE
C
C     START EVALUATION OF MIN OVERTURNING MOMENT
C
      IF(M.EQ.1.AND.IT.EQ.1)OM=OMSUM(M,1)
      IF(ABS(OM).LT.ABS(OMSUM(M,IT))) GO TO 1160
      OM=OMSUM(M,IT)
      MSTART=M
      ITSTRT=IT
 1160 CONTINUE
 1170 CONTINUE
C
C     SET ACTUAL VALUE ON FHWAVE
C
      FHWAVE=FHSUM(MAXO,ITMAXO)
C
C     PRINT SWITCH
C
      IF(IOP4.EQ.0) GO TO 1180
      TCRIT(1)=ITMAX*TSTEP+TMIN-TSTEP
      TCRIT(2)=ITMIN*TSTEP+TMIN-TSTEP
      TCRIT(3)=ITMAXO*TSTEP+TMIN-TSTEP
      TCRIT(4)=ITMINO*TSTEP+TMIN-TSTEP
      WRITE(6,8600)FHMAX,MAX,OMSUM(MAX,ITMAX),TCRIT(1)
      WRITE(6,8610)FHMIN,MIN,OMSUM(MIN,ITMIN),TCRIT(2)
      WRITE(6,8620)OMMAX,MAXO,FHSUM(MAXO,ITMAXO),TCRIT(3)
      WRITE(6,8630)OMMIN,MINO,FHSUM(MINO,ITMINO),TCRIT(4)
 1180 CONTINUE
C
C     CALCULATE WIND FORCE, LEVERARM WIND AND OVERTURNING
C     MOMENT FOR WIND.
C
      FHWIND=0.293*0.01*GAMW/G*AW*VWI*ABS(VWI)
      IF(IOP2.EQ.1)FHWIND=-ABS(FHWIND)
      ARMW=DARMW+AG+DW
      OMWIND=FHWIND*ARMW
```

```
C
C     PRINT SWITCH
C
      IF(IOP4.EQ.0) GO TO 1190
      WRITE(6,8640)
      WRITE(6,8650)FHWIND,ARMW,OMWIND
      WRITE(6,8550)
 1190 CONTINUE
C     NEW PAGE
 8550 FORMAT('1')
C     PRINT HEADING
 8560 FORMAT(1X,20A4)
C     PRINT WAVEDATA
 8570 FORMAT(' WATER DEPTH                                              =',E10.3/
     1       ' WAVE HEIGHT                                              =',E10.3/
     2       ' WAVE STEEPNESS                                           =',F10.3/
     3       ' WAVE PERIOD                                              =',F10.3/
     4       ' WAVE LENGTH                                              =',E10.3/)
C     PRINT HEADING FOR TABLE
 8580 FORMAT(1X,///,1X,'SUMMARY OF WAVE FORCES',//,1X,'TIME STEP',19X,
     .'HORIZONTAL  FORCES',37X,'OVERTURNING  MOMENTS',//,3X,'NO.',5X,
     .'LEG 1',7X,'LEG 2',7X,'LEG 3',7X,'LEG 4',7X,'TOTAL',13X,'LEG 1',
     .7X,'LEG 2',7X,'LEG 3',7X,'LEG 4',7X,'TOTAL',/)
C     PRINT VALUES IN TABLE
 8590 FORMAT(2X,I3,1X,E11.3,1X,E11.3,1X,E11.3,1X,E11.3,3X,E11.3,5X,
     .E11.3,1X,E11.3,1X,E11.3,1X,E11.3,3X,E11.3)
C     PRINT SPECIAL EVALUATED VALUES
 8600 FORMAT(/////,1X,'MAX POSITIV HORIZONTAL FORCE=',E11.3,3X,
     1'AT STEP NO.',I3
     2,10X,'CORRESPONDING OVERTURNING MOMENT=',E11.3,/
     3,1X,'WAVE PERIOD FOR MAX POSITIV HORIZONTAL FORCE =',F4.1)
 8610 FORMAT(/,1X,'MAX NEGATIVE HORIZONTAL FORCE=',E11.3,3X,
     1'AT STEP NO.',I3
     2,10X,'CORRESPONDING OVERTURNING MOMENT=',E11.3,/
     3,1X,'WAVE PERIOD FOR MAX NEGATIVE HORIZONTAL FORCE =',F4.1)
 8620 FORMAT(/,1X,'MAX POSITIVE OVERTURNING MOMENT=',E11.3,3X,
     1'AT STEP NO.',I3
     2,10X,'CORRESPONDING HORIZONTAL FORCE=',E11.3,/
     3,1X,'WAVE PERIOD FOR MAX POSITIVE OVERTURNING MOMENT =',F4.1)
 8630 FORMAT(/,1X,'MAX NEGATIVE OVERTURNING MOMENT=',E11.3,3X,
     1'AT STEP NO.',I3
     2,10X,'CORRESPONDING HORIZONTAL FORCE=',E11.3,/
     3,1X,'WAVE PERIOD FOR MAX NEGATIVE OVERTURNING MOMENT =',F4.1)
 8640 FORMAT(//,10X,'WIND FORCE:')
 8650 FORMAT(' MAX HORIZONTAL WINDFORCE                                 =',E10.3/
     1       ' LEVERARM                                                 =',F10.3/
     2       ' CORRESPONDING OVERTURNING MOMENT                          '      /
     3       ' RELATED TO MUDLINE                                       =',E10.3/)
       RETURN
       END
```

```
      SUBROUTINE FHZCAP(PI,HTIP,HCON,HSID,DTIP,DTOT,NPRO,SUO,
     *RK,SUH,PHI,TH,RR,GAMMSO,FVERT,AO,FHSIDE,FHULT)
C
C     PURPOSE:  THIS ROUTINE CALCULATE THE LATERAL CAPACITY
C               OF A SPUD-CAN  DEFINED BY THE PARAMETERS IN THE
C               ARGUMENTLIST ABOVE.THE SPUD-CAN IS PLACED AT THE
C               FINAL PENETRATION DEPTH IN A GIVEN SOIL PROFILE.
C
C     METHOD:   DESCIBED IN FINAL REPORT FOR PP2 IN THE PROJECT
C               CALLED: " FOUNDATION STABILITY OF JACK-UP PLATFORMS.
C
C     INPUT ARGUMENT:
C     PI       = PI(3.14).
C     HTIP     = VERTICAL HEIGHT OF SPUDCANTIP.
C     HCON     = VERTICAL HEIGHT OF SPUDCANCON.
C     HSID     = VERTICAL HEIGHT OF SPUDCANSIDE.
C     DTIP     = TIP DIAMETER OF SPUDCAN.
C     DTOT     = TOTAL DIAMETER OF SPUDCAN.
C     FVERT    = VERTICAL LOAD ON ACTUAL LEG.
C     NPRO     = SOIL PROFILE TYPE.
C     SUO      = UNDRAINED SHEAR STRENGTH OF SOFT CLAY LAYER
C                AT TOP OF LAYER.
C     RK       = SHEAR STRENGTH GRADIENT, I.E. INCREASE IN
C                UNDRAINED SHEAR STRENGTH PER UNIT DISTANCE
C                BELOW TOP OF LAYER.
C     SUH      = UNDRAINED SHEAR STRENGTH OF HARD CLAY LAYER
C                OR COHESION OF SAND LAYER.
C     PHI      = FRICTION ANGLE OF SAND LAYER.
C     TH       = THICKNESS OF TOP LAYER.
C     RR       = ROUGHNESS RATIO BETWEEN STEEL AND SOIL.
C     GAMMSO   = TOTAL UNIT WEIGHT OF SOIL.
C
C     INTERNAL PARAMETERS:
C     ASIDET   = AREA EXPOSED TO PASSIVE EARTHPRESSURE IN TOP LAYER.
C     ASIDED   = AREA EXPOSED TO PASSIVE EARTHPRESSURE IN DEEP LAYER.
C     ABT      = BASE AREA EXPOSED TO PASSIVE EARTHPRESSURE
C                IN TOP LAYER.
C     ABD      = BASE AREA EXPOSED TO PASSIVE EARTHPRESSURE
C                IN DEEP LAYER.
C     AS(1)    = TOTAL EXPOSED SIDEAREA OF SPUD-CAN IN BOTH LAYERS.
C     AS(2)    = EXPOSED SIDEAREA OF SPUD-CAN IN DEEP LAYER.
C     AB(1)    = TOTAL EXPOSED BASEAREA OF SPUD-CAN IN BOTH LAYERS.
C     AB(2)    = EXPOSED BASEAREA IN DEEP LAYER
C     SUST     = UNDRAINED SHEAR STRENGTH FOR ASIDET.
C     SUSD     = UNDRAINED SHEAR STRENGTH FOR ASIDED.
C     SUBT     = UNDRAINED SHEAR STRENGTH FOR ABT.
C     SUBD     = UNDRAINED SHEAR STRENGTH FOR ABD.
C     RNCT     = BEARING CAPACITY FACTOR TOP LAYER.
C     RNCD     = BEARING CAPACITY FACTOR DEEP LAYER.
C     RNCMAX   = MAXIMUM VALUE FOR RNC-FACTORS.
C     RNQ      = BEARING CAPACITY FACTOR.
C     RKP      = PASSIVE EARTHPRESSURE FACTOR.
C     PU       = LATERAL RESISTANCE PER SQUARE UNIT.
C
C     OUTPUT PARAMETERS:
C     AO       = FOUNDATION AREA.
C                IF ZPFIN .LE. TH   AO = BASE AREA TOP LAYER.
C                IF ZPFIN .GT. TH   AO = BASE AREA DEEP LAYER.
C     FHSIDE   = ULTIMATE LATERAL CAPACITY ON SIDEWALL.
C     FHULT    = ULTIMATE LATERAL CAPACITY.
C
```

```
      IMPLICIT DOUBLE PRECISION (A-H,O-Z), INTEGER (I-N)
      COMMON/JU7/ZPFIN
      DIMENSION R(2),AS(2),AB(2)
C
      RNCMAX=10.0
      PHIRAD=PHI*PI/180.0
      RKP=(1.0+SIN(PHIRAD))/(1.0-SIN(PHIRAD))
      RNQ=(TAN(45.0*PI/180.0+PHIRAD/2.0))**2.0*EXP(PI*TAN(PHIRAD))
C
C     SET ALL GEOMETRICAL SPUD-CAN PARAMETERS EQUAL TO CIRO
C
      ASIDET  = 0.0
      ASIDED  = 0.0
      ABT     = 0.0
      ABD     = 0.0
      SUST    = 0.0
      SUSB    = 0.0
      SUSD    = 0.0
      SUBD    = 0.0
C
C     CALCULATE TOTAL BASE AND SIDE AREA EXPOSED TO SOIL.
C
C     IF SPUD-CAN HAS PENETRATED INTO DEEP LAYER, EXPOSED BASE
C     AND SIDE AREA IN DEEP LAYER, EXPOSED BASE AND SIDE AREA IN
C     DEEP LAYER IS CALCULATED. THEN THE EXPOSED AREA FOR DEEP LAYER IS
C     SUBTRACTED FROM THE TOTAL EXPOSED AREAS.
C
      DO 200 I=1,2
      Z=ZPFIN
      IF(I.EQ.2)Z=ZPFIN-TH
      IF(Z.LE.0.0) GO TO 200
      IF(Z.GT.HTIP) GO TO 20
C
C     SPUD-CAN TIP PARTLY PENETRATED IN LAYER.
C
      R(I)=0.5*(DTIP/HTIP)*Z
      AS(I)=R(I)*Z
      AB(I)=PI*R(I)**2.0
      GO TO 200
   20 CONTINUE
      IF(Z.GT.(HTIP+HCON)) GO TO 30
C
C     SPUD-CAN CONE PARTLY PENETRATED IN LAYER
C
      R(I)=DTIP/2.0+(Z-HTIP)*(DTOT-DTIP)/(2.0*HCON)
      AS(I)=0.5*HTIP*DTIP+0.5*(2.0*R(I)+DTIP)*(Z-HTIP)
      AB(I)=PI*R(I)**2.0
      GO TO 200
C
   30 CONTINUE
      IF(Z.GT.(HTIP+HCON+HSID)) GO TO 40
C
C     SPUD-CAN SIDE-WALL PARTLY PENETRATED IN LAYER.
C
      R(I)=DTOT/2.0
      AS(I)=0.5*HTIP*DTIP+0.5*(DTOT+DTIP)*HCON+DTOT*(Z-HTIP-HCON)
      AB(I)=PI*R(I)**2.0
      GO TO 200
C
   40 CONTINUE
C
```

```
C       SPUD-CAN FULLY PENETRATED IN LAYER
C
        AS(I)=0.5*HTIP*DTIP+0.5*(DTOT+DTIP)*HCON+DTOT*HSID
        AB(I)=PI*R(I)**2.0
C
C       END OF DO-LOOP
C
  200   CONTINUE
C
C       FINAL CALCULATION OF EXPOSED AREAS IN TOP AND DEEP LAYER.
C
C       TOP LAYER
C
        ASIDET=AS(1)-AS(2)
        ABT=AB(1)-AB(2)
        ZST=ASIDET/R(1)/4.0
        ZBT=ZST*2.0
        IF(ZPFIN.LT.TH) GO TO 250
C
C       DEEP LAYER
C
        ASIDED=AS(2)
        ABD=AB(2)
        ZSD=ASIDED/R(2)/4.0+TH
        ZBD=ABD/R(2)/2.0+TH
  250   CONTINUE
C
C       ASSIGN VALUES TO SOIL VARIABLES
C
        IF(NPRO.GT.1) GO TO 300
C
C       SOIL PROFILE NR.1
C
        SUST=SU0+RK*ZST
        SUSB=SU0+RK*ZBT
        RNCT=2.0+(RNCMAX-2.0)*ZST/R(1)/5.0
        IF(RNCT.GT.RNCMAX)RNCT=RNCMAX
        PUT=0.0
        SUSD=0.0
        SUSB=0.0
        RNCD=0.0
        PUD=0.0
        PHIRT=0.0
        PHIRD=0.0
C
        IF(ZPFIN.LT.TH) GO TO 500
C
C       FINAL PENETRATION IN DEEP LAYER SOIL PROFILE NO. 1
C
        IF(PHI.GT.0.0) GO TO 270
C
C       CLAY IN DEEP LAYER
C
        SUSD=SUH
        SUSB=SUH
        PHIRD=0.0
        RNCD=2.0+(RNCMAX-2.0)*(ZSD-TH)/R(2)/5.0
        IF(RNCD.GT.RNCMAX)RNCD=RNCMAX
        GO TO 500
  270   CONTINUE
C
```

```
C     SAND IN DEEP LAYER
C
      PU=(GAMMSO*ZSD)*(3.0*RKP+(RNQ-3.0*RKP)*(ZSD-TH)/R(2)/6.0)
      PHIRD=PHIRAD
      GO TO 500
  300 CONTINUE
      IF(NPRO.GT.2) GO TO 400
C
C     SOIL PROFILE NO.2
C
      SUST=SUH
      SUBT=SUH
      RNCT=2.0+(RNCMAX-2.0)*ZST/R(1)/5.0
      IF(RNCT.GT.RNCMAX)RNCT=RNCMAX
      PUT=0.0
      SUSD=0.0
      SUBD=0.0
      RNCD=0.0
      PUD=0.0
      PHIRT=0.0
      PHIRD=0.0
      IF(ZPFIN.LT.TH) GO TO 500
C
C     FINAL PENETRATION IN DEEP LAYER SOIL PROFILE NR.2
C
      SUSD=SU0+(ZSD-TH)*RK
      SUSB=SU0+(ZBD-TH)*RK
      RNCD=2.0+(RNCMAX-2.0)*(2*ZSD-TH)/R(2)/5.C
      IF(RNCD.GT.RNCMAX)RNCD=RNCMAX
      GO TO 500
  400 CONTINUE
      IF(NPRO.NE.3) GO TO 1999
C
C     SOIL PROFILE NO.3
C
      SUST=0.0
      SUBT=0.0
      RNCT=0.0
      PUT=(GAMMSO*ZST)*(3.0*RKP+(RNQ-3.0*RKP)*ZST/R(1)/6.0)
      SUSD=0.0
      SUBD=0.0
      RNCD=0.0
      PUD=0.0
      PHIRT=PHIRAD
      IF(ZPFIN.LT.TH) GO TO 500
C
C     FINAL PENETRATION IN DEEP LAYER SOIL PROFILE NO.3
C
      SUSD=SU0+(ZSD-TH)*RK
      SUBD=SU0+(ZBD-TH)*RK
      RNCD=2.0+(RNCMAX-2.0)*(ZSD-TH)/R(2)/5.0
      IF(RNCD.GT.RNCMAX)RNCD=RNCMAX
C
  500 CONTINUE
C
C     CALCULATION OF ULTIMATE HORIZONTAL CAPACITY.
C
C
C     CAPACITY OF SIDEWALL IN TOP LAYER.
C
      FHST=ASIDET*(RNCT*SUST+PUT)
```

```
C
C     CAPACITY OF SIDEWALL IN DEEEP LAYER.
C
      FHSD=ASIDED*(RNCD*SUSD+PUD)
C
C     TOTAL CAPACITY OF SIDEWALL BOTH IN TOP AND DEEP LAYER.
C
      FHSIDE=FHST+FHSD
C
C     CAPACITY OF BASE IN TOP LAYER.
C
      FHBT=(ABT*SUBT+FVERT*TAN(PHIRT))*RR
C
C     CAPACITY OF BASE IN DEEP LAYER.
C
      FHBD=(ABD*SUBD+FVERT*TAN(PHIRAD))*RR
C
C     TOTAL CAPACITY OF BASE.
C
      FHBASE=FHBT+FHBD
C
C     ULTIMATE HORIZONTAL CAPACITY.
C
      FHULT=FHSIDE+FHBASE
      RETURN
C
 1999 CONTINUE
      WRITE(6,9000)
 9000 FORMAT(5X,'ERROR IN SOILFROFIL NUMBER')
      RETURN
      END
```

```
      SUBROUTINE FIFOR(M,MLOAD,MSTART,GAMMSO,STF,STF1,STF2,STF3,STFKZG,
     *STFKZI,STFKXI,STFKRI,PI,HTIP,HCON,HSID,DTIP,DTOT,NPRO,SUO,RK,SUH,
     *PHI,ZP,IPEN,TH,RR,NL,AO,BTOT,FIX,FZ,FH,DISP,FORCE,SFS,SFB,FIVECT,
     *TOTFOR,TDISP,STFKZ,STFKX,STFKR)
C
C     PURPOSE:
C
C     METHOD:
C
C     INPUT ARGUMENTS:
C
C     INTERNAL PARAMETERS:
C
C     OUTPUT ARGUMENTS:
C
      IMPLICIT DOUBLE PRECISION (A-H,O-Z), INTEGER (I-N)
      COMMON/JU6/FVULTI
      COMMON/JU7/ZPFIN
      DIMENSION STF(12,12),RINSTF(12,12),STF1(6,6),STF2(6,6),
     *STF3(6,6),FIVECT(12,22),FIX(3,12,22),FORCE(12,22),DISP(12,22),
     *STFKZG(4),STFKZI(4),STFKXI(4),STFKRI(4),STFKZ(4),STFKX(4),
     *STFKR(4),FULT(12,22),FHCAP(4,22),EULT(2),SFB(2,22),SFS(2,22),
     *FZ(4),TOTFOR(12,22),TDISP(12,22),ZP(50)
C
C     TEMPORARY PRINT STATEMENT
C
C      DO 1000 I=1,12
C      WRITE(6,7000)(STF(I,J),J=1,12)
C 1000 CONTINUE
      CALL INVM12(STF,RINSTF,12,0.00000001,IERR)
C
C     PRINT COMMENT AND STOP PROGRAM EXECUTION IF THE INVERTED
C     MATRIX IS CLOSE TO SINGULARITY, PROGRAM IS STOPPED IN
C     SUBROUTINE INVM12
C
C     TEMPORARY PRINT STATEMENT
C
C      DO 1010 I=1,12
C      WRITE(6,7000)(STF(I,J),J=1,12)
C 1010 CONTINUE
C
C     TEMPORARY PRINT STATEMENT
C
C      DO 1020 I=1,12
C      WRITE(6,7000)(RINSTF(I,J),J=1,12)
C 1020 CONTINUE
C
C     CALCULATE DISPLACEMENT DISP(1-12) FOR CHOSEN WAVECYCLE
C
      DO 1200 I=1,12
      DO 1150 N=1,12
      IF(MLOAD.GE.2) GO TO 1100
      DISP(I,MLOAD)=DISP(I,MLOAD)+RINSTF(I,N)*FIVECT(N,1)
      GO TO 1150
 1100 CONTINUE
      IF(MSTART.EQ.3.AND.M.EQ.3) GO TO 1140
      IF(M.EQ.3) GO TO 1110
      IF(MLOAD.EQ.3) GO TO 1140
      DISP(I,MLOAD)=DISP(I,MLOAD)+RINSTF(I,N)*(FIVECT(N,M)-
     *FIVECT(N,M-1))
      GO TO 1150
```

```
[illegible]0 CONTINUE
      DISP(I,MLOAD)=DISP(I,MLOAD)+RINSTF(I,N)*(FIVECT(N,3)-
     *FIVECT(N,22))
      GO TO 1150
 1140 CONTINUE
      DISP(I,MLOAD)=DISP(I,MLOAD)+RINSTF(I,N)*FIVECT(N,MSTART+2)
 1150 CONTINUE
 1200 CONTINUE
      IF(MLOAD.GT.1) GO TO 1260
      DO 1250 NR=1,4
      STFKZ(NR)=STFKZG(NR)
      STFKX(NR)=STFKXI(NR)
      STFKR(NR)=STFKRI(NR)
 1250 CONTINUE
 1260 CONTINUE
C       IF(NL.EQ.4) GO TO 1300
C
C     CALCULATE FINAL FORCES IN SOILSPRINGS FOR THREE LEGED PLATFORM
C
C       FORCE(1,MLOAD)=DISP(1,MLOAD)*STFKX(1)
C       FORCE(2,MLOAD)=DISP(2,MLOAD)*STFKZ(1)
C       FORCE(3,MLOAD)=DISP(3,MLOAD)*STFKR(1)
C
C       FORCE(10,MLOAD)=DISP(10,MLOAD)*(STFKX(2)+STFKX(3))
C       FORCE(11,MLOAD)=DISP(11,MLOAD)*(STFKZ(2)+STFKZ(3))
C       FORCE(12,MLOAD)=DISP(12,MLOAD)*(STFKR(2)+STFKR(3))
C       GO TO 1310
C
C     CALCULATE FINAL FORCES IN SOILSPRINGS FOR FOUR LEGED PLATFORM
C
C 1300  CONTINUE
C       FORCE(1,MLOAD)=DISP(1,MLOAD)*(STFKX(1)+STFKX(3))
C       FORCE(2,MLOAD)=DISP(2,MLOAD)*(STFKZ(1)+STFKZ(3))
C       FORCE(3,MLOAD)=DISP(3,MLOAD)*(STFKR(1)+STFKR(3))
C
C       FORCE(10,MLOAD)=DISP(10,MLOAD)*(STFKX(2)+STFKX(4))
C       FORCE(11,MLOAD)=DISP(11,MLOAD)*(STFKZ(2)+STFKZ(4))
C       FORCE(12,MLOAD)=DISP(12,MLOAD)*(STFKR(2)+STFKR(4))
C
C 1310  CONTINUE
C
C     STRUCTURE IS SUBJECTED TO THE INCREMENTAL LOAD VECTOR
C     NUMBER M. THREE POSSIBLE WAYS TO ARRIVE AT
C     SOLUTION.
C
      IF(M.GT.1.AND.M.NE.MSTART+2) GO TO 1312
C
C     CALCULATE FINAL SOIL REACTION FORCE ON SPUDCANS FOR
C     BOTH THREE AND FOUR LEG TYPE, FOR M=1, OR
C     M=MSTART+2.
C
C
      FORCE(1,MLOAD)=FIX(1,1,M)+STF1(1,1)*DISP(1,MLOAD)+
     *STF1(1,3)*DISP(3,MLOAD)+STF1(1,4)*DISP(4,MLOAD)+
     *STF1(1,6)*DISP(6,MLOAD)
C
      FORCE(2,MLOAD)=FIX(1,2,M)+STF1(2,2)*DISP(2,MLOAD)+
     *STF1(2,5)*DISP(5,MLOAD)
C
      FORCE(3,MLOAD)=FIX(1,3,M)+STF1(3,1)*DISP(1,MLOAD)+
     *STF1(3,3)*DISP(3,MLOAD)+STF1(3,4)*DISP(4,MLOAD)+
```

```
     .STF1(3,6)*DISP(6,MLOAD)
C
C
      FORCE(10,MLOAD)=FIX(3,10,M)+STF3(4,1)*DISP(7,MLOAD)+
     .STF3(4,3)*DISP(9,MLOAD)+STF3(4,4)*DISP(10,MLOAD)+
     .STF3(4,6)*DISP(12,MLOAD)
C
      FORCE(11,MLOAD)=FIX(3,11,M)+STF3(5,2)*DISP(8,MLOAD)+
     .STF3(5,5)*DISP(11,MLOAD)
C
      FORCE(12,MLOAD)=FIX(3,12,M)+STF3(6,1)*DISP(7,MLOAD)+
     .STF3(6,3)*DISP(9,MLOAD)+STF3(6,4)*DISP(10,MLOAD)+
     .STF3(6,6)*DISP(12,MLOAD)
C
C
C     CALCULATE FINAL FORCES IN LEGS FOR M=1 OR M=MSTART+2.
C
      FORCE(4,MLOAD)=FIX(1,4,M)+FIX(2,4,M)+STF1(4,1)*DISP(1,MLOAD)
     .+STF1(4,3)*DISP(3,MLOAD)+STF1(4,4)*DISP(4,MLOAD)
     .+STF1(4,6)*DISP(6,MLOAD)
C
      FORCE(5,MLOAD)=FIX(1,5,M)+FIX(2,5,M)+STF1(5,2)*DISP(2,MLOAD)
     .+STF1(5,5)*DISP(5,MLOAD)
C
      FORCE(6,MLOAD)=FIX(1,6,M)+FIX(2,6,M)+STF1(6,1)*DISP(1,MLOAD)
     .+STF1(6,3)*DISP(3,MLOAD)+STF1(6,4)*DISP(4,MLOAD)
     .+STF1(6,6)*DISP(6,MLOAD)
C
      FORCE(7,MLOAD)=FIX(2,7,M)+FIX(3,7,M)+STF3(1,1)*DISP(7,MLOAD)
     .+STF3(1,3)*DISP(9,MLOAD)+STF3(1,4)*DISP(10,MLOAD)
     .+STF3(1,6)*DISP(12,MLOAD)
C
      FORCE(8,MLOAD)=FIX(2,8,M)+FIX(3,8,M)+STF3(2,2)*DISP(8,MLOAD)
     .+STF3(2,5)*DISP(11,MLOAD)
C
      FORCE(9,MLOAD)=FIX(2,9,M)+FIX(3,9,M)+STF3(3,1)*DISP(7,MLOAD)
     .+STF3(3,3)*DISP(9,MLOAD)+STF3(3,4)*DISP(10,MLOAD)+STF3(3,6)*
     .DISP(12,MLOAD)
      GO TO 1314
C
 1312 CONTINUE
C
C     CALCULATE SOIL REACTION FORCE ON SPUDCAN BOTH FOR
C     THREE AND FOUR LEG TYPE. M .GT. 1
C
C     IF WAVESTEP VALUE EQUAL 1 THAT IS HERE M=3,(REMEMBER
C     M=MSTART+2) , AND LOADCOUNTER NOT EQUAL 3 (MLOAD NE 3),
C     GO TO 1313.
C
      IF(M.EQ.3.AND.MLOAD.NE.3) GO TO 1313
C
      FORCE(1,MLOAD)=FIX(1,1,M)-FIX(1,1,M-1)+STF1(1,1)*DISP(1,MLOAD)+
     .STF1(1,3)*DISP(3,MLOAD)+STF1(1,4)*DISP(4,MLOAD)+
     .STF1(1,6)*DISP(6,MLOAD)
C
      FORCE(2,MLOAD)=FIX(1,2,M)-FIX(1,2,M-1)+STF1(2,2)*DISP(2,MLOAD)+
     .STF1(2,5)*DISP(5,MLOAD)
C
      FORCE(3,MLOAD)=FIX(1,3,M)-FIX(1,3,M-1)+STF1(3,1)*DISP(1,MLOAD)+
     .STF1(3,3)*DISP(3,MLOAD)+STF1(3,4)*DISP(4,MLOAD)+
     .STF1(3,6)*DISP(6,MLOAD)
```

```
C
C
      FORCE(10,MLOAD)=FIX(3,10,M)-FIX(3,10,M-1)
     *+STF3(4,1)*DISP(7,MLOAD)+
     *STF3(4,3)*DISP(9,MLOAD)+STF3(4,4)*DISP(10,MLOAD)+
     *STF3(4,6)*DISP(12,MLOAD)
C
      FORCE(11,MLOAD)=FIX(3,11,M)-FIX(3,11,M-1)
     *+STF3(5,2)*DISP(8,MLOAD)+
     *STF3(5,5)*DISP(11,MLOAD)
C
      FORCE(12,MLOAD)=FIX(3,12,M)-FIX(3,12,M-1)
     *+STF3(6,1)*DISP(7,MLOAD)+
     *STF3(6,3)*DISP(9,MLOAD)+STF3(6,4)*DISP(10,MLOAD)+
     *STF3(6,6)*DISP(12,MLOAD)
C
C
C     CALCULATE FINAL FORCES IN LEGS
C
      FORCE(4,MLOAD)=FIX(1,4,M)-FIX(1,4,M-1)+FIX(2,4,M)-FIX(2,4,M-1)
     *+STF1(4,1)*DISP(1,MLOAD)
     *+STF1(4,3)*DISP(3,MLOAD)+STF1(4,4)*DISP(4,MLOAD)
     *+STF1(4,6)*DISP(6,MLOAD)
C
      FORCE(5,MLOAD)=FIX(1,5,M)-FIX(1,5,M-1)+FIX(2,5,M)-FIX(2,5,M-1)
     *+STF1(5,2)*DISP(2,MLOAD)
     *+STF1(5,5)*DISP(5,MLOAD)
C
      FORCE(6,MLOAD)=FIX(1,6,M)-FIX(1,6,M-1)+FIX(2,6,M)-FIX(2,6,M-1)
     *+STF1(6,1)*DISP(1,MLOAD)
     *+STF1(6,3)*DISP(3,MLOAD)+STF1(6,4)*DISP(4,MLOAD)
     *+STF1(6,6)*DISP(6,MLOAD)
C
      FORCE(7,MLOAD)=FIX(2,7,M)-FIX(2,7,M-1)+FIX(3,7,M)-FIX(3,7,M-1)
     *+STF3(1,1)*DISP(7,MLOAD)
     *+STF3(1,3)*DISP(9,MLOAD)+STF3(1,4)*DISP(10,MLOAD)
     *+STF3(1,6)*DISP(12,MLOAD)
C
      FORCE(8,MLOAD)=FIX(2,8,M)-FIX(2,8,M-1)+FIX(3,8,M)-FIX(3,8,M-1)
     *+STF3(2,2)*DISP(8,MLOAD)
     *+STF3(2,5)*DISP(11,MLOAD)
C
      FORCE(9,MLOAD)=FIX(2,9,M)-FIX(2,9,M-1)+FIX(3,9,M)-FIX(3,9,M-1)
     *+STF3(3,1)*DISP(7,MLOAD)
     *+STF3(3,3)*DISP(9,MLOAD)+STF3(3,4)*DISP(10,MLOAD)+STF3(3,6)*
     *DISP(12,MLOAD)
      GO TO 1314
C
 1313 CONTINUE
C
C     CALCULATE SOIL REACTION FORCES ON SPUDCANS FOR M=3 AND MLOAD NE 3.
C
      FORCE(1,MLOAD)=FIX(1,1,M)-FIX(1,1,22)+STF1(1,1)*DISP(1,MLOAD)+
     *STF1(1,3)*DISP(3,MLOAD)+STF1(1,4)*DISP(4,MLOAD)+
     *STF1(1,6)*DISP(6,MLOAD)
C
      FORCE(2,MLOAD)=FIX(1,2,M)-FIX(1,2,22)+STF1(2,2)*DISP(2,MLOAD)+
     *STF1(2,5)*DISP(5,MLOAD)
C
      FORCE(3,MLOAD)=FIX(1,3,M)-FIX(1,3,22)+STF1(3,1)*DISP(1,MLOAD)+
     *STF1(3,3)*DISP(3,MLOAD)+STF1(3,4)*DISP(4,MLOAD)+
```

```
     *STF1(3,6)*DISP(6,MLOAD)
C
C
      FORCE(10,MLOAD)=FIX(3,10,M)-FIX(3,10,22)+STF3(4,1)*DISP(7,MLOAD)+
     *STF3(4,3)*DISP(9,MLOAD)+STF3(4,4)*DISP(10,MLOAD)+
     *STF3(4,6)*DISP(12,MLOAD)
C
      FORCE(11,MLOAD)=FIX(3,11,M)-FIX(3,11,22)+STF3(5,2)*DISP(8,MLOAD)+
     *STF3(5,5)*DISP(11,MLOAD)
C
      FORCE(12,MLOAD)=FIX(3,12,M)-FIX(3,12,22)+STF3(6,1)*DISP(7,MLOAD)+
     *STF3(6,3)*DISP(9,MLOAD)+STF3(6,4)*DISP(10,MLOAD)+
     *STF3(6,6)*DISP(12,MLOAD)
C
C
C     CALCULATE FINAL FORCES IN LEGS. M=3 AND MLOAD .NE. 3.
C
      FORCE(4,MLOAD)=FIX(1,4,M)-FIX(1,4,22)+FIX(2,4,M)-FIX(2,4,22)
     *+STF1(4,1)*DISP(1,MLOAD)
     *+STF1(4,3)*DISP(3,MLOAD)+STF1(4,4)*DISP(4,MLOAD)
     *+STF1(4,6)*DISP(6,MLOAD)
C
      FORCE(5,MLOAD)=FIX(1,5,M)-FIX(1,5,22)+FIX(2,5,M)-FIX(2,5,22)
     *+STF1(5,2)*DISP(2,MLOAD)
     *+STF1(5,5)*DISP(5,MLOAD)
C
      FORCE(6,MLOAD)=FIX(1,6,M)-FIX(1,6,22)+FIX(2,6,M)-FIX(2,6,22)
     *+STF1(6,1)*DISP(1,MLOAD)
     *+STF1(6,3)*DISP(3,MLOAD)+STF1(6,4)*DISP(4,MLOAD)
     *+STF1(6,6)*DISP(6,MLOAD)
C
      FORCE(7,MLOAD)=FIX(2,7,M)-FIX(2,7,22)+FIX(3,7,M)-FIX(3,7,22)
     *+STF3(1,1)*DISP(7,MLOAD)
     *+STF3(1,3)*DISP(9,MLOAD)+STF3(1,4)*DISP(10,MLOAD)
     *+STF3(1,6)*DISP(12,MLOAD)
C
      FORCE(8,MLOAD)=FIX(2,8,M)-FIX(2,8,22)+FIX(3,8,M)-FIX(3,8,22)
     *+STF3(2,2)*DISP(8,MLOAD)
     *+STF3(2,5)*DISP(11,MLOAD)
C
      FORCE(9,MLOAD)=FIX(2,9,M)-FIX(2,9,22)+FIX(3,9,M)-FIX(3,9,22)
     *+STF3(3,1)*DISP(7,MLOAD)
     *+STF3(3,3)*DISP(9,MLOAD)+STF3(3,4)*DISP(10,MLOAD)+STF3(3,6)*
     *DISP(12,MLOAD)
C
 1314 CONTINUE
C
C     CALCULATION OF TOTAL FORCES AND TOTAL DISPLACEMENTS
C
      IF(MLOAD.GT.1) GO TO 1330
      TOTFOR(1,1)=FORCE(1,1)
      TDISP(1,1)=DISP(1,1)
      TOTFOR(2,1)=FORCE(2,1)+FZ(1)
      TDISP(2,1)=DISP(2,1)
      TOTFOR(3,1)=FORCE(3,1)
      TDISP(3,1)=DISP(3,1)
      TOTFOR(4,1)=FORCE(4,1)
      TDISP(4,1)=DISP(4,1)
      TOTFOR(5,1)=FORCE(5,1)
      TDISP(5,1)=DISP(5,1)
      TOTFOR(6,1)=FORCE(6,1)
```

```
      TDISP(6,1)=DISP(6,1)
      TOTFOR(7,1)=FORCE(7,1)
      TDISP(7,1)=DISP(7,1)
      TOTFOR(8,1)=FORCE(8,1)
      TDISP(8,1)=DISP(8,1)
      TOTFOR(9,1)=FORCE(9,1)
      TDISP(9,1)=DISP(9,1)
      TOTFOR(10,1)=FORCE(10,1)
      TDISP(10,1)=DISP(10,1)
      TOTFOR(11,1)=FORCE(11,1)+FZ(2)
      TDISP(11,1)=DISP(11,1)
      TOTFOR(12,1)=FORCE(12,1)
      TDISP(12,1)=DISP(12,1)
      IF(NL.EQ.4) GO TO 1320
      TOTFOR(11,1)=TOTFOR(11,1)+FZ(3)
      GO TO 1350
 1320 CONTINUE
      TOTFOR(2,1)=TOTFOR(2,1)+FZ(3)
      TOTFOR(11,1)=TOTFOR(11,1)+FZ(4)
      GO TO 1350
 1330 CONTINUE
C
C     CALCULATE TOTAL FORCE FOR LOADSTEP WHEN MLOAD IS GREATER
C     THAN 1.
C
      DO 1340 I=1,12
      TOTFOR(I,MLOAD)=TOTFOR(I,MLOAD-1)+FORCE(I,MLOAD)
      TDISP(I,MLOAD)=TDISP(I,MLOAD-1)+DISP(I,MLOAD)
 1340 CONTINUE
 1350 CONTINUE
C
C
C     CALCULATE NEW SOILSTIFFNESS
C
C
C     FHZCAP IS THE SAME FOR ALL SPUDCANS(IF CLAY) SINCE THE
C     SAME DEPT AND SOIL IS ASSUMED.
C
      IF(NPRO.EQ.3) GO TO 1351
      CALL FHZCAP(PI,HTIP,HCON,HSID,DTIP,DTOT,NPRO,SUO,RK,SUH,PHI,TH,
     *RR,GAMMSO,FVERT,AOFIN,FHSIDE,FHULT)
      FHCAP(1,MLOAD)=FHULT
      FHCAP(2,MLOAD)=FHULT
      FHCAP(3,MLOAD)=FHULT
      IF(NL.EQ.4) FHCAP(4,MLOAD)=FHULT
C
      GO TO 1353
 1351 CONTINUE
C
C     CALCULATION OF LATERAL RESITANCE FOR NPRO EQUAL THREE
C
      IF(NL.EQ.4) GO TO 1352
C
C     CALCULATION FOR THREE LEG TYPE AND NPRO EQUAL THREE.
C
C     CALCULATE VERTICAL LOAD FOR LEG NO.1
C
      FVERT=TOTFOR(2,MLOAD)
      CALL FHZCAP(PI,HTIP,HCON,HSID,DTIP,DTOT,NPRO,SUO,RK,SUH,PHI,TH,
     *RR,GAMMSO,FVERT,AOFIN,FHSIDE,FHULT)
C
```

```
C     HORIZONTAL CAPACITY FOR LEG NO.1
C
      FHCAP(1,MLOAD)=FHULT
C
C     CALCULATE VERTICAL LOAD FOR LEG NO.2 AND LEG NO.3
C
      FVERT=TOTFOR(11,MLOAD)/2.0
      CALL FHZCAP(PI,HTIP,HCON,HSID,DTIP,DTOT,NPRO,SUO,RK,SUH,PHI,TH,
     *RR,GAMMSO,FVERT,AOFIN,FHSIDE,FHULT)
C
C     HORIZONTAL CAPASITY FOR LEG NO.2 AND NO.3
C
      FHCAP(2,MLOAD)=FHULT
      FHCAP(3,MLOAD)=FHULT
C
C     ALL LOADS ARE RELATED TO ONE SPUDCAN AT EACH LEG
C     TRANSFORMATION TO CORRECT PLATFORM TYPE LATER. SEE STFMTX.
C
      GO TO 1353
 1352 CONTINUE
C
C     CALCULATION FOR FOUR LEG TYPE AND NPRO EGUAL THREE.
C
C     CALCULATE VERTICAL LOAD FOR LEG NO.1 AND NO.3.
C
      FVERT=TOTFOR(2,MLOAD)/2.0
      CALL FHZCAP(PI,HTIP,HCON,HSID,DTIP,DTOT,NPRO,SUO,RK,SUH,PHI,TH,
     *RR,GAMMSO,FVERT,AOFIN,FHSIDE,FHULT)
C
C     LATERAL CAPACITY FOR LEG NO.1 AND NO.3
C
      FHCAP(1,MLOAD)=FHULT
      FHCAP(3,MLOAD)=FHULT
C
C     CALCULATE VERTICAL LOAD FOR LEG NO.2 AND NO.4
C
      FVERT=TOTFOR(11,MLOAD)/2.0
      CALL FHZCAP(PI,HTIP,HCON,HSID,DTIP,DTOT,NPRO,SUO,RK,SUH,PHI,TH,
     *RR,GAMMSO,FVERT,AOFIN,FHSIDE,FHULT)
C
C     LATERAL CAPACITY FOR LEG NO.2 AND NO.4
C
      FHCAP(2,MLOAD)=FHULT
      FHCAP(4,MLOAD)=FHULT
C
 1353 CONTINUE
      IF(NL.EQ.4)GO TO 1360
C
C     CALCULATION OF ULTIMATE BEARING CAPASITY.
C
C
C     LEG NR.1(THREE LEG TYPE) = ELEMENT NR.1
C
      FH=ABS(TOTFOR(1,MLOAD))-ABS(FHSIDE)
      IF(ABS(FHSIDE).GT.ABS(TOTFOR(1,MLOAD))) FH=0.0
      CALL PREPEN
      FULT(2,MLOAD)=FVULTI
      GO TO 1370
 1360 CONTINUE
C
C     LEG NR.1 (FOUR LEG TYPE) = HALF ELEMENT NR.1
```

```
C
      FH=ABS(TOTFOR(1,MLOAD)/2.0)-ABS(FHSIDE)
      IF(ABS(FHSIDE).GT.ABS(TOTFOR(1,MLOAD)/2.0)) FH=0.0
      CALL PREPEN
      FULT(2,MLOAD)=FVULTI
 1370 CONTINUE
C
C     LEG NR.2 (BOTH FOR THREE AND FOUR LEG TYPE)
C
      FH=ABS(TOTFOR(10,MLOAD)/2.0)-ABS(FHSIDE)
      IF(ABS(FHSIDE).GT.ABS(TOTFOR(10,MLOAD)/2.0)) FH=0.0
      CALL PREPEN
      FULT(11,MLOAD)=FVULTI
C
      IF(NL.EQ.4) GO TO 1380
C
C     CALCULATION FOR MOMENT CAPASITY FOR ELEMENT NR.1 AND NR.2
C     THREE LEG TYPE
C
      EULT(1)=(AOFIN-ABS(AOFIN*TOTFOR(2,MLOAD)/FULT(2,MLOAD)))/
     *(2.0*SQRT(AOFIN))
C
C     EULT(1) CAN NEVER BEE NEGATIVE
C
      IF(EULT(1).LT.0.0)EULT(1)=0.0
      EULT(2)=(AOFIN-ABS(AOFIN*TOTFOR(11,MLOAD)/2.0/FULT(11,MLOAD)))/
     *(2.0*SQRT(AOFIN))
C
C     EULT(2) CAN NEVER BEE NEGATIVE.
C
      IF(EULT(2).LT.0.0)EULT(2)=0.0
C
C     MOMENT:
C
      FULT(3,MLOAD)=TOTFOR(2,MLOAD)*EULT(1)
      FULT(12,MLOAD)=TOTFOR(11,MLOAD)*EULT(2)
C
      GO TO 1390
 1380 CONTINUE
C
C     CALCULATION FOR MOMENT CAPASITY FOR ELEMENT NR.1 AND NR.2
C     FOUR LEG TYPE
C
      EULT(1)=(AOFIN-ABS(AOFIN*TOTFOR(2,MLOAD)/2.0/FULT(2,MLOAD)))/
     *(2.0*SQRT(AOFIN))
C
C     EULT(1) CAN NEVER BEE NEGATIVE
C
      IF(EULT(1).LT.0.0) EULT(1)=0.0
      EULT(2)=(AOFIN-ABS(AOFIN*TOTFOR(11,MLOAD)/2.0/FULT(11,MLOAD)))/
     *(2.0*SQRT(AOFIN))
C
C     IF(EULT(2).0.0) EULT(2)=0.0
C
C
C     MOMENT:
C
      FULT(3,MLOAD)=TOTFOR(2,MLOAD)*EULT(1)
      FULT(12,MLOAD)=TOTFOR(11,MLOAD)*EULT(2)
C
 1390 CONTINUE
```

```
C
      IF(NL.EQ.4) GO TO 1500
C
C     CALCULATION OF SAFETY FACTORS (THREE LEG TYPE):
C
      IF(ABS(TOTFOR(1,MLOAD)).LE.0.001) GO TO 1410
      SFS(1,MLOAD)=ABS(FHCAP(1,MLOAD)/TOTFOR(1,MLOAD))
      GO TO 1420
 1410 CONTINUE
      SFS(1,MLOAD)=999
 1420 CONTINUE
      IF(ABS(TOTFOR(10,MLOAD)).LE.0.001) GO TO 1430
      SFS(2,MLOAD)=ABS(FHCAP(2,MLOAD)*2.0/TOTFOR(10,MLOAD))
      GO TO 1440
 1430 CONTINUE
      SFS(2,MLOAD)=999
 1440 CONTINUE
      IF(ABS(TOTFOR(2,MLOAD)).LE.0.001) GO TO 1450
      SFB(1,MLOAD)=ABS(FULT(2,MLOAD)/TOTFOR(2,MLOAD))
      GO TO 1460
 1450 CONTINUE
      SFB(1,MLOAD)=999
 1460 CONTINUE
      IF(ABS(TOTFOR(11,MLOAD)).LE.0.001) GO TO 1470
      SFB(2,MLOAD)=ABS(FULT(11,MLOAD)*2.0/TOTFOR(11,MLOAD))
      GO TO 1480
 1470 CONTINUE
      SFB(2,MLOAD)=999
 1480 CONTINUE
C
C     NEW STIFFNESS THREE LEGS
C
      IF(1.0/SFB(1,MLOAD).GE.0.95) GO TO 1481
      IF(ABS(TOTFOR(2,MLOAD)/FULT(2,MLOAD)).GE.1.0) GO TO 1481
      STFKZ(1)=STFKZI(1)*(1.0-ABS(TOTFOR(2,MLOAD)/FULT(2,MLOAD)))**2.0
      GO TO 1482
 1481 CONTINUE
      STFKZ(1)=0.001*STFKZI(1)
 1482 CONTINUE
      IF(1.0/SFS(1,MLOAD).GE.0.95) GO TO 1483
      IF(ABS(TOTFOR(1,MLOAD)/FHCAP(1,MLOAD)).GE.1.0) GO TO 1483
      STFKX(1)=STFKXI(1)*(1.0-ABS(TOTFOR(1,MLOAD)/FHCAP(1,MLOAD)))**2.0
      GO TO 1484
 1483 CONTINUE
      STFKX(1)=0.001*STFKXI(1)
 1484 CONTINUE
      IF(FULT(1).LE.0.001)GO TO 1485
      IF(ABS(TOTFOR(3,MLOAD)/FULT(3,MLOAD)).GE.0.95) GO TO 1485
      IF(ABS(TOTFOR(3,MLOAD)/FULT(3,MLOAD)).GE.1.0) GO TO 1485
      STFKR(1)=STFKRI(1)*(1.0-ABS(TOTFOR(3,MLOAD)/FULT(3,MLOAD)))**2.0
      GO TO 1486
 1485 CONTINUE
      STFKR(1)=0.001*STFKRI(1)
 1486 CONTINUE
      IF(1.0/SFB(2,MLOAD).GE.0.95) GO TO 1487
      IF(ABS(TOTFOR(11,MLOAD)/2.0/FULT(11,MLOAD)).GE.1.0)GO TO 1487
      STFKZ(2)=STFKZI(2)*(1.0-ABS(TOTFOR(11,MLOAD)/2.0/
     .FULT(11,MLOAD)))**2.0
      GO TO 1488
 1487 CONTINUE
      STFKZ(2)=0.001*STFKZI(2)
```

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 1488 CONTINUE
      IF(1.0/SFS(2,MLOAD).GE.0.95) GO TO 1489
      IF(ABS(TOTFOR(10,MLOAD)/2.0/FHCAP(2,MLOAD)).GE.1.0) GO TO 1489
      STFKX(2)=STFKXI(2)*(1.0-ABS(TOTFOR(10,MLOAD)/2.0/
     *FHCAP(2,MLOAD)))**2.0
      GO TO 1490
 1489 CONTINUE
      STFKX(2)=0.001*STFKXI(2)
 1490 CONTINUE
      IF(FULT(2).LE.0.001)GO TO 1491
      IF(ABS(TOTFOR(12,MLOAD)/FULT(12,MLOAD)).GE.0.95) GO TO 1491
      IF(ABS(TOTFOR(12,MLOAD)/FULT(12,MLOAD)).GE.1.0) GO TO 1491
      STFKR(2)=STFKRI(2)*(1.0-ABS(TOTFOR(12,MLOAD)/
     *FULT(12,MLOAD)))**2.0
      GO TO 1492
 1491 CONTINUE
      STFKR(2)=0.001*STFKRI(2)
 1492 CONTINUE
      STFKZ(3)=STFKZ(2)
      STFKX(3)=STFKX(2)
      STFKR(3)=STFKR(2)
      GO TO 1600
 1500 CONTINUE
C
C     CALCULATION OF SAFETY FACTORS (FOUR LEG TYPE):
C
      IF(ABS(TOTFOR(1,MLOAD)).LE.0.001) GO TO 1510
      SFS(1,MLOAD)=ABS(FHCAP(1,MLOAD)*2.0/TOTFOR(1,MLOAD))
      GO TO 1520
 1510 CONTINUE
      SFS(1,MLOAD)=999
 1520 CONTINUE
      IF(ABS(TOTFOR(10,MLOAD)).LE.0.001) GO TO 1530
      SFS(2,MLOAD)=ABS(FHCAP(2,MLOAD)*2.0/TOTFOR(10,MLOAD))
      GO TO 1540
 1530 CONTINUE
      SFS(2,MLOAD)=999
 1540 CONTINUE
      IF(ABS(TOTFOR(2,MLOAD)).LE.0.001) GO TO 1550
      SFB(1,MLOAD)=ABS(FULT(2,MLOAD)*2.0/TOTFOR(2,MLOAD))
      GO TO 1560
 1550 CONTINUE
      SFB(1,MLOAD)=999
 1560 CONTINUE
      IF(ABS(TOTFOR(11,MLOAD)).LE.0.001) GO TO 1570
      SFB(2,MLOAD)=ABS(FULT(11,MLOAD)*2.0/TOTFOR(11,MLOAD))
      GO TO 1580
 1570 CONTINUE
      SFB(2,MLOAD)=999
 1580 CONTINUE
C
C     NEW STIFFNESS FOUR LEGS
C
      IF(1.0/SFB(1,MLOAD).GE.0.95) GO TO 1581
      IF(ABS(TOTFOR(2,MLOAD)/2.0/FULT(2,MLOAD)).GE.1.0) GO TO 1581
      STFKZ(1)=STFKZI(1)*(1.0-ABS(TOTFOR(2,MLOAD)/2.0/
     *FULT(2,MLOAD)))**2.0
      GO TO 1582
 1581 CONTINUE
      STFKZ(1)=0.001*STFKZI(1)
 1582 CONTINUE
```

```
      IF(1.0/SFS(1,MLOAD).GE.0.95) GO TO 1583
      IF(ABS(TOTFOR(1,MLOAD)/2.0/FHCAP(1,MLOAD)).GE.1.0) GO TO 1583
      STFKX(1)=STFKXI(1)*(1.0-ABS(TOTFOR(1,MLOAD)/2.0/
     .FHCAP(1,MLOAD)))**2.0
      GO TO 1584
 1583 CONTINUE
      STFKX(1)=0.001*STFKXI(1)
 1584 CONTINUE
      IF(EULT(1).LE.0.001) GO TO 1585
      IF(ABS(TOTFOR(3,MLOAD)/FULT(3,MLOAD)).GE.0.95) GO TO 1585
      IF(ABS(TOTFOR(3,MLOAD)/FULT(3,MLOAD)).GE.1.0) GO TO 1585
      STFKR(1)=STFKRI(1)*(1.0-ABS(TOTFOR(3,MLOAD)/
     .FULT(3,MLOAD)))**2.0
      GO TO 1586
 1585 CONTINUE
      STFKR(1)=0.001*STFKRI(1)
 1586 CONTINUE
      STFKZ(3)=STFKZ(1)
      STFKX(3)=STFKX(1)
      STFKR(3)=STFKR(1)
      IF(1.0/SFB(2,MLOAD).GE.0.95) GO TO 1587
      IF(ABS(TOTFOR(11,MLOAD)/2.0/FULT(11,MLOAD)).GE.1.0) GO TO 1587
      STFKZ(2)=STFKZI(2)*(1.0-ABS(TOTFOR(11,MLOAD)/2.0/
     .FULT(11,MLOAD)))**2.0
      GO TO 1588
 1587 CONTINUE
      STFKZ(2)=0.001*STFKZI(2)
 1588 CONTINUE
      IF(1.0/SFS(2,MLOAD).GE.0.95) GO TO 1589
      IF(ABS(TOTFOR(10,MLOAD)/2.0/FHCAP(2,MLOAD)).GE.1.0) GO TO 1589
      STFKX(2)=STFKXI(2)*(1.0-ABS(TOTFOR(10,MLOAD)/2.0/
     .FHCAP(2,MLOAD)))**2.0
      GO TO 1590
 1589 CONTINUE
      STFKX(2)=0.001*STFKXI(2)
 1590 CONTINUE
      IF(EULT(2).LE.0.001) GO TO 1591
      IF(ABS(TOTFOR(12,MLOAD)/FULT(12,MLOAD)).GE.0.95) GO TO 1591
      IF(ABS(TOTFOR(12,MLOAD)/FULT(12,MLOAD)).GE.1.0) GO TO 1591
      STFKR(2)=STFKRI(2)*(1.0-ABS(TOTFOR(12,MLOAD)/
     .FULT(12,MLOAD)))**2.0
      GO TO 1592
 1591 CONTINUE
      STFKR(2)=0.001*STFKRI(2)
 1592 CONTINUE
      STFKZ(4)=STFKZ(2)
      STFKX(4)=STFKX(2)
      STFKR(4)=STFKR(2)
 1600 CONTINUE
C
C     FORMAT STATEMENT FOR TEMPORARY PRINT STATEMENT
C
C7000 FORMAT(1X,12E11.3)
      RETURN
      END
```

```
      SUBROUTINE INITAL(FZ)
C
C     PURPOSE:
C     CALCULATION OF INITIAL STIFFNESSES.
C     METHOD:
C     RULES FOR CONSTRUCTION AND INSPECTION OF OFFSHORE STRUCTURES, MADE BY
C     DET NORSKE VERITAS.(APP. G - DYNAMIC ANALYSIS.)
C
C     INPUT ARGUMENTS:
C     POIRA1    = POISSONS RATIO FOR TOP LAYER.
C     POIRA2    = POISSONS RATIO FOR DEEP LAYER.
C     GMOD1     = G-MODULUS(SHEARMODULUS) FOR TOP LAYER
C     GMOD2     = G-MODULUS(SHEARMODULUS) FOR DEEP LAYER.
C     FVACT     = ACTUAL VERTICAL LOAD.
C     HTIP      = TIP HEIGHT OF SPUDCAN.
C     HCON      = CONE HEIGHT OF SPUDCAN.
C     HSID      = SIDE HEIGHT OF SPUDCAN.
C     DTOT      = TOTAL DIAMETER.
C     BL        = BREATH BETWEEN LEGS.
C     WG        = TOTAL WEIGHT INCLUDING LEGS AND FIXED EQUIPMENT.
C     GL        = GRAVITY LEVER ARM.
C     WGV       = VARIABLE LOAD.
C     NL        = NUMBER OF LEGS.
C     ZPFIN     = FINAL PENETRATION DEPTH.
C
C     INTERNAL PARAMETERS:
C     GMODR     = G-MODULUS FOR ROCKING RESPONSE.
C     GMODX     = G-MODULUS FOR HORIZONTAL RESPONSE.
C     GMODZ     = G-MODULUS FOR VERTICAL RESPONSE.
C     HPARA     = CALCULATED PARAMETER USED FOR CALCULATING CORRECT DEPTH.
C     POIRAR    = POISSONS RATIO FOR ROCKING RESPONSE
C     POIRAX    = POISSONS RATIO FOR HORIZONTAL RESPONSE.
C     POIRAZ    = POISSONS RATIO FOR VERTICAL RESPONSE.
C
C
C     OUTPUT ARGUMENTS:
C     STIKZI    = STIFFNESS FOR VERTICAL RESPONSE.
C     STIKXI    = STIFFNESS FOR HORIZONTAL RESPONSE.
C     STIKRI    = STIFFNESS FOR ROCKING RESPONSE.
C
C
      IMPLICIT DOUBLE PRECISION (A-H,O-Z), INTEGER (I-N)
      COMMON/JU4/POIRA1,POIRA2,GMOD1,GMOD2,FVACT,STFKZG(4),
     *STFKZI(4),STFKXI(4),STFKRI(4)
      COMMON/JU7/ZPFIN
C
      COMMON/JU1/IOP1,IOP2,IOP3,IOP4,IOP5,IOP6,TSET,BL,DW,AG,NL,WG,
     *GL,DI,DD,AW,PRECAP,DISPL,SPEPRE,WGLEG,ZPDEF,DTIP,DTOT,HTOP,HSID,
     *HCON,HTIP,HTOT,DMET,VWI,DARMW,HWM,TMIN,TMAX,TSTEP,GAMW,GAMMSO,
     *CI,CD,
     *VH1,VH2,WGV,VL,NPRO,SUO,RK,SUH,PHI,TH,RR,ALFA,
     *ZPD(50),ZPEN,ATOP(50),ADEP(50),ASIDD(50),ASIDT(50),
     *AO,ITR,COST,COSD,CO,COH,
     *FVULT(50),FVSU,FVPHI,FVSGUE,FVPUNC,BTOT,BEFF,FH,
     *FTOT(20,4,10),OMTOT(20,4,10),RLAR(20,4,10),FPRE(20,10),
     *FIVECT(12,22),OVTMP(20,10),RLARP(20,10),SPRNR,
     *PI,G,MSTART,ITSTRT,FHWIND
C
      COMMON/JU8/ZP(50),IPEN
      DIMENSION F(2),FZ(4)
      IF(ZPFIN.GE.TH)GO TO 1000
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      POIRAZ=POIRA1
      POIRAX=POIRA1
      POIRAR=POIRA1
      GMODZ=GMOD1
      GMODX=GMOD1
      GMODR=GMOD1
      GO TO 1500
 1000 CONTINUE
      HPARA=HTIP+HCON+HSID/2.0
      IF(ZPFIN.LE.(TH+HPARA)) GO TO 1200
 1100 CONTINUE
      POIRAZ=POIRA2
      POIRAX=POIRA2
      POIRAR=POIRA2
      GMODZ=GMOD2
      GMODX=GMOD2
      GMODR=GMOD2
      GO TO 1500
 1200 CONTINUE
      IF(ZPFIN.GE.(TH+HTIP)) GO TO 1210
      POIRAZ=POIRA2
      POIRAX=(POIRA1+POIRA2)/2.0
      POIRAR=POIRA1
      GMODZ=GMOD2
      GMODX=(GMOD1+GMOD2)/2.0
      GMODR=GMOD1
      GO TO 1500
 1210 CONTINUE
      IF(ZPFIN.GE.(TH+HTIP+HCON)) GO TO 1220
      POIRAZ=POIRA2
      POIRAX=POIRA2
      POIRAR=(POIRA1+POIRA2)/2.0
      GMODZ=GMOD2
      GMODX=GMOD2
      GMODR=(GMOD1+GMOD2)/2.0
      GO TO 1500
 1220 CONTINUE
      GO TO 1100
 1500 CONTINUE
C
C     CALCULATION OF ACTUAL DIAMETER FOR SOILSPRINGS.
C
      DACTU=2.0*SQRT(AO/3.14)
      STIKZI=4.0*GMODZ*DACTU/2.0*1.0/(1.0-POIRAZ)
      STIKXI=8.0*GMODX*DACTU/2.0*1.0/(2.0-POIRAX)
      STIKRI=8.0*GMODR*(DACTU/2.0)**3.0*1.0/(3.0*(1-POIRAR))
C
C     CALCULATION OF LOADS FROM GRAVITY
C
      IF(NL.EQ.4) GO TO 1550
C
C     CALCULATION FOR THREE LEGS
C
      A1=2.0*BL/3.0+GL
      B1=BL-A1
      A2=2.0*BL/3.0+VL
      B2=BL-A2
      GO TO 1560
 1550 CONTINUE
C
C     CALCULATION FOR FOUR LEGS
```

```
C
      A1=BL/2.0+GL
      B1=BL-A1
      A2=BL/2.0+VL
      B2=BL-A2
 1560 CONTINUE
      F(1)=+(WG-(NL*WGLEG))*B1/BL+WGV*B2/BL
      F(2)=+(WG-(NL*WGLEG))*A1/BL+WGV*A2/BL
      IF(NL.EQ.4) GO TO 1570
      FZ(1)=F(1)+WGLEG
      FZ(2)=0.5*F(2)+WGLEG
      FZ(3)=FZ(2)
      FZ(4)=0.0
      GO TO 1580
 1570 CONTINUE
      FZ(1)=0.5*F(1)+WGLEG
      FZ(2)=0.5*F(2)+WGLEG
      FZ(3)=FZ(1)
      FZ(4)=FZ(2)
 1580 CONTINUE
      STFKZG(1)=STIKZI*(1.0-FZ(1)/FVACT)**2.0
      STFKZG(2)=STIKZI*(1.0-FZ(2)/FVACT)**2.0
      STFKZG(3)=STIKZI*(1.0-FZ(3)/FVACT)**2.0
C
      STFKZI(1)=STIKZI
      STFKZI(2)=STIKZI
      STFKZI(3)=STIKZI
C
      STFKXI(1)=STIKXI
      STFKXI(2)=STIKXI
      STFKXI(3)=STIKXI
C
      STFKRI(1)=STIKRI
      STFKRI(2)=STIKRI
      STFKRI(3)=STIKRI
C
      IF(NL.EQ.3) GO TO 1590
      STFKZG(4)=STFKZG(2)
      STFKZI(4)=STFKZI(2)
      STFKXI(4)=STFKXI(2)
      STFKRI(4)=STFKRI(2)
 1590 CONTINUE
      RETURN
      END
```

```
      SUBROUTINE INPOL(FVACT,FVULT,ZPDEF,INPO,ZP)
C
C     PURPOSE:
C
C     METHOD:
C
C     INPUT ARGUMENTS:
C
C     INTERNAL PARAMETERS:
C
C     OUTPUT ARGUMENTS:
C
      IMPLICIT DOUBLE PRECISION (A-H,O-Z), INTEGER (I-N)
      COMMON/JU7/ZPFIN
      DIMENSION ZP(50),FVULT(50)
C
C     IF THE SOIL HAS HIGHER BEARING CAPASITY THAN MAX VERTICAL LOAD
C     DURING PRELOADING
C
      IF(FVACT.GE.FVULT(1)) GO TO 1000
      INPO=1
      GO TO 1200
 1000 CONTINUE
      DO 1100 IPEN=2,50
C
      IF(FVULT(IPEN).LE.0.0) GO TO 1300
      IF(FVULT(IPEN).LT.FVULT(IPEN-1)) GO TO 1100
      IF(FVACT.GT.FVULT(IPEN)) GO TO 1100
      INPO=IPEN
      GO TO 1200
 1100 CONTINUE
 1200 CONTINUE
      DELTA=(FVACT-FVULT(INPO-1))/(FVULT(INPO)-FVULT(INPO-1))
      DIFF=ZP(INPO)-ZP(INPO-1)
      ZPFIN=ZP(INPO-1)+DIFF*DELTA
      RETURN
 1300 CONTINUE
      WRITE(6,8050)FVACT
      WRITE(6,8100)ZPDEF
 8050 FORMAT(' VERTICAL LOAD ON ONE LEG DURING PRELOADING  =',E10.3/)
 8100 FORMAT(' LOAD ON SPUDCAN EXCEED BEARING CAPASITY DOWN '       /
     1       ' TO SPECIFIED DEPTH                          =',E10.3/)
      RETURN
      END
```

```
      SUBROUTINE INPUT
C
C     PURPOSE:THIS ROUTINE READ INPUT DATA AND PRINT THEM WITH SUPPLEMENTING
C     COMMENTS.
C     METHOD:
C     NONE.
C     INPUT ARGUMENTS:
C     INTERNAL PARAMETERS:
C     OUTPUT ARGUMENTS:
      IMPLICIT DOUBLE PRECISION (A-H,O-Z),INTEGER (I-N)
      CHARACTER*4 ITEXT1,ITEXT2(20)
      COMMON/JU2/ITEXT1(20)
      COMMON/JU4/POIRA1,POIRA2,GMOD1,GMOD2,FVACT,ZPFIN,STFKZI(4),
     .STFKXI(4),STFKRI(4)
      COMMON/JU5/PSTEEL,E,EIB,EAB,EI(4),EA(4),GA(4),
     .STFKZ(4),STFKX(4),STFKR(4),STF(12,12),
     .STF1(6,6),STF2(6,6),STF3(6,6)
      COMMON/JU1/IOP1,IOP2,IOP3,IOP4,IOP5,IOP6,TSET,BL,DW,AG,NL,WG,
     .GL,DI,DD,AW,PRECAP,DISPL,SPEPRE,WGLEG,ZPDEF,DTIP,DTOT,HTOP,HSID,
     .HCON,HTIP,HTOT,DMET,VWI,DARMW,HWM,TMIN,TMAX,TSTEP,GAMW,GAMMSO,
     .CI,CD,
     .VH1,VH2,WGV,VL,NPRO,SUO,RK,SUH,PHI,TH,RR,ALFA,
     .ZPD(50),ZPEN,ATOP(50),ADEP(50),ASIDD(50),ASIDT(50),
     .AO,ITR,COST,COSD,CO,COH,
     .FVULT(50),FVSU,FVPHI,FVSQUE,FVPUNC,BTOT,BEFF,FH,
     .FTOT(20,4,10),OMTOT(20,4,10),RLAR(20,4,10),FPRE(20,10),
     .FIVECT(12,22),OVTMP(20,10),RLARP(20,10),SPRNR,
     .PI,G,MSTART,ITSTRT,FHWIND
      COMMON/JU8/ZP(50),IPEN
C*************************************************************************
C     READ AND PRINT CONTROL DATA
C*************************************************************************
      READ(5,7000)(ITEXT1(I),I=1,20)
      READ(5,7000)(ITEXT2(I),I=1,20)
      READ(5,7010)IOP1,IOP2,IOP3,IOP4,IOP5,IOP6
      IF(IOP3.EQ.0) GO TO 1000
      WRITE(6,8000)(ITEXT1(I),I=1,20),(ITEXT2(I),I=1,20)
      WRITE(6,8010)IOP1,IOP2,IOP3,IOP4,IOP5,IOP6
 1000 CONTINUE
      IF(IOP1.EQ.0) GO TO 1010
      READ(5,7020)TSET
      IF(IOP3.EQ.0) GO TO 1010
      WRITE(6,8020)TSET
 1010 CONTINUE
C*************************************************************************
C     READ AND PRINT STRUCTURE GEOMETRY CARD
C*************************************************************************
      READ(5,7040)BL,DW,AG,NL,WG,GL,DI,DD
      IF(IOP3.EQ.0) GO TO 1020
      WRITE(6,8040)BL,DW,AG,NL,WG,GL,DI,DD
 1020 CONTINUE
      READ(5,7045)AW,PRECAP,SPEPRE,DISPL,WGLEG,ZPDEF
      IF(IOP3.EQ.0) GO TO 1030
      WRITE(6,8050)AW,PRECAP,SPEPRE,DISPL,WGLEG,ZPDEF
 1030 CONTINUE
C*************************************************************************
C     READ AND PRINT BARGE STIFFNESS CARD
C*************************************************************************
      READ(5,7050)EIB,EAB,GAB,E,PSTEEL
      IF(IOP3.EQ.0) GO TO 1040
      WRITE(6,8055)EIB,EAB,GAB,E,PSTEEL
```

```
 1040 CONTINUE
C*****************************************************************************
C      READ AND PRINT LEG STIFFNESS CARD
C*****************************************************************************
       IF(IOP3.EQ.0) GO TO 1045
       WRITE(6,8057)
 1045 CONTINUE
       DO 1050 I=1,NL
       READ(5,7060)EI(I),EA(I),GA(I)
       IF(IOP3.EQ.0) GO TO 1050
       WRITE(6,8060)I,EI(I),EA(I),GA(I)
 1050 CONTINUE
C*****************************************************************************
C      READ AND PRINT FOOTING GEOMETRI
C*****************************************************************************
       READ(5,7070) DTIP,DTOT,HTOP,HSID,HCON,HTIP,HTOT
       IF(IOP3.EQ.0) GO TO 1060
       WRITE(6,8070)DTIP,DTOT,HTOP,HSID,HCON,HTIP,HTOT
 1060 CONTINUE
C*****************************************************************************
C      READ AND PRINT ENVIROMENTAL CARDS
C*****************************************************************************
       READ (5,7080)VWI,DARMW,G
       READ (5,7090)HWM,TMAX,TMIN,TSTEP,GAMW,CI,CD
       IF(IOP3.EQ.0) GO TO 1070
       WRITE(6,8080)VWI,DARMW,G
       WRITE(6,8090)HWM,TMAX,TMIN,TSTEP,GAMW,CI,CD
 1070 CONTINUE
C*****************************************************************************
C      READ AND PRINT DYNAMIC CARD
C*****************************************************************************
C      READ (5,7100)TNAT,DAMP
C      IF(IOP3.EQ.0) GO TO 1080
C      WRITE(6,8100)TNAT,DAMP
C1080 CONTINUE
C*****************************************************************************
C      READ AND PRINT CURRENT CARD
C*****************************************************************************
       READ (5,7100)VH1,VH2
       IF(IOP3.EQ.0) GO TO 1090
       WRITE(6,8110)VH1,VH2
 1090 CONTINUE
C*****************************************************************************
C      READ AND PRINT VARIABLE LOAD
C*****************************************************************************
       READ (5,7120)WGV,VL
       IF(IOP3.EQ.0) GO TO 1100
       WRITE(6,8120)WGV,VL
 1100 CONTINUE
C*****************************************************************************
C      READ AND PRINT SOIL PROFILE DATA
C*****************************************************************************
       READ (5,7130)NPRO,SUO,RK,SUH,PHI,TH,RR,ALFA
       IF(IOP3.EQ.0) GO TO 1110
       WRITE(6,8130)NPRO,SUO,RK,SUH,PHI,TH,RR,ALFA
 1110 CONTINUE
C*****************************************************************************
C      READ AND PRINT SOIL STIFFNESS CARD
C*****************************************************************************
       READ (5,7140)POIRA1,POIRA2,GMOD1,GMOD2,GAMMSO
       IF(IOP3.EQ.0) GO TO 1120
```

```
      WRITE(6,8140)POIRA1,POIRA2,GMOD1,GMOD2,GAMMSO
 1120 CONTINUE
C*****************************************************************************
C     MAKE A TESTROUTINE FOR INPUT DATA.
C*****************************************************************************
C
C
C*****FORMAT*******************************************************************
 7000 FORMAT(20A4)
 7010 FORMAT(6I10)
 7020 FORMAT(F10.0)
 7040 FORMAT(3F10.0,I10,4E10.3)
 7045 FORMAT(6E10.3)
 7050 FORMAT(5E10.3)
 7060 FORMAT(3F10.3)
 7070 FORMAT(7F10.0)
 7080 FORMAT(3F10.0)
 7090 FORMAT(7F10.0)
 7100 FORMAT(2F10.0)
 7  0 FORMAT(E10.3,F10.0)
 7130 FORMAT(I10,7F10.0)
 7140 FORMAT(5E10.3)
 8000 FORMAT(1H1,3X,' PROGRAM "SAIL"'/' VERSION 8'/' SEPTEMBER 1983'/
     1       ' PROGRAMMED BY L. A. KRISTIANSEN, DET NORSKE VERITAS',///
     2       '********************************************************************
     3************************'/
     4       '*',20A4,/
     5       '*',20A4,/
     6       '********************************************************************
     7************************'//)
 8010 FORMAT(' INPUT DATA PRINTOUT:'/
     1       ' **********************'//
     2       ' CONTROL DATA                            '      /
     3       ' ************                            '      /
     4       ' OPTION NR.1,IOP1                       =',I10/
     5       ' (IOP1=0, CALCULATES ENVIROMENTAL FORCES ONLY.',/
     6       '  IOP1=1, CALCULATES ENVIROMENTAL FORCES AND ',/
     7       '  REST OF PROGRAM.)                        '//
     8       ' OPTION NR.2,IOP2                       =',I10/
     9       ' (IOP2=0, STRUCTURE IS EXPOSED TO WAVES MOVING '/
     1       '  IN POSITIVE X-DIRECTION.                     '/
     2       '  IOP2=1, STRUCTURE IS EXPOSED TO WAVES MOVING '/
     3       '  IN NEGATIVE X-DIRECTION.)                   '//
     4       ' OPTION NR.3,IOP3                       =',I10/
     5       ' (IOP3=0, NO PRINT OF INPUT DATA.            '/
     6       '  IOP3=1, PRINTS INFUT DATA.)               '//
     7       ' OPTION NR.4,IOP4                       =',I10/
     8       ' (IOP4=0, NO PRINT OF WAVE FORCES.          '/
     9       '  IOP4=1, PRINTS WAVE FORCES.)             '//
     1       ' OPTION NR.5,IOP5                       =',I10/
     2       ' (IOP5=0, NO PRINT OF PENETRATION.          '/
     3       '  IOP5=1, PRINTS PENETRATION DATA.)        '//
     4       ' OPTION NR.6,IOP6                       =',I10/
     5       ' (IOP6=0, TABULATED OVERVIEW OF TOTAL FORCES '/
     6       '  IOP6=1, PRINTS EACH LOADSTEP PLUS OVERVIEW.)'//)
 8020 FORMAT(' CHOSEN CRITICAL WAVE PERIOD            =',E10.3//)
 8 40 FORMAT(' STRUCTURE GEOMETRY                     '      /
     1       ' ******************                     '      /
     2       ' BREADTH BETWEEN LEGS, BL               =',E10.3/
     3       ' DEPTH OF WATER, DW                     =',E10.3/
     4       ' AIR GAP, AG                            =',E10.3/
```

```
     5         ' NUMBER OF LEGS, NL                        =',  I10/
     6         ' TOTAL STRUCTURE WEIGHT, WG                =',E10.3/
     7         ' GRAVITY LEVERARM, GL                      =',E10.3/
     8         ' EFFECTIVE DIAMETER INERTIA, DI            =',E10.3/
     9         ' EFFECTIVE DIAMETER DRAG, DD               =',E10.3)
 8050 FORMAT(' WIND AREA, AW                             =',E10.3/
     1         ' PRELOADING CAPACITY, PRECAP               =',E10.3/
     2         ' SPECEFIED PRELOAD, SPEPRE                 =',E10.3/
     3         ' TOTAL AVAILABLE LEGLENGTH, DISPL          =',E10.3/
     4         ' LEG WEIGHT, WGLEG                         =',E10.3/
     5         ' CALC. BEAR.CAP. DOWN TO, ZPDEF            =',E10.3//)
 8055 FORMAT(' BARGE STIFFNESS:                           '       /
     1         ' *****************                          '       /
     2         ' BENDING STIFFNESS, EIB                    =',E10.3/
     3         ' AXIAL STIFFNESS,   EAB                    =',E10.3/
     4         ' SHEAR STIFFNESS,   GAB                    =',E10.3/
     5         ' E MODULUS FOR STEEL, E                    =',E10.3/
     6         ' POISSONS RATIO, PSTEEL                    =',E10.3//)
 8057 FORMAT(' LEG STIFFNESS:                             '       /
     1         ' ***************                            '       /)
 8060 FORMAT(' LEG NR. ',I3,                                      /
     1         ' BENDING STIFFNESS, EI                     =',E10.3/
     2         ' AXIAL STIFNESS,    EA                     =',E10.3/
     3         ' SHEAR STIFFNESS,   GA                     =',E10.3//)
 8070 FORMAT(' FOOTING GEOMETRI:                          '       /
     1         ' *****************                          '       /
     2         ' TIP DIAMETER,     DTIP                    =',E10.3/
     3         ' TOTAL DIAMETER,   DTOT                    =',E10.3/
     4         ' TOP HEIGHT,       HTOP                    =',E10.3/
     5         ' SIDE HEIGHT,      HSID                    =',E10.3/
     6         ' CONE HEIGHT,      HCON                    =',E10.3/
     7         ' TIP HEIGHT,       HTIP                    =',E10.3/
     8         ' TOTAL HEIGHT,     HTOT                    =',E10.3//)
 8080 FORMAT(' ENVIROMENTAL DATA:                         '       /
     1         ' ******************                         '       /
     2         ' WIND VELOCITY, VW                         =',E10.3/
     3         ' LEVERARM WIND, DARMW                      =',E10.3/
     4         ' GRAVITY CONSTANT, G                       =',E10.3//)
 8090 FORMAT(' WAVE DATA:                                 '       /
     1         ' **********                                 '       /
     2         ' MAXIMUM WAVE HEIGHT, HWM                  =',E10.3/
     3         ' MAXIMUM WAVE PERIOD, TMAX                 =',E10.3/
     4         ' MINIMUM WAVE PERIOD, TMIN                 =',E10.3/
     5         ' PERIOD STEP                               =',E10.3/
     6         ' UNIT WEIGHT OF SEA WATER,GAMW             =',E10.3/
     7         ' INERTIA COEFFICIENT, CI                   =',E10.3/
     8         ' DRAG COEFFICIENT, CD                      =',E10.3//)
 8100 FORMAT(' DYNAMIC DATA:                              '       /
     1         ' *************                              '       /
     2         ' NATURAL FREQUENCY, TNAT                   =',E10.3/
     3         ' DAMPING COEFFICIENT, DAMP                 =',E10.3//)
 8110 FORMAT(' CURRENT DATA:                              '       /
     1         ' *************                              '       /
     2         ' TIDAL CURRENT AT S.W.L, VH1               =',E10.3/
     3         ' WIND GENERATED CURR. AT S.W.L, VH2        =',E10.3//)
 8120 FORMAT(' VARIABLE WEIGHT DATA:                      '       /
     1         ' *********************                      '       /
     2         ' VARIABLE WEIGHT, WGV                      =',E10.3/
     3         ' LEVERARM VARIABLE WEIGHT, VL              =',E10.3//)
 8130 FORMAT(' SOIL PROFILE DATA:                         '       /
     1         ' ******************                         '       /
```

```
     2        ' PROFILE NUMBER, NPRO                  =',  I10/
     3        ' SHEAR STRENGTH, SUO                   =',E10.3/
     4        ' SHEAR STRENGTH GRADIENT, RK           =',E10.3/
     5        ' SHEAR STRENGTH HARD LAYER, SUH        =',E10.3/
     6        ' INTERNAL FRICTION, PHI                =',E10.3/
     7        ' THICKNESS OF UPPER LAYER, TH          =',E10.3/
     8        ' ROUGHNESS RATIO, RR                   =',E10.3/
     9        ' PRESSURE INFLUENCE ANGEL, ALFA        =',E10.3//)
8140 FORMAT(' SOIL STIFFNESS DATA:                     '       /
     1        ' ********************                   '       /
     2        ' POISSONS RATIO UPPER LAYER, POIRA1 =',E10.3/
     3        ' POISSONS RATIO DEEP LAYER,  POIRA2 =',E10.3/
     4        ' G-MODULUS UPPER LAYER, GMOD1          =',E10.3/
     5        ' G-MODULUS DEEP LAYER,  GMOD2          =',E10.3/
     6        ' UNIT WEIGHT OF SOIL                   =',E10.3//)
      RETURN
      END
```

```
      SUBROUTINE INVM12(A,B,MA,EPS,IERR)
C
      IMPLICIT DOUBLE PRECISION (A-H,O-Z),INTEGER(I-N)
C
C*****************************************************************************
C
C
C
C     INVM12 INVERTS A SQUARE MATRIX  A  AND STORES THE RESULT IN  B
C
C
C
C*****************************************************************************
C
      DIMENSION A(MA,MA),B(MA,MA)
C
      COMMON/SAFE/ISAFE(50)
      COMMON/COM/IP(100),JP(200)
C
C      ISAFE(2)=6
C      ISAFE(11)=1
      IA = MA
      IB = 1
      IOP = -1
      IW = ISAFE(2)
      C = 0.
C
C     PRINT ?
C
      IF(ISAFE(11))10,10,5
    5 WRITE(IW,6000) MA
C
C     TRIANGULAR DECOMPOSITION OF  A
C
   10 CALL TRIM12(A,MA,EPS,IERR)
      IF(IERR)20,25,25
   20 WRITE(IW,6010)
      GO TO 200
C
   25 IF(MA-1)50,50,60
C
   50 B(1,1) = 1./A(1,1)
      GO TO 200
C
C     INTERCHANGE ROWS ON THE RIGHT-HAND SIDE
C
   60 DO 70 I=1,MA
   70 JP(I) = I
C
      DO 80 I=1,MA
      J = IP(I)
      K = JP(I)
      JP(I) = JP(J)
   80 JP(J) = K
C
C     SOLVE FOR THE INVERSE MATRIX
C
      MA1 = MA-1
C
      DO 150 K=1,MA1
      J = JP(K)
```

```
C
C     FORWARD SUBSTITUTION (COLUMN J)
C
      B(K,J) = 1./A(K,K)
      II = K+1
      DO 110 I=II,MA
      N = I-II+1
      E = PRAC10(A(I,K),B(K,J),C,IA,IB,N,IOP)
  110 B(I,J) = E/A(I,I)
C
C     BACKWARD SUBSTITUTION (COLUMN J)
C
      II = MA-K
      DO 120 N=1,II
      I = MA-N
      L = I+1
  120 B(I,J) = PRAC10(A(I,L),B(L,J),B(I,J),IA,IB,N,IOP)
      IF(K-1)150,150,130
  130 II = II+1
      DO 140 N=II,MA1
      I = MA-N
      L = I+1
  140 B(I,J) = PRAC10(A(I,L),B(L,J),C,IA,IB,N,IOP)
  150 CONTINUE
C
C     THE LAST COLUMN
C
      J = JP(MA)
      B(MA,J) = 1./A(MA,MA)
      DO 160 N=1,MA1
      I = MA-N
      L = I+1
  160 B(I,J) = PRAC10(A(I,L),B(L,J),C,IA,IB,N,IOP)
C
C     PRINT ?
C
  200 IF(ISAFE(11))300,300,210
  210 WRITE(IW,6020)
C
C
  300 CONTINUE
C
C     STOP EXECUTION IF IERR=-2
C
      IF(IERR.EQ.-2) GO TO 350
      RETURN
  350 CONTINUE
      STOP
C
 6000 FORMAT(///25H ENTERING INVM12 :    MA =,I4)
 6010 FORMAT(1X,///,' *** ERROR RETURN FROM INVM12 ***',/,
     1'  PROGRAM EXECUTION IS STOPPED BECAUSE THE INVERTED MATRIX ',/,
     2'  IS CLOSE TO SINGULARITY. YOU CAN TRY TO GIVE A LOWER VALUE ',/,
     3'  OF EPS IN THE ARGUMENT LIST WHEN CALLING SUBROUTINE INVM12. ')
 6020 FORMAT(/15H LEAVING INVM12)
C
      END
```

```
      SUBROUTINE LEGLEN(DW,AG,ZPFIN,DISPL)
C
C     PURPOSE:
C     THIS ROUTINE CALCULATES NECESSARY LEGLENGTH
C
C     METHOD:SIMPLE ADDITION
C
      DLEG=DW+AG+ZPFIN
      IF(DISPL.GT.DLEG)GO TO 1000
      WRITE(6,8000)DLEG
 8000 FORMAT('NECESSARY LEGLENGTH                              =',E10.3/
     1      ' ** WARNING! ** LEG IS TOO SHORT **               '       )
      GO TO 1010
 1000 CONTINUE
      WRITE(6,8100)DLEG
 8100 FORMAT(' NECESSARY LEGLENGTH                             =',E10.3)
 1010 CONTINUE
      RETURN
      END
```

```
      SUBROUTINE LOVECT(FIX)
C
C     PURPOSE:
C     THIS ROUTINE CREATES THE LOADVECTOR FOR THE DEFINED JACKUP MODEL.
C     GRAVITY IS ALREADY INCLUDED IN SUBROUTINE INITAL.
C     WINDFORCES IS ADDED TO SYSTEM IN TWO STEPS. THEN WAVE AND
C     CURRENT FORCES IS INTRODUCED.
C     METHOD:
C     STANDARD STATIC SOLUTION. FIRST CREATE A LOADVECTOR FOR WIND ONLY,
C     AND ONE FOR WAVE AND CURRENT. THEN COMBIEN THESE TWO INTO A
C     FINAL LOADVECTOR FOR ONE WAVE CYCLE.
C     INPUT ARGUMENTS:
C
C     INTERNAL PARAMETERS:
C
C     OUTPUT ARGUMENTS:
C
      IMPLICIT DOUBLE PRECISION (A-H,O-Z),INTEGER(I-N)
C     COMMON BLOCKS
      COMMON/JU1/IOP1,IOP2,IOP3,IOP4,IOP5,IOP6,TSET,BL,DW,AG,NL,WG,
     *GL,DI,DO,AW,PRECAP,DISPL,SPEPRE,WGLEG,ZPDEF,DTIP,DTOT,HTOP,HSID,
     *HCON,HTIP,HTOT,DMET,VWI,DARMW,HWM,TMIN,TMAX,TSTEP,GAMW,GAMMSO,
     *CI,CD,
     *VH1,VH2,WGV,VL,NPRO,SUO,RK,SUH,PHI,TH,RR,ALFA,
     *ZPD(50),ZPEN,ATOP(50),ADEP(50),ASIDD(50),ASIDT(50),
     *AO,ITR,COST,COSD,CO,COH,
     *FVULT(50),FVSU,FVPHI,FVSQUE,FVPUNC,BTOT,EEFF,FH,
     *FTOT(20,4,10),OMTOT(20,4,10),RLAR(20,4,10),FPRE(20,10),
     *FIVECT(12,22),OVTMP(20,10),RLARP(20,10),SPRNR,
     *PI,G,MSTART,ITSTRT,FHWIND
      COMMON/JU3/FORSEG(20,4,10),SEGARM(20,4,10)
      COMMON/JU7/ZPFIN
      COMMON/JU8/ZP(50),IPEN
      SAVE /JU1/,/JU3/
      DIMENSION FIX(3,12,22)
C
C     CALCULATE FIXED END FORCES AND MOMENTS FOR ELEMENTS.
C
C     ELEMENT NO.1
C
C     CALCULATE LEGLENGTH
C
      RLEG=AG+DW+(ZPFIN-HTIP-HCON)
C
C     CALCULATE FORCES ON ELEMENT FROM WAVE AND GRAVITY
C
      DO 1005 M=1,20
      DO 1000 NZ=1,10
      AFL=SEGARM(M,1,NZ)+(ZPFIN-HTIP-HCON)
      BFL=RLEG-AFL
C
C     FIXED END FORCES AND MOMENTS ON LEG NO.1
C
      FIX(1,1,M+2)=FIX(1,1,M+2)-FORSEG(M,1,NZ)*(BFL**2.0)*
     *(3.0*RLEG-2.0*BFL)/(RLEG**3.0)
      FIX(1,3,M+2)=FIX(1,3,M+2)-FORSEG(M,1,NZ)*AFL*(BFL**2.0)/
     *(RLEG**2.0)
      FIX(1,4,M+2)=FIX(1,4,M+2)-FORSEG(M,1,NZ)*(AFL**2.0)*
     *(3.0*RLEG-2.0*AFL)/(RLEG**3.0)
      FIX(1,6,M+2)=FIX(1,6,M+2)+FORSEG(M,1,NZ)*(AFL**2.0)*BFL/
     *(RLEG**2.0)
```

```
      IF(NL.EQ.4) GO TO 990
      GO TO 1000
  990 CONTINUE
C
C     A FOURLEGED JACKUP HAVE TWO LEGS ON LEFT SIDE OF MODEL
C     LEG NO.1 AND LEG NO.3
C
      AFL=SEGARM(M,3,NZ)+(ZPFIN-HTIP-HCON)
      BFL=RLEG-AFL
C
C     FIXED END FORCES AND MOMENTS ON LEG NO.3 ON FOUR LEG TYPE
C
      FIX(1,1,M+2)=FIX(1,1,M+2)-FORSEG(M,3,NZ)*(BFL**2.0)*
     .(3.0*RLEG-2.0*BFL)/(RLEG**3.0)
      FIX(1,3,M+2)=FIX(1,3,M+2)-FORSEG(M,3,NZ)*AFL*(BFL**2.0)/
     .(RLEG**2.0)
      FIX(1,4,M+2)=FIX(1,4,M+2)-FORSEG(M,3,NZ)*(AFL**2.0)*
     .(3.0*RLEG-2.0*AFL)/(RLEG**3.0)
      FIX(1,6,M+2)=FIX(1,6,M+2)+FORSEG(M,3,NZ)*(AFL**2.0)*BFL/
     .(RLEG**2.0)
 1000 CONTINUE
      IF(NL.EQ.4) GO TO 1001
      FIX(1,2,M+2)=0.0
      FIX(1,5,M+2)=0.0
      GO TO 1005
 1001 CONTINUE
      FIX(1,2,M+2)=0.0
      FIX(1,5,M+2)=0.0
 1005 CONTINUE
C
C     ELEMENT NO.2
C
C     THIS ELEMENT IS EXPOSED TO WIND AND GRAVITY,
C     GRAVITY IS ALREADY INCLUDED IN SUBROUTINE INITAL
C
C
      IF(NL.EQ.4) GO TO 1020
      IF(NL.NE.3) GO TO 1100
C
C     CALCULATION FOR THREE LEGS.
C
      FIX(2,4,1)=-FHWIND/(3.0*2.0)
      FIX(2,4,2)=2.0*FIX(2,4,1)
      FIX(2,6,1)=-FHWIND/3.0*DARMW/2.0
      FIX(2,6,2)=2.0*FIX(2,6,1)
      FIX(2,7,1)=-FHWIND*2.0/(3.0*2.0)
      FIX(2,7,2)=2.0*FIX(2,7,1)
      FIX(2,9,1)=-FHWIND*2.0/3.0*DARMW/2.0
      FIX(2,9,2)=2.0*FIX(2,9,1)
      GO TO 1030
C
C     CALCULATION FOR FOUR LEGS.
C
 1020 CONTINUE
      FIX(2,4,1)=-FHWIND/(2.0*2.0)
      FIX(2,4,2)=2.0*FIX(2,4,1)
      FIX(2,6,1)=-FHWIND/2.0*DARMW/2.0
      FIX(2,6,2)=2.0*FIX(2,6,1)
      FIX(2,7,1)=FIX(2,4,1)
      FIX(2,7,2)=2.0*FIX(2,7,1)
      FIX(2,9,1)=FIX(2,6,1)
```

```
      FIX(2,9,2)=2.0*FIX(2,9,1)
C
C     ELEMENT NO.3
C
C     CALCULATE FORCES FROM WAVE  AND  GRAVITY
C
 1030 CONTINUE
      RALEG=AG+DW+(ZPFIN-HTIP-HCON)
      DO 1055 M=1,20
      DO 1050 NZ=1,10
C
C     CALCULATES FORCES ON LEG NO.2 FOR BOTH THREE AND FOUR LEG TYPE.
C
      AAL=(ZPFIN-HCON-HTIP)+SEGARM(M,2,NZ)
      BAL=RALEG-AAL
      FIX(3,7,M+2)=FIX(3,7,M+2)-FORSEG(M,2,NZ)*(AAL**2.0)*
     *(3.0*RALEG-2.0*AAL)/(RALEG**3.0)
      FIX(3,9,M+2)=FIX(3,9,M+2)+FORSEG(M,2,NZ)*(AAL**2.0)*BAL/
     *(RALEG**2.0)
      FIX(3,10,M+2)=FIX(3,10,M+2)-FORSEG(M,2,NZ)*(BAL**2.0)*
     *(3.0*RALEG-2.0*BAL)/
     *(RALEG**3.0)
      FIX(3,12,M+2)=FIX(3,12,M+2)-FORSEG(M,2,NZ)*AAL*(BAL**2.0)/
     *(RALEG**2.0)
      IF(NL.EQ.4) GO TO 1040
C
C     CALCULATE FORCES ON LEG NO.3 FOR A THREE LEG TYPE
C
      AAL=(ZPFIN-HCON-HTIP)+SEGARM(M,3,NZ)
      BAL=RALEG-AAL
      FIX(3,7,M+2)=FIX(3,7,M+2)-FORSEG(M,3,NZ)*(AAL**2.0)*
     *(3.0*RALEG-2.0*AAL)/(RALEG**3.0)
      FIX(3,9,M+2)=FIX(3,9,M+2)+FORSEG(M,3,NZ)*(AAL**2.0)*BAL/
     *(RALEG**2.0)
      FIX(3,10,M+2)=FIX(3,10,M+2)-FORSEG(M,3,NZ)*(BAL**2.0)*
     *(3.0*RALEG-2.0*BAL)/
     *(RALEG**3.0)
      FIX(3,12,M+2)=FIX(3,12,M+2)-FORSEG(M,3,NZ)*AAL*(BAL**2.0)/
     *(RALEG**2.0)
      GO TO 1050
 1040 CONTINUE
C
C     CALCULATE FORCES ON LEG NO.4 FOR A FOUR LEG TYPE.
C
      AAL=(ZPFIN-HCON-HTIP)+SEGARM(M,4,NZ)
      BAL=RALEG-AAL
      FIX(3,7,M+2)=FIX(3,7,M+2)-FORSEG(M,4,NZ)*(AAL**2.0)*
     *(3.0*RALEG-2.0*AAL)/(RALEG**3.0)
      FIX(3,9,M+2)=FIX(3,9,M+2)+FORSEG(M,4,NZ)*(AAL**2.0)*BAL/
     *(RALEG**2.0)
      FIX(3,10,M+2)=FIX(3,10,M+2)-FORSEG(M,4,NZ)*(BAL**2.0)*
     *(3.0*RALEG-2.0*BAL)/
     *(RALEG**3.0)
      FIX(3,12,M+2)=FIX(3,12,M+2)-FORSEG(M,4,NZ)*AAL*(BAL**2.0)/
     *(RALEG**2.0)
 1050 CONTINUE
      FIX(3,8,M+2)=0.0
      FIX(3,11,M+2)=0.0
 1055 CONTINUE
C
C     LOADVECTOR FOR BOTH THREE AND FOUR LEG TYPE.
```

```
C
      DO 1060 I=1,2
      FIVECT(4,I)=-FIX(2,4,I)
      FIVECT(6,I)=-FIX(2,6,I)
      FIVECT(7,I)=-FIX(2,7,I)
      FIVECT(9,I)=-FIX(2,9,I)
 1060 CONTINUE
C
      DO 1070 M=3,22
      FIVECT(1,M)=-FIX(1,1,M)
      FIVECT(2,M)=-FIX(1,2,M)
      FIVECT(3,M)=-FIX(1,3,M)
      FIVECT(4,M)=-FIX(1,4,M)
      FIVECT(5,M)=-FIX(1,5,M)
      FIVECT(6,M)=-FIX(1,6,M)
      FIVECT(7,M)=-FIX(3,7,M)
      FIVECT(8,M)=-FIX(3,8,M)
      FIVECT(9,M)= FIX(3,9,M)
      FIVECT(10,M)=-FIX(3,10,M)
      FIVECT(11,M)=-FIX(3,11,M)
      FIVECT(12,M)=-FIX(3,12,M)
 1070 CONTINUE
C
C     TEMPORARY PRINT STATEMENT
C
C      DO 1080 M=1,22
C      WRITE(6,7000)(FIVECT(I,M),I=1,12)
C      1080 CONTINUE
      RETURN
 1100 CONTINUE
C 7000 FORMAT(1X,12E11.3)
      WRITE(6,8000)NL
 8000 FORMAT(3X,'ERROR IN INPUT DATA. NL=',I5)
      RETURN
      END
```

```
C***********************************************************************
      SUBROUTINE OVSTAB(WG,GL,BL,DTOT,WGV,VL,OMPPEN,IOP5)
C***********************************************************************
C                                                     DET NORSKE VERITAS
C     PURPOSE: CALCULATE AND CHECK OF OVERTURNING STABILITY
C
C     METHOD: DNV-RULES FOR CLASSIFICATION OF MOBILE OFFSHORE
C             UNITS, 1981
C
C     INPUT ARGUMENTS:
C     WG= STRUCTURE WEIGHT THAT REST ON SEABED.
C     GL= LEVERARM WEIGHT.
C     BL= BREADTH BETWEEN LEGS.
C     DTOT=TOTAL DIAMETER OF SPUDCAN.
C     WGV= VARIABLE LOAD TO BEE ELEVATED OR UNDER OPERATION.
C     VL= LEVERARM VARIABLE LOAD.
C     OMPPEN= MAXIMUM OVERTURNING MOMENT FOR ACTUAL PENETRATION.
C     IOP5=OPTION FOR NO PENETRATION PRINT
      IMPLICIT DOUBLE PRECISION (A-H,O-Z),INTEGER (I-N)
C
C     CALCULATE STABLIZING MOMENT
C
      STM=WG*(BL/3.0+DTOT/2.0-ABS(GL))+WGV*(BL/3.0+DTOT/2.0-ABS(VL))
      FOSAFE=ABS(STM/OMPPEN)
      IF(FOSAFE.GE.1.5)GO TO 1010
      IF(IOP5.EQ.0) GO TO 1000
      WRITE(6,8000)FOSAFE
 1000 CONTINUE
      GO TO 1020
 1010 CONTINUE
      IF(IOP5.EQ.0) GO TO 1020
      WRITE(6,8010)FOSAFE
 1020 CONTINUE
 8000 FORMAT(' ** WARNING! **                                      '   /
     1       ' SAFETY AGAINST OVERTURNING EXCEEDED, FOSAFE =',E10.3)
 8010 FORMAT(' SAFETY AGAINST OVERTURNING,INCLUDE VARIABLE    '      /
     1       ' AS SPECIFIED IN INPUT DATA, FOSAFE          =',E10.3)
      RETURN
      END
```

```
      SUBROUTINE PENETR(OMPREI,OMPPEN)
C
C     PURPOSE:PICK OUT VARIOUS DEFTH WHERE ONE WANT TO CALCULATE THE
C     BEARING CAPASITY.
C     INPUT ARGUMENTS:
C     OUTPUT ARGUMENTS:
      IMPLICIT DOUBLE PRECISION (A-H,O-Z), INTEGER (I-N)
C     COMMONBLOCKS
      COMMON/JU1/IOP1,IOP2,IOP3,IOP4,IOP5,IOP6,TSET,BL,DW,AG,NL,WG,
     *GL,DI,DD,AW,PRECAP,DISPL,SPEPRE,WGLEG,ZPDEF,DTIP,DTOT,HTOP,HSID,
     *HCON,HTIP,HTOT,DMET,VWI,DARMW,HWM,TMIN,TMAX,TSTEP,GAMW,GAMMSO,
     *CI,CD,
     *VH1,VH2,WGV,VL,NPRO,SUO,RK,SUH,PHI,TH,RR,ALFA,
     *ZPD(50),ZPEN,ATOP(50),ADEP(50),ASIDD(50),ASIDT(50),
     *AO,ITR,COST,COSD,CO,COH,
     *FVULT(50),FVSU,FVPHI,FVSQUE,FVPUNC,BTOT,BEFF,FH,
     *FTOT(20,4,10),OMTOT(20,4,10),RLAR(20,4,10),FPRE(20,10),
     *FIVECT(12,22),OVTMP(20,10),RLARP(20,10),SPRNR,
     *PI,G,MSTART,ITSTRT,FHWIND
      COMMON/JU8/ZP(50),IPEN
      IPEN=IPEN+1
C
C     LIMIT FOR PENETRATION CALCULATIONS EQUAL 50
C
      IF(IPEN.GT.49) GO TO 9000
      IF(IPEN.NE.1) GO TO 1000
      IF(ZP(IPEN).GE.HTIP) GO TO 1100
      ZP(IPEN)=HTIP/5.0
      GO TO 1400
 1000 CONTINUE
      IF(ZP(IPEN-1).GE.HTIP) GO TO 1100
      ZP(IPEN)=ZP(IPEN-1)+HTIP/5.0
      GO TO 1400
 1100 CONTINUE
      IF(IPEN.EQ.1) GO TO 1150
      IF(ZP(IPEN-1).GE.(HCON+HTIP)) GO TO 1200
      ZP(IPEN)=ZP(IPEN-1)+HCON/5.0
      GO TO 1400
 1150 CONTINUE
C     HERE IPEN=1, HTIP=0.
      ZP(IPEN)=HTIP+HCON/5.0
      GO TO 1400
 1200 CONTINUE
      IF(IPEN.EQ.0) GO TO 1250
      IF(ZP(IPEN-1).GE.(HTIP+HSID+HCON)) GO TO 1300
      ZP(IPEN)=ZP(IPEN-1)+HSID/5.0
      GO TO 1400
 1250 CONTINUE
C
C     HERE IPEN=1 AND HTIP=HCON=0.0
C
      ZP(IPEN)=HSID
      GO TO 1400
 1300 CONTINUE
      IF(ZP(IPEN-1).GT.ZPDEF) GO TO 1600
      ZP(IPEN)=ZP(IPEN-1)+(ZPDEF-HTIP-HCON-HSID)/25.0
      GO TO 1400
 1400 CONTINUE
      IF(IOP5.EQ.0) GO TO 1500
      WRITE(6,8000)ZP(IPEN)
 1500 CONTINUE
```

```
      OMPPEN=OMPREI*(ZP(IPEN)+DW)/DW
 1600 CONTINUE
      RETURN
 9000 CONTINUE
      WRITE(6,9999)IPEN
 8000 FORMAT('                                                    '////
     1        ' PENETRATION, ZP                                  =',E10.3)
 9999 FORMAT('*** ERROR MESSAGE ***                         IPEN=',I3,/,
     1        ' ZPDEF IS TOO LARGE, OVERFLOW IN STORAGE FOR  ',/,
     2        ' PENETRATION DEPTHS, MAX NUMBER = 48          ',/)
      STOP
      END
```

```
      FUNCTION PRAC10(A,B,C,INCA,INCB,N,IOP)
C
      IMPLICIT DOUBLE PRECISION (A-H,O-Z),INTEGER(I N)
C
C*********************************************************************
C
C
C
C     PRAC10 ACCUMULATES INNERPRODUCTS UTILIZING DOUBLE PRECISION
C     DEPENDING UPON INPUT PARAMETER IOP, THE RESULTING SUM MAY
C     BE SUBTRACTED FROM OR ADDED TO THE VALUE OF PARAMETER C.
C
C
C
C*********************************************************************
C
      DIMENSION A(1),B(1)
      DOUBLE PRECISION D,E,F
C
      IF (IOP) 10,30,20
   10 D=-C
      GO TO 40
   20 D=C
      GO TO 40
   30 D=0.
   40 IA=1
      IB=1
      DO 50 I=1,N
      E=A(IA)
      F=B(IB)
      D=D+E*F
      IA=IA+INCA
      IB=IB+INCB
   50 CONTINUE
      IF (IOP) 60,70,70
   60 D=-D
   70 PRAC10=D
      RETURN
      END
```

```
      SUBROUTINE PREL (WG,WGV,NL,BL,PRECAP,IOP5,OMPPEN,FVMAX)
C
C     PURPOSE: CALCULATE FVMAX AND NECESSARY PRELOAD.
C
C     METHOD:MOMENT EQULIBRIUM
C
C     INPUT ARGUMENTS:
C     WG= STRUCTURE WEIGHT THAT REST ON SEABED.
C     WGV= VARIABLE WEIGHT TO BEE ELEVATED OR UNDER OPERATION.
C     NL= NUMBER OF LEGS.
C     BL= BREADTH BETWEEN LEGS.
C     PRECAP=PRELOADCAPASITY=BALLAST TANKS.
C     IOP5= OPTION FOR STOPPING PENETRATION PRINT
C     IF IOP5=1 PRINT OF PENETRATION DATA.
C     IF IOP5=0 NO PRINT OF PENETRATION DATA.
C
C     INTERNAL PARAMETERS:
C     OMPPEN= MAKSIMUM OVERTURNING MOMENT ON STRUCTURE GENERATED BY
C             ENVIROMENTAL FORCES.
C     PRE=ADDITION LOAD NEEDED FOR PRELOAD IN ADDITION TO WEIGHT OF
C         STRUCTURE AND VARIABLE LOAD.
C     DIFF=EXTRA TEMPORARY LOAD NEEDED FOR PRELOAD, THE VARIABLE LOAD
C          IS ALREADY SUBTRACTED.
C
C     OUTPUT PARAMETERS:
C     FVMAX=MAKSIMUM VERTICAL LOAD.
C
      IMPLICIT DOUBLE PRECISION (A-H,O-Z),INTEGER (I-N)
      IF(NL.EQ.4) GO TO 1000
      FVMAX=(WG+WGV)/3+ABS(OMPPEN/BL)
      GO TO 1050
 1000 CONTINUE
      FVMAX=(WG+WGV)/4+ABS(OMPPEN/BL/2.0)
 1050 CONTINUE
      IF(IOP5.EQ.0) GO TO 2000
      WRITE(6,8000)FVMAX
 2000 CONTINUE
      PRE=(FVMAX-WG/NL-WGV/NL)*NL
      IF(IOP5.EQ.0) GO TO 2010
      WRITE(6,8010)PRE
 2010 CONTINUE
      DIFF=PRE-PRECAP
      IF(DIFF.GT.0.0)GO TO 2030
      GO TO 2040
 2030 CONTINUE
      WRITE(6,8020)DIFF
 2040 CONTINUE
 8000 FORMAT(' MAX VERTICAL FORCE ON ONE LEG, FVMAX        =',E10.3)
 8010 FORMAT(' TOTAL LOAD NEEDED FOR PRELOADING              ',/,
     1       ' INCLUDING SEAWATER AND VARIABLE              =',E10.3)
 8020 FORMAT(' TEMPORARY LOAD NEEDED FOR PRELOAD,           ',/,
     1       ' BECAUSE PRELOADCAPACITY ARE EXCEEDED,        ',/,
     2       ' GIVEN VARIABLE LOAD IS ALREADY SUBTRACTED   =',E10.3)
      RETURN
      END
```

```
      SUBROUTINE PREPEN
C
C     PURPOSE: THIS ROUTINE AND THE CONNECTED SUBROUTINES
C              CALCULATES THE BEARING CAPACITY OF A SPUD-CAN
C              SHAPED FOOTING PLACED IN ONE OF THE THREE
C              DESCRIBED SOIL PROFILES(SEE INPUT MANUAL)
C              AND PLACED IN A GIVEN DEPTH.
C
C     METHOD: VARIUOS MODIFIED BRINCH HANSEN'S FORMULAS.
C
C     INPUT ARGUMENTS:
C     INTERNAL VARIABLES:
C     OUTPUT ARGUMENTS:
C
      IMPLICIT DOUBLE PRECISION (A-H,O-Z),INTEGER (I N)
      COMMON/JU1/IOP1,IOP2,IOP3,IOP4,IOP5,IOP6,TSET,BL,DW,AG,NL,WG,
     *GL,DI,DO,AW,PRECAP,DISPL,SPEPRE,WGLEG,ZPCEF,DTIP,DTOT,HTOP,HSID,
     *HCON,HTIP,HTOT,DMET,VWI,DARMW,HWM,TMIN,TMAX,TSTEP,GAMW,GAMMSO,
     *CI,CD,
     *VH1,VH2,WGV,VL,NPRO,SUO,RK,SUH,PHI,TH,RR,ALFA,
     *ZPD(50),ZPEN,ATOP(50),ADEP(50),ASIDO(50),ASIDT(50),
     *AO,ITR,COST,COSD,CO,COH,
     *FVULT(50),FVSU,FVPHI,FVSQUE,FVPUNC,BTOT,BEFF,FH,
     *FTOT(20,4,10),OMTOT(20,4,10),RLAR(20,4,10),FPRE(20,10),
     *FIVECT(12,22),OVTMP(20,10),RLARP(20,10),SPRNR,
     *PI,G,MSTART,ITSTRT,FHWIND
      COMMON/JU6/FVULTI
      COMMON/JU7/ZPFIN
      COMMON/JU8/ZP(50),IPEN
      SAVE /JU1/,/JU2/,/JU3/
C
      IF(NPRO.GT.1)GO TO 1000
      CALL SPUD
      IF(ZP(IPEN).GT.TH)GO TO 500
      DEPT=SQRT(ATOP(IPEN))*0.5
      IF(ZP(IPEN).GE.(HTIP+HCON))DEPT=ZP(IPEN)-HCON-HTIP+DEPT
      IF(DEPT.GT.TH)GO TO 100
      COTOP=SUO+RK*DEPT
      GO TO 110
  100 CONTINUE
      COTOP=SUO+RK*TH
  110 CONTINUE
C
C     CALCULATION OF COHESION FOR SIDEFRICTION
C
      COST=SUO+(ZP(IPEN)-HTOT/2.0)*RK
      CO=COTOP
      COSD=0.0
      AO=ATOP(IPEN)
      CALL BHSU
      T=TH-ZP(IPEN)-HTIP-HCON
      IF(T.LE.(0.3*BEFF))GO TO 120
      FVSQUE=0.0
      GO TO 130
  120 CONTINUE
      ZPACT=ZP(IPEN)
      CALL BHSQUE(NPRO,BTOT,ZPACT,SUO,RK,TH,FH,GAMMSO,
     *AO,ASIDT,HTIP,HCON,HSID,IOP5,FVSQUE)
      CALL PUNCH
  130 CONTINUE
      IF(FVSU.GE.FVSQUE) FVUL=FVSU
```

```
      IF(FVSQUE.GE.FVSU)FVUL=FVSQUE
      IF(FVPUNC.LE.0.0) GO TO 140
      IF(FVPUNC.LT.FVUL) GO TO 150
  140 CONTINUE
      FVULT(IPEN)=FVUL
      GO TO 160
  150 CONTINUE
      FVULT(IPEN)=FVPUNC
  160 CONTINUE
      IF(IOP5.EQ.0) GO TO 170
      WRITE(6,6000)FVULT(IPEN)
  170 CONTINUE
      FVULTI=FVULT(IPEN)
      RETURN
  500 CONTINUE
      IF(PHI.GT.0.0)GO TO 700
      IF((ATOP(IPEN)-ADEP(IPEN)).GT.0.0) GO TO 510
C
C     SPUD-CAN BASE FULLY PENETRATED IN DEEP LAYER.
C
      COSD=SUH
      COST=SUO+RK*TH
      AO=ADEP(IPEN)
      CO=SUH
      CALL BHSU
      FVULT(IPEN)=FVSU
      GO TO 520
C
C     SPUD-CAN BASE PARTLY IN TOP LAYER, PARTLY IN DEEP LAYER.
C
  510 CONTINUE
      COST=SUO+RK*TH
      COSD=0.0
      CO=SUH
      AO=ATOP(IPEN)+ADEP(IPEN)
      CALL BHSU
      FVULT(IPEN)=FVSU
  520 CONTINUE
      IF(IOP5.EQ.0)GO TO 530
      WRITE(6,6000)FVULT(IPEN)
  530 CONTINUE
      FVULTI=FVULT(IPEN)
      RETURN
  700 CONTINUE
      COH=SUH
      COST=SUO+RK*TH
      IF((ATOP(IPEN)-ADEP(IPEN)).GT.0.0) GO TO 710
      AO=ADEP(IPEN)
      CALL TRANSF
      CALL BHPHI
      GO TO 720
  710 CONTINUE
      AO=ATOP(IPEN)+ADEP(IPEN)
      CALL TRANSF
      CALL BHPHI
  720 CONTINUE
      FVPHI=FVPHI+ASIDT(IPEN)*COST*RR
      FVULT(IPEN)=FVPHI
      IF(IOP5.EQ.0) GO TO 730
      WRITE (6,6000)FVULT(IPEN)
  730 CONTINUE
```

```
      FVULTI=FVULT(IPEN)
      RETURN
 1000 CONTINUE
      IF(NPRO.GT.2) GO TO 2000
      CALL SPUD
      IF(ZP(IPEN).GT.TH)GO TO 1500
      COTOP=SUH
      COST=SUH
      CO=COTOP
      COSD=0.0
      AO=ATOP(I)
      CALL BHSU
      CALL PUNCH
      IF(FVSU.GT.FVPUNC)GO TO 1100
      FVULT(IPEN)=FVSU
      GO TO 1200
 1100 CONTINUE
      FVUL=FVPUNC
      FVULT(IPEN)=FVUL
 1200 CONTINUE
      IF(IOP5.EQ.0) GO TO 1300
      WRITE (6,6000)FVULT(IPEN)
 1300 CONTINUE
      FVULTI=FVULT(IPEN)
      RETURN
 1500 CONTINUE
      IF((ATOP(IPEN)-ADEP(IPEN)).GT.0.0) GO TO 1600
C
C     SPUD-CAN BASE FULLY PENETRATED IN DEEP LAYER.
C
      DEP=SQRT(ADEP(IPEN))*0.5
      DEPT=DEP+ZP(IPEN)-TH-HCON-HTIP
      COTOP=0.0
      CODEP=SUO+RK*DEPT
      CO=CODEP
      COSD=SUO+RK*(ZP(IPEN)-TH-HTIP-HCON)
      COST=SUH
      AO=ADEP(IPEN)
      CALL BHSU
      FVULT(IPEN)=FVSU
      GO TO 1800
 1600 CONTINUE
C
C     SPUD-CAN BASE PARTLY PENETRATED IN DEEP LAYER.
C
      AO=(ATOP(IPEN)+ADEP(IPEN))
      DEPT=ZP(IPEN)+0.5*SQRT(AO)-HTIP-HCON
      IF(DEPT.LE.TH) GO TO 1650
      CODEP=SUO+RK*DEPT
      GO TO 1700
 1650 CONTINUE
      CODEP=SUO
 1700 CONTINUE
      COST=SUH
      COSD=0.0
      CO=CODEP
      CALL BHSU
      FVULT(IPEN)=FVSU
 1800 CONTINUE
      IF(IOP5.EQ.0) GO TO 1900
      WRITE(6,6000)FVULT(IPEN)
```

```
1[illegible]0 CONTINUE
      FVULTI=FVULT(IPEN)
      RETURN
 2000 CONTINUE
      IF(NPRO.GT.3)GO TO 3000
      CALL SPUD
      IF(ZP(IPEN).GT.TH)GO TO 2500
      COST=COH
      COSD=0.0
      CALL PUNCH
      CALL BHPHI
C
C     NO CONTRIBUTION TO BEARING CAPACITY FROM FRICTION
C     BETWEEN SAND AND SIDE-WALL OF SPUD-CAN.
C     THIS IS DONE BECAUSE NORMAL DESIGNED SPUD-CANS WILL NOT
C     PENETRATE WITH THE SPUD-CAN SIDE IN SAND.
C
      IF(FVPHI.GE.FVPUNC)GO TO 2100
      FVUL=FVPHI
      GO TO 2200
 2100 CONTINUE
      FVUL=FVPUNC
 2200 CONTINUE
      FVULT(IPEN)=FVUL
      IF(IOP5.EQ.0) GO TO 2300
      WRITE(6,6000)FVULT(IPEN)
 2300 CONTINUE
      FVULTI=FVULT(IPEN)
      RETURN
 2500 CONTINUE
      IF((ATOP(IPEN)-ADEP(IPEN)).GT.0.0) GO TO 2600
C
C     SPUD-CAN BASE FULLY PENETRATED IN DEEP LAYER.
C
      DEP=SQRT(ADEP(IPEN))*0.5
      DEPT=DEP+ZP(IPEN)-TH-HCON-HTIP
      COTOP=0.0
      CODEP=SUO+RK*DEPT
      CO=CODEP
      COSD=SUO+RK*(ZP(IPEN)-TH-HTIP-HCON)
      COST=0.0
      AO=ADEP(IPEN)
      CALL BHSU
      FVULT(IPEN)=FVSU
      GO TO 2800
 2600 CONTINUE
C
C     SPUD-CAN BASE PARTLY PENETRATED IN DEEP LAYER.
C
      AO=(ATOP(IPEN)+ADEP(IPEN))
      DEPT=ZP(IPEN)+0.5*SQRT(AO)-HTIP-HCON
      IF(DEPT.LE.TH) GO TO 2650
      CODEP=SUO+RK*DEPT
      GO TO 2700
 2650 CONTINUE
      CODEP=SUO
 2700 CONTINUE
      COST=0.0
      COSD=0.0
      CO=CODEP
      CALL BHSU
```

```
      FVULT(IPEN)=FVSU
 2800 CONTINUE
      IF(IOP5.EQ.0) GO TO 2900
      WRITE(6,6000)FVULT(IPEN)
 2900 CONTINUE
      FVULTI=FVULT(IPEN)
      RETURN
 3000 CONTINUE
      WRITE(6,9000)
      RETURN
 9000 FORMAT(' ** ERROR IN PROFILENUMBER FOR SOILDATA! **      '      )
 6000 FORMAT(' BEARING CAPACITY VERTICAL LOADING,               ',/
     1       ' FOR ONE SINGLE LEG,  FVULT                     =',E10.3)
      END
```

```
      SUBROUTINE PRINT(IOP6,M,MLOAD,DISP,FORCE,SFS,SFB,
     *TOTFOR,TDISP,IOP5)
C
C     PURPOSE:
C
C     INPUT ARGUMENTS:
C
C     INTERNAL PARAMETERS:
C
C     OUTPUT ARGUMENTS:
C
      IMPLICIT DOUBLE PRECISION (A-H,O-Z), INTEGER (I-N)
      DIMENSION DISP(12,22),FORCE(12,22),SFS(2,22),SFB(2,22),
     *TOTFOR(12,22),TDISP(12,22)
C
C     PRINT SWITCH
C
      IF(IOP6.EQ.0) GO TO 1015
      WRITE(6,8045)
      IF(M.LE.2) GO TO 900
      MPRINT=M-2
  900 CONTINUE
      IF(MLOAD.GT.1) GO TO 1000
      IF(MLOAD.LT.1) GO TO 1040
      WRITE(6,8000)
      WRITE(6,8010)FORCE(1,MLOAD),TOTFOR(1,MLOAD),FORCE(2,MLOAD),
     *TOTFOR(2,MLOAD),FORCE(3,MLOAD),TOTFOR(3,MLOAD),DISP(1,MLOAD),
     *TDISP(1,MLOAD),DISP(2,MLOAD),TDISP(2,MLOAD),DISP(3,MLOAD),
     *TDISP(3,MLOAD),SFS(1,MLOAD),SFB(1,MLOAD),FORCE(6,MLOAD),
     *TOTFOR(6,MLOAD)
      WRITE(6,8020)FORCE(10,MLOAD),TOTFOR(10,MLOAD),FORCE(11,MLOAD),
     *TOTFOR(11,MLOAD),FORCE(12,MLOAD),TOTFOR(12,MLOAD),DISP(10,MLOAD),
     *TDISP(10,MLOAD),DISP(11,MLOAD),TDISP(11,MLOAD),DISP(12,MLOAD),
     *TDISP(12,MLOAD),SFS(2,MLOAD),SFB(2,MLOAD),FORCE(9,MLOAD),
     *TOTFOR(9,MLOAD)
      IOP5=0
      GO TO 1050
 1000 CONTINUE
      IF(MLOAD.GT.2) GO TO 1010
      IF(MLOAD.LT.2) GO TO 1040
      WRITE(6,8030)
      WRITE(6,8010)FORCE(1,MLOAD),TOTFOR(1,MLOAD),FORCE(2,MLOAD),
     *TOTFOR(2,MLOAD),FORCE(3,MLOAD),TOTFOR(3,MLOAD),DISP(1,MLOAD),
     *TDISP(1,MLOAD),DISP(2,MLOAD),TDISP(2,MLOAD),DISP(3,MLOAD),
     *TDISP(3,MLOAD),SFS(1,MLOAD),SFB(1,MLOAD),FORCE(6,MLOAD),
     *TOTFOR(6,MLOAD)
      WRITE(6,8020)FORCE(10,MLOAD),TOTFOR(10,MLOAD),FORCE(11,MLOAD),
     *TOTFOR(11,MLOAD),FORCE(12,MLOAD),TOTFOR(12,MLOAD),DISP(10,MLOAD),
     *TDISP(10,MLOAD),DISP(11,MLOAD),TDISP(11,MLOAD),DISP(12,MLOAD),
     *TDISP(12,MLOAD),SFS(2,MLOAD),SFB(2,MLOAD),FORCE(9,MLOAD),
     *TOTFOR(9,MLOAD)
      GO TO 1050
 1010 CONTINUE
      WRITE(6,8040)MLOAD,MPRINT
      WRITE(6,8010)FORCE(1,MLOAD),TOTFOR(1,MLOAD),FORCE(2,MLOAD),
     *TOTFOR(2,MLOAD),FORCE(3,MLOAD),TOTFOR(3,MLOAD),DISP(1,MLOAD),
     *TDISP(1,MLOAD),DISP(2,MLOAD),TDISP(2,MLOAD),DISP(3,MLOAD),
     *TDISP(3,MLOAD),SFS(1,MLOAD),SFB(1,MLOAD),FORCE(6,MLOAD),
     *TOTFOR(6,MLOAD)
      WRITE(6,8020)FORCE(10,MLOAD),TOTFOR(10,MLOAD),FORCE(11,MLOAD),
     *TOTFOR(11,MLOAD),FORCE(12,MLOAD),TOTFOR(12,MLOAD),DISP(10,MLOAD),
```

```
     .TDISP(10,MLOAD),DISP(11,MLOAD),TDISP(11,MLOAD),DISP(12,MLOAD),
     .TDISP(12,MLOAD),SFS(2,MLOAD),SFB(2,MLOAD),FORCE(9,MLOAD),
     .TOTFOR(9,MLOAD)
 1015 CONTINUE
      IOP5=0
      IF(MLOAD.LT.22) GO TO 1035
      WRITE(6,8045)
      WRITE(6,8050)
      DO 1020 MLOAD=1,22
      WRITE(6,8060)MLOAD,TOTFOR(1,MLOAD),TOTFOR(2,MLOAD),
     1TOTFOR(3,MLOAD),TDISP(1,MLOAD),TDISP(2,MLOAD),TDISP(3,MLOAD),
     2SFS(1,MLOAD),SFB(1,MLOAD)
 1020 CONTINUE
      WRITE(6,8070)
      DO 1030 MLOAD=1,22
      WRITE(6,8060)MLOAD,TOTFOR(10,MLOAD),TOTFOR(11,MLOAD),
     1TOTFOR(12,MLOAD),TDISP(10,MLOAD),TDISP(11,MLOAD),TDISP(12,MLOAD),
     2SFS(2,MLOAD),SFB(2,MLOAD)
 1030 CONTINUE
 1035 CONTINUE
      IF(IOP6.EQ.0) GO TO 1060
      GO TO 1050
 1040 CONTINUE
      WRITE(6,9000) MLOAD
 1050 CONTINUE
 1060 CONTINUE
      RETURN
C     FORMAT STATEMENTS
C
 8000 FORMAT(1X,///
     1      ' LOADCASE NO.1                                      '    /
     2      ' STRUCTURWEIGHT AND HALF WINDLOAD                   '    /)
 8010 FORMAT(' LOADS ON LEFT SPUDCAN:                STEP VALUES '
     1      '        TOTAL VALUES '                                   //
     2      ' HORIZONTAL FORCE                       FH    =',E10.3,
     3      '   TOTFH   ='                                    ,E10.3/
     4      ' VERTICAL FORCE                         FV    =',E10.3,
     5      '   TOTFV   ='                                    ,E10.3/
     6      ' MOMENT FORCE                           M     =',E10.3,
     7      '   TOTMOM ='                                     ,E10.3//
     8      ' DISPLACEMENT ON LEFT SPUDCAN:                   '       /
     9      ' HORIZONTAL DISPLACEMENT                DH    =',E10.3,
     1      '   TOTDH   ='                                    ,E10.3/
     2      ' VERTICAL DISPLACEMENT                  DV    =',E10.3,
     3      '   TOTDV   ='                                    ,E10.3/
     4      ' ROTATION                               R     =',E10.3,
     5      '   TOTR    ='                                    ,E10.3/
     6      ' SAFETY AGAINST SLICING                        ',10X,
     7      '   SFS     ='                                    ,E10.3/
     8      ' SAFETY AGAINST VERTICAL FAILURE               ',10X,
     9      '   SFB     ='                                    ,E10.3///
     1      ' TOP LEG MOMENT                         TLM   =',E10.3,
     2      '   TOTTLM ='                                     ,E10.3/)
 8020 FORMAT(1X,///
     1      ' LOADS ON RIGHT SPUDCAN:               STEP VALUES '
     2      '                TOTAL VALUES '                           //
     3      ' HORIZONTAL FORCE                       FH    =',E10.3,
     4      '   TOTFH   ='                                    ,E10.3/
     5      ' VERTICAL FORCE                         FV    =',E10.3,
     6      '   TOTFV   ='                                    ,E10.3/
     7      ' MOMENT FORCE                           M     =',E10.3,
```

```
     8         '   TOTMOM ='                                           ,E10.3//
     9         ' DISPLACEMENT ON RIGHT SPUDCAN:                         '      /
     1         ' HORIZONTAL DISPLACEMENT                      DH    =',F10.3,
     2         '   TOTDH  ='                                           ,E10.3/
     3         ' VERTICAL DISPLACEMENT                        DV    =',E10.3,
     4         '   TOTDV  ='                                           ,E10.3/
     5         ' ROTATION                                     R     =',F10.3,
     6         '   TOTR   ='                                           ,E10.3/
     7         ' SAFETY AGAINST SLIDING                              ',10X,
     8         '   SFS    ='                                           ,F10.3/
     9         ' SAFETY AGAINST VERTICAL FAILURE                     ',10X,
     1         '   SFB    ='                                           ,E10.3///
     2         ' TOP LEG MOMENT                               TLM   =',F10.3,
     3         '   TOTTLM ='                                           ,E10.3/)
8030 FORMAT(1X,///
     1         ' LOADCASE NO.2                                          '      /
     1         ' STRUCTURE WEIGHT AND FULL WINDLOAD                     '      /)
8040 FORMAT(1X,///
     1          ' LOADCASE NR.',I2,' STRUCTURE WEIGHT,FULL WINDLOAD'
     2          ' AND WAVELOAD (STEP NUMBER M=',I2,')'                          /)
8045 FORMAT('1')
8050 FORMAT(1X,/,' SOIL REACTION FORCES ON LEFT SPUDCAN:     '    //
     1  ' LOADCASE     FHTOT         FVTOT         MTOT          DHTOT    '
     2  '    DVTOT        TOTROT        SFS           SFB        '      /)
8060 FORMAT(5X,I3,8E12.3)
8070 FORMAT(1X,///,' SOIL REACTION FORCES ON RIGHT SPUDCAN:   '    //
     1  ' LOADCASE     FHTOT         FVTOT         MTOT          DHTOT    '
     2  '   DVTOT         TOTROT        SFS           SFB        '      /)
9000 FORMAT(' ERROR IN MLOAD NUMBER - MLOAD =',I5     )
     END
```

```
      SUBROUTINE PUNCH
C
C     PURPOSE: THIS ROUTINE CALCULATE THE BEARING CAPACITY OF A
C     HARD LAYER OVERLAYING A SOFT LAYER.
C
C     METHOD:TAKEN FROM A DNV-REPORT
C     INPUT ARGUMENTS:
C     INTERNAL VARIABLES:
C     OUTPUT ARGUMENTS:
      IMPLICIT DOUBLE PRECISION (A-H,O-Z),INTEGER (I-N)
      COMMON/JU1/IOP1,IOP2,IOP3,IOP4,IOP5,IOP6,TSET,BL,DW,AG,NL,WG,
     *GL,DI,DO,AW,PRECAP,DISPL,SPEPRE,WGLEG,ZPDEF,DTIP,DTOT,HTOP,HSID,
     *HCON,HTIP,HTOT,DMET,VWI,DARMW,HWM,TMIN,TMAX,TSTEP,GAMW,GAMMSO,
     *CI,CD,
     *VH1,VH2,WGV,VL,NPRO,SUO,RK,SUH,PHI,TH,RR,ALFA,
     *ZPD(50),ZPEN,ATOP(50),ADEP(50),ASIDD(50),ASIDT(50),
     *AO,ITR,COST,COSD,CO,COH,
     *FVULT(50),FVSU,FVFHI,FVSQUE,FVPUNC,BTOT,BEFF,FH,
     *FTOT(20,4,10),OMTOT(20,4,10),RLAR(20,4,10),FPRE(20,10),
     *FIVECT(12,22),OVTMP(20,10),RLARP(20,10),SPRNR,
     *PI,G,MSTART,ITSTRT,FHWIND
      COMMON/JU8/ZP(50),IPEN
C     CALCULATE BOTH EFFECTIVE FOUNDATION AREA AND TOTAL
C     FOUNDATION AREA
      CALL TRANSF
C     IF NPRO=1, SOFT SOILLAYER OVERLAYING A HARD ONE,
C     STRESSDISTRIBUTION ANGLE IS SET EQUAL TO 0.1 DEGREES.
      IF(NPRO.EQ.1) ALFA=0.1
C     CALCULATE ANGEL IN RADIANS.
      ALFAR=ALFA*PI/180.0
C     EVALUATE DEPT TO USE IN CALCULATION.
      IF(ZP(IPEN).LT.HTIP) GO TO 10
      IF(ZP(IPEN).LT.HTIP+HCON) GO TO 20
      THRED=TH-ZP(IPEN)+HTIP+HCON
      GO TO 30
   10 CONTINUE
      THRED=TH-ZP(IPEN)
      GO TO 30
   20 CONTINUE
      THRED=TH-ZP(IPEN)+HTIP
   30 CONTINUE
      BMEFF=BEFF+2.0*THRED*TAN(ALFAR)
      BMTOT=BTOT+2.0*THRED*TAN(ALFAR)
      AOP=BMEFF*BMTOT
      AO=AOP
      IF(NPRO.EQ.1) GO TO 200
      IF(NPRO.EQ.2) GO TO 100
      IF(NPRO.EQ.3) GO TO 100
      IF(NPRO.GT.3) GO TO 900
  100 CONTINUE
      CO=SUO+0.5*RK*SQRT(AO/3.14)
      GO TO 300
  200 CONTINUE
      IF(PHI.GE.0.01) GO TO 250
      CO=SUH
      GO TO 300
  250 CONTINUE
      CALL BHPHI
C
C     ASSUME NO CONTRIBUTION FROM SIDEFRICTION ON SPUDCANSIDE IN SAND
C     BECAUSE USUALLY NO PENETRATION OF SPUDCAN-BASE IN SANDLAYER
```

```
C
      FVPUNC=FVPHI+(TH-THRED)*GAMMSO*ATOP(IPEN)
      GO TO 400
  300 CONTINUE
C     COMPUTE COEFFICIENTS TO BEE USED IN BRINCH HANSENS FORMULA.
      DCA=0.3*ATAN(TH/BMEFF)
C
C     TO AVOID NEGATIVE VALUE IN SQUAREROOT.
C
      IF(ABS(FH/(AO*CO)).GT.1.0) GO TO 325
      CICA=0.5-0.5*SQRT(1.0-ABS(FH/(AO*CO)))
      GO TO 350
  325 CONTINUE
      CICA=0.5
  350 CONTINUE
      SCA=0.2*(1.0-2.0*CICA)*BMEFF/BMTOT
      RNC=5.14
      IF(NPRO.EQ.1) GO TO 360
C
C     BRINCH HANSENS FORMULA FOR SU-MATERIAL:
C
      QSU=RNC*CO*(1.0+SCA+DCA-CICA)+GAMMSO*(TH-THRED)
     *+ASIDT(IPEN)*SUH
      GO TO 380
  360 CONTINUE
      IF(ZPEN.GT.(HTIP+HCON+HSID)) GO TO 365
      ZSFRIC=ZPEN-HTIP-HCON-0.5*(ZPEN-HTIP-HCON)
      GO TO 370
  365 CONTINUE
      ZSFRIC=ZPEN-HTIP-HCON-HSID/2.0
  370 CONTINUE
      SUSIDE=SUO+RK*ZSFRIC
      QSU=RNC*CO*(1.0+SCA+DCA-CISA)+GAMMSO*(TH-THRED)
     *+ASIDT(IPEN)*SUSIDE
  380 CONTINUE
      FVPUNC=QSU*AO
  400 CONTINUE
      IF(IOP5.EQ.0) GO TO 500
      WRITE(6,8000) FVPUNC
  500 CONTINUE
      AO=ATOP(IPEN)
      RETURN
  900 CONTINUE
      WRITE(6,9000)
      RETURN
 8000 FORMAT(' PUNCH THROUGH CAPACITY, FVPUNC              =',E10.3)
 9000 FORMAT(' THIS ROUTINE SHOULD ONLY BE USED FOR SOIL    '/
     1       ' PROFILE NR.2 AND NR.3                        ')
      END
```

```
      SUBROUTINE SPUD
C
C     PURPOSE: CALCULATE FOUNDATIONAREA FROM SPUDCANDATA
C
C     INTERNAL VARIABLES:
C     PI        = PI(3.14).
C     HTIP      = VERTICAL HEIGHT OF SPUDCAN TIP.
C     HCON      = VERTICAL HEIGHT OF SPUDCAN CONE.
C     HSID      = VERTICAL HEIGHT OF SPUDCAN SIDE.
C     DTIP      = TIP DIAMETER OF SPUDCAN.
C     DTOT      = TOTAL DIAMETER OF SPUDCAN.
C     ZP(IPEN)= ACTUAL PENETRATION DEPTH.
C
C     OUTPUT ARGUMENTS:
C     ASIDT(IPEN)= EXPOSED SIDE AREA IN TOP LAYER.
C     ASIDD(IPEN)= EXPOSED SIDE AREA IN DEEP LAYER.
C     AB(1)       = TOTAL EXPOSED BASE AREA IN BOTH LAYERS.
C     AB(2)       = EXPOSED BASE AREA IN DEEP LAYER.
C     ATOP(IPEN)  = EXPOSED AREA IN TOP LAYER.
C     ADEP(IPEN)  = EXPOSED AREA IN DEEP LAYER.
C     R(1)        = RADIUS OF TOTAL EXPOSED FOUNDATION
C     R(2)        = RADIUS OF FOUNDATION AREA IN DEEP LAYER.
C
      IMPLICIT DOUBLE PRECISION (A-H,O-Z), INTEGER(I-N)
      COMMON/JU1/IOP1,IOP2,IOP3,IOP4,IOP5,IOP6,TSET,BL,DW,AG,NL,WG,
     *GL,DI,DO,AW,PRECAP,DISPL,SPEPRE,WGLEG,ZPDEF,DTIP,DTOT,HTOP,HSID,
     *HCON,HTIP,HTOT,DMET,VWI,DARMW,HWM,TMIN,TMAX,TSTEP,GAMW,GAMMSO,
     *CI,CD,
     *VH1,VH2,WGV,VL,NPRO,SUO,RK,SUH,PHI,TH,RR,ALFA,
     *ZPD(50),ZPEN,ATOP(50),ADEP(50),ASIDD(50),ASIDT(50),
     *AO,ITR,COST,COSD,CO,COH,
     *FVULT(50),FVSU,FVPHI,FVSQUE,FVPUNC,BTOT,BEFF,FH,
     *FTOT(20,4,10),OMTOT(20,4,10),RLAR(20,4,10),FPRE(20,10),
     *FIVECT(12,22),OVTMP(20,10),RLARP(20,10),SPRNR,
     *PI,G,MSTART,ITSTRT,FHWIND
      COMMON/JU8/ZP(50),IPEN
C
      DIMENSION R(2),AB(2),AS(2)
      IF (SUM.GT.0.0) GO TO 6010
      SUM=0.0
      SUM=HTOP+HSID+HCON+HTIP
      IF(HTOT.LT.0.0)GO TO 6000
      IF(HSID.LT.0.0)GO TO 6000
      IF(HCON.LT.0.0)GO TO 6000
      IF(HTIP.LT.0.0)GO TO 6000
      IF(DTIP.LT.0.0)GO TO 6000
      IF(DTOT.LT.0.0)GO TO 6000
      DIFF=ABS(HTOT-SUM)
      IF(DIFF.GT.0.01)GO TO 6000
      IF(DTIP.GT.DTOT)GO TO 6000
      GO TO 6010
 6000 CONTINUE
      WRITE(6,9000)
 6010 CONTINUE
      Z=ZP(IPEN)
C
C     SET ALL AREA CALCULATIONS TO CIRO
C
      AS(1)=0.0
      AS(2)=0.0
      AB(1)=0.0
```

```
      AB(2)=0.0
      DO 100 I=1,2
      IF(I.EQ.2)Z=Z-TH
      IF(Z.LE.0.0) GO TO 100
      IF(Z.GT.HTIP)GO TO 6020
C
C     SPUD-CAN TIP PARTLY PENETRATED IN LAYER.
C
      R(I)=0.5*(DTIP/HTIP)*Z
      GO TO 6200
 6020 CONTINUE
      IF(Z.GT.(HTIP+HCON)) GO TO 6030
C
C     SPUD-CAN CONE PARTLY PENETRATED IN LAYER
C
      R(I)=DTIP/2.0+(Z-HTIP)*(DTOT-DTIP)/(2.0*HCON)
      GO TO 6200
 6030 CONTINUE
C
C     SPUD-CAN SIDE-WALL PARTLY PENETRATED IN LAYER
C
      R(I)=DTOT/2.0
      IF((Z-HTIP-HCON).GT.HSID) GO TO 6040
      AS(I)=2.0*PI*R(I)*(Z-HTIP-HCON)
      GO TO 6200
 6040 CONTINUE
C
C     SPUD-CAN TOTALLY EMBEDDED WITH SIDE
C
      AS(I)=2.0*PI*R(I)*HSID
 6200 CONTINUE
      AB(I)=PI*R(I)**2.0
C
C     END OF DO-LOOP
C
  100 CONTINUE
      ATOP(IPEN)=AB(1)-AB(2)
      ASIDT(IPEN)=AS(1)-AS(2)
      IF(ZP(IPEN).LT.TH) GO TO 6300
C
C     DEEP LAYER
C
      ADEP(IPEN)=AB(2)
      ASIDD(IPEN)=AS(2)
 6300 CONTINUE
      IF(IOP5.EQ.0) GO TO 6700
      IF(ZP(IPEN).GT.TH) GO TO 6500
      IF(ASIDT(IPEN).GT.0.0) GO TO 6400
      WRITE(6,8000)ATOP(IPEN)
      GO TO 6700
 6400 CONTINUE
      WRITE(6,8010) ATOP(IPEN),ASIDT(IPEN)
      GO TO 6700
 6500 CONTINUE
      IF(ASIDD(IPEN).GT.0.0) GO TO 6600
      WRITE(6,8020)ASIDT(IPEN),ATOP(IPEN),ADEP(IPEN)
      GO TO 6700
 6600 CONTINUE
      WRITE(6,8030)ASIDT(IPEN),ASIDD(IPEN),ADEP(IPEN)
 6700 CONTINUE
      IF(ASIDT(IPEN).LT.0.0) GO TO 6900
```

```
      IF(ASIDD(IPEN).LT.0.0) GO TO 6900
      GO TO 6950
 6900 CONTINUE
      WRITE(6,9010)
 6950 CONTINUE
      RETURN
 8000 FORMAT(' FOUNDATION AREA TOP LAYER                                   =',E10.3)
 8010 FORMAT(' FOUNDATION AREA TOP LAYER                                   =',E10.3/
     1       ' SIDE AREA TOP LAYER                                         =',E10.3)
 8020 FORMAT(' SIDE AREA TOP LAYER                                         =',E10.3/
     1       ' FOUNDATION AREA TOP LAYER                                   =',E10.3/
     2       ' FOUNDATION AREA DEEP LAYER                                  =',E10.3)
 8030 FORMAT(' SIDE AREA TOP LAYER                                         =',E10.3/
     1       ' SIDE AREA DEEP LAYER                                        =',E10.3/
     2       ' FOUNDATION AREA DEEP LAYER                                  =',E10.3)
 9000 FORMAT(//' ** ERROR IN SPUDCAN INPUT DATA **                         '//)
 9010 FORMAT(//' ERROR IN CALCULATION OF SIDEAREA OF SPUDCAN '//)
      END
```

```
      SUBROUTINE STFMTX(MLOAD,STFKZG,STFKZI,STFKXI,STFKRI)
C
C     PURPOSE:
C
C     METHOD:
C
C     INPUT ARGUMENTS:
C
C     INTERNAL PARAMETERS:
C
C     OUTPUT ARGUMENTS:
C
      IMPLICIT DOUBLE PRECISION (A-H,O-Z),INTEGER(I-N)
      COMMON/JU5/PSTEEL,E,EIB,EAB,EI(4),EA(4),GA(4),
     *STFKZ(4),STFKX(4),STFKR(4),
     *STF(12,12),STF1(6,6),STF2(6,6),STF3(6,6)
      COMMON/JU7/ZPFIN
      COMMON/JU1/IOP1,IOP2,IOP3,IOP4,IOP5,IOP6,TSET,BL,DW,AG,NL,WG,
     *GL,DI,DO,AW,PRECAP,DISPL,SPEPRE,WGLEG,ZPCEF,DTIP,DTOT,HTOP,HSID,
     *HCON,HTIP,HTOT,DMET,VWI,DARMW,HWM,TMIN,TMAX,TSTEF,GAMW,GAMMSO,
     *CI,CO,
     *VH1,VH2,WGV,VL,NPRO,SUO,RK,SUH,PHI,TH,RR,ALFA,
     *ZPD(50),ZPEN,ATOP(50),ADEP(50),ASIDO(50),ASIDT(50),
     *AO,ITR,COST,COSD,CO,COH,
     *FVULT(50),FVSU,FVPHI,FVSQUE,FVPUNC,BTOT,EEFF,FH,
     *FTOT(20,4,10),OMTOT(20,4,10),RLAR(20,4,10),FPRE(20,10),
     *FIVECT(12,22),OVTMP(20,10),RLARP(20,10),SPRNR,
     *PI,G,MSTART,ITSTRT,FHWIND
      COMMON/JU8/ZP(50),IPEN
      SAVE /JU1/,/JU5/,/JU7/
      DIMENSION RALFA(3),BETA(3),RI(3),GAMSH(4),A(3),S(3),STFKZG(4),
     *STFKZI(4),STFKXI(4),STFKRI(4)
C
C     CALCULATION OF LEGLENGTH
C
      RLEG=ZPFIN-HTIP-HCON+DW+AG
      IF(NL.EQ.4) GO TO 1000
      IF(NL.NE.3) GO TO 1100
C
C     CALCULATION FOR THREE LEGS
C
      RI(1)=EI(1)/E
      RI(2)=EIB/E
      RI(3)=(EI(2)+EI(3))/E
C
      A(1)=EA(1)/E
      A(2)=EAB/E
      A(3)=(EA(2)+EA(3))/E
      GO TO 1010
 1000 CONTINUE
C
C     CALCULATION FOR FOUR LEGS
C
      RI(1)=(EI(1)+EI(3))/E
      RI(2)=EIB/E
      RI(3)=(EI(2)+EI(4))/E

      A(1)=(EA(1)+EA(3))/E
      A(2)=EAB/E
      A(3)=(EA(2)+EA(4))/E
 1010 CONTINUE
```

```
      GAMSHB=0.9
C
C     CALCULATE BETA VALUES
C
C     VALUES FOR GAMSH IF NL EQUALS THREE.
      GAMSH(1)=GA(1)/(2.6*EA(1))
      GAMSH(3)=(GA(2)/(2.6*EA(2))+GA(3)/(2.6*EA(3)))
      IF(NL.EQ.4)GAMSH(1)=(GAMSH(1)+GA(4)/(2.6*EA(4)))
      IF(NL.EQ.4)GAMSH(3)=(GA(2)/(2.6*EA(2))+GA(4)/(2.6*EA(4)))
      BETA(1)=24.0*(1.0+PSTEEL)*RI(1)*GAMSH(1)/(A(1)*(RLEG**2.0))
      BETA(2)=24.0*(1.0+PSTEEL)*RI(2)*GAMSHB/(A(2)*(BL**2.0))
      BETA(3)=24.0*(1.0+PSTEEL)*RI(3)*GAMSH(3)/(A(3)*(RLEG**2.0))
C
      RALFA(1)=(1.0/12.0)*A(1)*(RLEG**2.0)*(1.0+BETA(1))/RI(1)
      RALFA(2)=(1.0/12.0)*A(2)*(BL**2.0)*(1.0+BETA(2))/RI(2)
      RALFA(3)=(1.0/12.0)*A(3)*(RLEG**2.0)*(1.0+BETA(3))/RI(3)
C
      S(1)=E*RI(1)/(RLEG*(1.0+BETA(1)))
      S(2)=E*RI(2)/(BL*(1.0+BETA(2)))
      S(3)=E*RI(3)/(RLEG*(1.0+BETA(3)))
C
C     CALCULATION OF LOCAL MATRIXES
C
      DO 1012 J=1,6
      DO 1011 I=1,6
      STF1(I,J)=0.0
      STF2(I,J)=0.0
      STF3(I,J)=0.0
 1011 CONTINUE
 1012 CONTINUE
C
C     MATRIX ELEMENT NR.1
C
      STF1(1,1)=S(1)*12.0/(RLEG**2.0)
      STF1(1,3)=S(1)*6.0/RLEG
      STF1(1,4)=-STF1(1,1)
      STF1(1,6)=STF1(1,3)
      STF1(2,2)=STF1(1,1)*RALFA(1)
      STF1(2,5)=-STF1(2,2)
      STF1(3,3)=S(1)*(4.0+BETA(1))
      STF1(3,4)=-STF1(1,3)
      STF1(3,6)=S(1)*(2.0-BETA(1))
      STF1(4,4)=STF1(1,1)
      STF1(4,6)=STF1(3,4)
      STF1(5,5)=STF1(2,2)
      STF1(6,6)=STF1(3,3)
C
C     MATRIX ELEMENT NR.2
C
      STF2(1,1)=S(2)*12.0*RALFA(2)/(BL**2.0)
      STF2(1,4)=-STF2(1,1)
      STF2(2,2)=S(2)*12.0/(BL**2.0)
      STF2(2,3)=-S(2)*6.0/BL
      STF2(2,5)=-STF2(2,2)
      STF2(2,6)=STF2(2,3)
      STF2(3,3)=S(2)*(4.0+BETA(2))
      STF2(3,5)=-STF2(2,3)
      STF2(3,6)=S(2)*(2.0-BETA(2))
      STF2(4,4)=STF2(1,1)
      STF2(5,5)=STF2(2,2)
      STF2(5,6)=STF2(3,5)
```

```
      STF2(6,6)=STF2(3,3)
C
C     MATRIX ELEMENT NR,3
C
      STF3(1,1)=S(3)*12.0/(RLEG**2.0)
      STF3(1,3)=-S(3)*6.0/RLEG
      STF3(1,4)=-STF3(1,1)
      STF3(1,6)=STF3(1,3)
      STF3(2,2)=STF3(1,1)*RALFA(3)
      STF3(2,5)=-STF3(2,2)
      STF3(3,3)=S(3)*(4.0+BETA(3))
      STF3(3,4)=-STF3(1,3)
      STF3(3,6)=S(3)*(2.0-BETA(3))
      STF3(4,4)=STF3(1,1)
      STF3(4,6)=-STF3(1,3)
      STF3(5,5)=STF3(2,2)
      STF3(6,6)=STF3(3,3)
C
C     ALL MATRIXES ARE SYMMETRIC - READ VALUE FOR LOWER TRIANGEL
C
      DO 1014 I=2,6
      DO 1013 J=1,I-1
      STF1(I,J)=STF1(J,I)
      STF2(I,J)=STF2(J,I)
      STF3(I,J)=STF3(J,I)
 1013 CONTINUE
 1014 CONTINUE
C
C     GIVE INITIAL VALUES FOR SOILSPRINGS IF FIRST LOADING
C
      IF(MLOAD.GT.1) GO TO 1016
      DO 1015 NR=1,4
      STFKZ(NR)=STFKZG(NR)
      STFKX(NR)=STFKXI(NR)
      STFKR(NR)=STFKRI(NR)
 1015 CONTINUE
 1016 CONTINUE
      DO 1030 J=1,12
      DO 1020 I=1,12
      STF(I,J)=0.0
 1020 CONTINUE
 1030 CONTINUE
      STF(1,1)=S(1)*12.0/(RLEG**2.0)+STFKX(1)
      STF(1,3)=S(1)*6.0/RLEG
      STF(1,4)=-S(1)*12.0/(RLEG**2.0)
      STF(1,6)=STF(1,3)
      STF(2,2)=S(1)*12.0*RALFA(1)/(RLEG**2.0)+STFKZ(1)
      STF(2,5)=-S(1)*12.0*RALFA(1)/(RLEG**2.0)
      STF(3,3)=S(1)*(4.0+BETA(1))+STFKR(1)
      STF(3,4)=-STF(1,6)
      STF(3,6)=S(1)*(2.0-BETA(1))
      STF(4,4)=-STF(1,4)+S(2)*12.0*RALFA(2)/(BL**2.0)
      STF(4,6)=-STF(1,6)
      STF(4,7)=-S(2)*12.0*RALFA(2)/(BL**2.0)
      STF(5,5)=-STF(2,5)+S(2)*12.0/(BL**2.0)
      STF(5,6)=-S(2)*6.0/BL
      STF(5,8)=-S(2)*12.0/(BL**2.0)
      STF(5,9)=STF(5,6)
      STF(6,6)=S(1)*(4.0+BETA(1))+S(2)*(4.0+BETA(2))
      STF(6,8)=-STF(5,9)
      STF(6,9)=S(2)*(2.0-BETA(2))
```

```
      STF(7,7)=-STF(4,7)+S(3)*12.0/(RLEG**2.0)
      STF(7,9)=-S(3)*6.0/RLEG
      STF(7,10)=-S(3)*12.0/(RLEG**2.0)
      STF(7,12)=STF(7,9)
      STF(8,8)=-STF(5,8)+S(3)*12.0*RALFA(3)/(RLEG**2.0)
      STF(8,9)=STF(6,3)
      STF(8,11)=-S(3)*12.0*RALFA(3)/(RLEG**2.0)
      STF(9,9)=S(2)*(4.0+BETA(2))+S(3)*(4.0+BETA(3))
      STF(9,10)=-STF(7,9)
      STF(9,12)=S(3)*(2.0-BETA(3))
      STF(10,10)=-STF(7,10)+STFKX(2)+STFKX(3)
      STF(10,12)=STF(9,10)
      STF(11,11)=-STF(8,11)+STFKZ(2)+STFKZ(3)
      STF(12,12)=S(3)*(4.0+BETA(3))+STFKR(2)+STFKR(3)
      IF(NL.EQ.3) GO TO 1035
      STF(1,1)=STF(1,1)+STFKX(3)
      STF(2,2)=STF(2,2)+STFKZ(3)
      STF(3,3)=STF(3,3)+STFKR(3)
      STF(10,10)=STF(10,10)-STFKX(3)+STFKX(4)
      STF(11,11)=STF(11,11)-STFKZ(3)+STFKZ(4)
      STF(12,12)=STF(12,12)-STFKR(3)+STFKR(4)
C
C     SYMMETRIC MATRIX - READ ELEMENT-VALUES FOR LOWER TRIANGEL
C
 1035 CONTINUE
      DO 1050 I=2,12
      DO 1040 J=1,I-1
      STF(I,J)=STF(J,I)
 1040 CONTINUE
 1050 CONTINUE
      RETURN
 1100 CONTINUE
      WRITE(6,8000)NL
 8000 FORMAT(5X,'ERROR IN INPUT DATA. NL=',I5)
      END
```

```
      SUBROUTINE TRANSF
C     PURPOSE: THIS ROUTINE TRANSFORM THE FOUNDATION FROM A CIRLE
C     TO A SQUARE TO CALCULATE EFFECTIVE WIDTH AND EFFECTIV
C     FOUNDATION AREA
      IMPLICIT DOUBLE PRECISION (A-H,O-Z), INTEGER (I-N)
      COMMON/JU1/IOP1,IOP2,IOP3,IOP4,IOP5,IOP6,TSET,BL,DW,AG,NL,WG,
     *GL,DI,DD,AW,PRECAP,DISPL,SPEPRE,WGLEG,ZPDEF,DTIP,DTOT,HTOP,HSID,
     *HCON,HTIP,HTOT,OMET,VWI,DARMW,HWM,TMIN,TMAX,TSTEF,GAMW,GAMMSO,
     *CI,CO,
     *VH1,VH2,WGV,VL,NPRO,SUO,RK,SUH,PHI,TH,RR,ALFA,
     *ZPD(50),ZPEN,ATOP(50),ADEP(50),ASIDD(50),ASIDT(50),
     *AO,ITR,COST,COSD,CO,COH,
     *FVULT(50),FVSU,FVPHI,FVSQUE,FVPUNC,BTOT,BEFF,FH,
     *FTOT(20,4,10),OMTOT(20,4,10),RLAR(20,4,10),FPRE(20,10),
     *FIVECT(12,22),OVTMP(20,10),RLARP(20,10),SPRNR,
     *PI,G,MSTART,ITSTRT,FHWIND
      COMMON/JU8/ZP(50),IPEN
      BTOT=SQRT(ATOP(IPEN)+ADEP(IPEN))
      IF(PHI.LT.0.01) GO TO 1030
      GO TO 1050
 1030 CONTINUE
      AO=BTOT**2.0
      BEFF=BTOT
      IF(IOP5.EQ.0) GO TO 1040
      WRITE(6,8000)BTOT,AO
 1040 CONTINUE
      RETURN
 1050 CONTINUE
      GO TO 1030
C
C     DUMMY LOOP ABOVE UNTIL PROGRAM IS RUNNING SMOUTH
C
C      E=AM/FVMAX
C      BEFF=BTOT-2.0*E
C      AO=BEFF*BTOT
C      WRITE(6,8010)BEFF,AO
 8000 FORMAT(' NO AREA REDUCTION.                               '      /
     1       ' TOTAL BREADTH, BTOT                            =',E10.3/
     2       ' TOTAL FOUNDATION AREA, AO                      =',E10.3)
 8010 FORMAT(' EFFECTIV WIDTH, BEFF                           =',E10.3/
     1       ' EFFECTIV FOUNDATION AREA, AO                   =',E10.3)
C      RETURN
      END
```

```
      SUBROUTINE TRIM12(A,MA,EPS,IERR)
C
      IMPLICIT DOUBLE PRECISION (A-H,O-Z),INTEGER(I-N)
C
C***********************************************************************
C
C    N O R S A M   SUBROUTINE :    TRIM12
C
C
C     TRIM12 DECOMPOSES A SQUARE MATRIX  A  INTO A LOWER TRIANGULAR
C     MATRIX  L  AND AN UPPER TRIANGULAR MATRIX  U  ( A = LU ) BY THE
C     METHOD OF CROUT
C
C
C
C***********************************************************************
C
      DIMENSION A(MA,MA)
C
      COMMON/SAFE/ISAFE(50)
      COMMON/COM/IP(100),SCAL(100),B(100)
C
      IA = MA
      IB = 1
      IOP = -1
      IW = ISAFE(2)
C
C     PRINT ?
C
      IF(ISAFE(11))10,10,5
    5 WRITE(IW,6000) MA
C
C     DIMENSION PARAMETER O.K. ?
C
   10 IF(MA-1)1000,20,15
   15 IF(MA-100)25,25,1000
C
   20 IF(A(1,1))400,1100,400
C
C     SCALING FACTORS FOR THE PIVOT SEARCH
C
   25 DO 50 I=1,MA
      E = A(I,1)*A(I,1)
      DO 30 J=2,MA
   30 E = E + A(I,J)*A(I,J)
      IF(E)1100,1100,50
   50 SCAL(I) = 1./SQRT(E)
C
C     SEARCH FOR FIRST PIVOT ELEMENT
C
      J = 1
      E = SCAL(1)*ABS(A(1,1))
      DO 100 I=2,MA
      F = SCAL(I)*ABS(A(I,1))
      IF(F-E)100,100,75
   75 E = F
      J = I
  100 CONTINUE
C
      IF(E)1100,1100,110
  110 IF(J-1)150,150,120
```

```
C
C     INTERCHANGE ROWS
C
  120 E = SCAL(1)
      SCAL(1) = SCAL(J)
      SCAL(J) = E
      DO 125 I=1,MA
      E = A(J,I)
      A(J,I) = A(1,I)
  125 A(1,I) = E
  150 IP(1) = J
C
C     FIRST ROW OF U
C
      E = 1./A(1,1)
      DO 175 I=2,MA
  175 A(1,I) = A(1,I)*E
C
      MA1 = MA-1
      IF(MA-2)325,325,200
C
C     MAIN LOOP
C
  200 DO 300 I=2,MA1
      I1 = I-1
      E = 0.
      K = I
C
C     ITH COLUMN OF L, AND
C     SEARCH FOR ITH PIVOT ELEMENT
C
      DO 225 J=I,MA
      B(J) = PRAC10(A(J,1),A(1,I),A(J,I),IA,IB,I1,IOP)
      F = SCAL(J)*ABS(B(J))
      IF(F-E)225,225,220
  220 E = F
      K = J
  225 CONTINUE
C
C     SINGULAR OR NEAR-SINGULAR ?
C
      F = SCAL(K)*ABS(A(K,I))*EPS
      IF(E-F)1100,1100,240
  240 DO 250 J=I,MA
  250 A(J,I) = B(J)
C
      IF(K-I)280,280,260
C
C     INTERCHANGE ROWS
C
  260 E = SCAL(I)
      SCAL(I) = SCAL(K)
      SCAL(K) = E
      DO 270 J=1,MA
      E = A(K,J)
      A(K,J) = A(I,J)
  270 A(I,J) = E
  280 IP(I) = K
C
C     ITH ROW OF U
C
```

```
      K = I+1
      E = 1./A(I,I)
      DO 290 J=K,MA
      F = PRAC10(A(I,1),A(1,J),A(I,J),IA,IB,I1,IOP)
  290 A(I,J) = F*E
  300 CONTINUE
C
C     LAST ELEMENT OF L
C
  325 B(MA) = PRAC10(A(MA,1),A(1,MA),A(MA,MA),IA,IB,MA1,IOP)
      IP(MA) = MA
C
C     SINGULAR OR NEAR-SINGULAR ?
C
      E = ABS(B(MA))
      F = ABS(A(MA,MA))*EPS
      IF(E-F)1100,1100,350
C
  350 A(MA,MA) = B(MA)
C
  400 IERR = 0
      GO TO 2000
C
 1000 IERR = -1
      WRITE(IW,6010)
      WRITE(IW,6020) IERR
      WRITE(IW,6030) MA
      GO TO 2000
C
 1100 IERR = -2
      WRITE(IW,6010)
      WRITE(IW,6020) IERR
C
C
 2000 RETURN
C
 6000 FORMAT(///25H ENTERING TRIM12 :    MA =,I4)
 6010 FORMAT(///33H *** ERROR RETURN FROM TRIM12 ***)
 6020 FORMAT(/17H     ERROR FLAG =,I3)
 6030 FORMAT(/25H     INPUT PARAMETER MA =,I4)
C
      END
```

```
      SUBROUTINE WAVE (IOP1,IOP2,X,DW,M,LN,HW,T,TI,IT,CD,CI,DD,DI,
     *WL,VH1,VH2,PI,G,GAMW,FORCE,OVMOM,ARM)
C
C     PURPOSE:THIS ROUTINE CALCULATE LOADS ON A SPESIFIED BODY EXPOSED
C     TO WAVES AND CURRENT. WE USE MORISONS FORMULA ON LEGSEGMENTS AND
C     FIND THE TOTAL FORCE AND MOMENT BY INTEGRATION.
C
      IMPLICIT DOUBLE PRECISION (A-H,O-Z), INTEGER (I N)
      COMMON/JU3/FORSEG(20,4,10),SEGARM(20,4,10)
      DIMENSION ZL(10),FI(10),FD(10),RLARM(10),FSEG(10)
      SAVE/JU1/,/JU3/
      OMEG=2.0*PI/T
      RK=2.0*PI/WL
      ETA=HW/2.0*COS(RK*X-OMEG*TI)
      DE=DW+ETA
      FORCE=0.0
      OVMOM=0.0
      ARM=0.0
      DO 1000 NZ=1,10
      DZ=DE/10.0
      ZL(NZ)=ETA-NZ*DZ+DZ/2.0
      RLARM(NZ)=DW+ZL(NZ)
      VW=PI*HW/T*(COSH(2.*PI*(ZL(NZ)+DW)/WL)/(SINH(2.*PI*DW/WL)))
     **COS(2.*PI*(X/WL-TI/T))
      ACCW=2.0*PI**2.0*HW/(T**2.0)*(COSH(2.*PI*(ZL(NZ)+DW)/WL)/
     *(SINH(2.*PI*DW/WL)))*SIN(2.0*PI*(X/WL-TI/T))
      VC=ABS(VH1)*(RLARM(NZ)/DW)*EXP(0.14)+ABS(VH2)*RLARM(NZ)/DW
C
C     MAX CURRENT VELOCITY IS OBTAINED AT STILL WATER LEVEL
C     AND IS ASSUMED CONSTANT ABOVE THIS LEVEL
C
      VCMAX=ABS(VH1)*EXP(0.14)+ABS(VH2)
      IF(VC.GT.VCMAX)VC=VCMAX
      VTOT=VW+VC
C     MORISONS FORMULA AND VERITAS RULES
      FI(NZ)=GAMW/G*CI*ACCW*PI/4.0*DI**2.*DZ
      FD(NZ)=0.5*GAMW/G*CD*ABS(VTOT)*VTOT*DZ*DD
C     IF STRUCTURE IS EXPOSED TO WAVES PROPAGATING IN POSITIVE
C     X-DIRECTION IOP2=0 AND ALL FORCES ARE ACTING IN CORRECT
C     DIRECTION.
C     IF STRUCTURE IS EXPOSED TO WAVES PROPAGATING IN NEGATIVE
C     X-DIRECTION IOP2=1. THIS IS SIMULATED IN THE ENVIROMENTAL
C     SUBROUTINES BY TURNING THE JACKUP 180 DEGREES. IN THE LATER
C     SUBROUTINES THE JACKUP IS TURNED BACK TO ITS ORIGINAL POSITION
C     TO FIND THE STIFFNESS OF THE STRUCTURE AND THE FOUNDATION,
C     THE DISPLACEMENTS AND THE DISTRIBUTION OF THE FORCES.
C     SINCE THE WAVES IS PROPAGATING IN NEGATIVE X-DIRECTION WE HAVE
C     TO CHANGE THE SIGN OF THE WAVEFORCES.(IN THIS ROUTINE.)
C
      IF(IOP2.EQ.1) GO TO 900
      FSEG(NZ)=FI(NZ)+FD(NZ)
      GO TO 950
  900 CONTINUE
      FSEG(NZ)=-(FI(NZ)+FD(NZ))
  950 CONTINUE
      FORCE=FORCE+FSEG(NZ)
      OVMOM=OVMOM+FSEG(NZ)*RLARM(NZ)
C      WRITE(6,9900) NZ,ZL(NZ),VW,ACCW,VC,VTOT,FI(NZ),FD(NZ),RLARM(NZ),
C     *FSEG(NZ)
C 9900 FORMAT(1X,I3,9F10.3)
      IF(IOP1.NE.1)GO TO 1000
```

```
      FORSEG(M,LN,NZ)=FSEG(NZ)
      SEGARM(M,LN,NZ)=RLARM(NZ)
 1000 CONTINUE
      ARM=OVMOM/FORCE
      RETURN
      END
```

```
      SUBROUTINE ZPCORR(FVACT,FVULT,INPO)
C
C     METHOD: ITERATION.
C
C     INPUT ARGUMENT:
C     FVACT
C     FVULT
C     FVULTI
C     IPEN
C     INPO
C     ZP
C
C     ITERNAL PARAMETERS:
C     NR
C     ZPTEST
C     ZPFIN
C
C     OUTPUT ARGUMENTS:
C     NONE
C
      IMPLICIT DOUBLE PRECISION(A-H,O-Z),INTEGER(I-N)
      COMMON/JU6/FVULTI
      COMMON/JU7/ZPFIN
      COMMON/JU8/ZP(50),IPEN
      DIMENSION FVULT(50),Z(100)
      NR=1
      GO TO 1050
 1000 CONTINUE
      NR=NR+1
      ZPTEST=Z(NR-1)
      IF(NR.GE.2) GO TO 1100
 1050 CONTINUE
      IPEN=IPEN+1
      ZP(IPEN)=ZPFIN
      ZPTEST=ZPFIN
      CALL PREPEN
 1100 CONTINUE
      IF(FVULTI.GE.FVACT) GO TO 1200
      Z(NR)=ZPTEST+0.5*((ZP(INPO)-ZPTEST)/(FVULT(INPO)-FVULTI))*
     *(FVACT-FVULTI)
      GO TO 1500
 1200 CONTINUE
      Z(NR)=ZPTEST+0.5*((ZPTEST-ZP(INPO-1))/
     *(FVULTI-FVULT(INPO-1)))*(FVACT-FVULTI)
 1500 CONTINUE
      ZP(IPEN)=Z(NR)
      CALL PREPEN
      IF(NR.GT.198) GO TO 1600
      IF(NR.GT.30.AND.(Z(NR)-Z(NR-1)).LE.0.0009) GO TO 1550
      IF(ABS((FVACT-FVULTI)/FVACT).GT.0.03) GO TO 1000
 1550 CONTINUE
      ZPFIN=Z(NR)
      ZP(IPEN)=ZPFIN
      WRITE(6,8000)ZPFIN
      WRITE(6,8050)FVACT,FVULTI
      RETURN
 1600 CONTINUE
      WRITE(6,9000)
      RETURN
 8000 FORMAT(1X,//,
     1       ' FINAL PENETRATION AFTER PRELOADING            =',E10.3//
```

```
     2       ' PRELOADWEIGHT IS ASSUMED EQUAL TO:               '       /
     3       ' SPECIFIED PRELOAD(IF GIVEN)/                     '       /
     4       ' PRELOADCAPASITY(IF SPEPRE IS NOT GIVEN) +        '       /
     5       ' STRUCTUREWEIGHT + VARIABEL LOAD                  '       /
     6       ' AS GIVEN IN INPUT DATA.)                         '      /)
 8050 FORMAT(' VERTICAL LOAD ON ONE LEG DURING PRELOADING  =',E10.3/
     1       ' BEARING CAPASITY OF SOIL FOR ONE LEG         =',E10.3/)
 9000 FORMAT(1X,'**ERROR** PENETRATION ITERATION IN SUBROUTINE ZPCORR'
     *,/,'    DOES NOT CONVERG!!!')
      END
```

COMPUTER LISTING OF PROGRAM SAM

The listing comprises the main program SAM and the following subroutines:

MATPEN
MATBRH
WAVEPR
MATENV
MATWAV
TOTFOR
MATSLI
MATLOC
SHBEAR
DEBEAR

```
C       *****************************************************************************2**
C       MAIN PROGRAM SAM FOR ANALYSIS OF THE BEARING CAPACITY OF MAT
C       SUPPORTED JACK UP PLATFORMS SUBJECTED TO WIND, CURRENT AND WAVE
C       FORCES
C       *****************************************************************************
C
C       THIS PROGRAM TAKES COMBINED WAVE AND CURRENT FORCES ACTING ON 3
C       OR 4 LEGS INTO ACCOUNT. FURTHER THE WAVE PRESSURE IS INTEGRATED OVER
C       A SIMPLIFIED MAT STRUCTURE TO TAKE WAVE FORCES INTO ACCOUNT. DIFFRACTI-
C       ON EFFECTS ARE NOT TAKEN INTO ACCOUNT, NEITHER ARE DRAG FORCES ON
C       ANY MAT ELEMENT INCLUDED IN THE ANALYSIS
C
C       *****************************************************************************
C
C
C       COMMON BLOCKS
C
        COMMON/HEAD/TEXT(40)
        COMMON/MATDAT/TOTL,TOTW,SLOTL,SLOTW,HOLEL,HOLEW,TMAT,
       1ALE,BLE,YLE
        COMMON/SKIRT/SKH,SKTH,ALFASK
        COMMON/SETTLE/ZPEN
        COMMON/MATFOR/FVX(10,21),FX(10,21),OMY(10,21),FVY(10,21),
       1FY(10,21),OMX(10,21)
        COMMON/WAVDAT/WH,DW,HW(10),WL(10),WS(10),TI(10)
        COMMON/PERIOD/TMIN,TMAX,TSTEP
        COMMON/CONSTA/PI,GAMW,G,CI,CD
        COMMON/LEGDAT/NL,X(4),Y(4),HLEG(4),DI(4),DO(4),EI(4),FA(4),GA(4)
        COMMON/LEGFOR/FHLEGX(10,21),FHLEGY(10,21),OMLEGY(10,21),OMLEGX(10,
       121)
        COMMON/WEIGHT/GW,GVAR,GLEG,GMAT,GLX,GLY,GVARLX,GVARLY
        COMMON/CURR/VC1,VC2
        COMMON/WIND/VW,WAREAX,WAREAY,WARM,WFX,WFY,WOMY,WOMX
        COMMON/TOTFOR/FVXT(10,21),FXT(10,21),OMYT(10,21),FVYT(10,21),
       1FYT(10,21),OMXT(10,21)
        COMMON/SOIL/NPRO,SUO,RK,SUH,PHI,TH,RR,GAMS
        COMMON/AREADA/AREAT,YCGMAT,XCGMAT,WIYT,WIXT
        COMMON/SAFE/FSDMI(10,2),FSSMI(10,2),FSLMI(10,2),FSSLMI(10,2),
       1AREG(10,21,2)
C
C
C
C       START READING INPUT DATA
C
C*****************************************************************************
C
C       READ 2 HEADING CARDS
C
        READ(5,8000) (TEXT(I),I=1,40)
C*****************************************************************************
C
C       PHYSICAL CONSTANTS AND HYDRODYNAMIC COEFFICIENTS FOR MORISON'S EQU
C
        READ(5,8010) GAMW,G,CI,CD
        PI=3.14159265
C*****************************************************************************
C
C       WATER DEPTH AND AIR GAP
C
        READ(5,8010)DW,AG
C*****************************************************************************
```

```
C     WAVE DATA
C
      READ(5,8010) WH,TMIN,TMAX,TSTEP
C*****************************************************************
C
C     CURRENT DATA
C
      READ(5,8010) VC1,VC2
C*****************************************************************
C
C     WIND DATA
C
      READ(5,8010) VW,WAREAX,WAREAY,WARM
C*****************************************************************
C
C     MAT GEOMETRY
C
      READ(5,8010) TOTL,TOTW,SLOTL,SLOTW,HOLEL,HOLEW
      READ(5,8010) TMAT,ALF,BLF,YLE,SKH,SKTH,ALFASK
C*****************************************************************
C
C     LEG GEOMETRY AND STIFFNESS
C
      READ(5,8020) NL
      DO 100 I=1,NL
      READ(5,8015) X(I),Y(I),HLEG(I),DI(I),DO(I),EI(I),EA(I),GA(I)
  100 CONTINUE
C*****************************************************************
C
C     PLATFORM WEIGHTS AND LEVERARMS OF WEIGTHS
C
      READ(5,8010) GW,GLX,GLY,GVAR,GVARLX,GVARLY,GLEG,GMAT
C*****************************************************************
C
C     SOIL PROFILE DATA
C
      READ(5,8020) NPRO
      READ(5,8010) SUO,RK,SUH,PHI,TH,RR,GAMS
C*****************************************************************
C
C
C     PRINT INPUT DATA
C
      WRITE(6,9100)
      WRITE(6,9105) (TEXT(I),I=1,40)
      WRITE(6,9110) GAMW,G,CI,CD
      WRITE(6,9115) DW,AG
      WRITE(6,9120) WH,TMIN,TMAX,TSTEP
      WRITE(6,9122) VC1,VC2
      WRITE(6,9123) VW,WAREAX,WAREAY,WARM
      WRITE(6,9130) TOTL,TOTW,SLOTL,SLOTW,HOLEL,HOLEW,TMAT,ALF,BLE,YLE,
     1SKH,SKTH,ALFASK
      WRITE(6,9135) NL,(X(I),I=1,4),(Y(I),I=1,4),(HLEG(I),I=1,4),(DI(I),
     1I=1,4),(DO(I),I=1,4),(EI(I),I=1,4),(EA(I),I=1,4),(GA(I),I=1,4)
      WRITE(6,9140) GW,GLX,GLY,GVAR,GVARLX,GVARLY,GLEG,GMAT
      WRITE(6,9150) NPRO,SUO,RK,SUH,PHI,TH,RR,GAMS
C
C     *************************************************************
C
C     CALCULATE PENETRATION OF MAT WITH BEARING CAPACITY FORMULA
```

```
      CALL MATPEN
C     ***********************************************************************
C
C     CALCULATE FORCES ON MAT WITH SUBROUTINE WAVEPR
C
      CALL WAVEPR
C
C     ***********************************************************************
C
C     CALCULATE SHEAR FORCE AND OVERTURNING MOMENT DUE TO WIND AND
C     COMBINED WAVE AND CURRENT ACTING ON THE LEGS
C
      CALL MATENV
C
C     ***********************************************************************
C
C     CALCULATE TOTAL FORCE VECTORS INCLUDING WEIGHTS, WAVE PRESSURE
C     ON MAT, WIND FORCES, AND COMBINED WAVE AND CURRENT FORCES
C
      CALL TOTFOR
C
C     ***********************************************************************
C
C     CALCULATE SAFETY AGAINST OVERTURNING ABOUT EDGE OF MAT
C
C     CALL MATOTM
C
C     ***********************************************************************
C
C     CALCULATE SAFETY AGAINST SLIDING DIRECTLY UNDER MAT AND AT
C     INTERFACE BETWEEN HARD AND SOFT LAYER FOR SOIL PROFILE 2 AND 3
C
      CALL MATSLI
C
C     ***********************************************************************
C
C     CALCULATE SAFETY AGAINST LOCAL BEARING CAPACITY FAILURE UNDER
C     MOST STRESSED BEAM OF A-SHAPED MAT
C
      CALL MATLOC
C
C     ***********************************************************************
C
C     CALCULATE SAFETY AGAINST BEARING CAPACITY FAILURE UNDER EDGE
C     I.E. SHALLOW FAILURE SURFACE
C
      CALL SHBEAR
C
C     ***********************************************************************
C
C     CALCULATE SAFETY AGAINST DEEPSEATED BEARING CAPACITY FAILURE
C
      CALL DEBEAR
C
C     ***********************************************************************
C
C
C     PRINT SUMMARY TABLE OF SAFETY FACTORS FOR THE FOUR DIFFERENT
C     FOUNDATION FAILURE MODES ANALYZED FOR EACH WAVE PERIOD INVESTIGATED
C
```

```
      NTI=(TMAX-TMIN)/TSTEP+1
      WRITE(6,9200)
      DO 200 IT=1,NTI
      WRITE(6,9250) TI(IT),FSSLMI(IT,1),FSSLMI(IT,2),FSLMI(IT,1),
     1FSLMI(IT,2),FSSMI(IT,1),FSSMI(IT,2),FSOMI(IT,1),
     2FSOMI(IT,2)
  200 CONTINUE
      WRITE(6,9260)
 8000 FORMAT(20A4)
 8010 FORMAT(8F10.3)
 8015 FORMAT(5F10.3,3E10.3)
 8020 FORMAT(I5)
 9100 FORMAT(1H1,' PROGRAM SAM'/' VERSION 4/FEBRUARY 1983'/
     1' PROGRAMMED BY T.J. KVALSTAD'///)
 9105 FORMAT(' ********************************************************
     1*******************************'/
     2        ' * ',20A4,'  *'/
     3        ' * ',20A4,'  *'/
     4        ' ********************************************************
     5*******************************'//)
 9110 FORMAT(' INPUT DATA PRINTOUT'/
     1        ' ******************'//
     2        ' UNIT WEIGHT OF SEA WATER =',F10.3/
     3        ' GRAVITY CONSTANT, G      =',F10.3/
     4        ' INERTIA COEFFICIENT, CI  =',F10.3/
     5        ' DRAG COEFFICIENT, CD     =',F10.3//)
 9115 FORMAT(' WATER DEPTH               =',F10.3/
     1        ' AIR GAP                  =',F10.3//)
 9120 FORMAT(' WAVE HEIGHT               =',F10.3/
     1        ' MINIMUM WAVE PERIOD      =',F10.3/
     2        ' MAXIMUM WAVE PERIOD      =',F10.3/
     3        ' PERIOD STEP              =',F10.3//)
 9122 FORMAT(' TIDAL INDUCED CURRENT VC1=',F10.3/
     1        ' WIND INDUCED CURRENT VC2 =',F10.3//)
 9123 FORMAT(' WIND VELOCITY "AVERAGE"  =',F10.3/
     1        ' WIND AREA X-DIR,WAREAX   =',F10.3/
     2        ' WIND AREA Y-DIR,WAREAY   =',F10.3/
     3        ' WIND LEVER ARM           =',F10.3//)
 9130 FORMAT(' TOTAL LENGTH OF MAT       =',F10.3/
     1        ' TOTAL WIDTH OF MAT       =',F10.3/
     2        ' SLOTLENGTH               =',F10.3/
     3        ' SLOTWIDTH                =',F10.3/
     4        ' HOLELENGTH               =',F10.3/
     5        ' HOLEWIDTH                =',F10.3/
     6        ' THICKNESS OF MAT         =',F10.3/
     7        ' WIDTH OF END STRIP, ALE  =',F10.3/
     8        ' WIDTH OF MIDDLE STRIP,BLE=',F10.3/
     9        ' DISTANCE TO CG, YLE      =',F10.3//
     1        ' SKIRT HEIGHT, SKH        =',F10.3/
     2        ' SKIRT TIP THICKN. SKTH   =',F10.3/
     3        ' STEEL SOIL FRICTION RATIO=',F10.3//)
 9135 FORMAT(' NUMBER OF LEGS, NL        =',I6/
     1        '                           LEG NO.1     LEG NO.2     LEG N
     10.3     LEG NO.4    '/
     1        ' X-COORDINATE OF LEGS     =',4(F10.3,2X)/
     1        ' Y-COORDINATE OF LEGS     =',4(F10.3,2X)/
     1        ' TOTAL LEG LENGTH         =',4(F10.3,2X)/
     2        ' EFFECTIVE DIAMETER INERT.=',4(F10.3,2X)/
     3        ' EFFECTIVE DIAMETER DRAG  =',4(F10.3,2X)/
     4        ' BENDING STIFFNESS, EI    =',4(F10.3,2X)/
     5        ' AXIAL STIFFNESS,    EA   =',4(F10.3,2X)/
```

```
     6         ' SHEAR STIFFNESS,    GA     =',4(E10.3,2X)//)
 9140 FORMAT(' PLATFORM WEIGHT              =',F10.3/
     1         ' LEVER ARM OF G IN X-DIR.  =',F10.3/
     2         ' LEVER ARM OF G IN Y-DIR.  =',F10.3/
     3         ' VARIABEL LOAD, GVAR        =',F10.3/
     4         ' LEVER ARM OF GVAR, X-DIR.=',F10.3/
     5         ' LEVER ARM OF GVAR, Y-DIR.=',F10.3/
     6         ' WEIGHT PER LEG, GLEG       =',F10.3/
     7         ' WEIGHT OF MAT, GMAT        =',F10.3//)
 9150 FORMAT(' SOIL PROFILE TYPE            =',I6/
     1         ' UNDR. SHEAR STRENGTH OF'/
     2         ' SOFT CLAY LAYER ON TOP    =',F10.3/
     3         ' SHEAR STRENGTH GRADIENT   =',F10.3/
     4         ' UNDR. SHEAR STRENGTH OR'/
     5         ' COHESION OF HARD LAYER    =',F10.3/
     6         ' FRICTION ANGLE,HARD LAYER=',F10.3/
     7         ' THICKNESS OF TOP LAYER    =',F10.3/
     8         ' ROUGHN. RATIO SOIL/STEEL =',F10.3/
     9         ' SUBM. UNIT WEIGHT OF SOIL=',F10.3//)
 9200 FORMAT(1H1,'PROGRAM *SAM*     : STABILITY ANALYSIS OF MATS'/
     1         /' MAIN PROGRAM SAM : '////
     7' *********************************************'/
     8' * SUMMARY TABLE OF SAFETY FACTORS FOR THE FOUR *'/
     9' * DIFFERENT FOUNDATION FAILURE MODES ANALYZED  *'/
     1' *********************************************'//
     1' ***********************************************************
     1************'/
     2' * WAVE * SLIDING UNDER  * LOCAL BEARING  * SHALLOW FAILURE*   DE
     3EPSEATED   *'/
     4' *PERIOD*   SKIRT TIP    * FAILURE UNDER  *   UNDER EDGE   *   OV
     5ERTURNING  *'/
     6' * SEC. *                *  MOST LOADED   *(TOTAL FAILURE) *    F
     7AILURE     *'/
     8' *      *                * BEAM OF A-MAT  *                *
     9           *'/
     1' *      * X-DIR.  Y-DIR. * X-DIR.  Y-DIR. * X-DIR.  Y-DIR. * X DI
     2R.  Y-DIR. *'/
     3' *      *                *                *                *
     4           *')
 9250 FORMAT(' *',F4.1,'  *',4(2F8.2,'*'))
 9260 FORMAT(' *      *',4('                *')/
     1' ***********************************************************
     2************'/)
      END
```

```
      SUBROUTINE MATPEN
C     LAST CORRECTION BY:  T.J.KVALSTAD,   DECEMBER 30, 1982
C     DATE/VERSION      :  30.DEC.1982/1
C
C     PURPOSE OF SUBROUTINE:
C          THIS SUBROUTINE CALCULATES ALL REQUIRED INFORMATION
C          ABOUT FOUNDATION AREA, AND ESTIMATES THE PENETRATION
C          OF THE MAT INTO THE SEA BED UNDER THE WEIGHT OF THE
C          FLATFORM
C
C     METHOD/THEORY:
C          BEARING CAPACITY THEORY ACCORDING TO BRINCH-HANSEN
C          HAS BEEN IMPLEMENTED
C
C
C     PARAMETER LIST:
C          R:  REAL         I:  INTEGER      /XXXXXX/: COMMON BLOCK
C
C
C
C
      COMMON/MATDAT/TOTL,TOTW,SLOTL,SLOTW,HOLEL,HOLEW,TMAT,
     1ALE,BLE,YLE
      COMMON/SETTLE/ZPEN
      COMMON/WEIGHT/GW,GVAR,GLEG,GMAT,GLX,GLY,GVARLX,GVARLY
      COMMON/SOIL/NPRO,SUO,RK,SUH,PHI,TH,RR,GAMS
      COMMON/CONSTA/PI,GAMW,G
      COMMON/LEGDAT/NL
      COMMON/AREADA/AREAT,YCGMAT,XCGMAT,WIYT,WIXT
C
C     CALCULATE PLATFORM WEIGHT FOR PENETRATION ANALYSIS
C
      GTOT=GW+GVAR+NL*GLEG+GMAT
C
C     CALCULATE MAT AREA, AREAT
C
      AREA1=TOTL*TOTW
      AREA2=SLOTL*SLOTW
      AREA3=HOLEL*HOLEW
      AREAT=AREA1-AREA2-AREA3
C
C     CALCULATE THE MOMENT OF INERTIA ABOUT THE Y-AXIS, WIYT
C
      WIY1=TOTL*(TOTW**3.)/12.
      WIY2=SLOTL*(SLOTW**3.)/12.
      WIY3=HOLEL*(HOLEW**3.)/12.
      WIYT=WIY1-WIY2-WIY3
C
C     CALCULATE THE CENTROID COORDINATES YCGMAT,XCGMAT OF THE MAT
C
      YCGMAT=AREA1*TOTL/2.-AREA2*SLOTL/2.-AREA3*(SLOTL+BLE+HOLEL/2.)
      YCGMAT=YCGMAT/AREAT+YLE-TOTL
      XCGMAT=0.
C
C     CALCULATE THE MOMENT OF INERTIA ABOUT THE X-AXIS, WIXT
C
      WIX1=TOTW*(TOTL**3.)/12.+AREA1*(YLE-TOTL/2.)**2.
      WIX2=SLOTW*(SLOTL**3.)/12.+AREA2*(TOTL-YLE-SLOTL/2.)**2.
      WIX3=HOLEW*(HOLEL**3.)/12.+AREA3*(YLE-ALE-HOLEL/2.)**2.
      WIXT=WIX1-WIX2-WIX3
```

```
C     CALCULATE AVERAGE SOIL PRESSURE UNDER MAT DUE TO WEIGHTS
C     AND THEIR ECCENTRICITIES
C
      OMY=GW*GLX+GVAR*GVARLX
      OMX=GW*GLY+GVAR*GVARLY
      QAVW=GTOT/AREAT
      Q1=GAVW+OMY/WIYT*TOTW/2.+OMX/WIXT*YLE
      Q2=QAVW-OMY/WIYT*TOTW/2.+OMX/WIXT*YLE
      Q3=GAVW-OMY/WIYT*TOTW/2.-OMX/WIXT*(TOTL-YLE)
      Q4=GAVW+OMY/WIYT*TOTW/2.-OMX/WIXT*(TOTL-YLE)
C
C     PRINT INFORMATION ABOUT MAT AREA, MOMENTS OF INERTIA AND
C     AVERAGE VERTICAL STRESS
C
      WRITE(6,9000) AREA1,AREA2,AREA3,AREAT,WIYT,WIXT,XCGMAT,
     1YCGMAT,QAVW,Q1,Q2,Q3,Q4
C
C     CHECK BEARING CAPACITY/CALCULATE PENETRATION
C
C
C     CALCULATE BEARING CAPACITY FOR SOFT CLAY PROFILE
C
      IF(NPRO.NE.1) GO TO 200
      ZPEN=0.
C     DEFINE AVERAGE RATIO OF WIDTH/LENGTH OF FOOTING BOL
      BOL=TOTW/TOTL*AREAT/AREA1
C     DEFINE AVERAGE WIDTH OF FOOTING BAV
      BAV=BOL*TOTL
C     DEFINE AVERAGE UNDRAINED SHEAR STRENGTH AS THE UNDRAINED
C     SHEAR STRENGTH AT DEPTH 0.2*BAV BELOW THE MAT
  110 SUAV=SUO+RK*(ZPEN+0.2*BAV)
      CALL MATBRH(BAV,TOTL,AREAT,BOL,ZPEN,SUAV,0.,0.,GTOT,0.,0.,QULT)
C     CHECK THICKNESS OF SOFT LAYER VS AVERAGE WIDTH TO INCLUDE
C     EVENTUAL SQUEEZING EFFECT ON BEARING CAPACITY FACTOR
C      THICK=TH-ZPEN
C      IF(THICK.GT.0.3*BAV) GO TO 150
C      SUAVSQ=(SUO+(ZPEN+TH)/2*RK)
C      CALL MATSQZ(BAV,TOTL,AREAT,ZPEN,SUAVSQ,0.,0.,GTOT,0.,0.,QSQU)
C 150  CONTINUE
  200 CONTINUE
C      IF(QSQU.GT.QULT) QULT=QSQU
      IF(QULT.GT.QAVW) GO TO 600
      WRITE(6,9010) ZPEN,QULT,QSQU
      ZPEN=ZPEN+0.1
      GO TO 110
  600 CONTINUE
      WRITE(6,9050) ZPEN
      RETURN
 9000 FORMAT( 1H1,'SUBROUTINE MATPEN',//
     1          '  TOTAL AREA OF MAT             =',F10.2/
     2          ' -SLOT AREA                     =',F10.2/
     3          ' -HOLE AREA                     =',F10.2/
     4          ' =NET AREA OF MAT               =',F10.2//
     5          '  INERTIA MOMENT ABOUT Y-AXIS   =',E11.3/
     6          '  INERTIA MOMENT ABOUT X-AXIS   =',E11.3//
     7          '  X-COORD. OF CENTER OF GRAVIT=',F10.2/
     8          '  Y-COORD. OF CENTER OF GRAVIT=',F10.2//
     9          '  AVERAGE VERTICAL STRESS       =',F10.2/
     1          '  CORNER STRESS +X,+Y           =',F10.2/
     2          '  CORNER STRESS -X,+Y           =',F10.2/
     3          '  CORNER STRESS -X,-Y           =',F10.2/
```

9

```
4               '   CORNER STRESS +X,-Y          =',F10.2//
5' PENETRATION    BEARING    SQUEEZING'/
6'    OF MAT      CAPACITY  CAPACITY '//)
9010 FORMAT(F13.2,2E11.3)
9050 FORMAT(//' PENETRATION AFTER SET DOWN ESTIMATED TO BE, ZPEN=',
    1F10.1/)
     END
```

```
      SUBROUTINE MATBRH(BEFF,EFFL,AEFF,BOL,Z,SU,PHI,FH,FV,POEFF,GAMSEF,
     1QULT)
C
C     LAST CORRECTION BY:  T.J. KVALSTAD, JANUARY 10, 1983
C     DATE/VERSION       :  10. JAN. 1983/1
C
C     PURPOSE OF SUBROUTINE:
C          THIS SUBROUTINE CALCULATES THE BEARING CAPACITY
C          OF A FOOTING ON UNDRAINED OR DRAINED SOILS
C
C     METHOD:
C          THE BEARING CAPACITY METHOD ACCORDING TO BRINCH-HANSENS
C          FORMULA AS DESCRIBED IN APPENDIX F OF VERITAS RULES FOR
C          FIXED OFFSHORE PLATFORMS
C
C     PARAMETER LIST:
C          R: REAL             I: INTEGER        /XXXXXX/ COMMON BLOCK
C
C
C     BEFF    R                = EFFECTIVE WIDTH OF FOOTING
C     EFFL    R                = EFFECTIVE LENGTH OF FOOTING
C     AEFF    R                = EFFECTIVE AREA
C     Z       R                = DEPTH BELOW MUDLINE
C     SU      R                = UNDRAINED SHEAR STRENGTH OF CLAY TO BE
C                                USED IN CALCULATION
C     PHI     R                = FRICTION ANGLE OF COHESIONLESS LAYERS
C                                TO BE USED IN CALCULATION
C     FH      R                = HORIZONTAL FORCE
C     FV      R                = VERTICAL FORCE
C     POEFF   R                = EFFECTIVE OVERBURDEN PRESSURE
C     GAMSEF  R                = SUBMERGED UNIT WEIGHT OF SANDY SOIL
C     QULT    R                = VERTICAL BEARING CAPACITY OF FOUNDATION
C
C     DEFINE BEARING CAPACITY FACTORS RNC,RNQ,RNGAM,
C
      RNC=5.14
      PHIR=0.01745329*PHI
      RNQ=EXP(3.1415*TAN(PHIR))*(TAN(.785398+PHIR/2.))**2.
      RNGAM=2.*(RNQ+1)*TAN(PHIR)
C
C      WRITE(6,9600) RNQ,RNGAM,RNC
C 9600 FORMAT(4F10.2/)
C
C     DEFINE LOAD INCLINATION FACTORS
C
      RIQ=(1.0-(0.5*FH/FV))**5.
      RIGAM=(1.0-(0.7*FH/FV))**5.
      RICA=0.5-0.5*SQRT(1.-FH/AEFF/SU)
C      WRITE(6,9600) RIQ,RIGAM,RICA
C
C     DEFINE SHAPE FACTORS
C
      SQ=1.+RIQ*BOL*SIN(PHIR)
      SGAM=1.-.4*RIGAM*BOL
      SCA=0.2*(1.-2.*RICA)*BOL
C
C     DEFINE DEPTH FACTOR FOR CLAYEY SOILS
C
      DCA=0.3*ATAN(Z/BEFF)
C      WRITE(6,9600) SQ,SGAM,SCA,DCA
C
```

```
      IF(PHI.GT.0.01) GO TO 100
C
      QULT=AEFF*RNC*SU*(1+SCA+DCA*RICA)
      GO TO 200
C
  100 QULT=AEFF*(0.5*GAMSEF*BEFF*RNGAM*SGAM*RIGAM+POEFF*RNQ*SQ*RIQ)
C
  200 CONTINUE
      RETURN
      END
```

```
      SUBROUTINE WAVEPR
C
C
C     LAST CORRECTION BY:  T.J.KVALSTAD,    FEBRUARY 9, 1982
C     DATE/VERSION       :  15.DEC 1982/3
C
C     PURPOSE OF SUBROUTINE:
C          THIS SUBROUTINE CALCULATES THE WAVE PRESSURE
C          FORCES ON AT JACK-UP MAT FOUNDATION BY INTEGRATING
C          THE PRESSURE DUE TO LINEAR WAVE THEORY (AIRY)
C          OVER THE TOP AND THE SIDES OF THE MAT
C
C
C     PARAMETER LIST:
C          R: REAL     I: INTEGER   /XXXXXX/ COMMON BLOCK
C
C
C     WAVE DATA:
C     WH       R   /WAVDAT/   = WAVE HEIGHT
C     TMIN     R   /PERIOD/   = MINIMUM WAVE PERIOD
C     TMAX     R   /PERIOD/   = MAXIMUM PERIOD
C     TSTEP    R   /PERIOD/   = STEP IN PERIOD
C     NTI      I              = NUMBER OF DIFFERENT PERIODS TO BE
C                               EVALUATED
C     TI       R              = WAVE PERIOD
C     WL       R              = WAVE LENGTH
C     WLO      R              = DEEP WATER WAVE LENGTH
C     WL1      R              = ITERATED WAVE LENGTH
C     DIFFL    R              = RELATIVE DIFFERENCE BETWEEN
C                               WL AND WL1 IN ITERATION
C                               DIFFL.LE.0.001
C     IT       I              = COUNTING NUMBER WHEN VARYING PERIOD
C     T        R              = TIME IN SECONDS WHEN STEPPING THROUGH
C                               A WAVE PERIOD IN 21 STEPS
C     ITIME    I              = COUNTING NUMBER FOR TIME STEPS
C
C
C     PHYSICAL CONSTANTS:
C     PI       R   /CONSTA/   = PI=3.14
C     GAMW     R   /CONSTA/   = UNIT WEIGHT OF SEAWATER
C     G        R   /CONSTA/   = CONSTANT OF GRAVITY 9.81M/S**2
C
C     MAT GEOMETRY DATA:
C     TOTL     R   /MATDAT/   = TOTAL LENGTH OF MAT
C     TOTW     R   /MATDAT/   = TOTAL WIDTH OF MAT
C     SLOTL    R   /MATDAT/   = TOTAL LENGTH OF SLOT
C     SLOTW    R   /MATDAT/   = TOTAL WIDTH OF SLOT
C     HOLEL    R   /MATDAT/   = LENGTH OF HOLE
C     HOLEW    R   /MATDAT/   = WIDTH OF HOLE
C     TMAT     R   /MATDAT/   = THICKNESS OF MAT
C     ALE      R   /MATDAT/   = WIDTH OF END SECTION
C     BLE      R   /MATDAT/   = WIDTH OF MAT SECTION BETWEEN SLOT AND HOLE
C     YLE      R   /MATDAT/   = REFERENCE DISTANCE FROM END OF MAT TO X AXIS
C                               X AXIS IS DEFINED TO BE MIDWAY BETWEEN THE
C                               LEGS OF 4-LEGGED PLATFORMS AND 1/3 FROM THE
C                               AFT LEGS OF 3-LEGGED PLATFORMS, I.E.
C                               APPROXIMATELY IN THE CENTER OF GRAVITY OF
C                               THE DECK STRUCTURE
C
C     ZPEN     R   /SETTLE/   = PENETRATION OF MAT INTO SEA BOTTOM
C
```

```
C     XL1      R                  = START COORDINATE OF PRESSURE INTEGRATION FOR
C                                   A RECTANGLE IN DIRECTION OF WAVE TRAVEL
C     XL2      R                  = CORRESPONDING END COORDINATE
C     YL       R                  = BREADTH OF RECTANGLE
C
C
C     PRESSURE AND FORCE PARAMETERS:
C     FVX      R   /MATFOR/       = TOTAL VERTICAL FORCE ON MAT, WAVE IN X-DIRECTION
C     FVY      R   /MATFOR/       = TOTAL VERTICAL FORCE ON MAT, WAVE IN Y-DIRECTION
C     FX       R   /MATFOR/       = TOTAL HORIZONTAL FORCE ON MAT, WAVE IN X DIRECTION
C     FY       R   /MATFOR/       = TOTAL HORIZONTAL FORCE ON MAT, WAVE IN Y DIRECTION
C     OMY      R   /MATFOR/       = OVERTURNING MOMENT ABOUT Y-AXIS, WAVE IN X DIRECTION
C     OMX      R   /MATFOR/       = OVERTURNING MOMENT ABOUT X-AXIS, WAVE IN Y DIRECTION
C     FVMAT    R                  = VERTICAL FORCE ON RECTANGLE
C     FHOR     R                  = HORIZONTAL FORCE ON RECTANGLE SIDE
C     OMMAX    R                  = OVERTURNING MOMENT ON RECTANGLE
C
C                                    ^ Y-AXIS
C                                    Ø
C                                    Ø
C                      ************TOTW*************
C                                    Ø
C                      *****************************
C                *     *     A       Ø             *     *
C                *     *     L       Ø             *     *
C                Y     *     E    HOLEW            *     *
C                L     *     ************* H       *     *
C                E     *     *       Ø     * O     *     *
C                *     *     *       Ø     * L     *     T
C                *     *     *       Ø     * E     *     O
C                *     *     ************* L       *     T           X-AXIS
C     -----------------*-----B-------O-------------*-----L---------------->
C                      *     L       Ø             *     *
C                      *     E     SLOTW           *     *
C                      *    *************** S      *
C                      *    *        Ø     * L     *
C                      *    *        Ø     * O     *     *
C                      *    *        Ø     * T     *     *
C                      *    *        Ø     * L     *     *
C                      ******        Ø     *********
C                                    Ø
C
C
C                                    ^   Z-AXIS
C                                    Ø
C                                    Ø                           SWL
C     -------------------------------O----------------------------------
C                                    Ø
C                                    Ø
C                                    Ø
C                                    Ø
C                                    Ø
C                                    Ø
C                                    Ø
C                                    Ø
C                                    Ø
C                       ******************************
C                       *     *      Ø      *      *   TMAT        X-AXIS
C     ------------------******************************------------------->
C
```

```
C
      COMMON/MATDAT/TOTL,TOTW,SLOTL,SLOTW,HOLEL,HOLEW,TMAT,
     1ALE,BLE,YLE
      COMMON/WAVDAT/WH,DW,HW(10),WL(10),WS(10),TI(10)
      COMMON/PERIOD/TMIN,TMAX,TSTEP
      COMMON/CONSTA/PI,GAMW,G
      COMMON/MATFOR/FVX(10,21),FX(10,21),OMY(10,21),FVY(10,21),
     1FY(10,21),OMX(10,21)
      COMMON/SETTLL/ZPEN
C
      DIMENSION FVMAT(6),YL(6),XL1(6),XL2(6),FHOR(6),OMMAX(6)
C
C     ***********************************************************
C
C     START DO LOOP ON WAVE PERIODS STARTING AT TMIN AND STEPPING
C     WITH TSTEP UNTIL TMAX. PERIOD CALLED TI, TIME CALLED T
C
C     ***********************************************************
C
      NTI=(TMAX-TMIN)/TSTEP+1
      DO 900 IT=1,NTI
      TI(IT)=TMIN+(IT-1)*TSTEP
C
C     CALCULATE WAVELENGTH WL FOR GIVEN WATERDEPTH AND PERIOD
C     BY ITERATION STARTING AT WLO = DEEP WATER CONDITION
C
      WLO=G*TI(IT)*TI(IT)/2./PI
      WL1=WLO
   50 WL(IT)=WLO*TANH(2.*PI*DW/WL1)
      DIFFL=ABS(WL1-WL(IT))/WL1
      IF(DIFFL.LT.0.001) GO TO 60
      WL1=WL(IT)
      GO TO 50
   60 CONTINUE
C
C     CALCULATE WAVE STEEPNESS WS(IT)
C
      HW(IT)=WH
      WS(IT)=WH/WL(IT)
      WRITE(6,9090) WH,DW,TI(IT),WL(IT),WS(IT)
C
C     CHECK WAVEHEIGHT VS STEEPNESS
C
      HWMAX=WL(IT)/(5.4+0.023*WL(IT))
      IF(WL(IT).LT.200.0) HWMAX=WL(IT)/10.
      IF(HWMAX.LT.WH) GO TO 70
      GO TO 80
   70 CONTINUE
C
C     IF THE GIVEN WAVEHEIGHT EXCEEDS THE MAXIMUM POSSIBLE WAVEHEIGHT
C     BASED ON THE ABOVE MAX STEEPNESS CRITERIA, IE 1/10 FOR WAVE
C     LENGTHS LESS THAN 200 M AND SOMEWHAT LESS FOR LARGER WAVES,
C     THE WAVEHEIGHT IS REDUCED TO THE MAX. POSSIBLE FOR FURTHER
C     ANALYSIS AND A WARNING IS GIVEN
C
      HW(IT)=HWMAX
      WS(IT)=HW(IT)/WL(IT)
      WRITE(6,9000) WH,HWMAX,TI(IT),HW(IT),WS(IT)
   80 CONTINUE
C
C     ***********************************************************
```

```
C
C     START LOOP ON TIME STEPPING THROUGH WAVE PERIOD TI IN
C     21 STEPS STARTING AT TIME T=0 ENDING WITH TI
C
C     **************************************************************
C
      WRITE(6,9100)
      DT=TI(IT)/20.
      DO 800 ITIME=1,21
      T=(ITIME-1)*DT
C
C     CALCULATE PRESSURE AMPLITUDE ON TOP OF MAT, PAMPT
C     AND AT MID HEIGHT OF SIDE, PAMPS
C
      PAMPT=.5*GAMW*HW(IT)*(COSH(2.*PI*TMAT/WL(IT)))/
     1COSH(2.*PI*DW/WL(IT))
      PAMPS=.5*GAMW*HW(IT)*(COSH(2.*PI*TMAT/2./WL(IT)))/
     1COSH(2.*PI*DW/WL(IT))
C
C     CALCULATE VERTICAL FORCE ON MAT FOR WAVES TRAVELLING IN THE
C     X-DIRECTION,FVX(IT,ITIME) AND FOR WAVES TRAVELLING IN THE
C     Y-DIRECTION FVY(IT,ITIME)
C     INDEX 1 RELATES TO THE TOTAL MATAREA, WAVES IN X-DIRECTION
C     INDEX 2   "         "   SLOTAREA,          "     "     "
C     INDEX 3   "         "   HOLEAREA,          "     "     "
C     INDEX 4   "         "   TOTAL MATAREA, WAVES IN Y DIRECTION
C     INDEX 5   "         "   SLOTAREA,          "     "     "
C     INDEX 6   "         "   HOLEAREA,          "     "     "
C
      YL(1)=TOTL
      XL1(1)=-TOTW/2.
      XL2(1)=TOTW/2.
      YL(2)=SLOTL
      XL1(2)=-SLOTW/2.
      XL2(2)=SLOTW/2.
      YL(3)=HOLEL
      XL1(3)=-HOLEW/2.
      XL2(3)=HOLEW/2.
      YL(4)=TOTW
      XL1(4)=YLE-TOTL
      XL2(4)=YLE
      YL(5)=SLOTW
      XL1(5)=XL1(4)
      XL2(5)=XL1(5)+SLOTL
      YL(6)=HOLEW
      XL1(6)=YLE-HOLEL-ALE
      XL2(6)=XL1(6)+HOLEL
      DO 100 N=1,6
      FVMAT(N)=PAMPT*WL(IT)*YL(N)/2./PI*(SIN(2.*PI*(XL2(N)/WL(IT)-T/
     1TI(IT)))-SIN(2.*PI*(XL1(N)/WL(IT)-T/TI(IT))))
  100 CONTINUE
      FVX(IT,ITIME)=FVMAT(1)-FVMAT(2)-FVMAT(3)
      FVY(IT,ITIME)=FVMAT(4)-FVMAT(5)-FVMAT(6)
C
C     CALCULATE OVERTURNING MOMENTS DUE TO VERTICAL WAVE PRESSURE
C     ON TOP OF MAT RELATIVE TO Y-AXIS, OMY(IT,ITIME) AND RELATIVE TO
C     X-AXIS, OMX(IT,ITIME) FOR WAVES TRAVELLING IN THE X- AND
C     Y-DIRECTION RESPECTIVELY
C
      DO 150 N=1,6
```

```
      OMMAX(N)=PAMPT*YL(N)*(WL(IT)*WL(IT)/4./PI/PI*(COS(2.*PI*(XL2(N)/
     1WL(IT)-T/TI(IT)))-COS(2.*PI*(XL1(N)/WL(IT)-T/TI(IT))))+XL2(N)*
     2WL(IT)/2./PI*SIN(2.*PI*(XL2(N)/WL(IT)-T/TI(IT)))-XL1(N)*WL(IT)/
     32./PI*SIN(2.*PI*(XL1(N)/WL(IT)-T/TI(IT))))
  150 CONTINUE
      OMY(IT,ITIME)=OMMAX(1)-OMMAX(2)-OMMAX(3)
      OMX(IT,ITIME)=OMMAX(4)-OMMAX(5)-OMMAX(6)
C
C     CALCULATE HORIZONTAL FORCES DUE TO WAVE PRESSURE ON THE SIDE
C     OF THE MAT STRUCTURE IN THE X- AND Y-DIRECTION, FX(IT,ITIME)
C     AND FY(IT,ITIME)
C
      DO 200 N=1,6
      FHOR(N)=PAMPS*YL(N)*(COS(2.*PI*(XL1(N)/WL(IT)-T/TI(IT)))-COS(2.*
     1PI*(XL2(N)/WL(IT)-T/TI(IT))))*(TMAT-ZPEN)
  200 CONTINUE
      FX(IT,ITIME)=FHOR(1)-FHOR(2)-FHOR(3)
      FY(IT,ITIME)=FHOR(4)-FHOR(5)-FHOR(6)
      OMY(IT,ITIME)=OMY(IT,ITIME)+TMAT/2.*FX(IT,ITIME)
      OMX(IT,ITIME)=OMY(IT,ITIME)+TMAT/2.*FY(IT,ITIME)
C
C     PRINT OUTPUT TABLE
C
      WRITE(6,9110) ITIME,T,FVX(IT,ITIME),FX(IT,ITIME),OMY(IT,ITIME),
     1FVY(IT,ITIME),FY(IT,ITIME),OMX(IT,ITIME)
  800 CONTINUE
  900 CONTINUE
      RETURN
 9000 FORMAT(' **WARNING!  GIVEN WAVEHEIGHT WH   =',F10.3,' EXCEEDS'/
     1       '              MAX WAVEHEIGHT HWMAX  =',F10.3,' FOR'/
     2       '              WAVE PERIOD TI(IT)    =',F10.3,/
     3       '              ANALYSIS WILL BE'/
     4       '              PERFORMED WITH HW(IT) =',F10.3,' GIVING'/
     5       '              WAVE STEEPNESS WS(IT) =',F10.3//)
 9090 FORMAT(1H1,'PROGRAM *SAM*      : STABILITY ANALYSIS OF MATS'/
     1          /' SUBROUTINE WAVEPR: CALCULATION OF VERTICAL FORCES'
     2          /'                   HORIZONTAL FORCES AND'
     3          /'                   OVERTURNING MOMENTS DUE TO'
     4          /'                   WAVE PRESSURE ON THE SEABED'/
     5         //' WAVEHEIGHT   =',F10.1/
     6           ' WATERDEPTH   =',F10.1/
     7           ' WAVE PERIOD  =',F10.1/
     8           ' WAVE LENGTH  =',F10.1/
     9           ' WAVE STEEPN. =',F10.3//)
 9100 FORMAT(///,' TIME  TIME IN        WAVES IN X-DIRECTION
     1     WAVES IN Y-DIRECTION'/
     2          ' STEP  SECONDS VERTICAL HORIZONTAL OVERTURNING   VERTI
     3CAL HORIZONTAL OVERTURNING'/
     4          '  NO            FORCE     FORCE        MOMENT      FORC
     5E     FORCE      MOMENT'//)
 9110 FORMAT(1X,I3,2X,F5.2,2F10.2,F14.2,2F10.2,F14.2)
      END
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```
      SUBROUTINE MATENV
C
C
C     LAST CORRECTION BY:    T. J. KVALSTAD,    DECEMBER 19, 1982
C     DATE/VERSION       :   15.DEC.1982/2
C
C     PURPOSE OF SUBROUTINE:
C          THIS SUBROUTINE CALCULATES THE HORIZONTAL FORCES ACTING ON
C          THE LEGS OF A MAT SUPPORTED JACK-UP PLATFORM UNDER COMBINED
C          WAVE AND CURRENT ACTION.
C
C     METHOD/THEORY:
C          MORISON'S EQUATION IS USED AND THE FORCES ARE CALCULEATED FOR
C          THE CASE WHERE WAVE AND CURRENT IS ACTING IN THE SAME DIRECT-
C          ION. FORCE CALCULATION IS CARRIED OUT FOR WAVES AND CURRENT
C          ACTING IN THE X-DIRECTION AND IN THE Y-DIRECTION
C
C
C     PARAMETER LIST:
C          R: REAL       I: INTEGER      /XXXXXX/ COMMON BLOCK
C
C
C     WIND DATA:
C     VW      R  /WIND/     = AVERAGE WIND SPEED
C     WAREAX R  /WIND/     = WIND AREA IN X-DIRECTION
C     WAREAY R  /WIND/     = WIND AREA IN Y-DIRECTION
C     WARM    R  /WIND/     = WIND LEVER ARM ABOVE AIR GAP, IE
C                             TOTAL LEVER ARM EQUAL DW+AG+WARM
C
C     WIND FORCES AND OVERTURNING MOMENTS:
C     WFX     R  /WIND/     = WIND FORCE IN X-DIRECTION
C     WFY     R  /WIND/     = WIND FORCE IN Y-DIRECTION
C     WOMY    R  /WIND/     = OVERTURNING MOMENT ABOUT Y-AXIS DUE TO WFX
C     WOMX    R  /WIND/     = OVERTURNING MOMENT ABOUT X-AXIS DUE TO WFY
C
C     WAVE DATA:
C     WH      R  /WAVDAT/   = WAVE HEIGHT
C     TMIN    R  /PERIOD/   = MINIMUM WAVE PERIOD
C     TMAX    R  /PERIOD/   = MAXIMUM WAVE PERIOD
C     TSTEP   R  /PERIOD/   = STEP IN PERIOD
C     NTI     I             = NUMBER OF DIFFERENT PERIODS TO BE EVALUATED
C     TI(IT)  R             = WAVE PERIODS
C     WL(IT)  R             = WAVE LENGTHS
C     WLO     R             = DEEP WATER WAVE LENGTH
C     WL1     R             = ITERATED WAVE LENGTH
C     DIFFL   R             = RELATIVE DIFFERENCE BETWEEN WL AND WL1 IN
C                             ITERATIVE CALCULATION OF WL
C     IT      I             = COUNTING NUMBER WHEN VARYING PERIOD
C     T       R             = TIME IN SECONDS WHEN STEPPING THROUGH WAVE
C                             WAVE PERIOD IN 21 STEPS
C     ITIME   I             = COUNTING NUMBER FOR TIME STEPS
C
C     CURRENT DATA:
C     VC1     R  /CURR/     = TIDAL CURRENT VELOCITY AT STILL WATER LEVEL
C     VC2     R  /CURR/     = WAVE INDUCED CURRENT VELOCITY AT SWL
C
C
C     FORCES AND OVERTURNING MOMENTS DUE TO WAVE AND CURRENT:
C     FHLEGX R  /LEGFOR/   = TOTAL WAVE FORCE ON NL LEGS IN X DIRECTION
C     FHLEGY R  /LEGFOR/   = TOTAL WAVE FORCE ON NL LEGS IN Y DIRECTION
C     OMLEGY R  /LEGFOR/   = OVERTURNING MOMENT DUE TO FHLEGX
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C     OMLEGX R  /LEGFOR/  = OVERTURNING MOMENT DUE TO FHLEGY
C
C
C     PHYSICAL CONSTANTS AND COEFFICIENTS:
C     PI      R  /CONSTA/  = 3.14
C     GAMW    R  /CONSTA/  = UNIT WEIGHT OF SEAWATER
C     G       R  /CONSTA/  = GRAVITY ACCELERATION (9.81M/S**2
C     CI      R  /CONSTA/  = INERTIA COEFFICIENT
C     CD      R  /CONSTA/  = DRAG COEFFICIENT
C
C
C     LEG GEOMETRY DATA:
C     NL      I  /LEGDAT/  = NUMBER OF LEGS (MAX. 4)
C     X(I)    R  /LEGDAT/  = X COORDINATES OF LEGS
C     Y(I)    R  /LEGDAT/  = Y COORDINATES OF LEGS
C     HLEG(I)R   /LEGDAT/  = TOTAL LENGTHS OF LEGS
C     DI(I)   R  /LEGDAT/  = EFFECTIVE INERTIA DIAMETER OF LEGS
C     DD(I)   R  /LEGDAT/  = EFFECTIVE DRAG DIAMETER OF LEGS
C     EI(I)   R  /LEGDAT/  = BENDING STIFFNESS OF LEGS
C     EA(I)   R  /LEGDAT/  = AXIAL STIFFNESS OF LEGS
C     GA(I)   R  /LEGDAT/  = SHEAR STIFFNESS OF LEGS
C
C
C     COMMON BLOCKS
C
C
      COMMON/HEAD/TEXT(40)
      COMMON/WIND/VW,WAREAX,WAREAY,WARM,WFX,WFY,WOMY,WOMX
      COMMON/WAVDAT/WH,DW,HW(10),WL(10),WS(10),TI(10)
      COMMON/PERIOD/TMIN,TMAX,TSTEP
      COMMON/CONSTA/PI,GAMW,G,CI,CD
      COMMON/LEGDAT/NL,X(4),Y(4),HLEG(4),DI(4),DD(4),EI(4),EA(4),GA(4)
      COMMON/CURR/VC1,VC2
      COMMON/LEGFOR/FHLEGX(10,21),OMLEGY(10,21),FHLEGY(10,21),OMLEGX(10,
     121)
C
C
      DIMENSION FTOTX(21,4,15),FTOTY(21,4,15),OMTOTX(21,4,15),OMTOTY(21,
     14,15),RLARFX(21,4,15),RLARFY(21,4,15)
C
C
C     ******************************************************************
C
C     CALCULATE WIND FORCE, LEVERARM WIND AND OVERTURNING
C     MOMENT FOR WIND
C
C     ******************************************************************
C
      WFX=0.297*0.01*GAMW/G*WAREAX*VW*ABS(VW)
      WFY=0.297*0.01*GAMW/G*WAREAY*VW*ABS(VW)
      ARMW=WARM+AG-DW
      WOMY=WFX*ARMW
      WOMX=WFY*ARMW
      WRITE(6,9200) VW,WAREAX,WAREAY,ARMW
      WRITE(6,9210) WFX,WOMY,WFY,WOMX
C
C     ******************************************************************
C
C     START DO LOOP ON WAVE PERIODS STARTING AT TMIN AND STEPPING WITH
C     TSTEP UNTIL TMAX. PERIOD CALLED TI,TIME CALLED T
C
```

```
C     ****************************************************************************
C
C
      NTI=(TMAX-TMIN)/TSTEP+1
      IF(NTI.GT.10) NTI=10
      DO 600 IT=1,NTI
      TI(IT)=TMIN+(IT-1)*TSTEP
C
C     CALCULATE WAVELENGTH WL FOR GIVEN WATERDEPTH AND PERIOD BY
C     ITERATION STARTING WITH THE DEEPWATER VALUE WLO
C
      WL0=G*TI(IT)*TI(IT)/2./PI
      WL1=WL0
   50 WL(IT)=WL0*TANH(2.*PI*DW/WL1)
      DIFFL=ABS(WL1-WL(IT))/WL1
      IF(DIFFL.LT.0.001) GO TO 60
      WL1=WL(IT)
      GO TO 50
   60 CONTINUE
C
C     CALCULATE WAVE STEEPNESS WS(IT)
C
      WS(IT)=WH/WL(IT)
C
      WRITE(6,9090) WH,DW,TI(IT),WL(IT),WS(IT)
C
      HW(IT)=WH
C
C     CHECK WAVEHEIGHT AND WAVE STEEPNESS
C
C     HWMAX IS EQUAL TO A MAXIMUM WAVE HEIGTH FOR WAVES WITH WAVE-
C     LENGTH WL ASSUMING A MAXIMUM STEEPNESS OF 1/10 FOR WAVELENGTHS
C     LESS THAN 200 M AND ACCORDING TO THE FOLLOWING EXPRESSION FOR
C     WAVELENGTHS EXCEEDING 200 M.
C
      HWMAX=WL(IT)/(5.4+0.023*WL(IT))
      IF(WL(IT).LT.200.0) HWMAX=WL(IT)/10.0
      IF(HWMAX.LT.WH) GO TO 70
      GO TO 80
   70 CONTINUE
C
C     THE GIVEN WAVEHEIGHT, WH, EXCEEDS THE MAX. WAVEHEIGHT AS ESTIMATED
C     ABOVE. A WARNING IS GIVEN AND WAVE HEIGHT IS REDUCED TO THE
C     CALCULATED MAXIMUM VALUE HWMAX IN THE FOLLOWING CALCULATION OF
C     FORCES.
C
      HW(IT)=HWMAX
      WS(IT)=HW(IT)/WL(IT)
      WRITE(6,9000) WH,HWMAX,TI(IT),HW(IT),WS(IT)
   80 CONTINUE
C
C
C     ****************************************************************************
C
C     START LOOP ON TIMESTEPPING THROUGH WAVE PERIOD TI IN 21 STEPS
C     STARTING AT TIME T=0 AND ENDING WITH T=TI
C
C     ****************************************************************************
C
```

```
      DT=TI(IT)/20.
C
      WRITE(6,9100)
      DO 500 M=1,21
      T=(M-1)*DT
C
C     ****************************************************************************
C
C     CALL SUBROUTINE MATWAV FOR EACH LEG.
C
C     CALCULATE COMBINED WAVE AND CURRENT FORCES FOR WAVE ACTION IN X
C     DIRECTION, FTOTX(M,LN,IT)= TOTAL HORIZONTAL FORCE IN X DIRECTION
C     AT TIME STEP M ON LEG NO. LN FOR PERIOD STEP IT, RLARFX IS THE
C     LEVER ARM OF THIS FORCE RELATIVE TO SEA BED AND OMTOTY IS THE
C     CORRESPONDING OVERTURNING MOMENT ABOUT THE Y AXIS
C
      FHLEGX(IT,M)=0.
      OMLEGY(IT,M)=0.
      DO 100 LN=1,NL
      CALL MATWAV (X(LN),DW,LN,HW(IT),TI(IT),T,DO,DI,WL(IT),
     1FTOTX(M,LN,IT),OMTOTY(M,LN,IT),RLARFX(M,LN,IT),0)
      FHLEGX(IT,M)=FHLEGX(IT,M)+FTOTX(M,LN,IT)
      OMLEGY(IT,M)=OMLEGY(IT,M)+OMTOTY(M,LN,IT)
  100 CONTINUE
C
C     DO CORRESPONDINGLY FOR WAVES IN Y DIRECTION
C     FTOTY(M,LN,IT)= TOTAL HORIZONTAL FORCE IN Y DIRECTION AT TIME
C     STEP M ON LEG NO. LN FOR PERIOD STEP IT. RLARFY IS THE LEVER ARM
C     OF THIS FORCE RELATIVE TO SEA BED AND OMTOTX IS THE CORRESPONDING
C     OVERTURNING MOMENT ABOUT THE X-AXIS
C
      FHLEGY(IT,M)=0.
      OMLEGX(IT,M)=0.
      DO 150 LN=1,NL
      CALL MATWAV (Y(LN),DW,LN,HW(IT),TI(IT),T,DO,DI,WL(IT),
     1FTOTY(M,LN,IT),OMTOTX(M,LN,IT),RLARFY(M,LN,IT),0)
      FHLEGY(IT,M)=FHLEGY(IT,M)+FTOTY(M,LN,IT)
      OMLEGX(IT,M)=OMLEGX(IT,M)+OMTOTY(M,LN,IT)
  150 CONTINUE
C
C     PRINT FORCES AND MOMENTS
C
      WRITE(6,9110) M,T,FHLEGX(IT,M),OMLEGY(IT,M),FHLEGY(IT,M),OMLEGX(IT
     1,M)
  500 CONTINUE
  600 CONTINUE
      RETURN
 9000 FORMAT(' **WARNING!  GIVEN WAVEHEIGHT WH  =',F10.3,' EXCEEDS'/
     1       '              MAX WAVEHEIGHT HWMAX =',F10.3,' FOR     '/
     2       '              WAVE PERIOD TI(IT)   =',F10.3,'ANALYSIS'/
     3       '              PERFORMED WITH HW(IT)=',F10.3,'GIVING   '/
     4       '              WAVE STEEPNESS WS(IT)=',F10.3//)
 9090 FORMAT(1H1,'PROGRAM *SAM*     : STABILITY ANALYSIS OF MATS'/
     1       /' SUBROUTINE MATENV: CALCULATION OF HORIZONTAL FORCES'
     2       /'                    ON LEGS DUE TO COMBINED WAVE AND'
     3       /'                    CURRENT BY MORISONS EQUATION'/
     4        ' WAVEHEIGHT   =',F10.1/
     5        ' WATERDEPTH   =',F10.1/
     6        ' WAVE PERIOD  =',F10.1/
     7        ' WAVE LENGTH  =',F10.1/
     8        ' WAVE STEEPN. =',F10.4//)
```

```
9100 FORMAT(/,' TIME  TIME IN          WAVES IN X-DIRECTION
   1       WAVES IN Y-DIRECTION'/
   2        ' STEP  SECONDS VERTICAL HORIZONTAL OVERTURNING   VERTICA
   3L  HORIZONTAL  OVERTURNING'/
   4        ' NO                 FORCE      FORCE       MOMENT        FORCE
   5     FORCE        MOMENT'//)
9110 FORMAT(1X,I3,2X,F5.2,10X,F10.2,F14.2,10X,F10.2,F14.2)
9200 FORMAT(1H1,'PROGRAM *SAM*     : STABILITY ANALYSIS OF MATS'/
   1          /' SUBROUTINE MATENV: CALCULATION OF WIND FORCES'/
   2          ' WINDSPEED     =',F10.1/
   3          ' WINDAREAX     =',F10.1/
   4          ' WINDAREAY     =',F10.1/
   5          ' LEVERARM      =',F10.1,' RELATIVE TO SEA BED'///
   6          '  WIND IN X-DIRECTION          WIND IN Y-DIRECTION'/
   2          ' HORIZONTAL OVERTURNING    HORIZONTAL  OVERTURNING'/
   4          '    FORCE      MOMENT          FORCE        MOMENT'//)
9210 FORMAT(1X,F10.2,F12.2,4X,F10.2,F12.2)
     END
```

```
      SUBROUTINE MATWAV (X,DW,LN,HW,TI,T,DD,DI,WL,FORCE,OVMOM,ARM,IPR)
C     PURPOSE:
C        THIS ROUTINE CALCULATE LOADS ON A VERTICAL LEG EXPOSED
C        TO WAVES AND CURRENT
C
C     METHOD:
C        LINEAR WAVE THEORY AND MORISONS EQUATION FOR DRAG AND INERTIA
C        FORCES ON A CYLINDER WITH EFFECTIVE INERTIA DIAMETER DI AND
C        EFFECTIVE DRAG DIAMETER DD IS USED. THE SUBMERGED PART OF
C        THE LEG IS DIVIDED IN 10 SEGMENTS AND THE TOTAL FORCE AND
C        OVERTURNING MOMENT RELATIVE TO SEA BED IS CALCULATED BY
C        INTEGRATION OVER ALL 10 LEG SEGMENTS
C
      COMMON/CONSTA/PI,GAMW,G,CI,CD
      COMMON/CURR/VC1,VC2
C
C     CALCULATE ANGULAR FREQUENCY OMEG, WAVE NUMBER RK AND
C     INSTANTANEOUS LEVEL OF WATER IN POSITION X AT TIME TI
C
      OMEG=2.0*PI/TI
      RK=2.0*PI/WL
      ETA=HW/2.0*COS(RK*X-OMEG*T)
C
C     INSTANTANEOUS DEPTH OF WATER DE
C
      DE=DW+ETA
      FORCE=0.0
      OVMOM=0.0
      ARM=0.0

C
C     DIVIDE SUBMERGED PART OF LEG IN 10 SEGMENTS
C
      DZ=DE/10.
C
C     LOOP ON ALL SUBMERGED SEGMENTS
C
      DO 100 NZ=1,10
C
C     AVERAGE Z-COORDINATE OF SEGMENT
C
      ZL=ETA-NZ*DZ+DZ/2.0
C
C     CALCULATE LEVER ARM OF FORCE RELATIVE TO SEA BED
C
      RLARM=DW+ZL
C
C     WAVE INDUCED HORIZONTAL VELOCITY AND ACCELERATION
C
      VW=PI*HW/TI*(COSH(2.*PI*(ZL+DW)/WL)/(SINH(2.*PI*DW/WL)))
     1*COS(2.*PI*(X/WL-T/TI))
      ACCW=2.0*PI**2.0*HW/(TI**2.0)*(COSH(2.*PI*(ZL+DW)/WL)/
     1(SINH(2.*PI*DW/WL)))*SIN(2.0*PI*(X/WL-T/TI))
C
C     CURRENT VELOCITY AT MID OF SEGMENT USING VERITAS GUIDELINES
C     FOR CURRENT DISTRIBUTION WITH DEPTH OF TIDAL AND WAVE INDUCED
C     CURRENT
      VC=VC1*(RLARM/DW)*EXP(0 14)+VC2*RLARM/DW
C
C     COMBINED HORIZONTAL WATER PARTICLE VELOCITY DUE TO WAVES AND CURRENT
C
```

```
      VTOT=VW+VC
C
C     MORISONS FORMULA FOR INERTIA AND DRAG FORCES ON LEG SEGMENT
C
      FI=GAMW/G*CI*ACCW*PI/4.0*DI**2 *DZ
      FD=0.5*GAMW/G*CD*ABS(VTOT)*VTOT*DZ*DD
      FSEG=FI+FD
C
C     ACCUMULATED FORCE AND OVERTURNING MOMENT ON LEG
C
      FORCE=FORCE+FSEG
      OVMOM=OVMOM+FSEG*RLARM
      IF(IPR.EQ.0) GO TO 100
      WRITE(6,9900) NZ,ZL,VW,ACCW,VC,VTOT,FI,FD,RLARM,FSEG
 9900 FORMAT(1X,I3,9F10.3)
  100 CONTINUE
      IF(ABS(FORCE).LT.0.0001) GO TO 105
      ARM=OVMOM/FORCE
      GO TO 107
  105 FORCE=0.0
      OVMOM=0.0
      ARM=0.0
  107 CONTINUE
      IF(IPR.EQ.0) GO TO 110
      WRITE(6,9901) LN,FORCE,OVMOM,ARM
 9901 FORMAT(' LEG NO.   HOR.FORCE   OVERT.MOM   LEVERARM'/
     1/I5,5X,3F10.3)
  110 CONTINUE
      RETURN
      END
```

```
      SUBROUTINE TOTFOR
C
C     LAST CORRECTION BY:    T.J.KVALSTAD,    DECEMBER 30, 1982
C     DATE/VERSION       :   30.DEC.1982/1
C
C     PURPOSE OF SUBROUTINE:
C           THIS SUBROUTINE ADDS ALL FORCES FROM WEIGHTS,WAVEPRESSURE
C           ON MAT,WAVE AND CURRENT ON LEGS AND WIND FORCES INTO A
C           TOTAL LOAD VECTOR; A) FOR FORCES ACTING IN THE X DIRECTION
C           AND B) FOR FORCES ACTING IN THE Y-DIRECTION
C
C     METHOD/THEORY:
C           NONE
C
C     PARAMETER LIST:  SEE COMMENTS
C
C
C
      COMMON/TOTFOR/FVXT(10,21),FXT(10,21),OMYT(10,21),FVYT(10,21),
     1FYT(10,21),OMXT(10,21)
      COMMON/LEGFOR/FHLEGX(10,21),OMLEGY(10,21),FHLEGY(10,21),OMLEGX(10,
     121)
      COMMON/MATFOR/FVX(10,21),FX(10,21),OMY(10,21),FVY(10,21),
     1FY(10,21),OMX(10,21)
      COMMON/WIND/VW,WAREAX,WAREAY,WARM,WFX,WFY,WOMY,WOMX
      COMMON/WEIGHT/GW,GVAR,GLEG,GMAT,GLX,GLY,GVARLX,GVARLY
      COMMON/LEGDAT/NL,X(4),Y(4)
      COMMON/PERIOD/TMIN,TMAX,TSTEP
C
      NTI=(TMAX-TMIN)/TSTEP+1
C
C     PRINT HEADING FOR THIS SUBROUTINE
C
      WRITE(6,9000)
C     CALCULATE TOTAL WEIGHT OF PLATFORM,GTOT AND OVERTURNING MOMENTS
C     IN THE X-AND Y-DIRECTION, GMX AND GMY
C
      GTOT=GW+GVAR+3*GLEG+GMAT
      GMY=GW*GLX+GVAR*GVARLX
      GMX=GW*GLY+GVAR*GVARLY
      DO 10 LN=1,NL
      GMY=GMY+GLEG*X(LN)
      GMX=GMX+GLEG*Y(LN)
   10 CONTINUE
C
C     PRINT GTOT,GMY AND GMX
C
      WRITE(6,9010) GTOT,GMY,GMX
C
C     START LOOPING ON ALL DIFFERENT PERIODS
C
      DO 300 IT=1,NTI
C
C
      WRITE(6,9000)
      TI=TMIN+(IT-1)*TSTEP
      WRITE(6,9020) IT,TI
C
C     START LOOPING ON ALL 21 TIMESTEPS
C
      DO 200 M=1,21
```

```
C
      DT=TI/20.
      T=DT*(M-1)
C
C     SUM UP ALL VERTICAL FORCES FOR WIND,WAVE AND CURRENT ACTING
C     IN X-DIRECTION AND Y-DIRECTION RESPECTIVELY
C
      FVXT(IT,M)=GTOT+FVX(IT,M)
      FVYT(IT,M)=GTOT+FVY(IT,M)
C
C     SUM UP ALL HORIZONTAL FORCES
C
      FXT(IT,M)=FX(IT,M)+FHLEGX(IT,M)+WFX
      FYT(IT,M)=FY(IT,M)+FHLEGY(IT,M)+WFY
C
C     SUM UP ALL OVERTURNING MOMENTS
C
      OMYT(IT,M)=GMY+WOMY+OMY(IT,M)+OMLEGY(IT,M)
      OMXT(IT,M)=GMX+WOMX+OMX(IT,M)+OMLEGX(IT,M)
C
C     PRINT TOTAL FORCE VECTORS
C
      WRITE(6,9030) M,T,FVXT(IT,M),FXT(IT,M),OMYT(IT,M),FVYT(IT,M),
     1FYT(IT,M),OMXT(IT,M)
  200 CONTINUE
  300 CONTINUE
      RETURN
 9000 FORMAT(1H1,'PROGRAM *SAM*     : STABILITY ANALYSIS OF MATS'
     1           /' SUBROUTINE TOTFOR: CALCULATION OF TOTAL FORCE'
     2           /'                    VECTORS AND OVERTURNING MOM-'
     3           /'                    MENTS FOR WAVE+CURRENT,WIND'
     4           /'                    WAVE PRESSURE ON MAT AND'
     5           /'                    WEIGHT OF HULL,VARIABLE,LEGS'
     6           /'                    AND MAT'//)
 9010 FORMAT(' TOTAL WEIGHT OF PLATFORM,          GTOT    =',F10.3/
     1       ' TOTAL OVERTURNING MOMENT ABOUT              '/
     2       ' Y AXIS DUE TO WEIGHTS,             GMY     =',F10.3/
     3       ' TOTAL OVERTURNING MOMENT ABOUT              '/
     4       ' X AXIS DUE TO WEIGHTS,             GMX     =',F10.3//)
 9020 FORMAT(' PERIOD NO.                         IT      =',I7/
     1       ' WAVE PERIOD IN SECONDS             TI      =',F10.3//
     2' TIME  TIME IN        WAVES IN X-DIRECTION                  WAVE
     3S IN Y-DIRECTION'/
     4' STEP  SECONDS VERTICAL HORIZONTAL OVERTURNING   VERTICAL  HORIZO
     5NTAL OVERTURNING'/
     6'  NO.            FORCE      FORCE      MOMENT       FORCE      FORC
     7E        MOMENT'//)
 9030 FORMAT(1X,I3,2X,F5.2,6E12.3)
      END
```

```
      SUBROUTINE MATSLI
C
C     LAST CORRECTION BY: T.J.KVALSTAD     FEBRUARY 6, 1983
C     DATE/VERSION       : 6.FEB.1983/1
C
C     PURPOSE OF SUBROUTINE:
C          THIS SUBROUTINE CALCULATES THE SAFETY AGAINST SLIDING
C          FAILURE ALONG A HORIZONTAL SLIDING SURFACE DIRECTLY
C          UNDER THE TIP OF THE SKIRTS.
C     METHOD:
C          THE METHOD IS THE NORMAL WAY OF ANALYSING SLIDING
C          RESISTANCE FOR GRAVITY TYPE STRUCTURES. THE PASSIVE
C          EARTH PRESSURE DEVELOPED AGAINST THE FRONTAL SKIRTS AND
C          THE SIDE FRICTION ALONG THE SIDE OF THE SKIRTS  PARALLEL
C          TO THE DIRECTION OF SLIDING ARE TAKEN INTO ACCOUNT.
C          THE SOIL HAS A  SHEAR STRENGHT SUO AT THE SURFACE
C          INCREASING WITH A GRADIENT RK WITH DEPTH AND THE FOOTING
C          IS SQUARE WITH A WIDTH XL AND A LENGTH NORMAL TO THE
C          SLIDING DIRECTION EQUAL TO YL
C
C
C     PARAMETER LIST:   SEE COMMENTS
C
C
      COMMON/MATDAT/TOTL,TOTW,SLOTL,SLOTW,HOLEL,HOLEW,TMAT,
     1ALE,BLE,YLE
      COMMON/SKIRT/SKH,SKTH,ALFASK
      COMMON/SETTLE/ZPEN
      COMMON/WAVDAT/WH,DW,HW(10),WL(10),WS(10),TI(10)
      COMMON/PERIOD/TMIN,TMAX,TSTEP
      COMMON/CONSTA/PI,GAMW,G,CI,CD
      COMMON/MATFOR/FVX(10,21),FX(10,21),OMY(10,21),FVY(10,21),
     1FY(10,21),OMX(10,21)
      COMMON/TOTFOR/FVXT(10,21),FXT(10,21),OMYT(10,21),FVYT(10,21),
     1FYT(10,21),OMXT(10,21)
      COMMON/SOIL/NPRO,SUO,RK,SUH,PHI,TH,RR
      COMMON/SAFE/FSDMI(10,2),FSSMI(10,2),FSLMI(10,2),FSSLMI(10,2),
     1AREG(10,21,2)
C
      DIMENSION XL(3),YL(3),SIDL(3),XQP(3),XQA(3),QP(3),QA(3)
      DIMENSION EPR(3),EPL(3),EPRES(3),BRES(3),SSRES(3),SLIRES(21,2)
      DIMENSION SF(21,2),SFMIN(2),MMIN(2)
C
C     DEFINE HEIGHT OF ACTIVE AND PASSIVE EARTH PRESSURE WEDGES
C
      H=ZPEN+SKH
C
C     STEP THROUGH ALL PERIODS
C
      NTI=(TMAX-TMIN)/TSTEP+1
      DO 600 IT=1,NTI
      WRITE(6,9090) WH,DW,TI(IT),WL(IT),WS(IT)
C
      SFMIN(1)=1000.
      SFMIN(2)=1000.
C
C     STEP THROUGH ALL 21 TIMESTEPS IN WAVE
C
      DT=TI(IT)/20.
      DO 500 M=1,21
      T=(M-1)*DT
```

```
C
C     CALCULATE WAVE PRESSURE AMPLITUDE AT SEA BOTTOM
C
      PAMPB= 5*GAMW*HW(IT)/COSH(2.*PI*DW/WL(IT))
C
C     PERFORM SLIDING ANALYSIS FOR X-DIRECTION AND Y DIRECTION
C
      DO 400 L=1,2
      IF(L.EQ.2) GO TO 10
C
C     SET DIMENSIONS FOR WAVE IN X-DIRECTION
C     INITIAL 1 RELATES TO TOTAL MAT
C     INITIAL 2 RELATES TO THE HOLE AREA
C     INITIAL 3 RELATES TO THE SLOT AREA
C
      XL(1)=TOTW
      YL(1)=TOTL
      XL(2)=HOLEW
      YL(2)=HOLEL
      XL(3)=SLOTW
      YL(3)=SLOTL
      SIDL(1)=TOTW*2.-SLOTW
      SIDL(2)=HOLEW*2.
      SIDL(3)=SLOTW
      XGP(1)=XL(1)/2.+H/2.
      XGA(1)=-XGP(1)
      XGP(2)=-HOLEW/2.+H/2.
      XGA(2)=-XGP(2)
      XGP(3)=-SLOTW/2.+H/2.
      XGA(3)=-XGP(3)
      FH=FXT(IT,M)
      GO TO 20
C
C     SET DIMENSIONS FOR WAVE IN Y-DIRECTION
   10 CONTINUE
      XL(1)=TOTL
      YL(1)=TOTW
      XL(2)=HOLEL
      YL(2)=HOLEW
      XL(3)=SLOTL
      YL(3)=SLOTW
      SIDL(1)=TOTL*2.
      SIDL(2)=HOLEL*2.
      SIDL(3)=SLOTL*2.
      XGP(1)=YLE+H/2.
      XGA(1)=YLE-TOTL-H/2.
      XGP(2)=YLE-ALE-HOLEL+H/2.
      XGA(2)=YLE-ALE-H/2.
      XGP(3)=YLE-TOTL+H/2.
      XGA(3)=YLE-TOTL+SLOTL-H/2.
      FH=FYT(IT,M)
   20 CONTINUE
C
C     CHECK DIRECTION OF POSSIBLE FAILURE DEVELOPMENT
C     AND SET PARAMETER NCH POSITIVE FOR POSITIVE DRIVING
C     FORCE AT MUDLINE AND VICE VERSA.
C
      NCH=-1
      IF(FH.GE.0.) NCH=1
C
C     CALCULATE: - SLIDING RESISTANCE ALONG BASE
```

```
C                  * NET EARTH PRESSURE AGAINST SIDES
C                  * SIDE RESISTANCE ALONG SKIRTS
C      FOR TOTAL AREA, HOLE AREA AND SLOT AREA
C
       DO 100 J=1,3
C
C      CALCULATE AVERAGE WATER PRESSURE Q AT SEA BED
C      ACTING ON ACTIVE AND PASSIVE WEDGES. ASSUMING AN ACTIVE
C      AND A PASSIVE SLOPE ANGLE OF 45 DEGREES AND TAKING THE
C      PRESSURE AT DISTANCE H/2 FROM THE RESPECTIVE SIDE AS
C      AVERAGE VALUE.
C
       QP(J)=PAMPB*COS(2.*PI*(XQP(J)/WL(IT)-T/TI(IT)))
       QA(J)=PAMPB*COS(2.*PI*(XQA(J)/WL(IT)-T/TI(IT)))
C
C      ADD RESISTANCE DUE TO DIFFERENCE BETWEEN ACTIVE AND
C      PASSIVE EARTH PRESSURE ACTING ON OUTER SIDE OF MAT
C      IF ACTIVE PRESSURE IS NEGATIV ITS EFFECT IS NEGLECTED
C      I.E. GAPPING IS ASSUMED TO DEVELOP AND THE WAVE PRESSURE
C      IS ASSUMED TO ACT DIRECTLY AT THE EMBEDDED SIDE.
C
       SUAV=SUO+.5*H*RK
       EPR(J)=NCH*2.*SUAV*H*YL(J)+.5*GAMS*H*H*YL(J)+QP(J)*H*YL(J)
       EPL(J)=.5*GAMS*H*H*YL(J)+QA(J)*H*YL(J)-NCH*2.*SUAV*H*YL(J)
C
C       IF(M.NE.5) GO TO 777
C       WRITE(6,9660) J,QP(J),QA(J),EPR(J),EPL(J)
C 9660 FORMAT(' J,QP,QA,EPR,EPL',I5,4F10.3)
C  777 CONTINUE
C      CORRECT EARTH PRESSURE VALUES AT END OF SLOT WHEN WAVES
C      ACTING IN Y-DIRECTION
C
       IF(L.EQ.1) GO TO 22
       IF(J.NE.1) GO TO 21
       EPL(1)=EPL(1)-.5*GAMS*H*H*YL(3)-QA(1)*H*YL(3)
      1+NCH*2.*SUAV*H*YL(3)
   21  IF(J.NE.3) GO TO 22
       EPR(3)=0.
   22  CONTINUE
C
C      CHECK IF ACTIVE EARTH PRESSURE IS NEGATIVE AND IN THIS CASE
C      REPLACE WITH WATER PRESSURE AT SKIRT DUE TO GAPPING ASSUMPTION
C
       IF(NCH.EQ.1.AND.EPL(J).LT.0.) GO TO 25
       GO TO 30
   25  EPL(J)=QA(J)*H*YL(J)
C      CORRECTION FOR SLOT
       IF(L.EQ.2.AND.J.EQ.1) EPL(1)=EPL(1)-QA(1)*H*YL(3)
   30  CONTINUE
       IF(NCH.EQ.-1.AND.EPR(J).LT.0.) EPR(J)=QA(J)*H*YL(J)
       EPRES(J)=(EPR(J)-EPL(J))*NCH
C
C
C      CALCULATE RESISTANCE ALONG BASE AREA
C
       BRES(J)=XL(J)*YL(J)*(SUO+RK*H)
C
C      CALCULATE RESISTANCE DUE TO SIDE FRICTION ALONG PARALLEL SKIRTS
C      ASSUMING SKIRTS  ALONG ALL SIDES INCLUDING HOLE AND SLOT
C
       SSRES(J)=(SUO+.5*RK*H)*H*SIDL(J)
```

```
C
  100 CONTINUE
C
      IF(M.NE.5) GO TO 103
      DO 101  J=1,3
      WRITE(6,9665) J,EPR(J),EPL(J),EPRES(J),BRES(J),SSRES(J)
C 9665 FORMAT(' J,EPR,EPL,EPRES,BRES,SSRES',I5,5F10.3)
C  101 CONTINUE
C  103 CONTINUE
C
C     CHECK IF PASSIVE RESISTANCE PLUS SIDE FRICTION ON SKIRTS
C     IN HOLE AREA EXCEEDS SLIDING CAPACITY OF HOLE AREA
C
      IF(ABS(BRES(2)+SSRES(2)).GT.EPRES(2)) GO TO 130
      BRES(2)=0.
      EPRES(2)=0.
      SSRES(2)=0.
  130 CONTINUE
C
C     CHECK IF PASSIVE RESISTANCE IN SLOT AREA EXCEEDS SLIDING
C     CAPACITY OF SLOT AREA FOR WAVES ACTING IN X-DIRECTION
C
      IF(L.EQ.2) GO TO 140
      IF(L.EQ.1.AND.BRES(3).GT.EPRES(3)) GO TO 140
      BRES(3)=0.
      EPRES(3)=0.
  140 CONTINUE
C
C     CHECK IF SIDE FRICTION IN SLOT AREA EXCEEDS SLIDING
C     CAPACITY OF SLOT AREA FOR WAVES COMING IN Y-DIRECTION
C
      IF(L.EQ.1) GO TO 150
      IF(L.EQ.2.AND.BRES(3).GT.SSRES(3)) GO TO 150
      BRES(3)=0.
      SSRES(3)=0.
  150 CONTINUE
C
C
C      IF(M.NE.5) GO TO 203
C      DO 201  J=1,3
C      WRITE(6,9665) J,EPR(J),EPL(J),EPRES(J),BRES(J),SSRES(J)
C  201 CONTINUE
C  203 CONTINUE
C
C
C     CALCULATE TOTAL SLIDING RESISTANCE
C
      BREST=BRES(1)-BRES(2)-BRES(3)
      EPREST=EPRES(1)+EPRES(2)+EPRES(3)
      SSREST=SSRES(1)+SSRES(2)+SSRES(3)
      SLIRES(M,L)=BREST+EPREST+SSREST
C
C     CALCULATE REQUIRED BASE AREA TO MOBILIZE
C     SUFFICIENT HORIZONTAL CAPACITY
C
      FHBASE=ABS(FH)-SSREST-EPREST
C
C     AREG CAN BE NEGATIV IF SIDEFRICTION AND PASSIVE
C     EARTH PRESSURE EXCEED THE HORIZONTAL FORCE. THIS
C     MEANS THAT THE BEARING CAPACITY IN SUBR. MATLOC WILL
C     NOT BE REDUCED IN ANY ELEMENT.
```

```
C
      AREQ(IT,M,L)=FHBASE/(SUO+RK*H)
C
C     CALCULATE SAFETY FACTOR
C
      SF(M,L)=SLIRES(M,L)/FH
C
C     CHECK IF THIS IS MINIMUM
C
      IF(ABS(SF(M,L)).GT.ABS(SFMIN(L))) GO TO 400
      SFMIN(L)=SF(M,L)
C
C     PRINT DETAILED RESULTS FOR CHECK OF ANALYSIS
C
      NPRSLI=0
      IF(NPRSLI.EQ.0) GO TO 400
      IF(M.EQ.1.AND.L.EQ.1) WRITE (6,9500)
      IF(M.NE.5) GO TO 400
      WRITE(6,9510) M,L,BREST,EPREST,SSREST,SLIRES(M,L),
     1AREQ(IT,M,L),SF(M,L)
 9500 FORMAT(/' DETAILED PRINT FOR SLIDING ANALYSIS'/
     1' TIME  DIR.   BASE        EARTH     SIDE        TOTAL     AREA
     1 SAFETY'/
     2' STEP  1=X    RESIS-      PRESS. FRICTION    SLIDING MOBILIZED
     2 FACTOR'/
     3' NO.   2=Y    TANCE        .        .        RESIST.      .
     3   .  '/)
 9510 FORMAT(2I4,6F10.3)
  400 CONTINUE
  500 CONTINUE
C
C     PRINT RESULTS FOR EACH TIME STEP
      WRITE(6,9301)
      DO 510 M=1,21
      WRITE(6,9300) M,SLIRES(M,1),FXT(IT,M),SF(M,1),SLIRES(M,2),
     1FYT(IT,M),SF(M,2)
  510 CONTINUE
C
C     DEFINE MINIMUM VALUES FOR THIS PERIOD IN X DIRECTION
C     AND Y DIRECTION
C
      DO 530 L=1,2
  530 FSSLMI(IT,L)=10000.
      DO 550 L=1,2
      DO 540 M=1,21
      IF(ABS(FSSLMI(IT,L)).LT.ABS(SF(M,L))) GO TO 540
      FSSLMI(IT,L)=SF(M,L)
      MMIN(L)=M
  540 CONTINUE
  550 CONTINUE
C
C     PRINT RESULT FOR THIS PERIOD
C
      WRITE(6,9600) MMIN(1),FSSLMI(IT,1),MMIN(2),FSSLMI(IT,2)
C
  600 CONTINUE
      RETURN
 9090 FORMAT(1H1,'PROGRAM *SAM*    :STABILITY ANALYSIS OF MATS'/
     1         /' SUBROUTINE MATSLI:CALCULATION OF SAFETY AGAINST'/
     2          '                   SLIDING FAILURE AT SKIRT TIP'/
     3          ' WAVEHEIGHT   =',F10.1/
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```
     4          ' WATERDEPTH    =',F10.1/
     5          ' WAVE PERIOD   =',F10.1/
     6          ' WAVE LENGTH   =',F10.1/
     7          ' WAVE STEEPN.  =',F10.4//)
 9200 FORMAT(1H1,' WAVES IN X-DIRECTION: WAVE PERIOD NO.',I3/
     1          '                       TIME STEP   NO.',I3/)
 9210 FORMAT(1H1,' WAVES IN Y-DIRECTION: WAVE PERIOD NO.',I3/
     1          '                       TIME STEP   NO.',I3/)
 9301 FORMAT(//' RESULTS OF SLIDING RESISTANCE ANALYSIS'//
     1' TIME         X DIRECTION                        Y-DIRECTION        '/
     2'  STEP  RESIST-  HORIZ.   SAFETY   RESIST-  HORIZ.   SAFETY  '/
     3'   NO.   ANCE    FORCE    FACTOR    ANCE    FORCE    FACTOR'//)
 9300 FORMAT(1X,I5,2(2F10.1,F10.3))
 9600 FORMAT(/' MINIMUM SAFETY FACTORS FOR THIS PERIOD WERE:'/
     1' IN X-DIRECTION IN TIME STEP NO.',I5,' FSX =',F10.2/
     2' IN Y-DIRECTION IN TIME STEP NO.',I5,' FSY =',F10.2/)
      END
```

```
      SUBROUTINE MATLOC
C
C     LAST CORRECTION BY: T.J. KVALSTAD      SEPTEMBER 1, 1983
C     DATE/VERSION      : 1.SEP.1983/3
C
C     PURPOSE OF SUBROUTINE:
C          THIS SUBROUTINE CALCULATES THE SAFETY AGAINST LOCAL
C          FAILURE OF THE MOST STRESSED PART OF A MAT WITH
C          HOLE AND SLOT OF CERTAIN MINIMUM DIMENSIONS.
C
C     METHOD:
C
C          THE BASIC ASSUMPTION IS THAT A MAT WITH SUFFICIENT
C          SLIDING RESISTANCE MAY FAIL UNDER THE COMBINATION
C          OF HIGH WEIGHT AND MOMENT LOADING ON THE MOST STRESSED
C          SIDE OF THE MAT.
C
C          IN SUBROUTINE MATSLI THE REQUIRED AREA TO RESIST
C          THE HORIZONTAL FORCE HAS BEEN CALCULATED. THE
C          SOMEWHAT SIMPLIFIED APPROACH FOLLOWED IN THE
C          CALCULATION OF SAFETY AGAINST LOCAL FAILURE IS THAT
C          UNDER THE AREA REQUIRED FOR HORIZONTAL LOAD THE
C          BEARING CAPACITY FACTOR NC IS REDUCED TO 2.57
C          (ACCORDING TO BRINCH-HANSEN THEORY) WHILE UNDER THE
C          REST OF THE AREA THE FULL VERTICAL BEARING CAPACITY
C          WILL BE PRESENT.
C
C          MOMENT RESISTANCE ABOUT THE LEAST STRESSED EDGE
C          IS THEN CALCULATED WITH THE ABOVE ASSUMPTIONS FOR
C          VERTICAL BEARING CAPACITY AND COMPARED WITH THE
C          CORRESPONDING MOMENT DUE TO TOTAL VERTICAL LOAD
C          AND TOTAL OVERTURNING MOMENT.
C
C     PARAMETER LIST:   SEE COMMENTS
C
C
      COMMON/MATDAT/TOTL,TOTW,SLOTL,SLOTW,HOLEL,HOLEW,TMAT,
     1ALE,BLE,YLE
      COMMON/SKIRT/SKH,SKTH,ALFASK
      COMMON/SETTLE/ZPEN
      COMMON/WAVDAT/WH,DW,HW(10),WL(10),WS(10),TI(10)
      COMMON/PERIOD/TMIN,TMAX,TSTEP
      COMMON/CONSTA/PI,GAMW,G,CI,CD
      COMMON/MATFOR/FVX(10,21),FX(10,21),OMY(10,21),FVY(10,21),
     1FY(10,21),OMX(10,21)
      COMMON/TOTFOR/FVXT(10,21),FXT(10,21),OMYT(10,21),FVYT(10,21),
     1FYT(10,21),OMXT(10,21)
      COMMON/SOIL/NFRO,SUO,RK,SUH,PHI,TH,RR
      COMMON/SAFE/FSOMI(10,2),FSSMI(10,2),FSLMI(10,2),FSSLMI(10,2),
     1AREQ(10,21,2)
C
      DIMENSION IK(2,12),A(12),X(12),Y(12),BOL(12),ZAV(12),Q(12)
      DIMENSION EPR(3),EPL(3),EPRES(3),BRES(3),SSRES(3),SLIRES(21,2)
      DIMENSION SF(21,2),SFMIN(2),MMIN(2)
C
      DATA (IK(1,I),I=1,12)/1,3,7,9,4,10,5,11,2,6,8,12/
      DATA (IK(2,I),I=1,12)/1,2,3,4,5,6,7,8,9,10,11,12/
C
C     CALCULATE BASIC VALUES FOR PART AREAS
C
C     AREAS
```

```
C
      BFX=(TOTW-SLOTW)/2.
      BBX=(TOTW-HOLEW)/2.
      A(1)=BFX*SLOTL
      A(2)=A(1)
      A(3)=BFX*BLE
      A(4)=SLOTW*BLE/2.
      A(5)=A(4)
      A(6)=A(3)
      A(7)=HOLEL*BBX
      A(8)=A(7)
      A(9)=BBX*ALE
      A(10)=HOLEW*ALE/2.
      A(11)=A(10)
      A(12)=A(9)
C
C     COORDINATES OF CENTER OF AREAS
C
      X(1)=-(TOTW-BFX)/2.
      X(2)=-X(1)
      X(3)=X(1)
      X(4)=-SLOTW/2.
      X(5)=-X(4)
      X(6)=X(2)
      X(7)=-(TOTW-BBX)/2.
      X(8)=-X(7)
      X(9)=X(7)
      X(10)=-HOLEW/2.
      X(11)=-X(10)
      X(12)=X(8)
      Y(1)=YLF-TOTL+SLOTL/2.
      Y(2)=Y(1)
      Y(3)=YLF-ALE-HOLEL-BLE/2.
      Y(4)=Y(3)
      Y(5)=Y(3)
      Y(6)=Y(3)
      Y(7)=YLF-ALE-HOLEL/2.
      Y(8)=Y(7)
      Y(9)=YLF-ALE/2.
      Y(10)=Y(9)
      Y(11)=Y(9)
      Y(12)=Y(9)
C
C     CALCULATE APPOXIMATE B/L RATIOS FOR ALL AREAS FOR
C     LOADING IN X  AND Y DIRECTION
C
      DO 10 I=1,12
      BOL(I)=0.
   10 CONTINUE
      BOL(1)=BFX/SLOTL/2.
      IF(BOL(1).GT.1.) BOL(1)=1./BOL(1)
      BOL(2)=BOL(1)
      BOL(9)=BBX/ALE
      IF(BOL(9).GT.1.) BOL(9)=1./BOL(9)
      BOL(12)=BOL(9)
C
C     CALCULATE APROXIMATE DEPTH FOR
C     AVERAGE UNDRAINED SHEAR STRENGTH
C
      ZAV(1)=.2*BFX
      ZAV(2)=ZAV(1)
```

```
      ZAV(3)=.3*BFX
      ZAV(4)=.2*BLE
      ZAV(5)=ZAV(4)
      ZAV(6)=ZAV(3)
      ZAV(7)=.2*BBX
      ZAV(8)=.2*BBX
      ZAV(9)=.1*(BBX+ALE)
      ZAV(10)=ALE/2.
      ZAV(11)=ZAV(10)
      ZAV(12)=ZAV(9)
C
C     STEP THROUGH ALL PERIODS
C
      NTI=(TMAX-TMIN)/TSTEP+1
      DO 600 IT=1,NTI
      WRITE(6,9090) WH,DW,TI(IT),WL(IT),WS(IT)
C
C     PRINT DETAILS ABOUT PART AREAS
      WRITE(6,9095)
      DO 5 I=1,12
      WRITE(6,9096) I,A(I),X(I),Y(I),BOL(I),ZAV(I)
    5 CONTINUE
C
      SFMIN(1)=1000.
      SFMIN(2)=1000.
C
C     STEP THROUGH ALL 21 TIMESTEPS IN WAVE
C
      DT=TI(IT)/20.
      DO 500 M=1,21
      T=(M-1)*DT
C
C     CALCULATE WAVE PRESSURE AMPLITUDE AT SEA BOTTOM
C
      PAMPB=.5*GAMW*HW(IT)/COSH(2.*PI*DW/WL(IT))
C
C     PERFORM LOCAL BEARING CAPACITY ANALYSIS FOR
C     X-DIRECTION AND Y-DIRECTION
C
C
C     CALCULATE LOADS ACTING ON MAT. IT IS ASSUMED THAT
C     THE MAT CONSISTS OF SLENDER BEAMS FOR THIS FAILURE
C     TYPE. THE OVERTURNING MOMENT AND VERTICAL FORCE ON THE
C     TOP OF THE MAT WILL THUS BE COMPENSATED BY THE
C     INCREASE IN BEARING CAPACITY DUE TO WAVE PRESSURE
C     ON THE SEA BED NEXT TO THE BEAMS. THIS ASSUMPTION
C     IS ONLY VALID FOR RELATIVE SLENDER BEAMS, I.E. A
C     MAT WITH A RELATIVELY LARGE HOLE AND SLOT.
C     THE FORCES CONSIDERED WILL THUS BE THE TOTAL
C     FORCE AND MOMENT MINUS THE FORCE AND MOMENT DUE TO
C     WAVE PRESSURE ON TOP OF THE MAT.
C
      DO 400 L=1,2
      IF(L.EQ.2) GO TO 20
      FV=FVXT(IT,M)-FVX(IT,M)
      OM=OMYT(IT,M)-OMY(IT,M)
      GO TO 30
   20 CONTINUE
      FV=FVYT(IT,M)-FVY(IT,M)
      OM=OMXT(IT,M)-OMX(IT,M)
   30 CONTINUE
```

```
C
C     CHECK DIRECTION OF POSSIBLE FAILURE DEVELOPMENT
C     AND SET PARAMETER NCH POSITIVE FOR POSITIVE DRIVING
C     MOMENT AT MUDLINE AND VICE VERSA.
C
      NCH=-1
      IF(OM.GE.0.) NCH=1
C
C     CALCULATE OVERTURNING MOMENT RELATIVE TO EDGE
C
      IF(L.EQ.1) OMEDG=NCH*FV*TOTW/2.+OM
      IF(L.EQ.2.AND.NCH.EQ.1) OMEDG=OM+(TOTL-YLE)*FV
      IF(L.EQ.2.AND.NCH.EQ.-1) OMEDG=OM-FV*YLE
C
C     CALCULATE NUMBER OF ELEMENTS REQUIRED FOR HORIZONTAL
C     LOADING
C
      ARQ=AREQ(IT,M,L)
      NSTART=1
      NEND=12
      NNN=1
      IF(NCH.EQ.1) GO TO 40
      NSTART=12
      NEND=1
      NNN=-1
   40 CONTINUE
      ASUM=0.
      RESMOM=0.
      DO 100 NA=NSTART,NEND,NNN
      I=IK(L,NA)
      ASUM=ASUM+A(I)
      RICA=0.0
C
C     IF ACCUMULATED AREA LESS THAN REQUIRED FOR HORIZONTAL
C     FORCE SET INCLINATION FACTOR TO 0.5. ARQ CAN BE NEGATIVE
C     IF SIDE RESISTANCE ON SKIRTS PLUS PASSIVE EARTH PRESSURE
C     RESISTANCE EXCEED HORIZONTAL FORCE; IN THIS CASE THERE
C     WILL BE NO REDUCTION IN BEARING CAPACITY
C
      IF(ASUM.LE.ARQ) RICA=0.5
C
C     CALCULATE SHAPE FACTOR AND BEARING CAPACITY FACTOR
C     AND BEARING CAPACITY OF EACH AREA
C
      SCA=.2*(1.-2.*RICA)*BOL(I)
      RNC=5.14*(1+SCA-RICA)
      QVULT=A(I)*(RNC*(SUO+ZAV(I)*RK)+GAMS*ZPEN)
      IF(L.EQ.2) GO TO 70
      ARM=TOTW/2.+NCH*X(I)
      GO TO 80
   70 ARM=TOTL-YLE+Y(I)
      IF(NCH.EQ.-1) ARM=TOTL-ARM
   80 CONTINUE
      DMOM=ARM*QVULT
      RESMOM=RESMOM+DMOM
      IF(IPFLO.NE.1) GO TO 203
      IF(M.NE.5) GO TO 203
      DO 201 I=1,12
      WRITE(6,9665) L,NA,I,Q(I),SCA,RNC,QVULT,ARM,DMOM,RESMOM
  201 CONTINUE
  203 CONTINUE
```

```
  100 CONTINUE
C
C     CALCULATE SAFETY FACTOR
C
      SF(M,L)=RESMOM/OMEDG
C
C     CHECK IF THIS IS MINIMUM
C
      IF(ABS(SF(M,L)).GT.ABS(SFMIN(L))) GO TO 400
      SFMIN(L)=SF(M,L)
C
  400 CONTINUE
  500 CONTINUE
C
C     PRINT RESULTS FOR EACH TIME STEP
      WRITE(6,9301)
      DO 510 M=1,21
      WRITE(6,9300) M,SF(M,1),SF(M,2)
  510 CONTINUE
C
C     DEFINE MINIMUM VALUES FOR THIS PERIOD IN X DIRECTION
C     AND Y-DIRECTION
C
      DO 530 L=1,2
  530 FSLMI(IT,L)=10000.
      DO 550 L=1,2
      DO 540 M=1,21
      IF(ABS(FSLMI(IT,L)).LT.ABS(SF(M,L))) GO TO 540
      FSLMI(IT,L)=SF(M,L)
      MMIN(L)=M
  540 CONTINUE
  550 CONTINUE
C
C     PRINT RESULT FOR THIS PERIOD
C
      WRITE(6,9600) MMIN(1),FSLMI(IT,1),MMIN(2),FSLMI(IT,2)
C
  600 CONTINUE
      RETURN
 9090 FORMAT(1H1,'PROGRAM *SAM*     :STABILITY ANALYSIS OF MATS'/
     1          /' SUBROUTINE MATLOC:CALCULATION OF SAFETY AGAINST'/
     2          '                   LOCAL BEARING CAPACITY FAILURE'/
     2          '                   UNDER MOST STRESSED BEAM OF'/
     2          '                   AN A-SHAPED MAT '/
     3          ' WAVEHEIGHT   =',F10.1/
     4          ' WATERDEPTH   =',F10.1/
     5          ' WAVE PERIOD  =',F10.1/
     6          ' WAVE LENGTH  =',F10.1/
     7          ' WAVE STEEPN. =',F10.4//)
 9095 FORMAT(/' DETAILS ABOUT PART AREAS'/
     1' AREA  AREA     CENTER  COORDINATES   RATIO   AVERAGE'/
     2'  NO.  M**2.    X-COORD.   Y-COORD.     B/L     DEPTH '/)
 9096 FORMAT(I3,5F10.2)
 9300 FORMAT(3X,I4,F18.2,F18.2)
 9301 FORMAT(//' TIME STEP     SAFETY FACTOR          SAFETY FACTOR'/
     1         '    NO.         X-DIRECTION             Y-DIRECTION '/)
 9600 FORMAT(/' MINIMUM SAFETY FACTORS FOR THIS PERIOD WERE:'/
     1' IN X-DIRECTION IN TIME STEP NO.',I5,' FSX =',F10.2/
     2' IN Y DIRECTION IN TIME STEP NO.',I5,' FSY =',F10.2/)
 9665 FORMAT(3I3,7F10.2)
      END
```

```
      SUBROUTINE SHBEAR
C
C     LAST CORRECTION BY: T.J. KVALSTAD      FEBRUARY 6, 1983
C     DATE/VERSION      : 6.FEB.1983/1
C
C     PURPOSE OF SUBROUTINE:
C          THIS SUBROUTINE CALCULATES THE SAFETY AGAINST FAILURE
C          ALONG A SHALLOW SLIDING SURFACE CAUSED BY OVERSTRESSING
C          OF A TOP CLAY LAYER DUE TO THE COMBINED EFFECT OF
C          VERTICAL AND HORIZONTAL FORCES AND OVERTURNING MOMENT
C          THE SOIL HAS A  SHEAR STRENGHT SUO AT THE SURFACE
C          INCREASING WITH A GRADIENT RK WITH DEPTH AND THE FOOTING
C          IS SQUARE WITH A WIDTH XL AND A LENGTH NORMAL TO THE
C          SLIDING DIRECTION EQUAL TO YL
C
C     METHOD:
C          THE METHOD USED WAS DEVELOPED AS A PART OF THE STUDY PERFORMED
C          ON ANALYSIS METHODS FOR EVALUATION OF THE STABILITY OF JACK UP
C          FOUNDATIONS AND IS DESCRIBED ON THE FINAL REPORT OF PP2.
C          BASICALLY A CYLINDRIC FAILURE SURFACE IS ASSUMED WITH RADIUS
C          R. FOUR DIFFERENT CASES ARE ANALYZED. CASE 1 WITH THE CENTER
C          OF THE CIRCLE AT DEPTH R UNDER THE LEFT EDGE OF THE  FOOTING
C          AND CASE 2 WITH THE CENTER OF THE CIRCLE AT HEIGHT R OVER THE
C          RIGHT EDGE OF THE FOOTING WHILE CASE 3 AND 4 ARE SYMMETRIC
C          VERSIONS OF 1 AND 2. THE LATTER CASES ARE CONSIDERED  IF THE
C          DRIVING MOMENT DUE TO HORIZONTAL FORCE AND OVERTURNING MOMENT
C          ARE NEGATIVE
C
C          THE MINIMUM RADIUS THAT CAN BE ANALYSED IS EQUAL TO
C          WIDTH OF THE FOOTING, XL. THE SLIDING FOOTING AND SOIL MASS
C          ABOVE THE CYLINDER SURFACE PUSHES ON A PASSIVE EARTH PRESSURE
C          WEDGE ON THE RIGHT SIDE OF THE FOOTING  AT THE VERTICAL
C          INTERFACE BETWEEN THE SOIL MASS UNDER THE FOOTING AND THE
C          PASSIVE EARTH PRESSURE WEDGE THE FULL SHEAR STRENGTH OF THE
C          SOIL IS MOBILIZED, I.E. THE VERTICAL COMPONENT OF THE PASSIVE
C          EARTH PRESSURE. THE SIDE FRICTION FORCES ON THE SOIL MASS UNDER
C          THE FOOTING AS WELL AS ON THE PASSIVE EARTH PRESSURE WEDGE ARE
C          TAKEN INTO ACCOUNT AS WELL.
C
C
C     PARAMETER LIST:   SEE COMMENTS
C
      COMMON/MATDAT/TOTL,TOTW,SLOTL,SLOTW,HOLEL,HOLEW,TMAT,
     1ALE,BLE,YLE
      COMMON/SETTLE/ZPIN
      COMMON/WAVDAT/WH,DW,HW(10),WL(10),WS(10),TI(10)
      COMMON/PERIOD/TMIN,TMAX,TSTEP
      COMMON/CONSTA/FI,GAMW,G,CI,CD
      COMMON/MATFOR/FVX(10,21),FX(10,21),OMY(10,21),FVY(10,21),
     1FY(10,21),OMX(10,21)
      COMMON/TOTFOR/FVXT(10,21),FXT(10,21),OMYT(10,21),FVYT(10,21),
     1FYT(10,21),OMXT(10,21)
      COMMON/SOIL/NPRO,SUO,RK,SUH,PHI,TH,RR
      COMMON/SAFE/FSDMI(10,2),FSSMI(10,2),FSLMI(10,2),FSSLMI(10,2),
     1AREG(10,21,2)
C
      DIMENSION SFSHMI(21,2),RSFMIN(21,2)
C
C     STEP THROUGH ALL PERIODS
```

```
      NTI=(TMAX-TMIN)/TSTEP+1
      DO 600 IT=1,NTI
      WRITE(6,9090) WH,DW,TI(IT),WL(IT),WS(IT)
C
      SFDMIP=1000.
C
C     STEP THROUGH ALL 21 TIMESTEPS IN WAVE
C
      DT=TI(IT)/20.
      DO 500 M=1,21
      T=(M-1)*DT
C
C     CALCULATE WAVE PRESSURE AMPLITUDE AT SEA BOTTOM
C
      PAMPB=.5*GAMW*HW(IT)/COSH(2.*PI*DW/WL(IT))
C
C     PERFORM STABILITY ANALYSIS FOR X-DIRECTION AND Y-DIRECTION
C
      DO 400 L=1,2
      IF(L.EQ.2) GO TO 10
C
C     SET DIMENSIONS FOR WAVE IN X-DIRECTION
C
C      WRITE(6,9200)
      XL=TOTW
      YL=TOTL
      FV=FVXT(IT,M)
      FH=FXT(IT,M)
      OM=OMYT(IT,M)
      GO TO 20
C
C     SET DIMENSIONS FOR WAVE IN Y-DIRECTION
   10 CONTINUE
C      WRITE(6,9210)
      XL=TOTL
      YL=TOTW
      FV=FVYT(IT,M)
      FH=FYT(IT,M)
      OM=OMXT(IT,M)
   20 CONTINUE
C
C
C     FAILURE CASE 1. CENTER OF CIRCLE BELOW EDGE
C     ****************************************************
C
C     VARY RADIUS OF FAILURE SURFACE FROM MINIMUM POSSIBLE VALUE, XL TO
C     20 TIMES XL WITH A QUADRATIC STEP FUNCTION AND CALCULATE THE
C     OPENING ANGLE, ALFA AND THE HEIGHT OF THE PASSIVE WEDGE, H
C
      IF(IPRSH.EQ.1) WRITE(6,9852)
C
      SF1MIN=100000.
      SF2MIN=100000.
C
C     CALCULATE MINIMUM RADIUS, RMIN, TO AVOID PART OF FAILURE
C     SURFACE IN HARD LAYER, AND CALCULATE OPENING ANGLE, ALFA,
C     OF CYLINDER SURFACE AND HEIGHT OF PASSIVE WEDGE, H.
C
      RMIN=XL
      IF(TH.LT.XL) RMIN=(XL**2.+TH**2.)/(2.*TH)
      DO 100 K=1,20
```

```
      R=RMIN*(1.+(K-1)*.2/19.)
      SINALF=XL/R
      ALFA=ASIN(SINALF)
      H=R*(1-COS(ALFA))
C
C     CHECK DIRECTION OF POSSIBLE FAILURE DEVELOPMENT
C     AND SET PARAMETER NCH POSITIVE FOR POSITIVE DRIVING
C     MOMENT DUE TO OVERTURNING MOMENT AND HORIZONTAL FORCE
C     AT MUDLINE AND VICE VERSA.
C
      OMCH=OM+FH*R
      NCH=-1
      IF(OMCH.GE.0.) NCH=1
C
C     CHECK IF VERTICAL FORCE ON MAT UNDER WAVE TROUGH LEADS
C     TO RELATIVE UPLIFT WITH AN UPLIFT MOMENT GREATER THAN
C     THE MOMENT DUE TO OVERTURNING MOMENT AND HORIZONTAL
C     FORCE ON MAT. IF YES SET NUPCH=-1, IF NO SET NUPCH =1
C
      OMFV=FV*XL/2.
      NUPCH=1
      IF(OMFV.LT.0.0.AND.ABS(OMFV).GT.ABS(OMCH)) NUPCH=-1
C
C     CALCULATE AVERAGE WATER PRESSURE Q AT SEA BED
C     ACTING ON PASSIVE WEDGE. ASSUMING A PASSIVE SLOPE
C     ANGLE OF 35 DEGREES.
C
      X1=XL/2.
      X2=X1+1.4142*H
      IF(NCH.GT.0) GO TO 50
      X1=-X2
      X2=XL/2.
   50 DXX=X2-X1
      Q=PAMPB*WL(IT)/2./PI/DXX*(SIN(2.*PI*(X2/WL(IT)-T/TI(IT)))-
     1SIN(2.*PI*(X1/WL(IT)-T/TI(IT))))
      G=Q*NUPCH
C
C     CALCULATE RESISTANCE ALONG CYLINDER PART
C
      OMRES=YL*R*R*SUO*(ALFA+RK*R/SUO*(ALFA-SIN(ALFA)))
C
C     ADD RESISTANCE DUE TO PASSIVE EARTH PRESSURE
C
      OMRES=OMRES+(R-H/2.)*(2.8284*SUO*H+Q*H)*YL+
     1(R-0.6667*H)*(1.4142*RK*H*H+GAMS*H*H/2.)*YL
C
C     ADD RESISTANCE DUE TO VERTICAL COMPONENT OF PASSIVE PRESSURE
C
      OMRES=OMRES+XL*SUO*H*YL*(1.+RK*H/SUO/2.)
C
C     ADD RESISTANCE DUE TO SIDE FRICTION ON PASSIVE EARTH PRESSURE
C     WEDGE
C
      OMRES=OMRES+2.*SUO*H*H*R*1.4142*(.5+RK*H/6./SUO-1./3.*H/R
     1-1./8.*RK*H*H/SUO/R-0.3536*XL/R-0.11785*RK*H*XL/SUO/R)
C
C     ADD RESISTANCE OF SIDE FRICTION ON THE SIDE OF THE SOIL MASS
C     UNDER THE FOOTING (ABOVE THE CYLINDER SURFACE)
C
      OMRES=OMRES+XL*H*(SUO+RK*H/3.)*(R+H*COS(ALFA)/3.)-R*R*(ALFA-
```

```
     1SIN(ALFA))*(SUC+RK*H/2.)*R*(1.-.6667*(1.-COS(ALFA/2.)))
C
C     CALCULATE THE DRIVING MOMENT DUE TO HORIZONTAL FORCE, VERTICAL
C     FORCE AND OVERTURNING MOMENT PLUS THE WEIGHT OF THE SOIL MASS
C     UNDER THE FOOTING ABOVE THE FAILURE SURFACE
C
      OMDRI=FV*(XL/2.+NCH*OM/FV)+NCH*FH*R+GAMS*YL*(XL*XL*(R/6.+H/3.)
     1-R**3./3.*ALFA*SIN(ALFA/2.))
C
C     CALCULATE SAFETY FACTOR
C
      SF1=OMRES/OMDRI*NCH
      ALFAD=ALFA*180./3.1415927
      IF(IPRSH.EQ.1) WRITE(6,9800) K,R,ALFAD,H,OMRES,OMDRI,SF1
C
C     CHECK IF THIS IS MINIMUM
C
      IF(ABS(SF1).GT.ABS(SF1MIN)) GO TO 90
      SF1MIN=SF1
      RSF1MI=-R
   90 CONTINUE
  100 CONTINUE
```

```
C     FAILURE CASE 2  CENTER ABOVE RIGHT EDGE
C     ****************************************
C
      IF(IPRSH.EQ.1) WRITE(6,9853)
      DO 200 K=1,20
      R=RMIN*(1.+(K-1)**2/19.)
      SINALF=XL/R
      ALFA=ASIN(SINALF)
      H=R*(1-COS(ALFA))
      ZC=R*COS(ALFA)
C
C
C     CHECK DIRECTION OF POSSIBLE FAILURE DEVELOPMENT
```

```
     1SIN(2.*PI*(X1/WL(IT)-T/TI(IT))))
      G=Q*NUPCH
C
C     CALCULATE RESISTANCE ALONG CYLINDER PART
C
      OMRES=YL*R*R*SUO*(ALFA+RK*R/SUO*SIN(ALFA)-RK*ZC/SUO*ALFA)
C
C     ADD RESISTANCE DUE TO PASSIVE EARTH PRESSURE
C
      OMRES=OMRES+(R-H/2.)*(2.8284*SUO*H+Q*H)*YL+
     1(R*COS(ALFA)+0.6667*H)*(1.4142*RK*H*H+GAMS*H*H/2.)*YL
C
C     ADD RESISTANCE DUE TO SIDE FRICTION ON PASSIVE EARTH PRESSURE
C     WEDGE
C
      OMRES=OMRES+SUO*H*H*2.*1.4142*(.5*ZC/H+.33333+RK*ZC/6./SUO
     1+1./8.*RK*H/SUO)
C
C     ADD RESISTANCE OF SIDE FRICTION ON THE SIDE OF THE SOIL MASS
C     UNDER THE FOOTING (ABOVE THE CYLINDER SURFACE)
C
      IF(R.EQ.XL) GO TO 170
      ONEPTA=(1.+TAN(ALFA/2.))/(1.-TAN(ALFA/2.))
      ONEPTA=ABS(ONEPTA)
  170 OMRES=OMRES+2.*SUO*R**3.*(.33333*ALFA+.25*RK*R/SUO*SIN(ALFA)
     1-.33333*RK*ZC/SUO*ALFA)
      IF(R.EQ.XL) GO TO 180
      OMRES=OMRES-SUO*ZC**3.*(.3333-RK*ZC/SUO/12.)*(SIN(ALFA)/
     1(COS(ALFA))**2.+ALOG(ONEPTA))
  180 CONTINUE
C     CALCULATE THE DRIVING MOMENT DUE TO HORIZONTAL FORCE, VERTICAL
C     FORCE AND OVERTURNING MOMENT PLUS THE WEIGHT OF THE SOIL MASS
C     UNDER THE FOOTING ABOVE THE FAILURE SURFACE
C
      CO=COS(ALFA)
      OMDRI=FV*(XL/2.-NCH*OM/FV)+NCH*FH*R*COS(ALFA)+GAMS/3.*YL*R*R*R
     1*(1.-3./2.*COS(ALFA)+.5*CO*CO*CO)
C
C     CALCULATE SAFETY FACTOR
C
      SF2=OMRES/OMDRI*NCH
      ALFAD=ALFA*180./3.1415927
      IF(IPRSH.EQ.1) WRITE(6,9800) K,R,ALFAD,H,OMRES,OMDRI,SF2
C
C     CHECK IF THIS IS MINIMUM
C
      IF(ABS(SF2).GT.ABS(SF2MIN)) GO TO 190
      SF2MIN=SF2
      RSF2MI=R
  190 CONTINUE
  200 CONTINUE
      IF(IPRSH.EQ.0) GO TO 220
      WRITE(6,9000) SF2MIN,RSF2MI
  220 CONTINUE
C
C
C     DEFINE MINIMUM VALUE FOR THIS TIMESTEP AND DIRECTION
C
      SFSHMI(M,L)=SF1MIN
      RSFMIN(M,L)=RSF1MI
      IF(ABS(SF2MIN).LT.ABS(SFMIN)) SFSHMI(M,L)=SF2MIN
```

```
      IF(ABS(SF2MIN).LT.ABS(SFMIN)) RSFMIN(M,L)=RSF2MI
  400 CONTINUE
C
C
  500 CONTINUE
C
      WRITE(6,9301)
      WRITE(6,9300) (M,(RSFMIN(M,L),SFSHMI(M,L),L=1,2),M=1,21)
C
      DO 530 L=1,2
  530 FSSMI(IT,L)=10000.
      DO 550 M=1,21
      DO 540 L=1,2
      IF(ABS(FSSMI(IT,L)).GT.ABS(SFSHMI(M,L))) FSSMI(IT,L)=SFSHMI(M,L)
  540 CONTINUE
  550 CONTINUE
C
  600 CONTINUE
      RETURN
 9000 FORMAT(//' MINIMUM SAFETY FOUND SFMIN=',F10.3/
     1          ' WITH A RADIUS         RSFMIN=',F10.3/)
 9090 FORMAT(1H1,'PROGRAM *SAM*    :STABILITY ANALYSIS OF MATS'/
     1          /' SUBROUTINE SHBEAR:CALCULATION OF SAFETY AGAINST'/
     2          /'                   SHALLOW FOUNDATION FAILURE'/
     3           ' WAVEHEIGHT   =',F10.1/
     4           ' WATERDEPTH   =',F10.1/
     5           ' WAVE PERIOD  =',F10.1/
     6           ' WAVE LENGTH  =',F10.1/
     7           ' WAVE STEEPN. =',F10.4//)
C 9200 FORMAT(/' WAVES IN X-DIRECTION'/)
C 9210 FORMAT(/' WAVES IN Y-DIRECTION'/)
 9300 FORMAT(I5,4F15.2)
 9301 FORMAT(//' RESULTS OF SHALLOW BEARING CAPACITY ANALYSIS'//
     1' POSITIV RADIUS MEANS CENTER OF SLIP CYLINDER ABOVE FOOTING'/
     2' NEGATIV RADIUS MEANS CENTER OF SLIP CYLINDER BELOW FOOTING'/
     3' POSITIV SAFETY FACTOR MEANS SLIP IN POSITIV DIRECTION (RIGHT)'/
     4' NEGATIV SAFETY FACTOR MEANS SLIP IN NEGATIV DIRECTION (LEFT)'///
     5'   TIME          X-DIRECTION                        Y-DIRECTION      '/
     6'   STEP       RADIUS      SAFETY           RADIUS            SAFETY  '/
     7'    NO.                   FACTOR                             FACTOR'//)
 9800 FORMAT(1X,I5,3F10.3,2F15.3,F10.3)
 9852 FORMAT(//' FAILURE TYPE 1: CENTER OF CYLINDER SURFACE UNDER
     1 LEFT OR RIGHT EDGE OF FOOTING'//)
 9853 FORMAT(//' FAILURE TYPE 2: CENTER OF CYLINDER SURFACE'/
     1' ABOVE RIGHT OR LEFT EDGE OF FOOTING'//)
      END
```

```
      SUBROUTINE DEBEAR
C
C     LAST CORRECTION BY: T.J KVALSTAD,   FEBRUARY 1983
C     DATE/VERSION       : 1.FEB.1983/1
C
C     PURPOSE OF THIS SUBROUTINE:
C          THIS SUBROUTINE CALCULATES THE SAFETY AGAINST DEEPSEATED
C          BEARING CAPACITY FAILURE DUE TO COMBINED EFFECT OF WIND,
C          WAVE AND CURRENT FORCES ON HULL AND LEGS RESPECTIVELY AND
C          INCLUDES THE WAVE PRESSURE INDUCED MOMENT OVER THE ASSUMED
C          FAILURE AREA.
C
C     METHOD/THEORY:
C          LIMIT EQUILIBRIUM THEORY FOR A CYLINDRICAL FAILURE BODY
C          ROTATING ABOUT A CENTRE LINE WITH COORDINATES XC AND ZC
C          IN THE SAME COORDINATE SYSTEM AS THE MAT HAS BEEN SPECIFIED.
C          SOFT, COHESIVE SOIL ACCORDING TO SOIL PROFILE 1 IS INCLUDED,
C          OTHER PROFILES NOT INTERESTING FOR THIS DEEPSEATED FAILURE
C          SURFACE. THE CENTER COORDINATES ARE VARIED OVER A 11 TIMES
C          11 POINTS GRID SYSTEM WITH HORIZONTAL AND VERTICAL SPACING
C          EQUAL 1/5 OF THE WIDTH OF THE MAT. THE RADIUS OF THE CYLIN
C          DRIC FAILURE SURFACE IS VARIED IN TEN EQUAL STEPS BETWEEN A
C          MINIMUM VALUE REQUIRED TO KEEP THE FAILURE SURFACE OUTSIDE
C          THE MAT AND A MAXIMUM RADIUS TOUCHING THE SURFACE OF THE
C          STIFF CLAY/SAND LAYER AT DEPTH TH BELOW SEA BED. THIS MEANS
C          THAT TOTALLY 1331 DIFFERENT FAILURE SURFACES ARE CHECKED
C          FOR EACH TIME STEP WHICH SHOULD GIVE A VERY GOOD ESTIMATE
C          OF THE MINIMUM SAFETY FACTOR FOR THIS TYPE OF FAILURE.
C
C          WAVE PRESSURE AT SEA BED INTEGRATED OVER TOP SURFACE OF
C          SOIL MASS IN FAILURE.
C
C
C
C     PARAMETER LIST:
C          R:    REAL           I:   INTEGER        /XXXXXX/: COMMON BLOCK
C
C
      COMMON/MATDAT/TOTL,TOTW,SLOTL,SLOTW,HOLEL,HOLEW,TMAT,
     1ALE,BLE,YLE
      COMMON/SETTLE/ZPEN
      COMMON/WAVDAT/WH,DW,HW(10),WL(10),WS(10),TI(10)
      COMMON/PERIOD/TMIN,TMAX,TSTEP
      COMMON/CONSTA/PI,GAMW,G,CI,CD
      COMMON/MATFOR/FVX(10,21),FX(10,21),OMY(10,21),FVY(10,21),
     1FY(10,21),OMX(10,21)
      COMMON/TOTFOR/FVXT(10,21),FXT(10,21),OMYT(10,21),FVYT(10,21),
     1FYT(10,21),OMXT(10,21)
      COMMON/SOIL/NPRO,SUO,RK,SUH,PHI,TH,RR,GAMS
      COMMON/SAFE/FSDMI(10,2),FSSMI(10,2),FSLMI(10,2)
C
      DIMENSION SFDMIN(2,21),RSFDMI(2,21),XCSFDM(2,21),ZCSFDM(2,21)
      DIMENSION SFEDG(21),REDG(21)
C
      IPRSF=0
C
C     STEP THROUGH ALL PERIODS
C
C
C
      NTI=(TMAX-TMIN)/TSTEP+1
      DO 600 IT=1,NTI
      TI(IT)=TMIN+(IT-1)*TSTEP
```

```
      WRITE(6,9090) WH,DW,TI(IT),WL(IT),WS(IT)
C
C     CALCULATE FOR X DIRECTION AND FOR Y DIRECTION
C
      DO 550 K=1,2
C
      IF(K.EQ.1) WRITE(6,9095)
      IF(K.EQ.2) WRITE(6,9096)
      SFDMIP=1000.
      DO 110 I=1,21
      SFDMIN(K,I)=1000.
  110 CONTINUE
C
C
C     STEP THROUGH ALL 21 TIMESTEPS IN A WAVE
C
      DT=TI(IT)/20.
      WRITE(6,9100)
      DO 500 M=1,21
      T=(M-1)*DT
C
C
C     CALCULATE PRESSURE AMPLITUDE AT SEA BOTTOM
C
      PAMPB=.5*GAMW*HW(IT)/COSH(2.*PI*DW/WL(IT))
C
C      WRITE(6,9900) T,PAMPB
C 9900 FORMAT(' T PAMPB ',2F10.3/)
C
C     STEP THROUGH ALL XC VALUES
C
C
      TOTB=TOTW
      IF(K.EQ.2) TOTB=TOTL
      DO 450 IXC=1,5
      DXC=TOTB/2.
      XC=-TOTB+(IXC-1)*DXC
C
C     STEP THROUGH ALL ZC VALUES
C
      DO 400 IZC=1,4
      DZC=TOTB/5
      ZC=(IZC-1)*DZC
      RMIN=SQRT(ZC*ZC+(TOTB/2. +ABS(XC))**2.)
      RMAX=ZC+TH
C      WRITE(6,9901) XC,ZC,RMIN,RMAX
C 9901 FORMAT(' XC ZC RMIN RMAX ',4F10.3)
      IF(RMAX.LT.RMIN) GO TO 400
C
C
C     STEP THROUGH ALL R VALUES
C
      DR=(RMAX-RMIN)/5.
      DO 350 IR=1,5
      R=RMIN+(IR-1)*DR
C
C     CALCULATE VERTICAL FORCE AND OVERTURNING MOMENT DUE TO WAVE
C     PRESSURE ON SEA BED FOR WAVES COMING IN X DIRECTION AND THE
C     CORRESPONDING ECCENTRICITY RELATIV TO THE CENTER OF THE SLIP
C     CIRCLE.
C
```

```
      YL=TOTL
      IF(K.EQ.2) YL=TOTW
      DXL=SQRT(R*R-ZC*ZC)
      SURFL=2.*DXL
      IF(SURFL.GE.WL(IT)) GO TO 350
      XL1=XC-DXL
      XL2=XC+DXL
      TX1=2.*PI*(XL1/WL(IT)-T/TI(IT))
      TX2=2.*PI*(XL2/WL(IT)-T/TI(IT))
C      WRITE(6,9902) XL1,XL2,TX1,TX2
C 9902 FORMAT(' XL1  XL2  TX1  TX2 ',4F10.3)
      FVWP=PAMPB*WL(IT)*YL/2./PI*(SIN(TX2)-SIN(TX1))
      OMWP=PAMPB*YL*(WL(IT)**2./4./PI/PI*(COS(TX2)-COS(TX1))+XL2*WL(IT)
     1/2./PI*SIN(TX2)-XL1*WL(IT)/2./PI*SIN(TX1))
C
C      WRITE(6,9903) FVWP,OMWP
C 9903 FORMAT(' FVWP OMWP ',2F10.3)
C
C     CALCULATE TOTAL EXTERNAL FORCES ACTING ON THE SEABED SURFACE OF
C     THE SLIPPING CYLINDRICAL BODY AT COORDINATES X=0 AND Z=0 I.E.
C     ORIGO OF REFERENCE COORDINATE SYSTEM FOR MAT. THE VERTICAL WAVE
C     PRESSURE FORCE AND THE CORRESPONDING OVERTURNING MOMENT ACTING
C     ON THE MAT STRUCTURE ARE SUBTRACTED AND REPLACED BY THE SAME
C     FORCES FOR THE TOTAL SEA BED SURFACE OF THE SLIP BODY
C
      IF(K.EQ.2) GO TO 310
      TOTMO=OMYT(IT,M)-OMY(IT,M)+OMWP
      TOTFV=FVXT(IT,M)-FVX(IT,M)+FVWP
      TOTFH=FXT(IT,M)
      GO TO 320
  310 CONTINUE
      TOTMO=OMXT(IT,M)-OMX(IT,M)+OMWP
      TOTFV=FVYT(IT,M)-FVY(IT,M)+FVWP
      TOTFH=FYT(IT,M)
  320 CONTINUE
C
C     TRANSFORM ALL FORCES AND MOMENTS RELATIVE TO CENTER OF SLIP CIRCLE
C
      ECC=TOTMO/TOTFV
      OVMCMC=TOTFV*(ECC-XC)-TOTFH*ZC
C
C     CALCULATE RESISTING MOMENT ON THE TWO END SURFACES OF THE SLIP
C     BODY
C
      COSALF=ZC/R
      ALFA=ACOS(COSALF)
      OMRES=SUO*R**3.*(.6667*ALFA+.5*RK*R/SUO*SIN(ALFA)-.6667*RK*ZC/SUO
     1*ALFA)
      IF(ZC.EQ.0.) GO TO 330
      TANAH=(1+TAN(ALFA/2.))/(1.-TAN(ALFA/2.))
      OMRES=OMRES+SUO*ZC**3.*(.33333-RK*ZC/12./SUO)*(LOG(TANAH)+SIN(ALFA
     1)/COS(ALFA)/COS(ALFA))
  330 CONTINUE
      OMRES=2.*OMRES
C
      OMRES1=OMRES
C
C     CALCULATE RESISTING MOMENT ON CYLINDERSURFACE
C
      OMRES2=2.*YL*R*R*SUO*((1.-RK*ZC/SUO)*ALFA+R*RK/SUO*SIN(ALFA))
```

```
      OMRES=OMRES1+OMRES2
C
C      WRITE(6,9905) OMRES1,OMRES2
C 9905 FORMAT(' OMRES1  OMRES2 ',2F12.2)
C
C     CALCULATE SAFETY FACTOR
C
      SFD=ABS(OMRES/CVMOMC)
C
C     PRINT DATA FOR EACH INVESTIGATED SLIP SURFACE IF REQUIRED
      IF(IPRSF.EQ.0) GO TO 335
      WRITE(6,9950) XC,ZC,R,FVWP,OMWP,OVMOMC,OMRES,SFD
 9950 FORMAT(3F8.2,4E11.3,F10.2)
  335 CONTINUE
C
C     CHECK IF MINIMUM FOR EACH TIME STEP
C
      IF(SFD.LT.SFDMIP) SFDMIP=SFD
      IF(SFD.GE.SFDMIN(K,M)) GO TO 350
      SFDMIN(K,M)=SFD
      RSFDMI(K,M)=R
      XCSFDM(K,M)=XC
      ZCSFDM(K,M)=ZC
  350 CONTINUE
  400 CONTINUE
  450 CONTINUE
C
C     PRINT RESULTS FOR EACH TIME STEP
C
      WRITE(6,9110) M,XCSFDM(K,M),ZCSFDM(K,M),RSFDMI(K,M),SFDMIN(K,M)
C
C
  500 CONTINUE
C
C     DEFINE MINIMUM SAFETY AGAINST DEEPSEATED FAILURE FSDMI
C     FOR EACH DIRECTION AND PERIOD
C
      FSDMI(IT,K)=1000.
      DO 540 M=1,21
      IF(SFDMIN(K,M).LT.FSDMI(IT,K)) FSDMI(IT,K)=SFDMIN(K,M)
  540 CONTINUE
C
      WRITE(6,9120) FSDMI(IT,K)
C
  550 CONTINUE
C
  600 CONTINUE
C
C     CHECK IF MINIMUM FOR EACH PERIOD
C
C
C
C
C
C
C
      RETURN
 9090 FORMAT(1H1,'PROGRAM *SAM*     : STABILITY ANALYSIS OF MATS'/
     1          /' SUBROUTINE DEBEAR: CALCULATION OF SAFETY AGAINST'
     2          /'                    DEEPSEATED FOUNDATION FAILURE'/
     3           ' WAVEHEIGHT    =',F10.1/
```

```
     4           ' WATERDEPTH   =',F10.1/
     5           ' WAVE PERIOD  =',F10.1/
     6           ' WAVE LENGTH  =',F10.1/
     7           ' WAVE STEEPN. =',F10.4//)
 9095 FORMAT(' WAVES IN X-DIRECTION '/)
 9096 FORMAT(' WAVES IN Y-DIRECTION '/)
 9100 FORMAT(/' RESULTS OF STABILITY ANALYSIS FOR DEEP SEATED FAILURE SU
     1RFACE'/' INCLUDING WAVE PRESSURE EFFECTS OVER TOTAL MAT AREA INCLU
     2DIN HOLE AND SLOT AREA'//
     3'  TIME   CENTER COORDINATES    RADIUS        SAFETY'/
     4'  STEP      XC        YC       OF SLIP       FACTOR'/
     5'   NC       --        --       SURFACE         SFD '/)
 9110 FORMAT(I5,4F12.2)
 9120 FORMAT(//' MINIMUM SAFETY AGAINST DEEPSEATED FAILURE'/
     1          ' FOR THIS WAVE PERIOD AND WAVE DIRECTION = ',F5.2/)
      END
```

ADDENDUM TO PROGRAM LISTINGS OF SAIL AND SAM

Some statement labels are unfortunately illegible due to the punched holes in these copies of the program listings. Missing label numbers are listed below for each of the two programs with reference made to page number and line number on page.

SAIL

Page No.	Line No.	Missing Label No.
3	57	1025
11	39	1040
13	1	1120
21	1	1110
27	39	1380
29	1	1488
29	38	1540
33	20	1580
35	39	1000
37	19	7100
37	20	7120
37	57	8040
41	38	210
53	38	520
55	1	1900
63	58	6600
71	58	280
75	58	1600

SAM

Page No.	Line No.	Missing Label No.
9	4	9010
47	5	9095

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TECHNICAL REPORT

Date	December 31, 1983
VERITAS Report No.	83-0905
Subject Group	G3, P3
Department	23
Project No.	233103
Title of Report	FOUNDATION STABILITY OF JACKUP PLATFORMS APPENDICES TO PP3: PARAMETRIC STUDY
Approved by	
Client/Sponsor of project	Several Companies, see Preface
Client/Sponsor ref.	
Work carried out by	Lars A. Kristiansen Tore J. Kvalstad Knut O. Ronold
Reporters sign.	

Summary

4 Indexing terms

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Date of last rev.	Rev. No.	Number of pages
Oktober 23, 1984	1	66

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PROJECT: FOUNDATION STABILITY OF JACK-UP PLATFORMS

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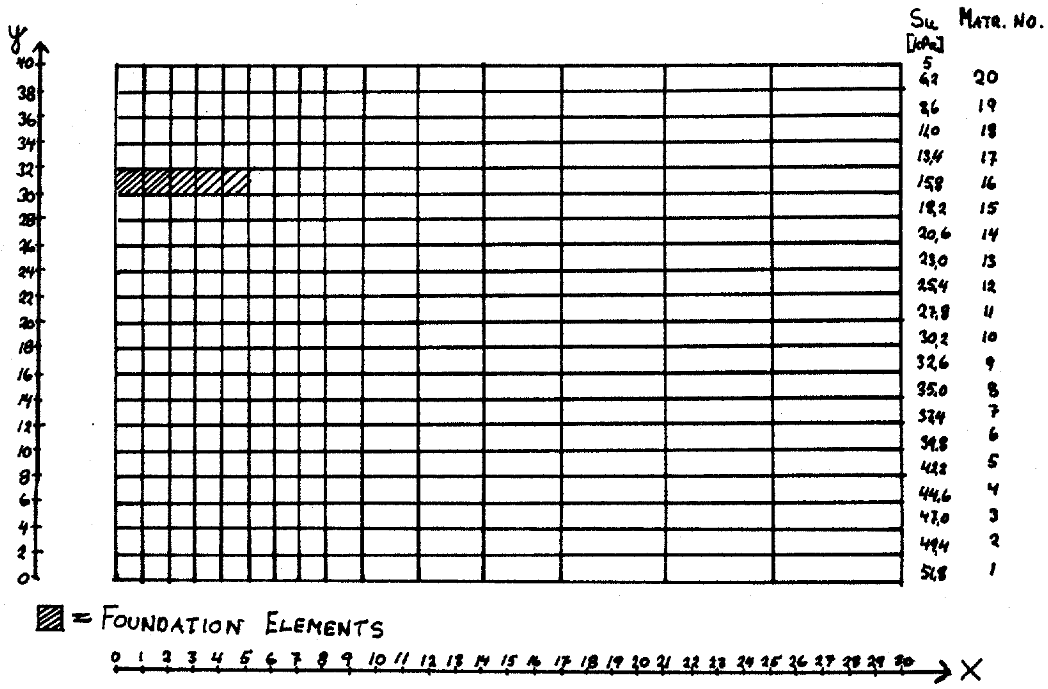
Effect of moment capacity on total vertical load.

APPENDIX A

Comparison between Brinch-Hansen's bearing capacity formula and solution by the finite element program AXIPLN for a deeply penetrated footing.

Element mesh:



Input data:

Ultimate load = Bearing capacity as calculated by Brinch-Hansen's formula.

$s_{uo} = 5$ kPa

k = 1.2 kPa/m

Load steps for AXIPLN analysis:

0) Initial stress
1) Excavation and 50% of ultimate load
2) 75% of ultimate load
3) 90% of ultimate load
4) 100% of ultimate load
5) 110% of ultimate load
6) 120% of ultimate load
7) 130% of ultimate load

The below sketches show how the failure develops from load step 3 to load step 6.

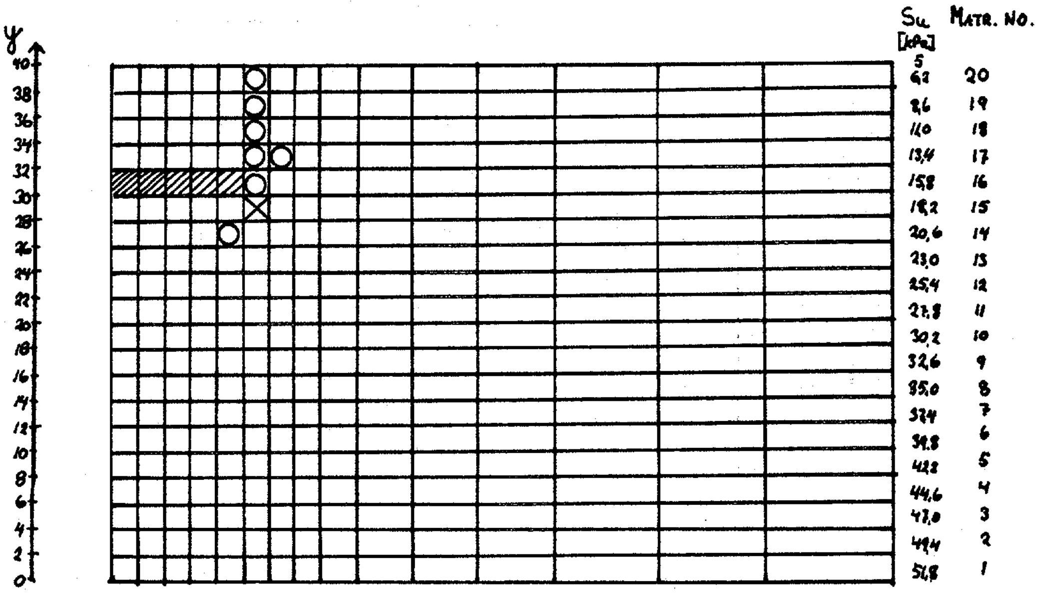
Notations X = Element that has reached failure at this load step.

0 = Element in failure from previous load step.

For load step 4 and 5

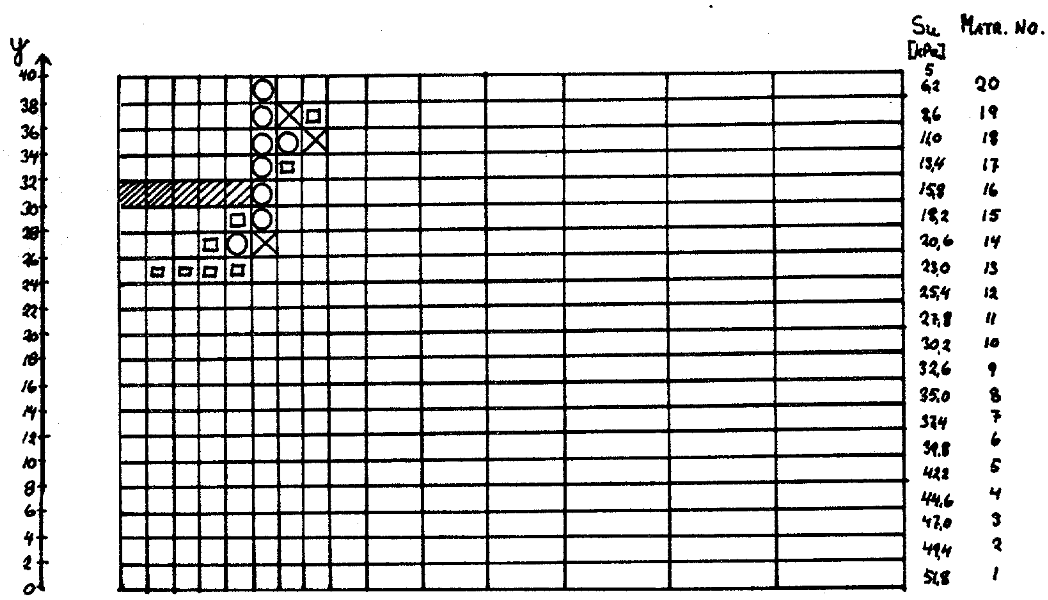
☐ = Element with a stress level between 0.90 and 0.99 (failure corresponds to a stress level of 1.0)

Load step 3:



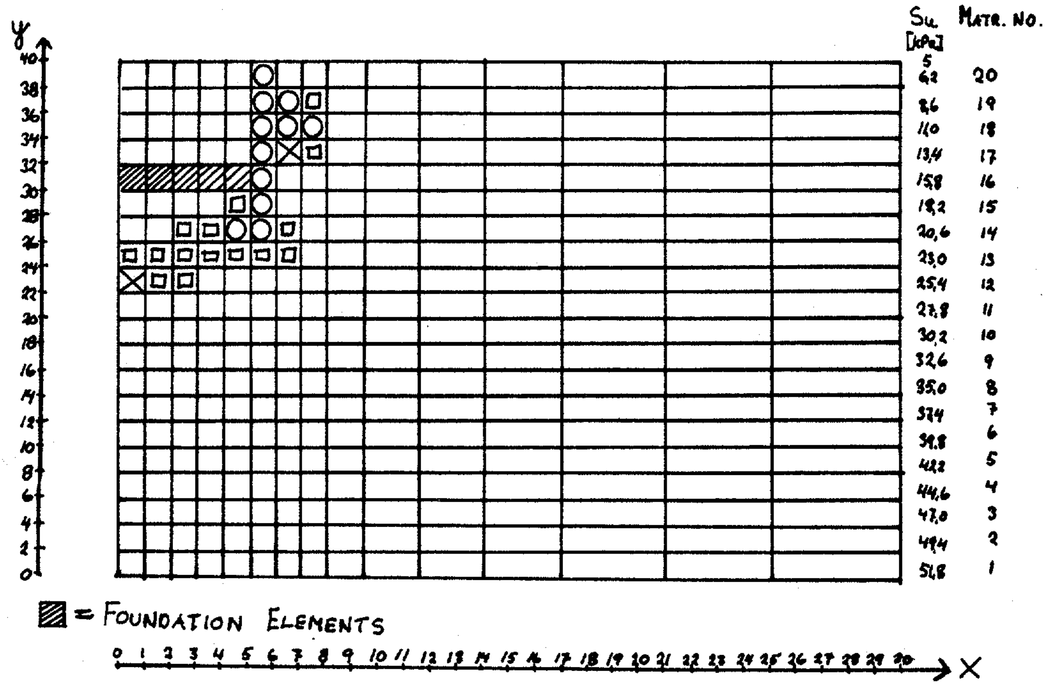
Failure has developed under edge of spudcan.

Load step 4:



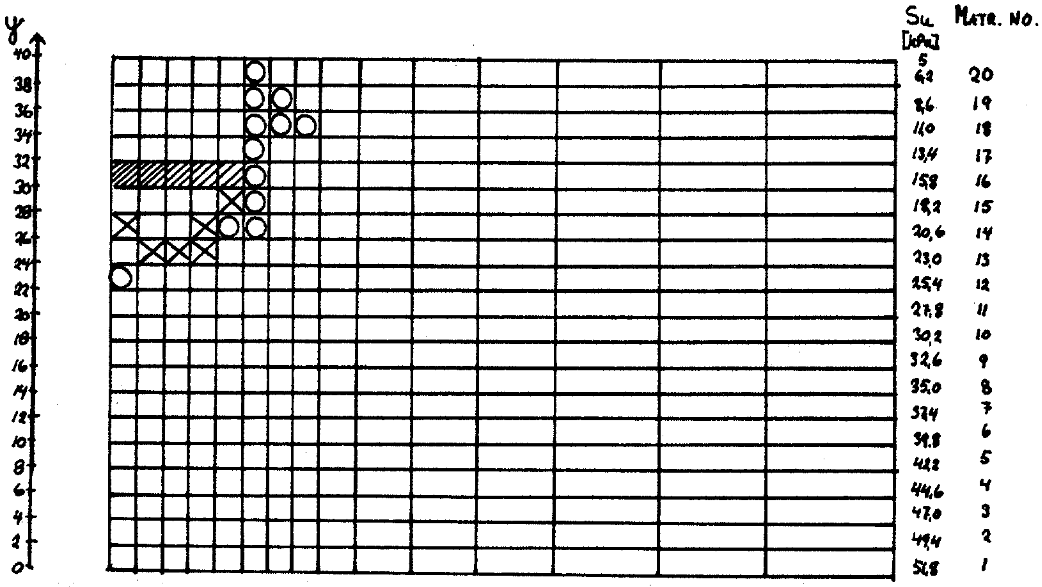
The failure propagates under the edge.

Load step 5:



Elements along the theoretical slip surface are heavily stressed. All these elements have a stress level of more than 0.90. Some elements, especially near the edge, have a stress level exceeding 1.0. This means that the program has not succeeded to redistribute the occurring forces to other elements, when one element has reached failure. This load step has therefore been defined as failure.

Load step 6:



Failure along slip surface, local failure below footing and under edge.

APPENDIX B

Description of diagram for estimation of penetration depth by means of bearing capacity formula.

Brinch-Hansen's bearing capacity formula for vertical loading, cohesive soil, states

$$q_u = s_u \cdot N_c \cdot (1 + s_{ca} + d_{ca})$$

When applied for a spudcan, overburden pressure and side friction must be taken into account in addition.

Overburden pressure $= \gamma' \cdot z$

Side friction $= 2\pi \cdot r \cdot h \cdot (s_{uo} + k(z - \frac{h}{c}))$

The total bearing capacity hence becomes

$$q_u = [s_{uo} + k(z + \frac{r}{2})] \cdot 5.14 \cdot (1.2 + 0.3 arctg \frac{1}{\sqrt{\pi}}) + \gamma' \cdot z + 2\pi rh(s_{uo} + k(z - \frac{h}{2}))$$

which gives the following normalized equation

$$\frac{q_u}{s_{uo}} = [1 + \frac{k \cdot r}{s_{uo}} \cdot (\frac{z}{r} + \frac{1}{2})] \cdot 5.14 \cdot (1.2 + 0.3 arctg(\frac{1}{\sqrt{\pi}}) \cdot \frac{z}{r}) + \frac{\gamma' \cdot r}{s_{uo}} \cdot (\frac{z}{r}) + \frac{2h}{r}(1 + \frac{k \cdot r}{s_{uo}}((\frac{z}{r}) - \frac{1}{2} \cdot (\frac{h}{r})))$$

where:

q_u = Ultimate bearing pressure.
s_{uo} = Undrained shear strength at mudline.
z = Penetration depth for base.
π = Pi (3.14).
γ' = Submerged unit weight of soil.
r = Radius of spudcan base.
k = Rate of increase in undrained shear strength with depth.

For legs with spudcans:
h = Height of spudcan side which is exposed to soil.

For legs without spudcans:
h = Height of penetrated leg side.

The derived expression for q_u/s_{uo} may be split into two parts, Q_1 and Q_2:

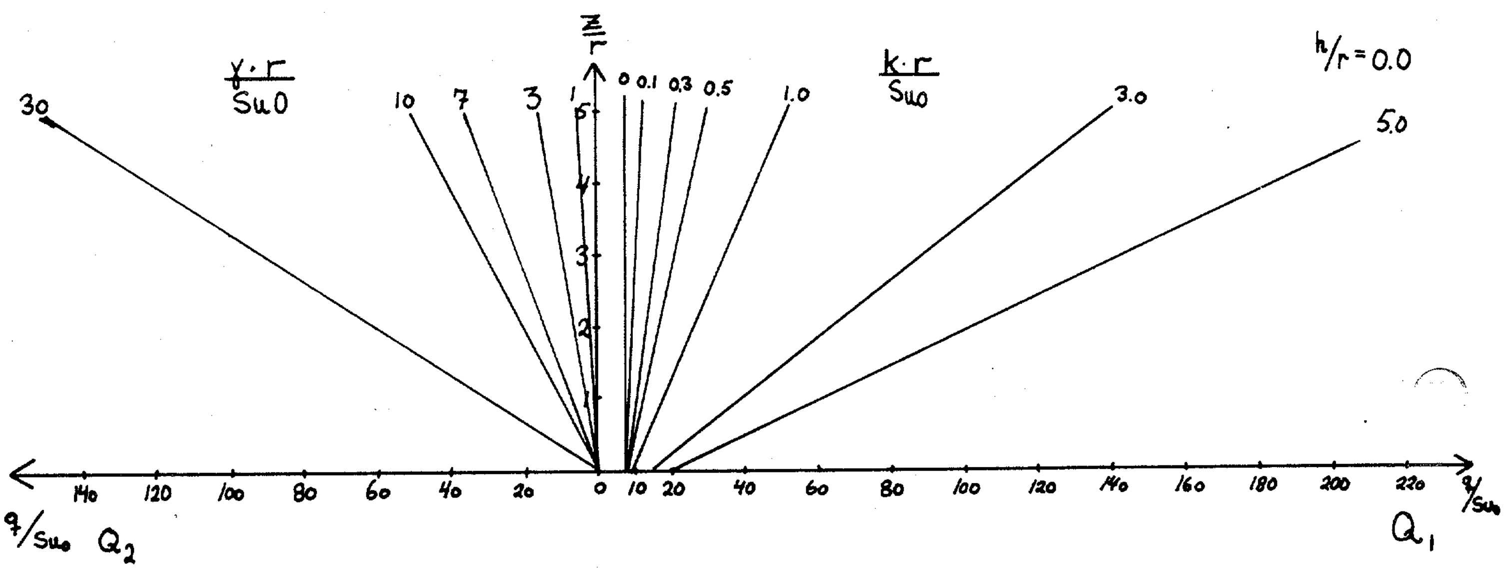
$$Q_1 = [1+\frac{k\cdot r}{s_{uo}}\cdot(\frac{z}{r}+\frac{1}{2})]\cdot 5.14\cdot(1.2+0.3arctg(\frac{1}{\sqrt{\pi}})\cdot\frac{z}{r})+\frac{2h}{r}(1+\frac{k\cdot r}{s_{uo}}((\frac{z}{r})-\frac{1}{2}(\frac{h}{r})))$$

$$Q_2 = \frac{\gamma'\cdot r}{s_{uo}}\cdot(\frac{z}{r})$$

Q_1 represents the bearing capacity of the exposed base and the side friction (without any embedment effect).

Q_2 represents the bearing capacity due to the overburden pressure (i.e. the embedment effect).

Bearing capacity for legs with spudcan.

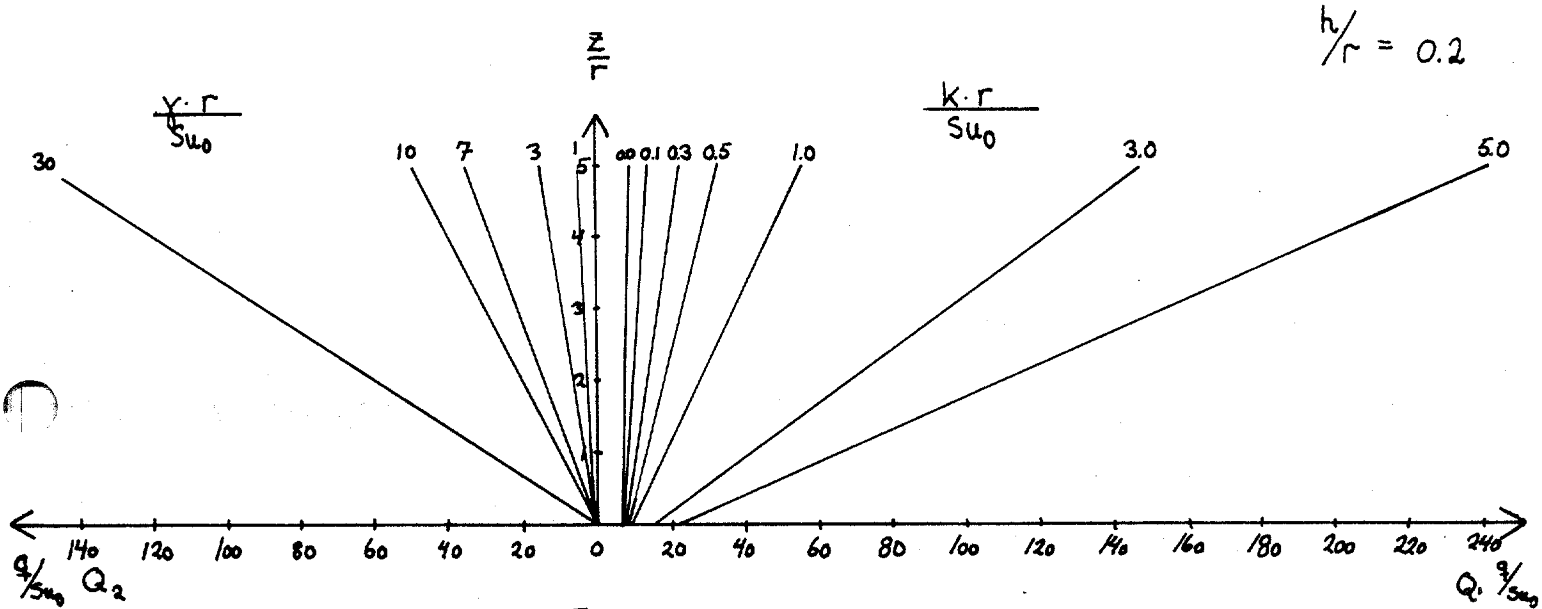


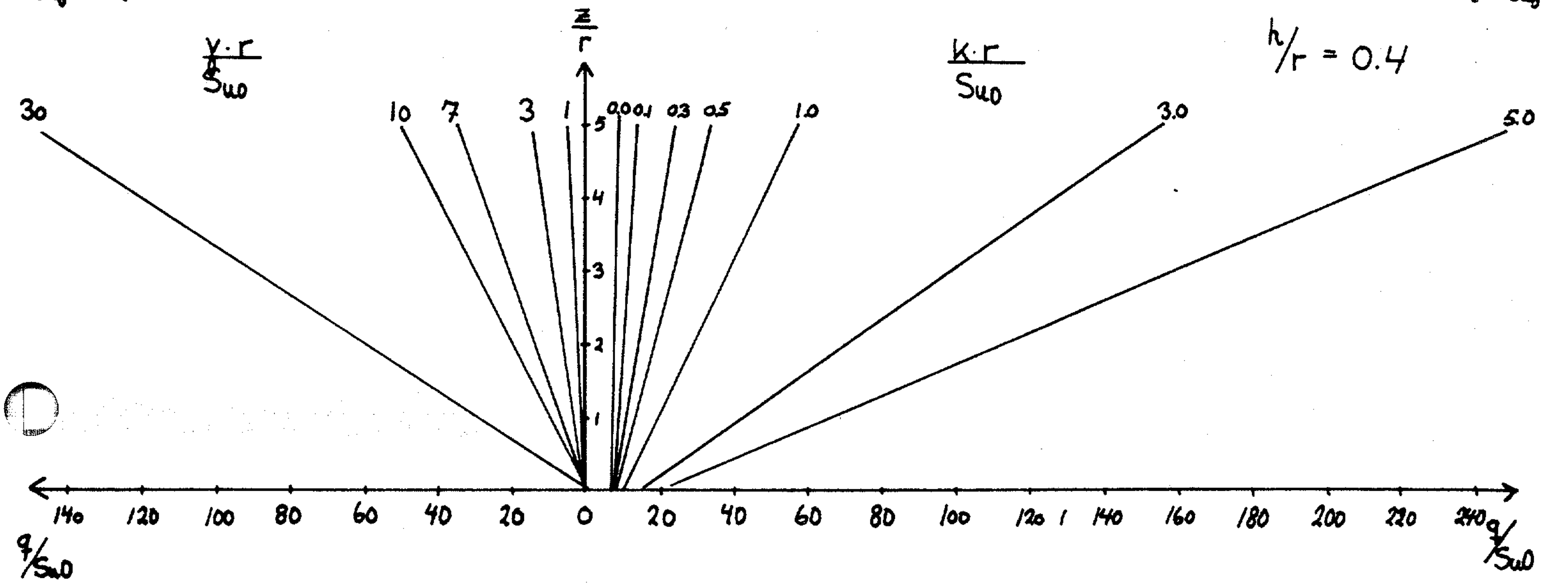
The left side of the diagram (Q_2), representing the overburden pressure, is the same for all h/r ratios.

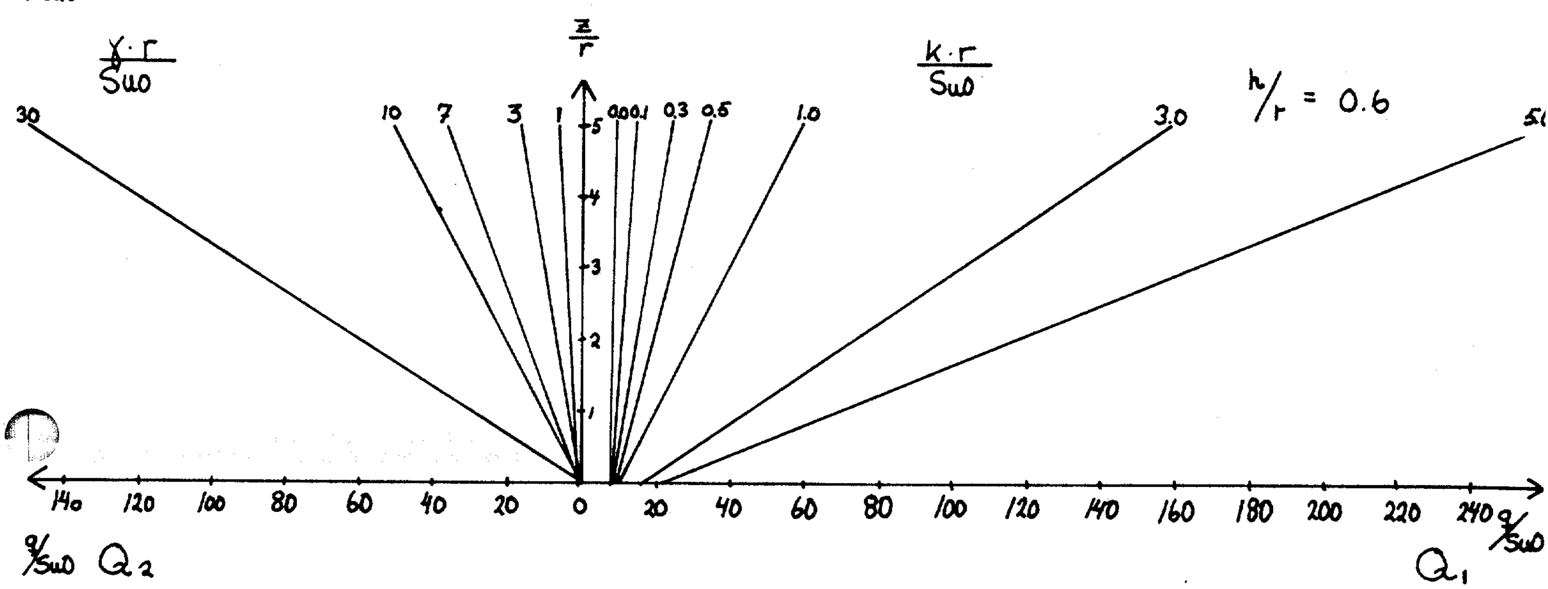
The right side of the diagram (Q_1), representing the bearing capacity of an ordinary slip surface and the side friction, differ from one h/r ratio to another. This is because the resulting side friction varies with the height of the vertical spudcan wall.

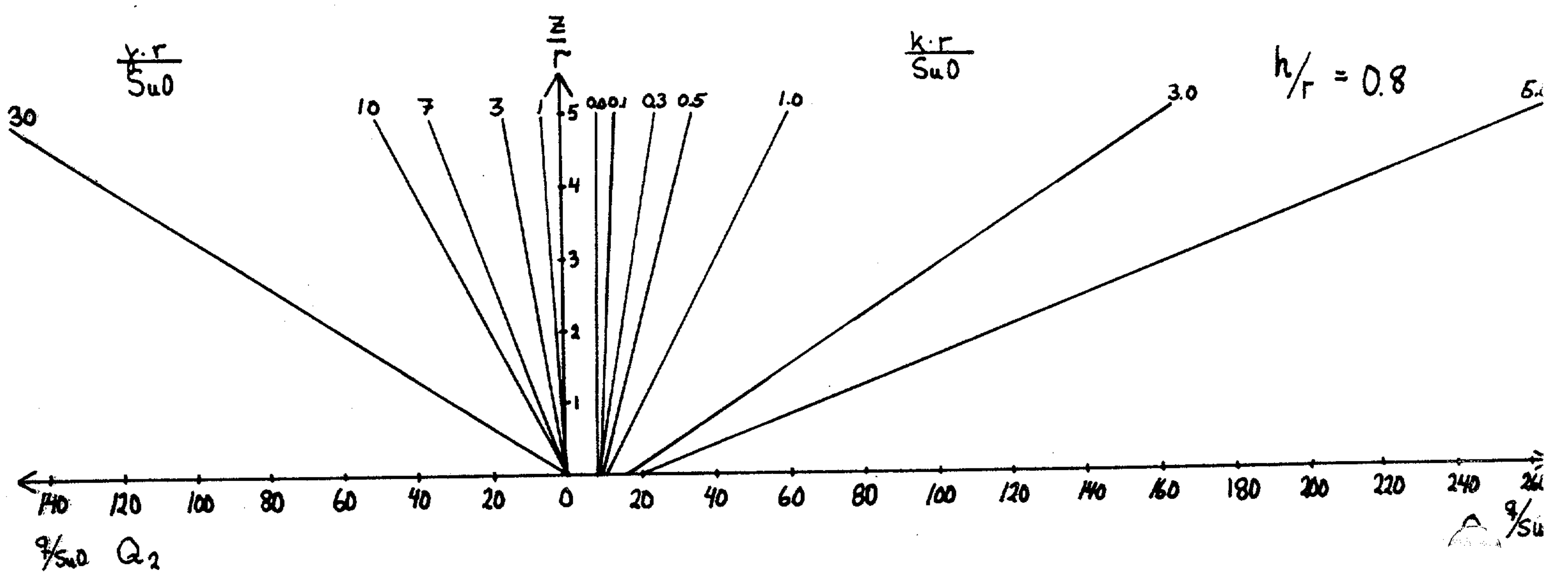
The use of the diagram is explained in Section 3.4.4.

Below diagrams are reproduced for h/r ratios 0.2, 0.4, 0.6, 0.8 and 1.0 for legs with spudcans.

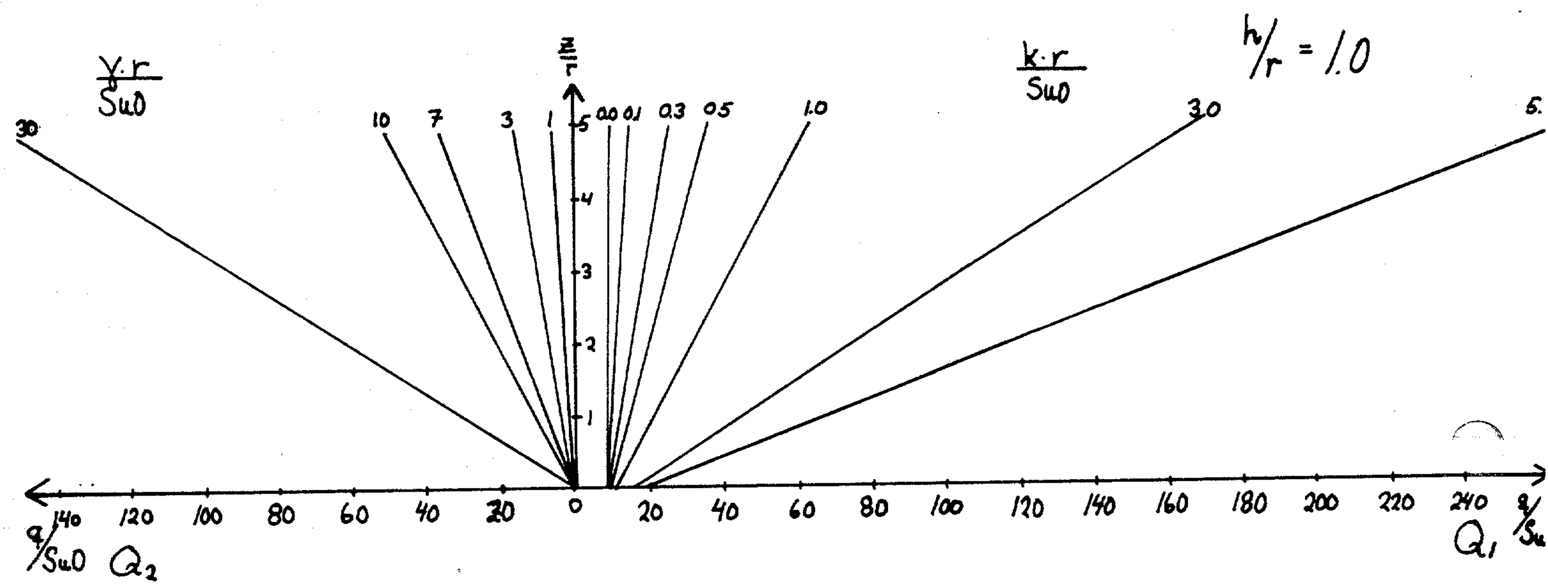






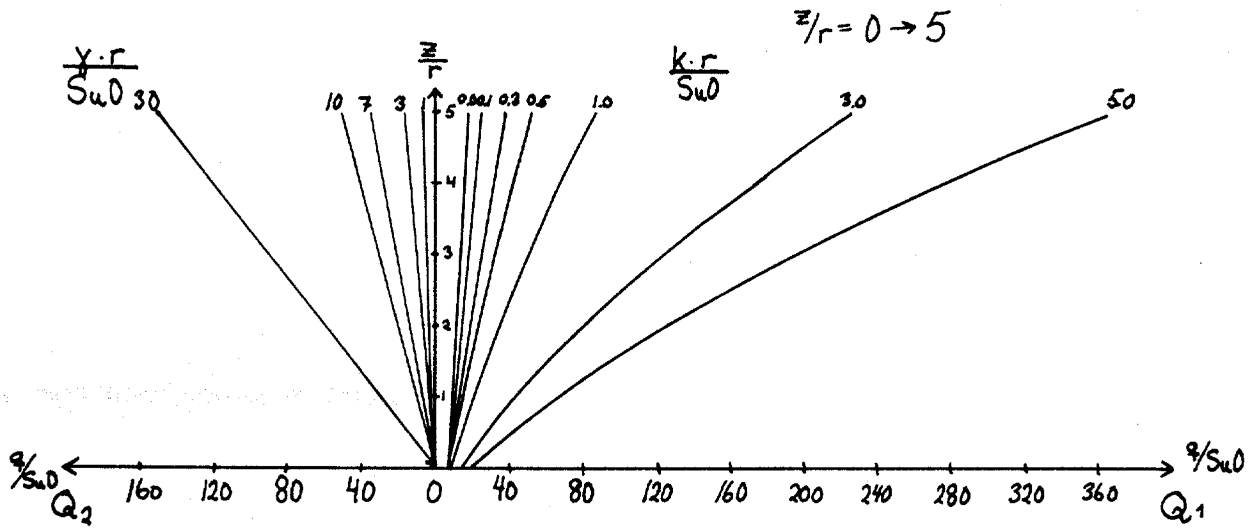


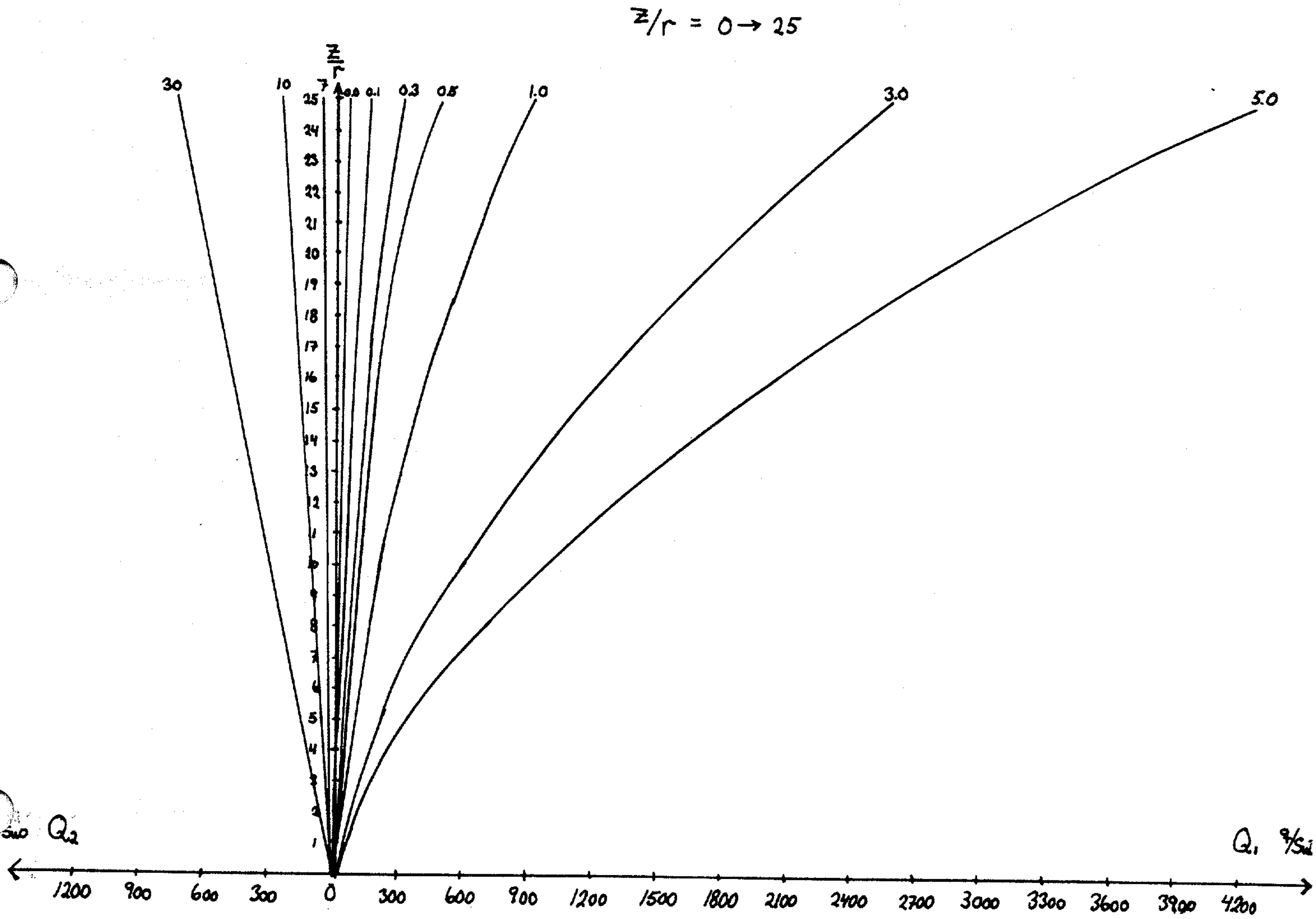
$\frac{\gamma \cdot r}{Su0}$
$\frac{z}{r}$
$\frac{k \cdot r}{Su0}$
h/r = 0.8
30
10
7
3
1
0.0
0.1
0.3
0.5
1.0
3.0
5.
1
2
3
4
5
140
120
100
80
60
40
20
0
20
40
60
80
100
120
140
160
180
200
220
240
260
q/Su0 Q2
q/Su



$\frac{\gamma \cdot r}{Su0}$
$\frac{z}{r}$
$\frac{k \cdot r}{Su0}$
h/r = 1.0
30
10
7
3
1
0.0
0.1
0.3
0.5
1.0
3.0
5.
1
2
3
4
5
140
120
100
80
60
40
20
0
20
40
60
80
100
120
140
160
180
200
220
240
q/Su0 Q2
Q1
q/Su

Below diagrams are reproduced for legs without spudcans with z/r ratios 0 → 5 and 0 → 25.



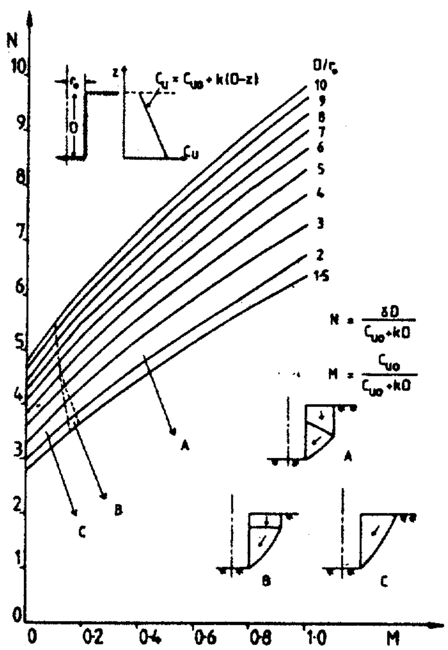


APPENDIX C

Description of diagram for estimation of stability of foundation hole.

The diagram is based on the paper "Stability of Axisymmetric Excavations in Clays" by Britto and Kusakabe, March 1982.

The below figure shows failure curves for various soil data and geometry properties, and corresponding failure shapes are indicated.



Plot of N, against M for Different D/r_o Ratios

The failure mode depends on the dimensionless parameters N and M and on the $(z-h)/r$ ratio.

Modified for a spudcan, the N and M ratios may be expressed as follows

$$N = \frac{\gamma'\cdot(z-h)}{s_{uo}+k\cdot(z-h)} \qquad M = \frac{s_{uo}}{s_{uo}+k\cdot(z-h)}$$

where:

r = Radius of spudcan base.

γ' = Submerged unit weight.

z = Depth from seabed to spudcan base.

h = Height of vertical wall on spudcan.

s_{uo} = Undrained shear strength at mudline.

k = Rate of increase in undrained shear strength with depth.

The most important parameter for the failure shape is M. For soft clays M will range from 0.15 to 1.0.

This means that failure mode A in the above figure is the most likely failure mode. The stability diagram, which has been developed and is reproduced below, is however also valid for failure modes B and C.

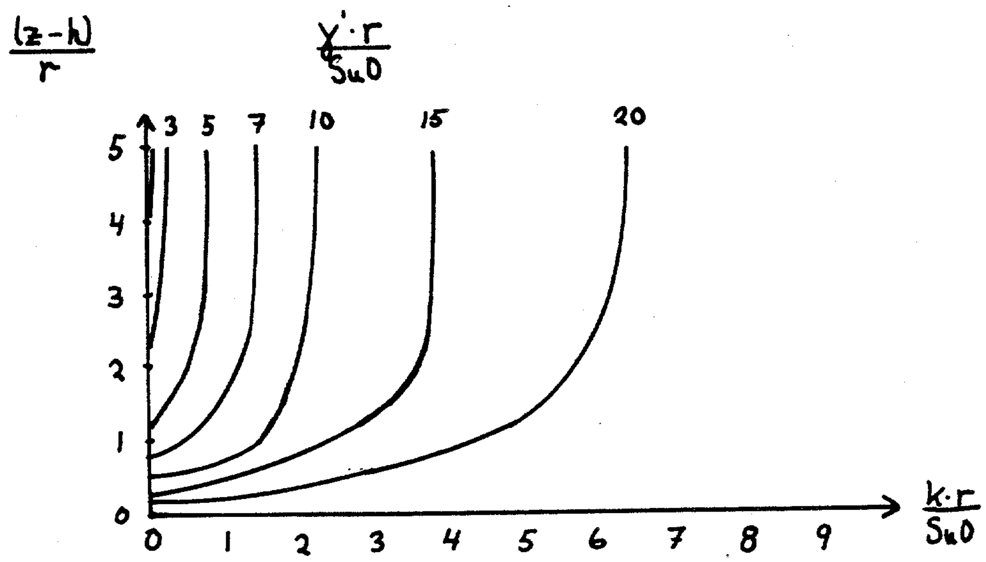


Table for $\frac{k \cdot r}{S_{u0}} = 0$.

Input $\frac{\gamma' \cdot r}{S_{u0}}$	1	2	3	5	7	10	15	20
Output $\frac{z-h}{r}$	10.3	4	2.3	1.2	0.8	0.5	0.3	0.2

Input to diagram:

$$\frac{\gamma' \cdot r}{s_{uo}}, \quad \frac{k \cdot r}{s_{uo}}$$

Output from diagram:

$\frac{(z-h)}{r}$ = critical depth, i.e. maximum penetration depth for a stable foundation hole above a topical spudcan, normalized with respect to the spudcan base radius.

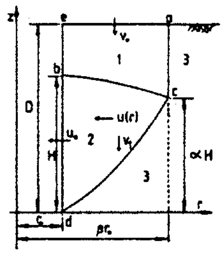
If the critical depth from the stability diagram equals or exceeds the penetration depth for the base minus the height of the vertical spudcan wall, the foundation hole is likely to fail.

For calculation of additional load due to failure see Appendix D.

APPENDIX D

Method for calculation additional load due to failure of foundation hole.

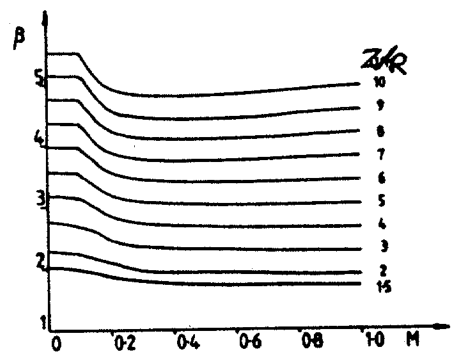
The most common failure mode for an open foundation hole above a spudcan in soft clay is indicated on the below sketch, referenced as failure mode A in Appendix C.



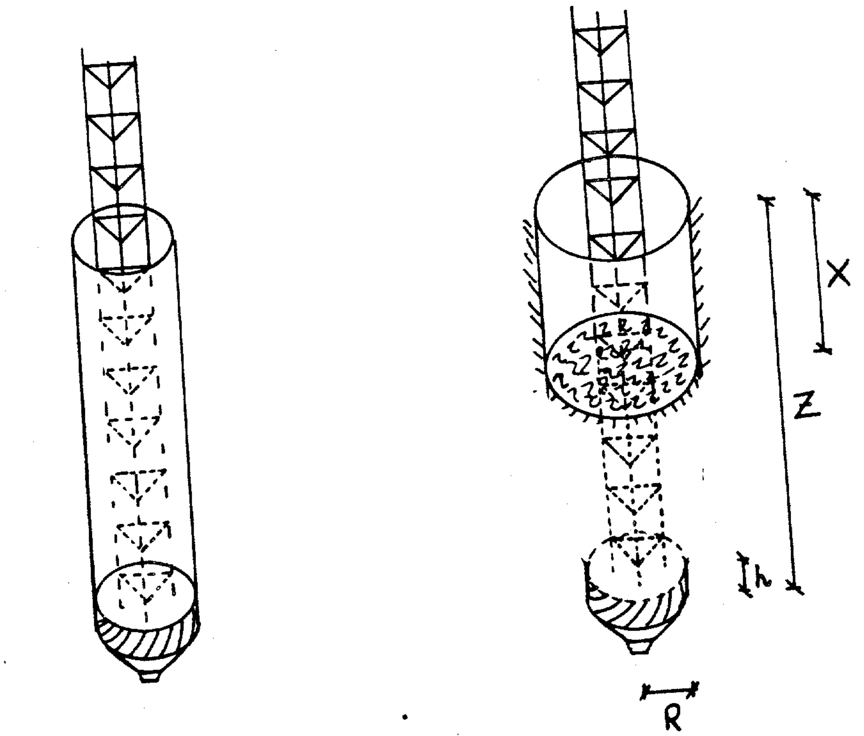
According to this sketch, the parameter β expresses the extent of the soil zone adjacent to the foundation hole, undergoing plastic deformations when the foundation hole fails. The β value may be expressed by a simple equation, $\beta=(0.5\cdot\frac{(z-h)}{r}+1)$. The below figure shows the variation in β with z/r and the M parameter, where

$$M=\frac{s_{uo}}{s_{uo}+k\cdot(z+h)}$$

For further description of notations, reference is made to Appendix C.



With reference to the below sketch, the additional load on a footing due to wall failure may be calculated as follows:



Initial situation.

After back-filling.

Initial situation: Open hole above a deeply penetrated spudcan.
After back-filling: The hole above the spudcan is closed with back-filled soil after wall failure.

where:

X = Distance from mudline down to surface of back-filled material after wall failure.

Z = Distance from mudline to spudcan base.

R = Radius of spudcan base.

h = Height of vertical spudcan wall.

V_s = Volume of back-filled slice.

V_F = Volume of soil resting on footing after wall failure.

$V_s = \pi\cdot(\beta R)^2\cdot X - \pi R^2\cdot X$

$V_F = \pi R^2\cdot(z - h - x)$

V_s equals V_F at failure which hence gives

$$\pi\cdot(\beta\cdot R)^2\cdot X - \pi R^2\cdot X = \quad \pi R^2\cdot(z - h - x)$$

$$\beta^2\cdot X - X = (z - h - x)$$

$$X = \frac{(z-h)}{\beta^2}$$

The additional weight ΔF_v on the footing due to wall failure hence becomes:

$$\Delta F_v \;=\; \gamma'\cdot V_F \;=\; \gamma'\cdot\pi R^2\cdot(z - h - \frac{z-h}{\beta^2})$$

$$\Delta F_v \;=\; \gamma'\pi R^2(z-h)(1-\beta^{-2})$$

$$\Delta F_v \;=\; \gamma'\pi R^2(z-h)(1-(0.5\frac{(z-h)}{R}+1)^{-2})$$

APPENDIX E

Modified Brinch Hansen's bearing capacity formula for estimation of punch-through capacity.

The following notations are applied:

F_v	=	Ultimate vertical load.
$q_{top,ult}$	=	Ultimate bearing capacity at foundation base in top layer.
$q_{deep,ult}$	=	Ultimate bearing capacity on top of deep layer.
s_{u0}	=	Undrained shear strength in top of soft deep layer.
α	=	Pressure influence angle. Set equal to 26.6°.
h	=	Penetrated side height of vertical spud-can wall.
F_{ac}	=	Factor describing the ratio between the exposed foundation area of spud-can and the foundation area in deep layer for the actual α.
H	=	Height of hard top layer.
N_c	=	Bearing capacity factor = 5.14
γ'	=	Submerged unit weight of soil.
r	=	Radius of spudcan base.
ρ	=	Friction angle for sand.
k	=	Rate of increase in undrained shear strength with depth.
s_{uh}	=	Undrained shear strength of hard layer, constant with depth.
$s_{u,actual}$	=	Average undrained shear strength for deep layer, calculated at a depth below the hard layer equal to half the diameter of the equivalent "foundation area" on top of the deep layer.
π	=	3.14

Stiff clay overlaying soft clay:

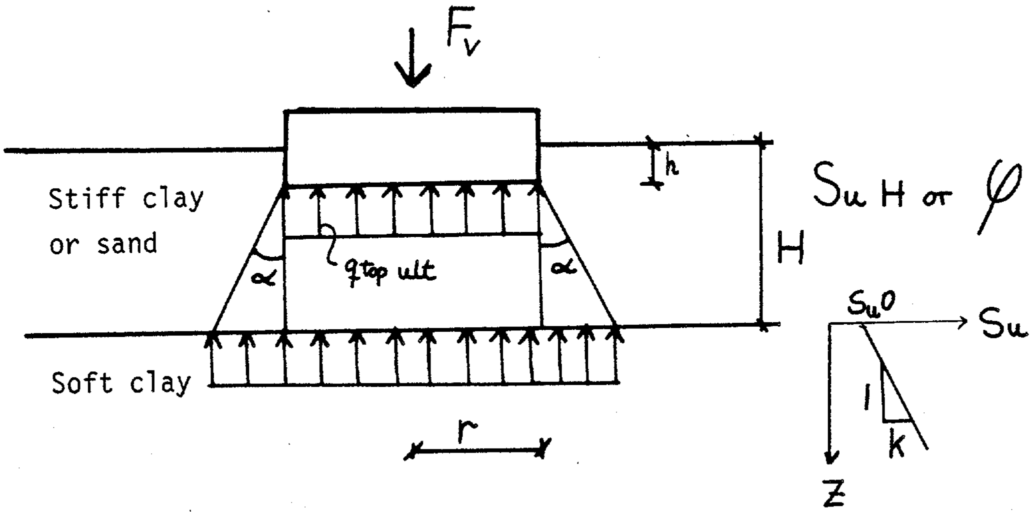


Figure E 1.: Idealized model for punch-through failure.

$$q_{top,ult} = F_{ac} \cdot q_{deep,ult} + \gamma' \cdot H + 2 \cdot \pi \cdot r \cdot h \cdot \frac{s_{uh}}{\pi r^2}$$

$$q_{top,ult} = \frac{\pi(r + (H-h)tg\alpha)^2}{\pi \cdot r^2} \cdot q_{deep,ult} + \gamma' \cdot H + \frac{2h \cdot s_{uh}}{r}$$

$$tg\alpha = \frac{x}{H-h} \rightarrow X = \frac{1}{2}(H-h) \text{ for } \alpha = 26.6^o$$

$$s_{u,actual} = (s_{uo} + \frac{1}{2} \cdot k(r + (H-h) \cdot tg\alpha))$$

$$q_{top,ult} = \frac{(r + 0.5(H-h))^2}{r^2}(s_{uo} + k(\frac{r}{2} + \frac{r}{4}(H-h))) \cdot N_c(1.2 + 0.3 arctg\frac{H}{r\sqrt{\pi}}) + \gamma' \cdot H + \frac{2h}{r} s_{uh}$$

For sand overlaying soft clay:

Contribution from side friction $= \dfrac{2\cdot\pi\cdot r\cdot h}{\pi\cdot r^2}\cdot y = \dfrac{h^2}{r}\cdot tg\varphi\cdot\gamma'$

$$y = RR\cdot tg\varphi\cdot K_p\cdot\gamma'\cdot\frac{h}{2} \approx 0.8\cdot tg\varphi\cdot 1.2\cdot\gamma'\cdot\frac{h}{2} = 0.48\,tg\varphi\gamma'\cdot h$$

where:

RR = Roughness ratio.

K_p = Passive earth pressure coefficient.

Formula for sand overlaying soft clay:

$$q_{top,ult} = \frac{(r+0.5(H-h))^2}{r^2}(s_{uo}+k(\frac{r}{2}+\frac{k}{4}(H-h)))N_c(1.2+0.3\,arctg\frac{H}{r\sqrt{\pi}})+\gamma'\cdot H+\frac{h^2}{r}\gamma'\cdot tg\varphi$$

APPENDIX F

Description of finite element analysis for layered soils.

Both thin sand layer overlaying soft clay and stiff clay overlaying soft clay have been studied.

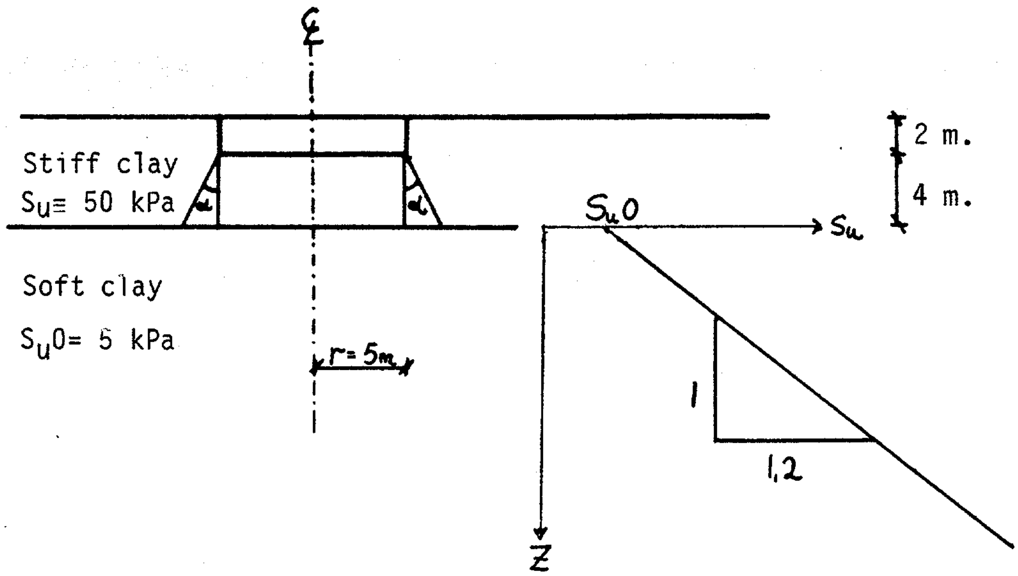


Figure F 1: Model used for comparison between "punch-through" formula and finite element program. Stiff clay overlaying soft clay.

The same element mesh and loading procedure is used for both cases:

Load step no.	Load increment (kPa)	Total load (kPa)
1	100	100
2	50	150
3	50	200
4	50	250
5	50	300

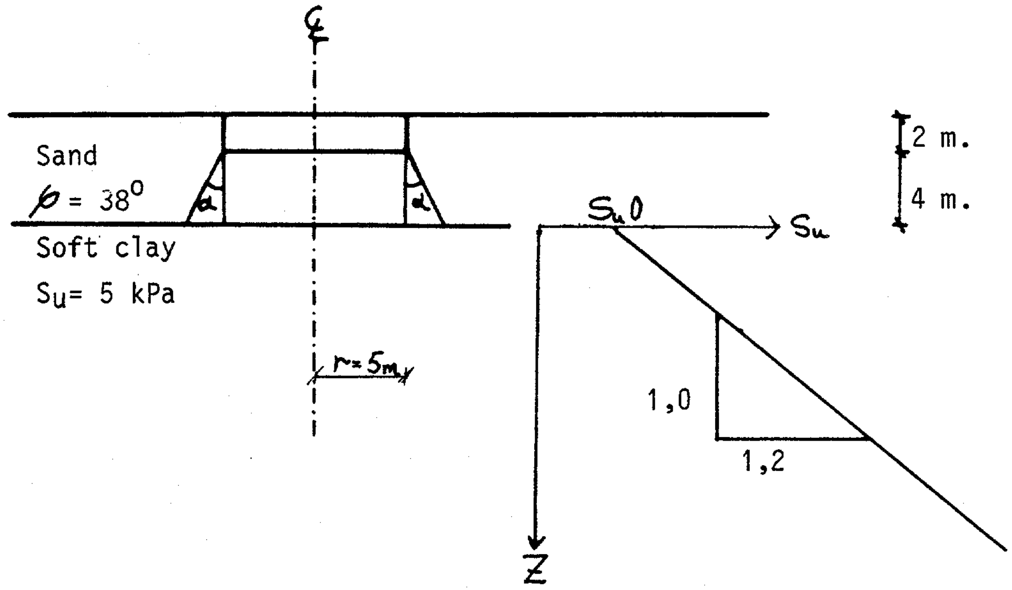
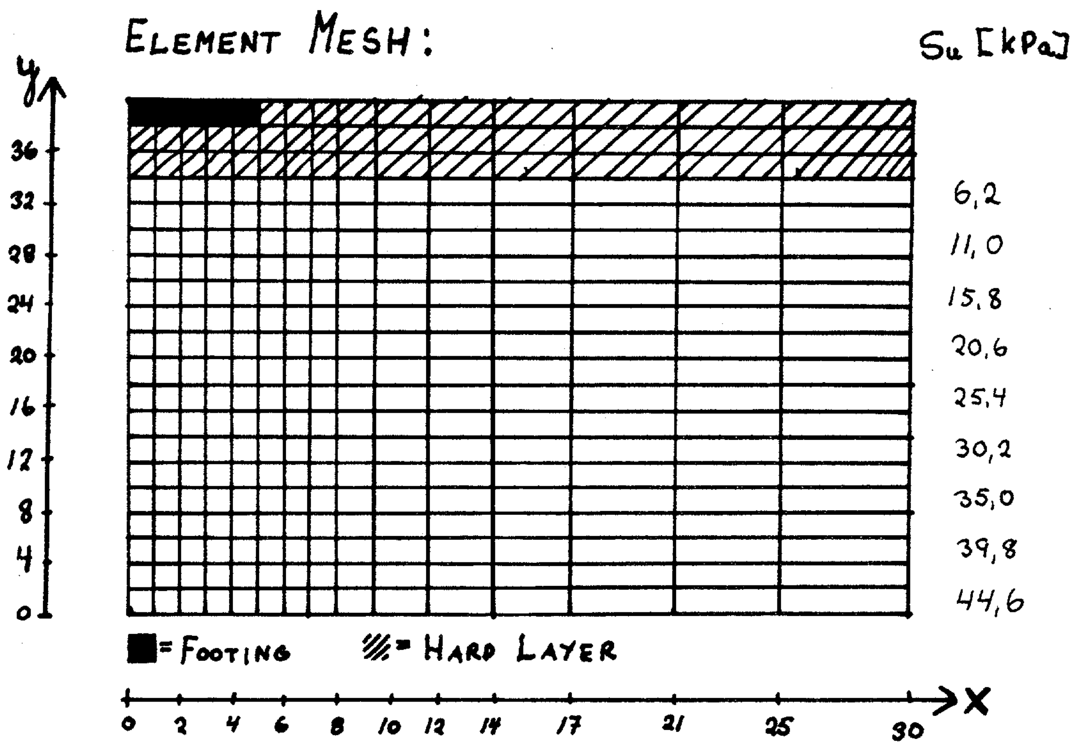


Figure F 2.: Model used for comparison between "punch-through" formula and finite element program. Sand overlaying soft clay.

The load was applied as water pressure loading on the foundation, and an element mesh with 336 nodes and 300 elements was used, see the below figure.



Stiff clay overlaying soft clay:

1) Development of failure

Symbols used

X = Failure during this step.

0 = Failure during last step.

□ = More than 90% of ultimate capacity mobilized.

Load step no. 1

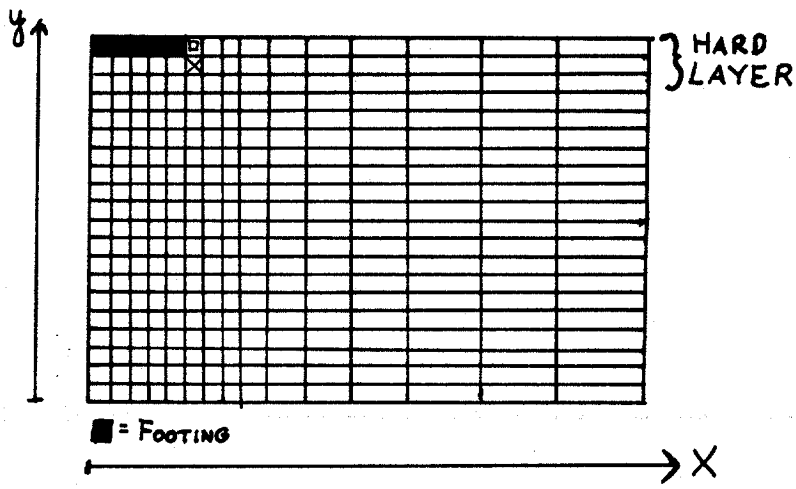
$q = 100\ kPa,\ \Delta q = 100\ kPa$

The most stressed element is element 291 with a stress level of 83% of ultimate capacity.

Load step no. 2

$q = 150\ kPa,\ \Delta q = 50\ kPa$

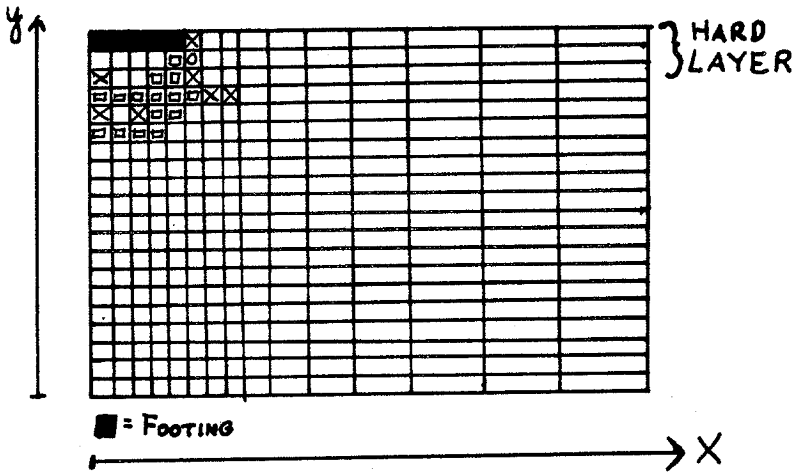
Failure develops in hard layer under edge of spudcan.



Load step no. 3.

$q = 200\ kPa,\ \Delta q = 50\ kPa$

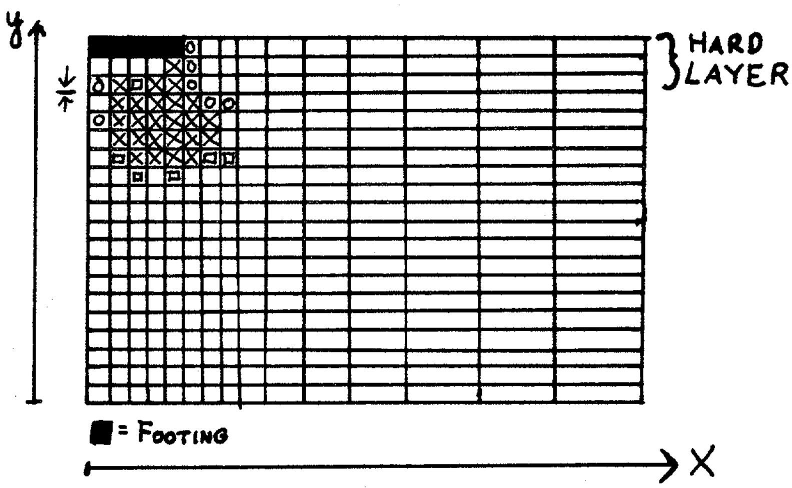
Failure under edge propagates and is fully developed from spudcan and down in soft layer. This load step therefore corresponds to a lower bound for punch-through failure. (See figure next page.)



Load step no. 4.

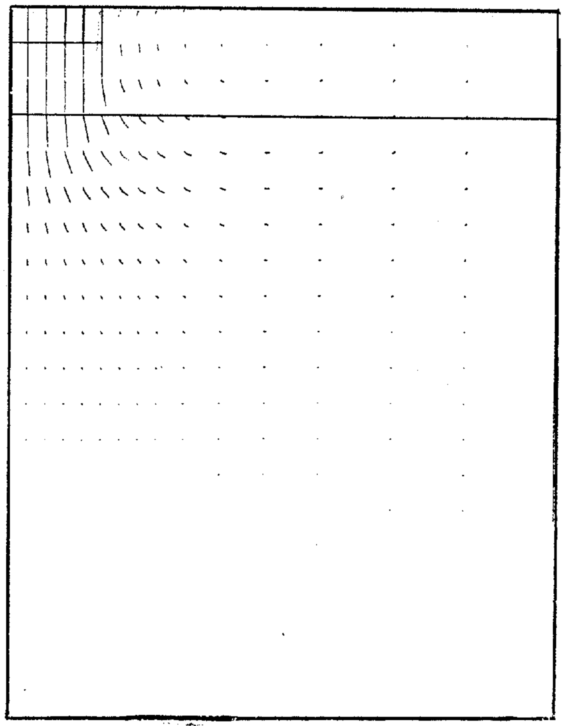
$q = 250\ kPa,\ \Delta q = 50\ kPa$

Large areas under spudcan are in failure. The punch-through capacity is far exceeded.

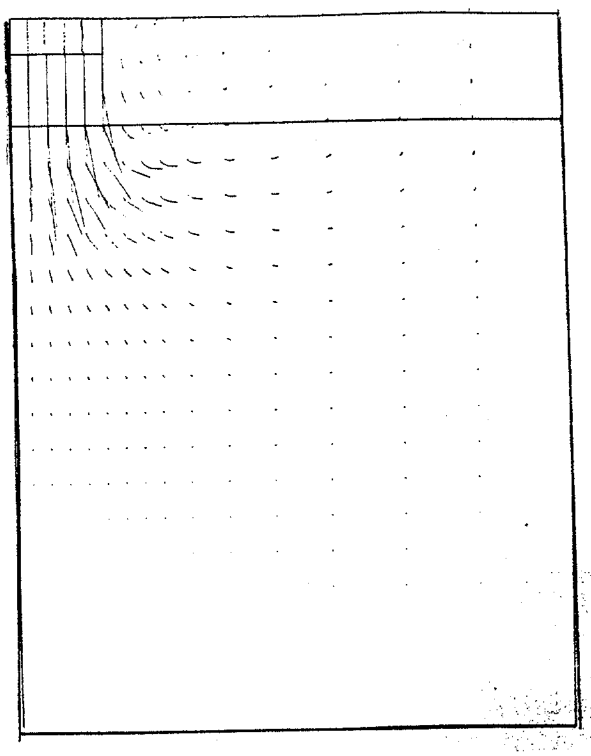


Deformations:

This plot shows the nodal deformations from the situation with no load applied till the load level equals the punch-through capacity. (BELOW)



This plot shows the nodal deformations for load steps 1 through 5. (BELOW)

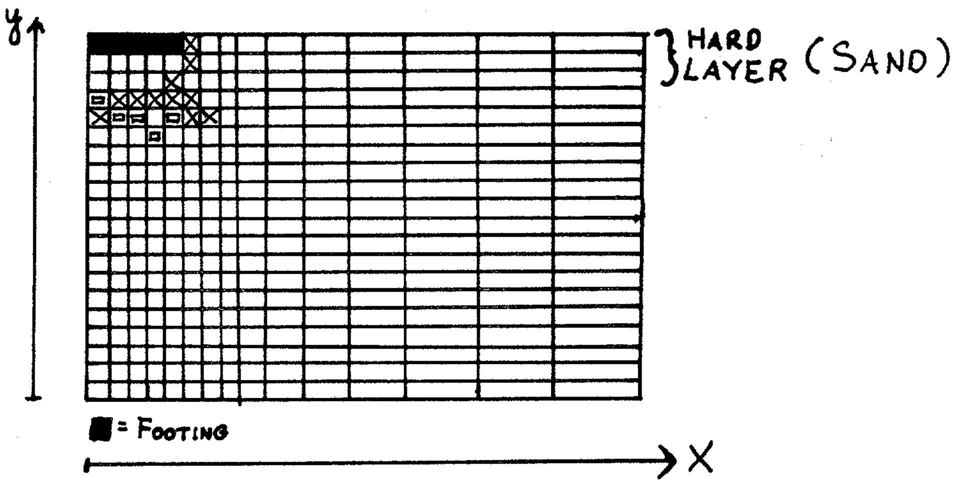


Sand overlaying soft clay:

1) Development of failure:

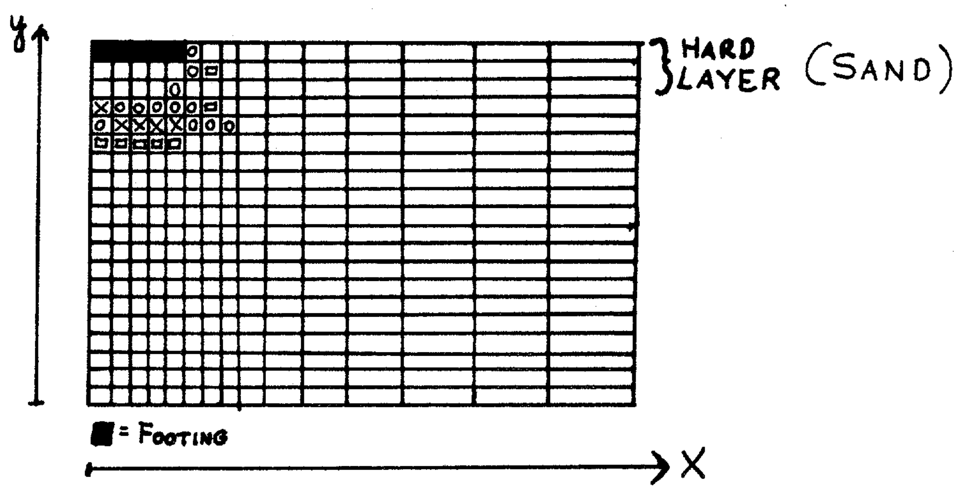
Load step no. 1

$q = \Delta q = 100\ kPa$



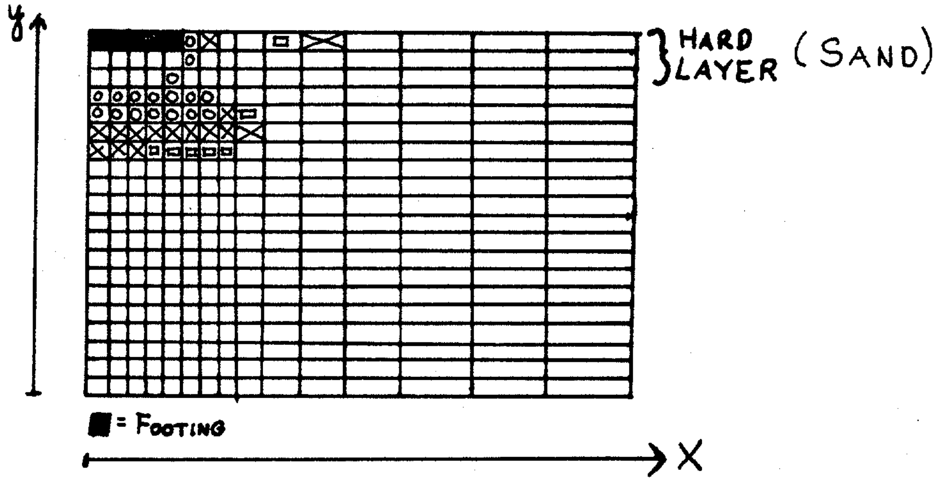
Load step no. 2

$q = 150\ kPa,\ \Delta q = 50\ kPa$



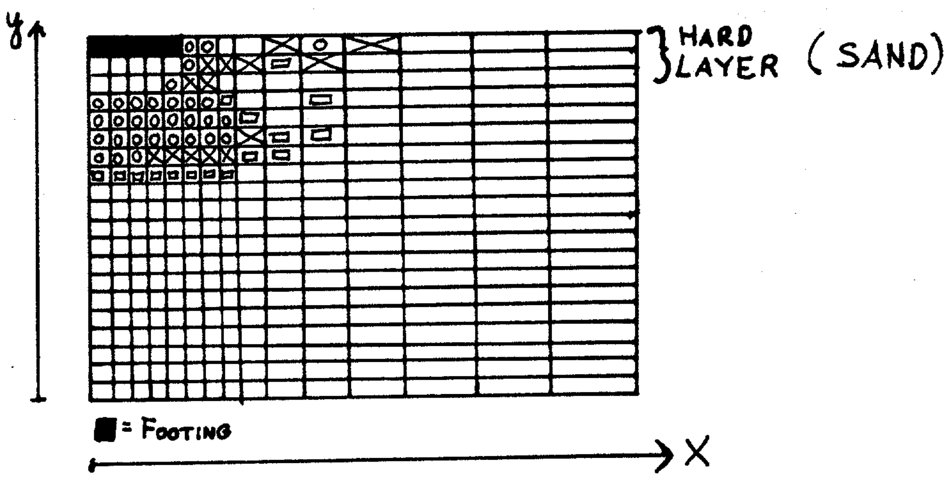
Load step no. 3

$q = 200 \ kPa$, $\Delta q = 50 \ kPa$



Load step no. 4

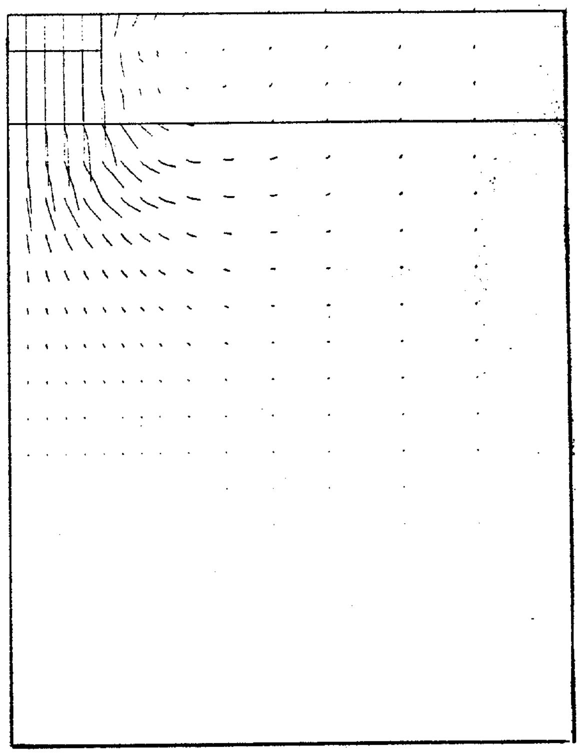
$q = 250 \ kpa$, $\Delta q = 50 \ kPa$



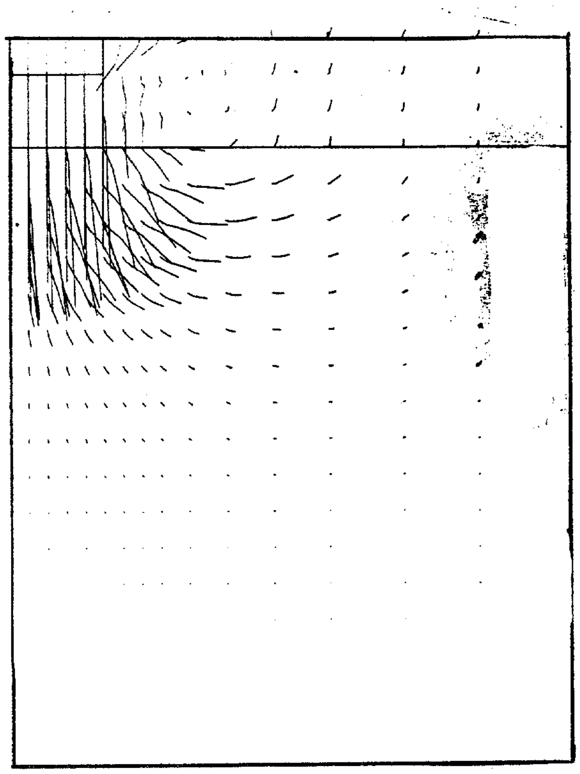
The failure starts under the edge of the footing and spreads out in top of the soft layer. Punch-through failure occurs at q = 150-200 kPa.

2) Deformations:

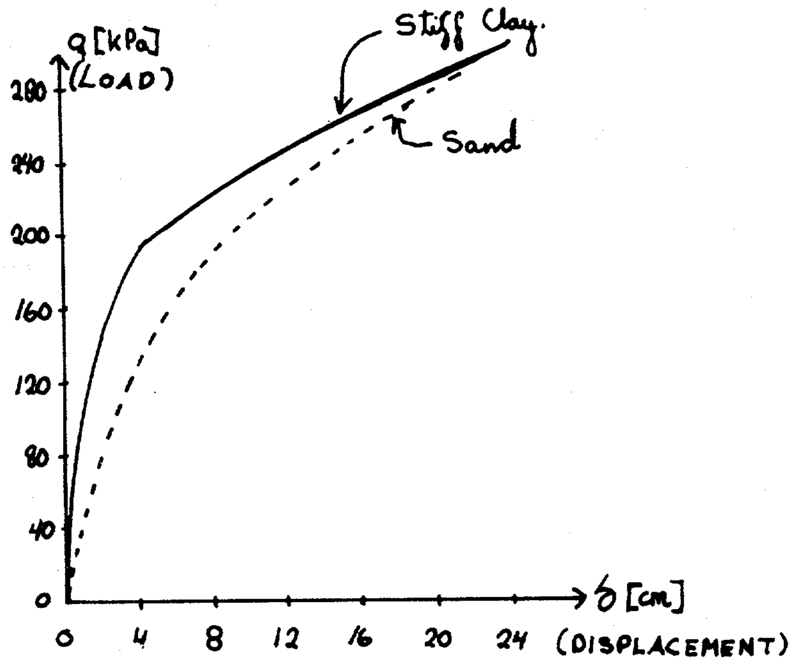
Nodal deformations for load steps 1 through 3.



Nodal deformations for load step 1 through 5.



Load-displacement curve.



APPENDIX G

Description of parametric study on safety against punch-through failure.

The following 7 cases have been studied:

Case I) Surface footing resting on stiff clay overlying soft clay
Case II) Shallowly penetrated footing in stiff clay overlying soft clay
Case III) Shallowly penetrated footing in very stiff clay overlying soft clay
Case IV) Surface footing on thin layer of sand overlying soft clay
Case V) Surface footing on medium thick sand overlying soft clay
Case VI) Surface footing on thick layer of sand clay overlying soft clay
Case VII) Effect of variation in foundation radius and height of side wall on spud-can.

The results of the studies are plotted in a coordinate system where the topical parameter value in percent of initial value is plotted along the horizontal axis and the factor of safety is plotted along the vertical axis.

Effects of parameters which are easy to estimate, such as T (thickness of hard layer) and γ' (submerged unit weight), are only studied for a variation of $\pm$ 20% of initial parameter value. Effects of parameters which usually are more uncertain, such as s_{uo}, k and s_{uH}, are tested for 50% and 150% of the initial parameter value.

Case I) Initial situation

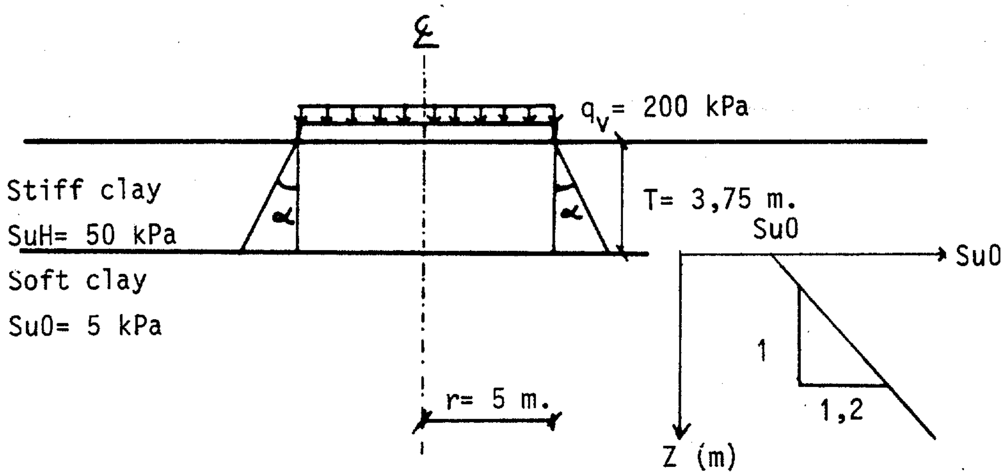
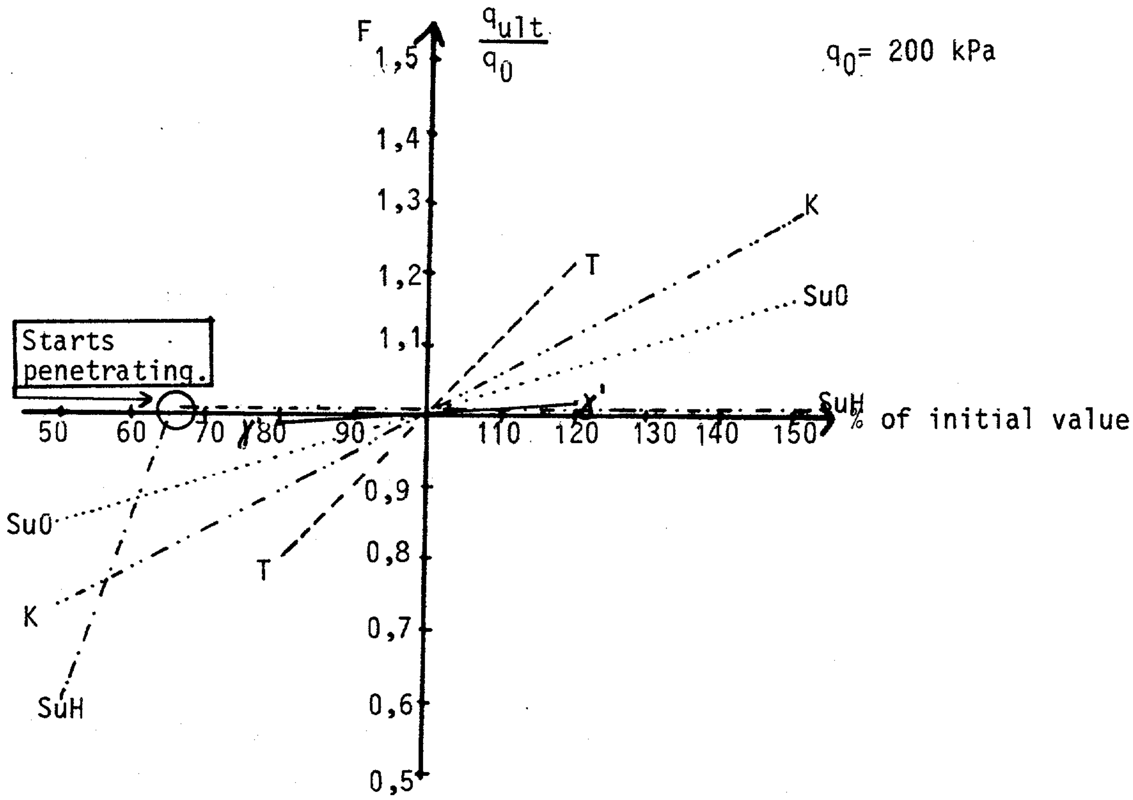


Figure G 1.: Initial situation for punch-through study case No. 1
Circular surface footing on stiff clay overlaying soft clay.



T=
Su0=
K=
γ'=
SuH=

where:
q_0= Initial punch-through capacity.
q_{ult}= Punch-through capacity for actual parameter.
F = Factor of safety against punch-through failure

Figure G 2.: Results case No. 1. Effect of parameter variation on safety against punch-through failure.

The effects of variation of the height h of the spudcan side and the penetration depth z have not been considered since the foundation base rests on the surface.

As can be seen from the above figure the bearing capacity and safety against punch-through are more sensitive to some of the parameters than to the others. The most important parameter is the thickness of the hard top layer. Then the increase in undrained shear strength with depth and undrained shear strength in top of underlaying soft soil. The variation in submerged unit weight does hardly affect the bearing capacity. Neither does the shear strength of the hard layer (for a surface footing), as long as it is not reduced below a value for which the bearing capacity will equal the load. If s_{uH} is reduced further below this value, the footing will start to penetrate in the hard layer, which has thus become somewhat softer.

Case II) Initial situation

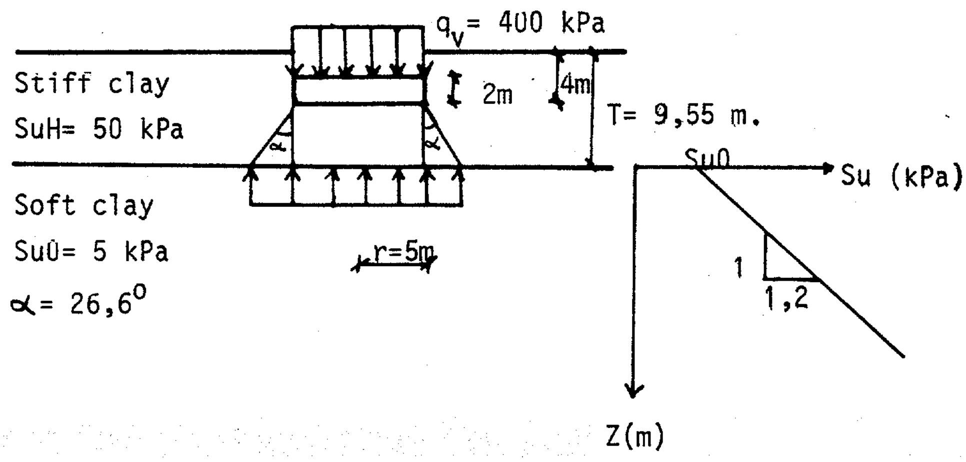
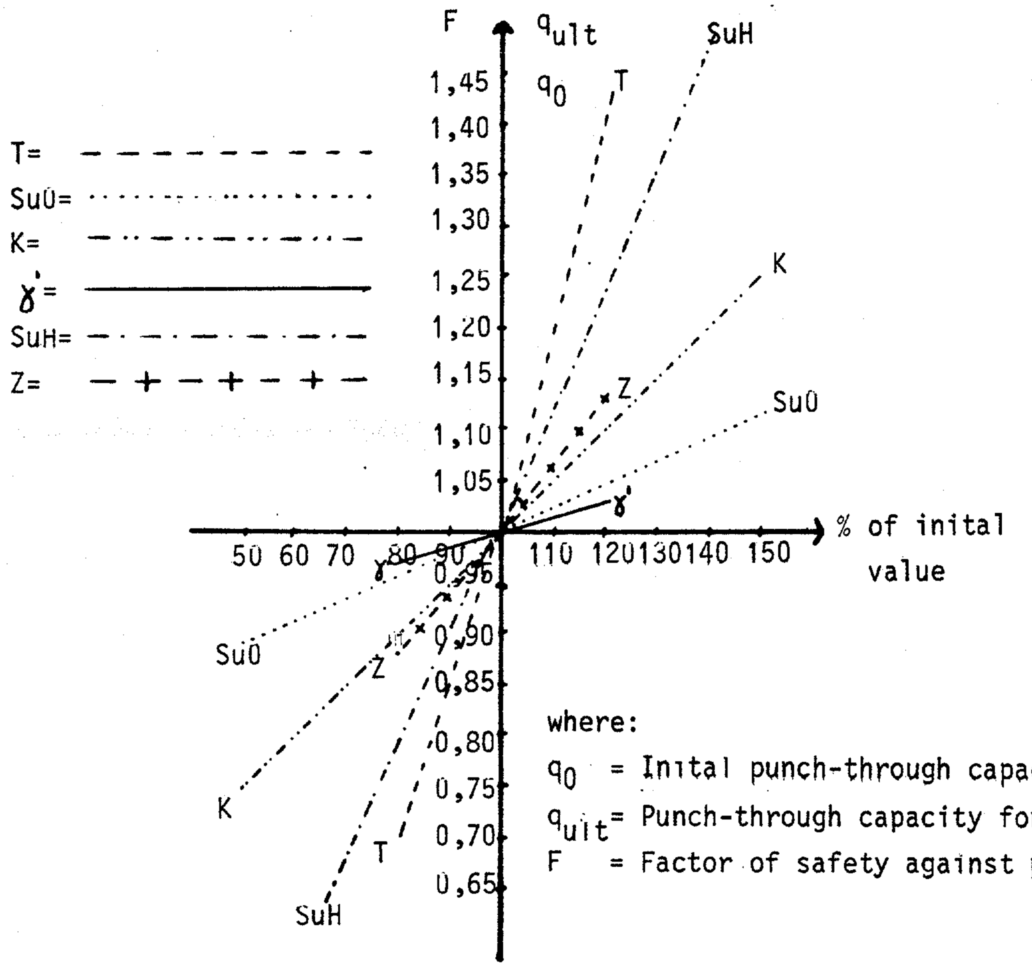


Figure G 3.: Initial situation for punch-through study case No.2.
Embedded circular footing in stiff clay overlaying soft clay.

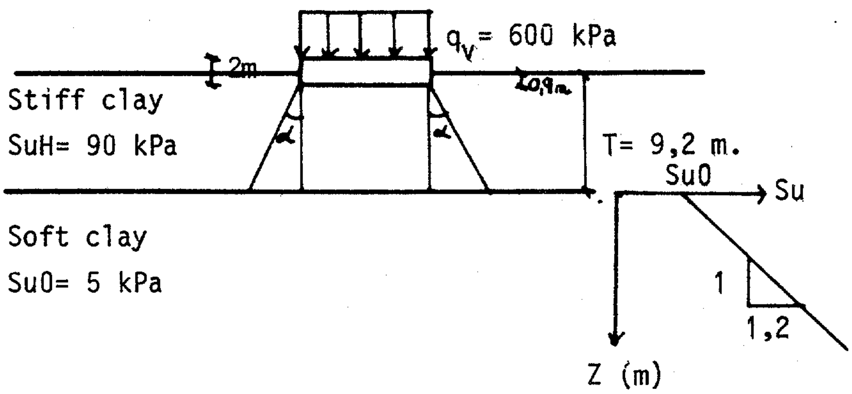


where:
q_0 = Inital punch-through capacity.
q_{ult} = Punch-through capacity for actual parameter.
F = Factor of safety against punch-through failure.

Figure G 4.: Results case No. 2. Effect of parameter variation on safety against punch-through failure.

For this case the effect of varying T, k, s_{uo} and γ' is the same as for Case I. Since the foundation has penetrated into the hard layer (Case II) the variation in s_{uH} is very important for the punch-through capacity. If s_{uH} is reduced, the footing produces a deeper penetration. Hence, the distance to the soft layer reduces and so does the transmitted "foundation area" on top of the soft soil. This increases the stress level on the theoretical foundation area on top of the soft soil and the safety against punch-through is reduced below 1.0, which means failure. If s_{uH} is increased, the result is a more shallow penetration - a wider transmitted foundation area on top of the soft soil and a reduced stress level which increases the safety against punch-through.

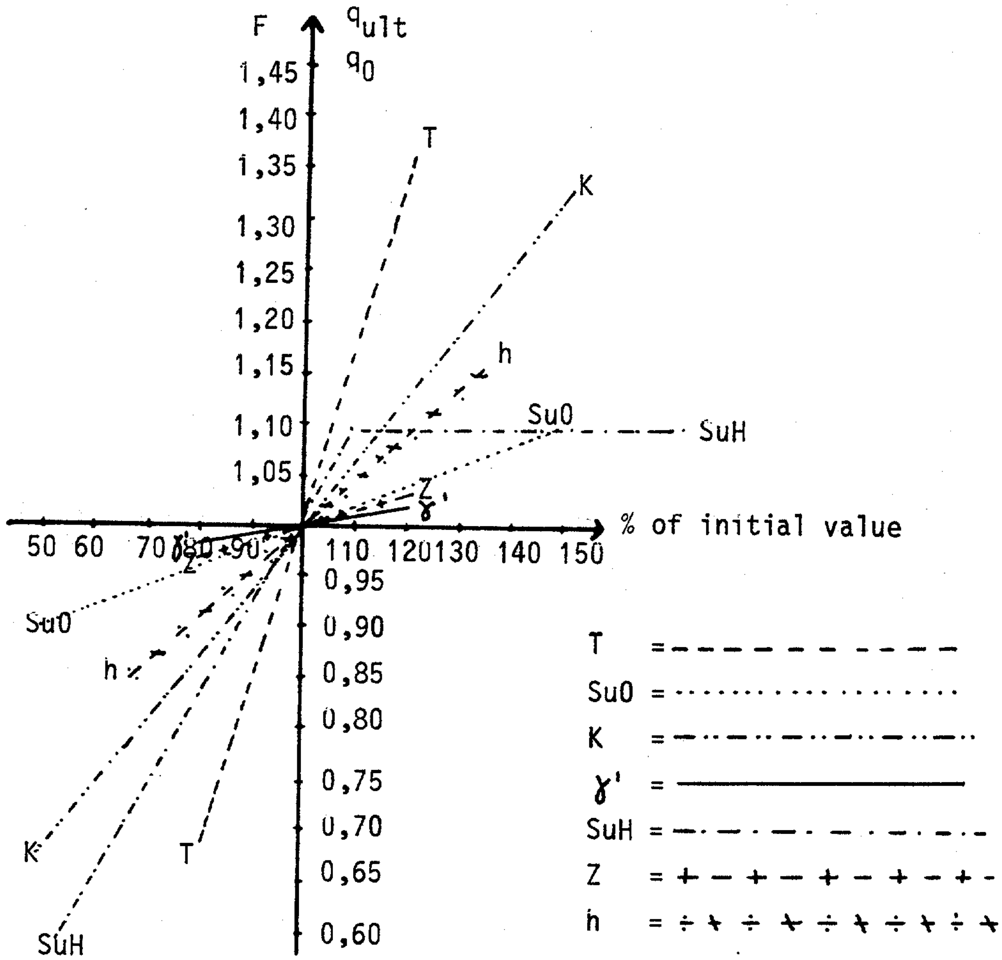
Case III) Initial situation.



α = 26,6°

Figure G 5.: Initial situation for punch-through study case No. 3. Partly embedded circular footing in very stiff clay overlaying soft clay.

To see the effect of higher bearing capacity, the undrained shear strength of the stiff clay is increased by 80%.



where:

T = Thickness of top layer.

Z = Base penetration depth.

h = Exposed height of side wall.

q_{ult} = Punch-through capacity for actual parameter.

q_0 = Inital punch-through capacity.

F = Factor of safety against punch-through failure.

Figure G 6.: Results case No. 3. Effect of parameter variation on safety against punch-through failure.

If we compare the above figure with the corresponding figure for Case II, 20% variation in z has more effect for Case II than for Case III. This is because the penetration in Case II is four times as large, while most of the other parameters are the same in the two cases. When s_{uH} (=shear strength for hard layer) increases, the result is a more shallow penetration and an increase in punch-through capacity. This continues until the s_{uH} is high enough to keep the foundation on the surface. Further increase in s_{uH} does not increase the punch-through capacity in our model.

Parameter study on punch-through failure for sand overlaying soft clay:

Bearing capacity in sand when no soft clay is present underneath:

$$q = \frac{1}{2}\gamma' \cdot b' \cdot N_\gamma \cdot s_\gamma \cdot d_\gamma \cdot i_\gamma + p'_o \cdot N_q \cdot s_q \cdot d_q \cdot i_q$$

$F_H = 0 \rightarrow i_\gamma = i_q = 1.0 \quad \varphi = 38^o \quad \gamma' = 10\ kN/m^3 \quad b' = 8.86\ m$

$N_\gamma = 56.2 \quad N_q = 48.9 \quad s_q = 1.62$

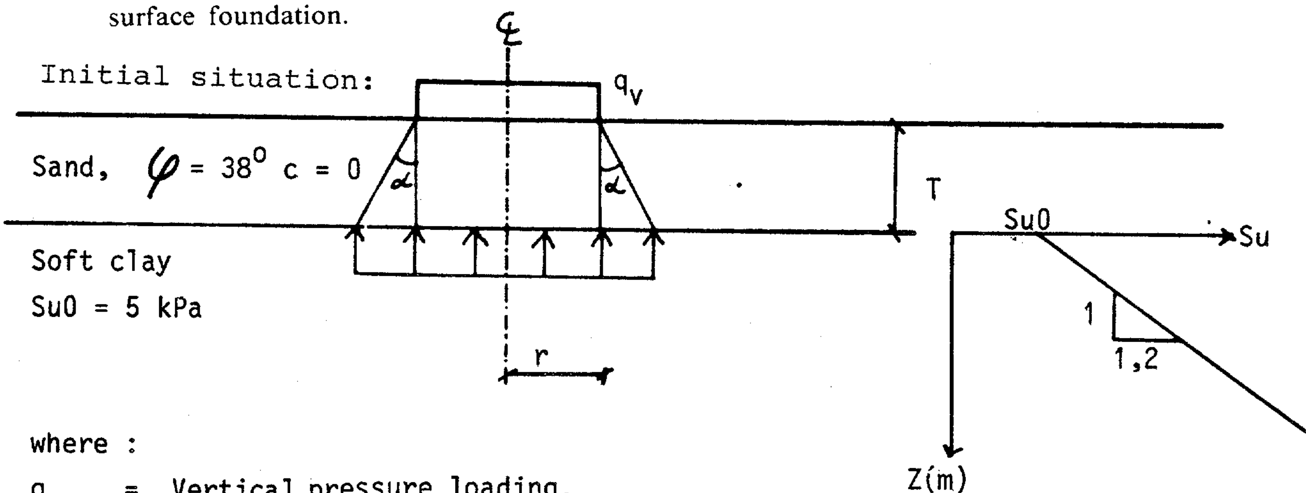
$$\rightarrow q = 0.5 \cdot 10 \cdot 8.86 \cdot 56.2 \cdot 0.6 + p'_o \cdot 48.9 \cdot 1.62 \cdot (1 + d_q \cdot 0.0156) \cdot 1$$

$$q = 1493\ kPa + 79.2 \cdot (1 + d_q \cdot 0.0156)\ kPa$$

For $\varphi = 30^o$ and footing on surface

$$q = 0.5 \cdot 10 \cdot 8.86 \cdot 15.07 \cdot 0.6 = 401\ kPa$$

This means that most jackup platforms will not penetrate in sand at all. But if the sand is overlaying soft clay, punch-through could be a serious problem. We will therefore consider three cases with vertical q_v = 200, 400 and 600 kPa, respectively, on a surface foundation.



where :

q_V = Vertical pressure loading.

T = Thickness of top layer.

r = Radius of foundation.

Figure G 7.: Inital situation for punch-through studie study case No. 4, 5 and 6.
Partly embedded circular footing on sand overlaying soft clay.

Formula for punch-through capacity of foundation on surface:

$$q_{punch-through} = \frac{(r+0.5\cdot(T-z))^2}{r^2}(s_{uo}+k\cdot(\frac{r}{2}+\frac{r}{4}\cdot(T-z)))\cdot N_c\cdot(1.2+0.3\cdot arctg\frac{T}{r\sqrt{\pi}})$$

which leads to

$$q_{punch-through} = \frac{(r+0.5\cdot T)^2}{r^2}(s_{u0}+k\cdot(\frac{r}{2}+\frac{r}{4}\cdot T))\cdot N_c\cdot(1.2+0.3 arctg\frac{T}{r\sqrt{\pi}})$$

where

r = Radius of foundation.

T = Thickness of top layer (sand layer).

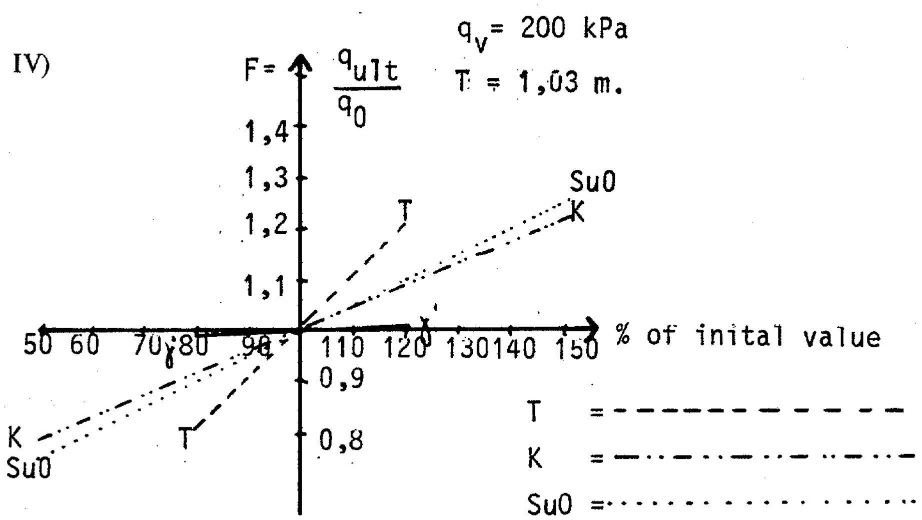
s_{uo} = Shear strength on top of soft clay.

k = The increase in shear strength with depth from top of soft layer.

N_c = Bearing Capacity factor = 5.14

π = Pi.

Case IV)



where:

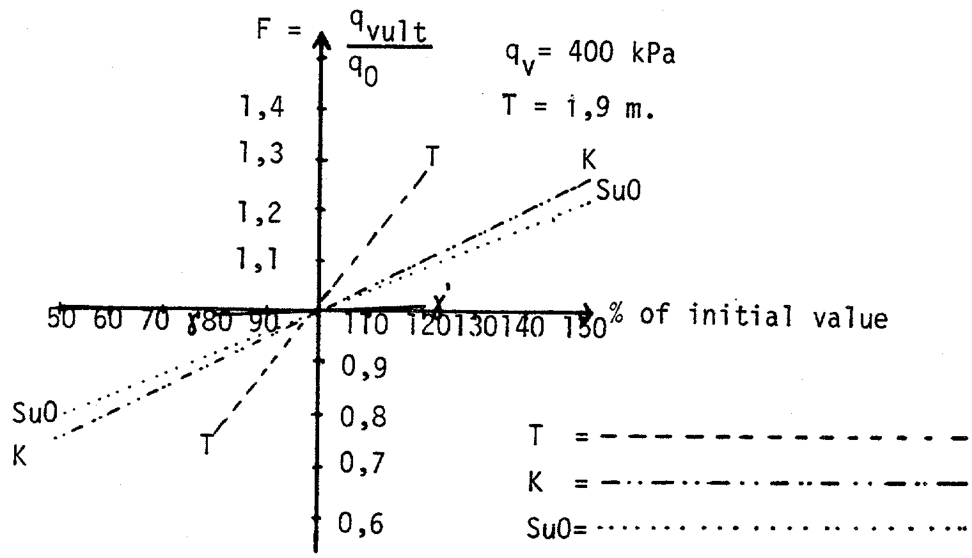
q_{ult} = Ultimate bearing capacity for punch-through.

q_0 = Punch-through capacity for initial case.

F = Factor of safety against punch-through failure.

Figure G 8.: Results case No. 4. Effect of parameter variation on safety against punch-through failure.
Vertical loadlevel q_v = 200 kPa.

Case V)



where:

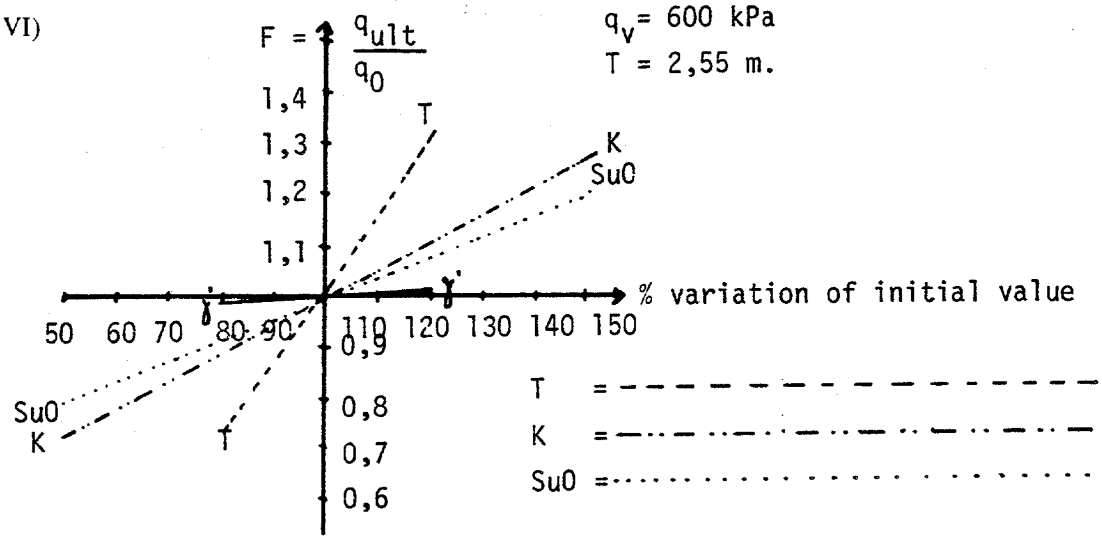
q_{vult} = Ultimate punch-through bearing capacity.

q_0 = Punch-through capacity for initial case.

F = Factor of safety against punch -through failure.

Figure G 9.: Results case No. 5. Effect of parameter variation on safety against punch-through failure. Vertical loadlevel q_v= 400 kPa.

Case VI)



where:

q_{ult}= Ultimate bearing capacity for punch-through.

q_0 = Punch-through capacity for initial case.

F = Factor of safety against punch-through failure.

Figure G 10.: Results case No. 6. Effect of parameter variation on safety against punch-through failure. Vertical loadlevel q_v= 600 kPa.

Jackup platforms supported by spudcans will normally not penetrate in sand. To find the punch-through capacity for a sand layer overlaying a soft clay, the most important parameter is the thickness of the sand layer. A thicker sand layer increases the punch-through capacity , while a thinner sand layer decreases the punch-through capacity. Other important parameters for punch-through are the undrained shear strength and the rate of increase in undrained shear strength with depth for the underlaying soft clay. Uncertainties in the submerged unit weight of the sand do hardly affect the punch-through capacity.

Case VII)

Effect of variation in foundation radius and height of side wall on spudcan.

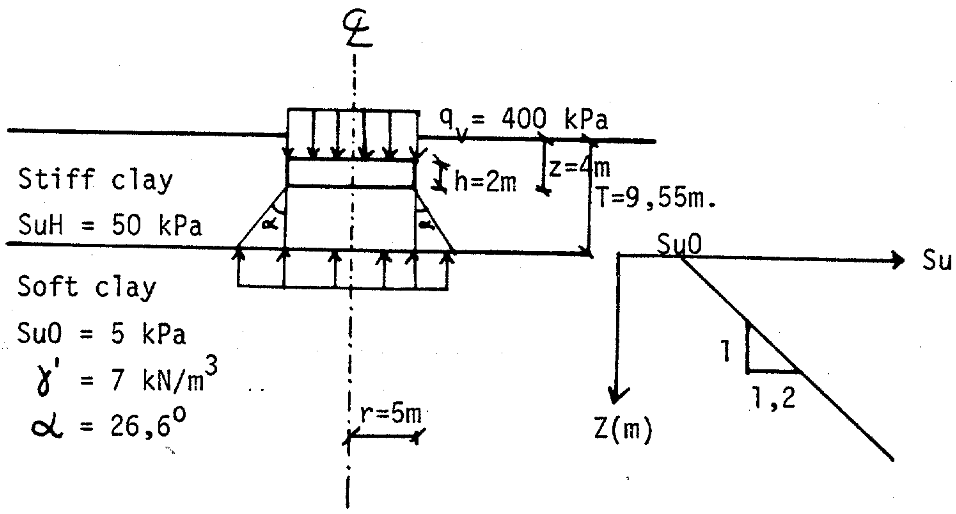
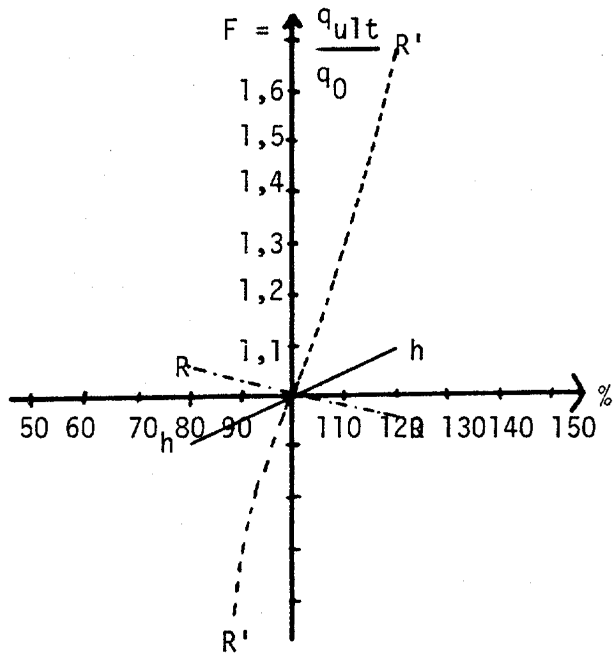


Figure G 11.: Initial condition for case No. 7.



where:

h = ———————— = Height of exposed side wall on spud-can.

R' = - - - - - - - - - - = Assuming same total load.

R = —·—·—·—·— = Assuming the same pressure loading level.

Figure G 12.: Results case No. 7. Effect of variation in foundation radius and height of side wall on spud-can.

If the same total load is assumed, an increase in the foundation area (obtained by increasing the radius) will lead to an increase in the punch-through capacity, while the punch-through capacity will decrease when the radius decreases.

If the same load level is assumed for the contact pressure, an increase in radius will decrease the safety against punch-through (and the total load will increase correspondingly).

The figure above shows that the bearing capacity is strongly dependent on the foundation radius.

APPENDIX H

Derivation of formulae used for evaluation of sliding resistance for a shallowly embedded spudcans.

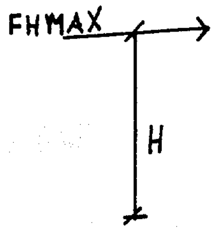
F_o = Factor of safety against overturning.

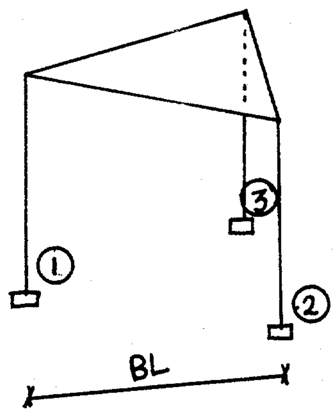
H = Lever arm for resulting environmental force.

BL = Breadth between legs.

WG = Structure weight.

WGV = Variable load.





Stabilizing moment $= \dfrac{WG+WGV}{3} \cdot BL$

Overturning moment $= F_H \cdot H$

Stabilizing moment $\geqslant F_o \cdot$ overturning moment (Fo > 1.5)

$$\frac{WG+WGV}{3} \cdot BL = F_o \cdot F_H \cdot H$$

$$FH_{MAX} = \frac{WG+WGV}{3} \cdot \frac{BL}{H} \cdot \frac{1}{F_o}$$

$$F_{v1} = \frac{(WG+WGV)}{3} - \frac{F_H \cdot H}{BL}$$

$FH = \dfrac{FH_{MAX}}{3}$, assuming one third of total horizontal load on each leg.

$$F_{v1} = \frac{(WG+WGV)}{3} \cdot (1 - \frac{1}{F_o})$$

$$F_{v2\ or\ 3} = \frac{WG+WGV}{3} + \frac{1}{2} \frac{FH_{MAX} \cdot H}{BL}$$

$$F_{v2\ or\ 3} = \frac{WG+WGV}{3} (1 + \frac{1}{2} F_o)$$

Environmental forces from left hand side:

$$\frac{FH_{MAXLegno.\ 1}}{F_{v1}} = \frac{BL}{3 \cdot H (F_o - 1)}$$

$$\frac{FH_{MAXLegno.\ 2or3}}{F_{v2\ or\ v3}} = \frac{BL}{3 \cdot H(F_o + 0.5)}$$

Environmental forces from right hand side:

$$\frac{FH_{MAXLegno.\ 1}}{F_{v1}} = \frac{BL}{3 \cdot H \cdot (F_o + 1)}$$

$$\frac{FH_{MAXLegno.\ 2or3}}{F_{v2\ or\ 3}} = \frac{BL}{3 \cdot H \cdot (F_o - 0.5)}$$

Ratio between max. horizontal force and actual vertical load for the three jack-up platform models:

Model	BL	H
1	33m	33m
2	37m	60m
3	43m	100m

$F_o = 1.5$ is assumed in the following:

Environmental force from left:

	Model 1	Model 2	Model 3
$\frac{FH_{MAX(Oneleg)}}{F_{v1}}$	0.66	0.41	0.29
$\frac{FH_{MAX(Oneleg)}}{F_{v2\ or\ v3}}$	0.16	0.11	0.07

Environmental forces from right:

	Model 1	Model 2	Model 3
$\frac{FH_{MAX(Oneleg)}}{F_{v1}}$	0.13	0.08	0.06
$\frac{FH_{MAX(Oneleg)}}{F_{v2\ or\ v3}}$	0.33	0.21	0.14

Formula for sliding resistance in sand:

$$FH_{ult} = FV_{ACTUAL} \cdot tg\varphi \cdot RR \text{ (Assuming cohesion } = 0.0)$$

$$SFSL = \frac{FH_{ult}}{FH_{MAX}}$$

where:

RR = Roughness ratio.

φ = Internal friction angle for soil.

FV_{ACTUAL} = Actual vertical load.

FH_{ult} = Ultimate lateral capacity.

SFSL = Factor of safety against sliding.

FH_{MAX} = Maximum horizontal force on actual leg.

Formula for sliding resistance in clay.

$$FH_{ult} = A \cdot s_u$$

where:

A = Foundation area.

s_u = Undrained shear strength.

$$FH_{MAXone\ \ leg} = \frac{1}{3} FH_{MAX-Total} = \frac{1}{3}\frac{(WG+WGV)}{3}\cdot\frac{BL}{H}\cdot\frac{1}{F_o}$$

$$A = \frac{F_v}{qv_{ult}} = \frac{(WG+WGV+PRE)}{3 \cdot N_c \cdot 1.2 \cdot s_u}$$

where:

qv_{ult} = Ultimate bearing pressure for plain vertical loading.

PRE = Ballast weight used for preload.

$$SFSL = \frac{FH_{ULT}}{FH_{MAX}} = \frac{(WG+WGV+PRE)}{3 \cdot N_c \cdot 1.2}\;\frac{9 \cdot H \cdot F_o}{(WG+WGV) \cdot BL}$$

$$\underline{\underline{SFSL = \frac{3 \cdot H \cdot F_o}{1.2 \cdot BL \cdot N_c} \cdot \left(1 + \frac{PRE}{WG+WGV}\right)}}$$

If we assume the required preload to be the ballast which is necessary to achieve vertical load on all legs equal to max vertical load on the most stressed leg when the structure is exposed to designed environmental loads, we have:

$$Fv_{(preload\,one\quad leg)} = \frac{FH \cdot H}{BL} = \frac{(WG\ +\ WGV)}{3} \cdot \frac{1}{F_o}$$

$$Required\,preload = \frac{WG + WGV}{F_o}$$

$$PRE\ =\ Used\ preload\ =\ \frac{K \cdot (WG + WGV)}{F_o}$$

where:

K = Correction factor with values from in the range $0 \rightarrow 1$ telling the degree of used required preload.

$$SFSL = \frac{3 \cdot H \cdot F_o}{1.2 \cdot BL \cdot N_c}(1 + \frac{PRE}{WG\ +\ WGV}) = \frac{3 \cdot H \cdot (F_o + K)}{BL \cdot 1.2 \cdot N_c}$$

$$SFSL = 0.49 \cdot \frac{H \cdot (F_o + K)}{BL}$$

To include cyclic effects and effect of very soft soil overlaying hard soil the following formula may be applied:

$$SFSL = 0.49\ \frac{N_c}{N_c^*} \frac{H \cdot (F_o + K)}{BL} \cdot \frac{s_{u\,cyclic}}{s_{u\,static}}$$

where:

N_c^* = Bearing capacity factor reflecting the effect of a deeper layer on the bearing capacity, giving the "real value" of N_c to be used together with soft layer's soil parameters.

$s_{u\,cyclic}$ = Undrained shear strength of soft layer for lateral cyclic loading.

$s_{u\,static}$ = Undrained shear strength of soft layer for vertical static loading.

APPENDIX I

Description of input data and results from the parametric study on lateral resistance and bearing capacity of shallowly embedded spudcans.

For structural and geometric data see Table 1 and 2 of Section 2.1

MODEL No. 1

Shallowly penetrated spudcans in clay. Soil profile no. 1

Initial sitution:

$s_{u0} = 50\ kPa, k = 1.2 kPa/m$

Penetration depth for tip = 0.96 m. Wave period T = 13 sec.

Wave height = 15m.

Factor for safety against sliding (minimum) = SFS_{Min} = 2.22

Factor for safety against bearing capacity failure (minimum) = $SFB_{\min}$ = 0.94

MODEL NO. 2.

Shallowly penetrated spudcans in clay. Soil profile no. 1

Initial situation:

Penetration depth for spud-can tip = 1.1m, $s_{u0} = 50 kPa, k = 1.2 kPa/m.$

Wave Period = 15 sek, Wave height = 20 m.

Factor of safety against sliding (minimum) = $SFS_{\min}$ = 2.11

Factor of safety against bearing capacity failure (minimum) = $SFB_{\min}$ = 0.89

MODEL NO. 3

Shallowly penetrated spudcans in clay. Soil profile no. 1

Initial situation:

Wave Period = 16s. Wave height = 25 m,s_{u0} = 60kPa , k = 1.2 kPa/m

Penetration depth for spudcan tip = 1.69 m.

Factor of safety against sliding (minimum) = $SFS_{\min}$ = 2.20 Factor of safety against bearing capacity failure (minimum) = $SFB_{\min}$ = 0.865

The tables on the following three pages cover the results for the three jackup platform models as referenced above.

MODEL NO. 1

CASE : SHALLOW PENETRATION IN CLAY. SOIL PROFILE NO. 1.
Su0 =50KPa ZPfin = 0,96 m SFSmin = 2,22 SFBmin = 0,94 (BASIC CASE)

Case no.	Parameter	Value	Variation	SFS min	± %	SFB min	± %	Comments
1	Wave Height	18	+20%	1,68	-24	0,85	-10	
2		12	-20%	2,9	+32	1,01	+8	
3	Wave Period	17	+30%	2,11	-4	0,92	-2	
4		15	+15%	2,14	-3	0,93	-1	
5	Wave Period	11	-15%	2,23	1	0,95	+1	
6		9	-30%	3,06	+39	1,02	+9	
7	Wind Speed	55	+10%	2,08	-5	0,91	-3	
8		45	-10%	2,33	+6	0,97	+3	
9	Current	1,1	+100%	1,80	-18	0,87	-7	
10		0,0	-100%	2,69	+22	1,0	+6	
11	Distance	37	+12%	2,23	+1	0,96	+2	
12	between legs.	29	-12%	2,15	-2	0,92	-2	
13	Platform	72	+20%	2,49	13	0,96	+2	
14	weight.	48	-20%	1,93	-12	0,92	-2	
15	Lever arm	14,7	+ 5%	2,20	0	0,94	0	
16	for wind	9,3	- 5%	2,20	0	0,94	0	
17	Drag diameter	6,0	+20%	1,95	-11	0,91	-3	
18	for legs.	4,0	-20%	2,53	+15	0,97	+3	
19	Inertia diam.	2,4	+20%	2,16	-2	0,93	-1	
20	for legs.	1,6	-20%	2,22	+1	0,95	+1	
21	EI Bending	96	+20%	2,21	0	0,94	0	
22	stiff legs.	64	-20%	2,20	0	0,94	0	
23	EA Axial	12	+20%	2,20	0	0,94	0	
24	stiff legs.	8	-20%	2,20	0	0,94	0	
25	GA Shear	24	+20%	2,20	0	0,94	0	
26	stiff legs.	16	-20%	2,20	0	0,94	0	
27	G-modulus	450 Su	+50%	2,20	0	0,94	0	
28	for soil.	150 Su	-50%	2,20	0	0,94	0	
29	DTIP for	1,5	+50%	2,20	0	0,94	0	ZPF= 0,96
30	spud-can	0,5	-50%	2,20	0	0,94	0	ZPF = 0,96
31	HSID for	2,03	+50%	2,20	0	0,94	0	
32	spud-can	0,68	-50%	2,20	0	0,94	0	
33	HCON for	0,75	+50%	2,24	2	0,95	+1	ZPF = 1,19 + 24%
34	spud-can.	0,25	-50%	2,17	-1	0,93	-1	
35	HTIP for	0,75	+50%	2,19	0	0,94	0	ZPF = 1,21 + 26%
36	spud-can	0,25	-50%	2,24	+2	0,94	0	ZPF = 0,92 - 4%
37	DTOT for	12,0	+ 9%	2,20	0	0,94	0	ZPF = 0,92 - 4%
38	spud-can	10,0	- 9%	2,33	+6	0,99	+5	ZPF = 1,48 +54%
39	Preload	21,6	+20%	2,27	+3	0,99	+5	ZPF = 0,97 + 1%
40		14,4	-20%	2,12	-4	0,89	-5	ZPF = 0,95 - 1%

MODEL NO. 2

CASE : SHALLOW PENETRATION IN CLAY. SOIL PROFILE NO. 1.

Su0 = 50 KPa ZPfin = 1,1 m SFSmin = 2,11 SFBmin = 0,89 (BASIC CASE)

Case no.	Parameter	Value	Variation	SFS min	± %	SFB min	± %	Comments	
1	Wave Height	24	+20%	1,63	-23	0,76	-15		
2		16	-20%	2,74	+30	0,99	+11		
3	Wave Period	19	+26,7%	2,01	-5	0,88	-1		
4		17	+13,3%	2,06	-2	0,88	-1		
5	Wave Period	13	-13,3%	2,16	+2	0,90	+1		
6		11	-26,7%	2,47	+17	0,94	+6		
7	Wind Speed	55	+10%	1,99	-6	0,86	-3		
8		45	-10%	2,22	+5	0,93	+4		
9	Current	1,1	+100%	1,72	-16	0,80	-10		
10		0,0	-100%	2,59	+23	0.98	+10		
11	Distance	41	+11%	2,13	+1	0,92	+3		
12	between legs.	33	-11%	2,09	-1	0,87	-2		
13	Platform	110	+22%	2,65	+26	0,97	+9	ZPF= 2,27	+106%
14	weight.	70	-22%	1,81	-14	0,85	-4	ZPF= 1,06	÷ 3%
15	Lever arm	16,4	+ 5%	2,10	0	0,89	0		
16	for wind	7,6	- 5%	2,11	0	0,89	0		
17	Drag diameter	6,0	+20%	1,86	-12	0,85	-4		
18	for legs.	4,0	-20%	2,42	+15	0,94	+6		
19	Inertia diam.	2,4	+20%	2,06	-2	0,89	0		
20	for legs.	1,6	-20%	2,14	+1	0,89	0		
21	EI Bending	156	+20%	2,11	0	0,89	0		
22	stiff legs.	104	-20%	2,11	0	0,89	0		
23	EA Axial	15,6	+20%	2,11	0	0,89	0		
24	stiff legs.	10,4	-20%	2,11	0	0,89	0		
25	GA Shear	2,4	+20%	2,11	0	0,89	0		
26	stiff legs.	1,6	-20%	2,11	0	0,89	0		
27	G-modulus	450Su	+50%	2,11	0	0,89	0		
28	for soil.	150Su	-50%	2,11	0	0,89	0		
29	DTIP for	1,88	+50%	2,11	0	0,89	0		
30	spud-can	0,63	-50%	2,10	0	0,89	0		
31	HSID for	2,4	+50%	2,11	0	0,89	0		
32	spud-can	0,8	-50%	2,11	0	0,89	0		
33	HCON for	0,75	+50%	2,16	+2	0,94	+6	ZPF=1,38	+ 25%
34	spud-can.	0,25	-50%	2,08	-1	0,88	-1	ZPF = 0,85	- 23%
35	HTIP for	0,9	+50%	2,09	-1	0,89	0	ZPF = 1,39	+ 26%
36	spud-can	0,3	-50%	2,13	1	0,90	+1	ZPF = 0,8	- 27%
37	DTOT for	14	+7,7%	2,08	-1	0,89	0	ZPF = 1,06	- 3%
38	spud-can	12	-7,7%	2,51	+19	0,97	9	ZPF = 3,16	+ 187%
39	Preload	54	+50%	2,59	+23	1,09	22	ZPF = 2,14	+ 95%
40		9	-50%	1,71	-19	0,68	-24	ZPF= 1,04	- 5%

MODEL NO. 3

CASE: SHALLOW PENETRATION IN CLAY. SOIL PROFILE NO. 1.

Su0 = 60 KPa ZPfin = 1,69 m SFSmin = 2,20 SFBmin = 0,87

Case no.	Parameter	Value	Variation	SFS min	± %	SFB min	± %	Comments
1	Wave Height	30	+20%	2,01	-9	0,82	-5	
2		20	-20%	2,9	+32	0,98	+13	
3	Wave Period	20	+25%	2,09	-5	0,86	-1	
4		18	+12,5%	2,14	-3	0,86	-1	
5	Wave Period	14	-12,5%	2,36	+7	0,88	+2	
6		12	-25%	2,83	+29	0,95	+10	
7	Wind Speed	55	+10%	2,08	-5	0,84	-3	
8		45	-10%	2,33	+6	0,9	+4	
9	Current	1,1	+100%	1,97	-10	0,84	-3	
10		0,0	-100%	2,79	+27	0,96	+11	
11	Distance	52	+11%	2,24	+2	0,9	+4	
12	between legs.	42	-11%	2,17	-1	0,83	-4	
13	Platform	144	+20%	2,46	+12	0,9	+4	ZPF = 1,74
14	weight.	96	-20%	1,94	-12	0,8	-3	ZPF = 1,64
15	Lever arm	18,6	+5%	2,21	0	0,86	-1	
16	for wind	5,0	-5%	2,22	+1	0,87	0	
17	Drag diameter	6,0	+20%	1,94	-12	0,81	-6	
18	for legs.	4,0	-20%	2,55	+16	0,92	+6	
19	Inertia diam.	2,4	+20%	2,17	-1	0,86	-1	
20	for legs.	1,6	-20%	2,24	+2	0,87	0	
21	EI Bending	2,4	+20%	2,20	0	0,87	0	
22	stiff legs.	1,6	-20%	2,20	0	0,87	0	
23	EA Axial	2,6	+20%	2,20	0	0,87	0	
24	stiff legs.	1,8	-20%	2,20	0	0,87	0	
25	GA Shear	2,4	+20%	2,20	0	0,88	+1	
26	stiff legs.	1,6	-20%	2,20	0	0,86	-1	
27	G-modulus	450Su	+50%	2,20	0	0,87	0	
28	for soil.	150Su	-50%	2,20	0	0,87	0	
29	DTIP for	2,63	+50%	2,22	+1	0,87	0	
30	spud-can	0,88	-50%	2,19	0	0,86	0	
31	HSID for	3,0	+50%	2,20	0	0,87	0	
32	spud-can	1,0	-50%	2,20	0	0,87	0	
33	HCON for	1,13	+50%	2,32	+5	0,95	10	ZPF = 2,05 +21%
34	spud-can.	0,38	-50%	2,16	-2	0,85	-2	ZPF = 1,35 -20%
35	HTIP for	1,5	+50%	2,18	-1	0,86	0	ZPF = 2,18 +29%
36	spud-can	0,5	-50%	2,23	+1	0,86	0	ZPF - 1,20 -29%
37	DTOT for	16	+ 7%	2,20	0	0,86	0	ZPF = 1,64 -3%
38	spud-can	14	- 7%	2,24	+2	0,88	+2	ZPF = 1,75 +4%
39	Preload	90	+50%	2,51	+14	1,05	+21	ZPF - 1,75 +4%
40		30	-50%	1,90	-14	0,71	-18	ZPF - 1,63 -4%

APPENDIX J:

Description of input on lateral resistance and bearing capacity of deeply penetrated spudcans.

For structural and geometric data see Table 1 and 2 of section 2.1.

MODEL NO. 1

Deeply penetrated spudcan in uniform clay. Soil profile no. 1.

Initial situation

Wave period = T = 13 sec. and wave height = 15 m

$s_{u0} = 20\ kPa,\ k = 1.2\ kPa/m$

Penetration depth for tip = 8.65 meter

Factor of safety against sliding (minimum) = $SFS_{\min}$ = 1.98

Factor of safety against bearing capacity failure = $SFB_{\min}$ = 1.0

MODEL NO. 2

Deeply penetrated spudcans in uniform clay. Soil profile no. 1

Initial situation:

Wave period = T = 15 sec and wave height = 20 m

$s_{u0} = 30\ kPa,\ k = 1.2\ kPa/m$

Penetration depth for spudcan tip = 6.95 m

Factor of safety against sliding (minimum) = $SFS_{\min}$ = 2.12

Factor of safety against bearing capacity failure = $SFB_{\min}$ = 0.95

MODEL NO. 3

Deeply penetrated spudcans in uniform clay. Soil profile no. 1.

Initial situation

Wave Period = T = 16 sec and wave height = 25 m

$s_{u0} = 20\ kPa,\ k = 1.2\ kPa/m.$

Penetration depth for spudcan tip = 12.4 m

Safety against sliding (minimum) = $SFS_{\min}$ = 1.93

Safety against bearing capacity failure (minimum) = $SFS_{\min}$ = 0.91

The tables on the following three pages cover the results for the three jackup platform models as referenced above.

MODEL NO. 1

CASE: DEEP PENETRATION IN CLAY. SOIL PROFILE NO.1

Su0 =20KPa ZPfin = 8,45 m SFSmin = 1,98 SFBmin = 1,0 (BASIC CASE)

Case no.	Parameter	Value	Variation	SFS min	± %	SFB min	± %	Comments
1	Wave Height	18	+20%	1,51	-24	0,9	-10	
2		12	-20%	2,60	+31	1,07	+7	
3	Wave Period	17	+30%	1,90	-4	0,98	-2	
4		15	+15%	1,92	-3	0,99	-1	
5	Wave Period	11	-15%	2,01	+2	1,0	0	
6		9	-30%	2,74	38	1,08	+8	
7	Wind Speed	55	+10%	1,87	-6	0,97	-3	
8		45	-10%	2,09	+6	1,03	+3	
9	Current	1,1	+100%	1,62	-18	0,93	-7	
10		0,0	-100%	2,42	+22	1,06	+6	
11	Distance	37	+12%	2,01	+2	1,02	+2	
12	between legs.	29	-12%	1,93	-3	0,97	-3	
13	Platform	72	+20%	2,20	+11	1,0	0	ZPF = 10,6m +25%
14	weight.	48	-20%	1,77	-11	0,99	-1	ZPF = 6,3m -25%
15	Lever arm	14,7	+ 5%	1,98	0	0,99	-1	
16	for wind	9,3	- 5%	1,98	0	1,0	0	
17	Drag diameter	6,0	+20%	1,76	-11	0,96	-4	
18	for legs.	4,0	-20%	2,27	+15	1,04	+4	
19	Inertia diam.	2,4	+20%	1,94	-2	0,99	-1	
20	for legs.	1,6	-20%	2,0	+1	1,0	0	
21	EI Bending	96	+20%	1,99	+1	1,0	0	
22	stiff legs.	64	-20%	1,97	-1	1,0	0	
23	EA Axial	12	+20%	1,98	0	1,0	0	
24	stiff legs.	8	-20%	1,98	0	1,0	0	
25	GA Shear	24	+20%	1,99	+1	1,02	+2	
26	stiff legs.	16	-20%	1,97	-1	0,99	-1	
27	G-modulus	450 Su	+50%	1,98	0	1,0	0	
28	for soil.	150 Su	-50%	1,98	0	1,0	0	
29	DTIP for	1,5	+50%	1,98	0	1,0	0	
30	spud-can	0,5	-50%	1,98	0	1,0	0	
31	HSID for	2,03	+50%	2,16	+8	1,03	+3	ZPF = 8,11m -4%
32	spud-can	0,68	-50%	1,78	-10	0,96	-4	ZPF = 8,80m +4%
33	HCON for	0,75	+50%	2,02	+2	1,0	0	ZPF = 8,69m +3%
34	spud-can.	0,25	-50%	1,93	-3	0,99	-1	ZPF = 8,21m -3%
35	HTIP for	0,75	+50%	1,99	+1	1,0	0	ZPF = 8,69m +3%
36	spud-can	0,25	-50%	1,97	-1	1,0	0	ZPF = 8,21m -3%
37	DTOT for	12,0	+ 9%	1,97	-1	1,01	+1	ZPF = 5,85m -31%
38	spud-can	10,0	- 9%	2,03	+3	0,99	-1	ZPF = 11,8m +40%
39	Preload	21,6	+20%	2,03	+3	1,05	+5	ZPF = 9,09m +8%
40		14,4	-20%	1,93	-3	0,95	-5	ZPF = 7,81m -8%

MODEL NO. 2

CASE : DEEP PENETRATION IN CLAY. SOIL PROFILE NO.1

Su0 =30 KPa ZPfin = 6,95 m SFSmin = 2,12 SFBmin = 0,95 (BASIC CASE)

Case no.	Parameter	Value	Variation	SFS min	± %	SFB min	± %	Comments
1	Wave Height	24	+20%	1,64	-23	0,83	-13	
2		16	-20%	2,75	+30	1,05	+11	
3	Wave Period	19	+26,7%	2,01	-5	0,94	-1	
4		17	+13,3%	2,07	-2	0,94	-1	
5	Wave Period	13	-13,3%	2,16	+2	0,96	+1	
6		11	-26,7%	2,47	+17	1,0	+5	
7	Wind Speed	55	+10%	2,0	-6	0,91	-4	
8		45	-10%	2,23	+5	0,99	+4	
9	Current	1,1	+100%	1,73	-18	0,86	-9	
10		0,0	-100%	2,61	+23	1,03	+8	
11	Distance	41	+11%	2,14	+1	0,98	+3	
12	between legs.	33	-11%	2,09	-1	0,92	-3	
13	Platform	110	+22%	2,33	+10	0,97	+2	ZPF=9,41m +35%
14	weight.	70	-22%	1,90	-10	0,93	-2	ZPF=4,51 m ÷ 29%
15	Lever arm	16,4	+ 5%	2,12	0	0,94	-1	
16	for wind	7,6	- 5%	2,12	0	0,96	+1	
17	Drag diameter	6,0	+20%	1,88	-11	0,91	-4	
18	for legs.	4,0	-20%	2,41	+14	1,0	+5	
19	Inertia diam.	2,4	+20%	2,06	-3	0,95	0	
20	for legs.	1,6	-20%	2,15	1	0,95	0	
21	EI Bending	156	+20%	2,12	0	0,95	0	
22	stiff legs.	104	-20%	2,11	0	0,95	0	
23	EA Axial	15,6	+20%	2,12	0	0,95	0	
24	stiff legs.	10,4	-20%	2,12	0	0,95	0	
25	GA Shear	2,4	+20%	2,12	0	0,96	+1	
26	stiff legs.	1,6	-20%	2,12	0	0,94	-1	
27	G-modulus	450Su	+50%	2,12	0	0,95	0	
28	for soil.	150Su	-50%	2,12	0	0,95	0	
29	DTIP for	1,88	+50%	2,12	0	0,95	0	
30	spud-can	0,63	-50%	2,11	0	0,95	0	ZPF=6,95 0%
31	HSID for	2,4	+50%	2,30	+8	0,98	+3	ZPF=6,54 -6%
32	spud-can	0,8	-50%	1,91	-10	0,91	-4	ZPF=7,39 +6%
33	HCON for	0,75	+50%	2,15	+1	0,96	+1	ZPF=7,2 +4%
34	spud-can.	0,25	-50%	2,08	-2	0,95	0	ZPF=6,7 -4%
35	HTIP for	0,9	+50%	2,12	0	0,95	0	ZPF=7,24 +4%
36	spud-can	0,3	-50%	2,11	0	0,95	0	ZPF=6,7 -4%
37	DTOT for	14	+7,7%	2,15	+1	0,96	+1	ZPF=4,47 -36%
38	spud-can	12	-7,7%	2,31	+9	0,96	+1	ZPF=10,0 +44%
39	Preload	54	+50%	2,32	+9	1,08	+13	ZPF=9,16 +32%
40		9	-50%	1,89	-11	0,83	-13	ZPF=365 -47%

MODEL NO. 3

CASE: DEEP PENETRATION IN CLAY. SOIL PROFILE NO.1

Su0 =20 KPa ZPfin = 12,4 m SFSmin = 1,93 SFBmin = 0,91 (BASIC CASE)

Case no.	Parameter	Value	Variation	SFS min	± %	SFB min	± %	Comments
1	Wave Height	30	+20%	1,76	-9	0,87	-4	
2		20	-20%	2,52	+31	1,03	+13	
3	Wave Period	20	+25%	1,81	-6	0,9	-1	
4		18	+12,5%	1,87	-3	0,905	0	
5	Wave Period	14	-12,5%	2,04	+6	0,93	+2	
6		12	-25%	2,45	+27	1,0	+9,9	
7	Wind Speed	55	+10%	1,83	-5	0,88	-3,3	
8		45	-10%	2,03	+5	0,95	+4	
9	Current	1,1	+100%	1,55	-20	0,8	-12	
10		0,0	-100%	2,42	+25	1,0	+12	
11	Distance	52	+11%	1,95	+1	0,95	+4,4	
12	between legs.	42	-11%	1,91	-1	0,87	-4,4	
13	Platform	144	+20%	2,11	+9	0,93	+2	ZPF= 14,7 +19%
14	weight.	96	-20%	1,74	-10	0,89	-2	ZPF = 10,0 -19%
15	Lever arm	18,6	+5%	1,93	0	0,91	0	
16	for wind	5,0	-5%	1,93	0	0,92	1	
17	Drag diameter	6,0	+20%	1,71	-11	0,86	-6	
18	for legs.	4,0	-20%	2,21	+15	0,98	+8	
19	Inertia diam.	2,4	+20%	1,89	-2	0,91	0	
20	for legs.	1,6	-20%	1,95	+1,0	0,91	0	
21	EI Bending	2,4	+20%	1,93	0	0,91	0	
22	stiff legs.	1,6	-20%	1,93	0	0,92	0	
23	EA Axial	2,6	+20%	1,93	0	0,91	0	
24	stiff legs.	1,8	-20%	1,93	0	0,91	0	
25	GA Shear	2,4	+20%	1,93	0	0,92	0	
26	stiff legs.	1,6	-20%	1,93	0	0,90	0	
27	G-modulus	450Su	+50%	1,93	0	0,91	0	
28	for soil.	150Su	-50%	1,92	0	0,91	0	
29	DTIP for	2,63	+50%	1,93	0	0,91	0	
30	spud-can	0,88	-50%	1,93	0	0,91	0	
31	HSID for	3,0	+50%	2,11	+9	0,94	+3	ZPF= 11,9 -4%
32	spud-can	1,0	-50%	1,71	-11	0,88	-3	ZPF = 12,8 +3%
33	HCON for	1,13	+50%	2,0	+4	0,92	0	ZPF = 12,7 +2%
34	spud-can.	0,38	-50%	1,9	-2	0,91	0	ZPF = 12,0 -3%
35	HTIP for	1,5	+50%	1,94	0	0,92	0	ZPF = 12,9 +3%
36	spud-can	0,5	-50%	1,91	-1	0,91	0	ZPF = 11,9 -4%
37	DTOT for	16	+ 7%	1,90	-2	0,92	0	ZPF = 9,88 -20%
38	spud-can	14	- 7%	1,98	+3	0,91	0	ZPF = 15,4 +24%
39	Preload	90	+50%	2,18	+13	1,07	+18	ZPF = 17,3 +40%
40		30	-50%	1,67	-13	0,76	-17	ZPF = 9,4 -24%

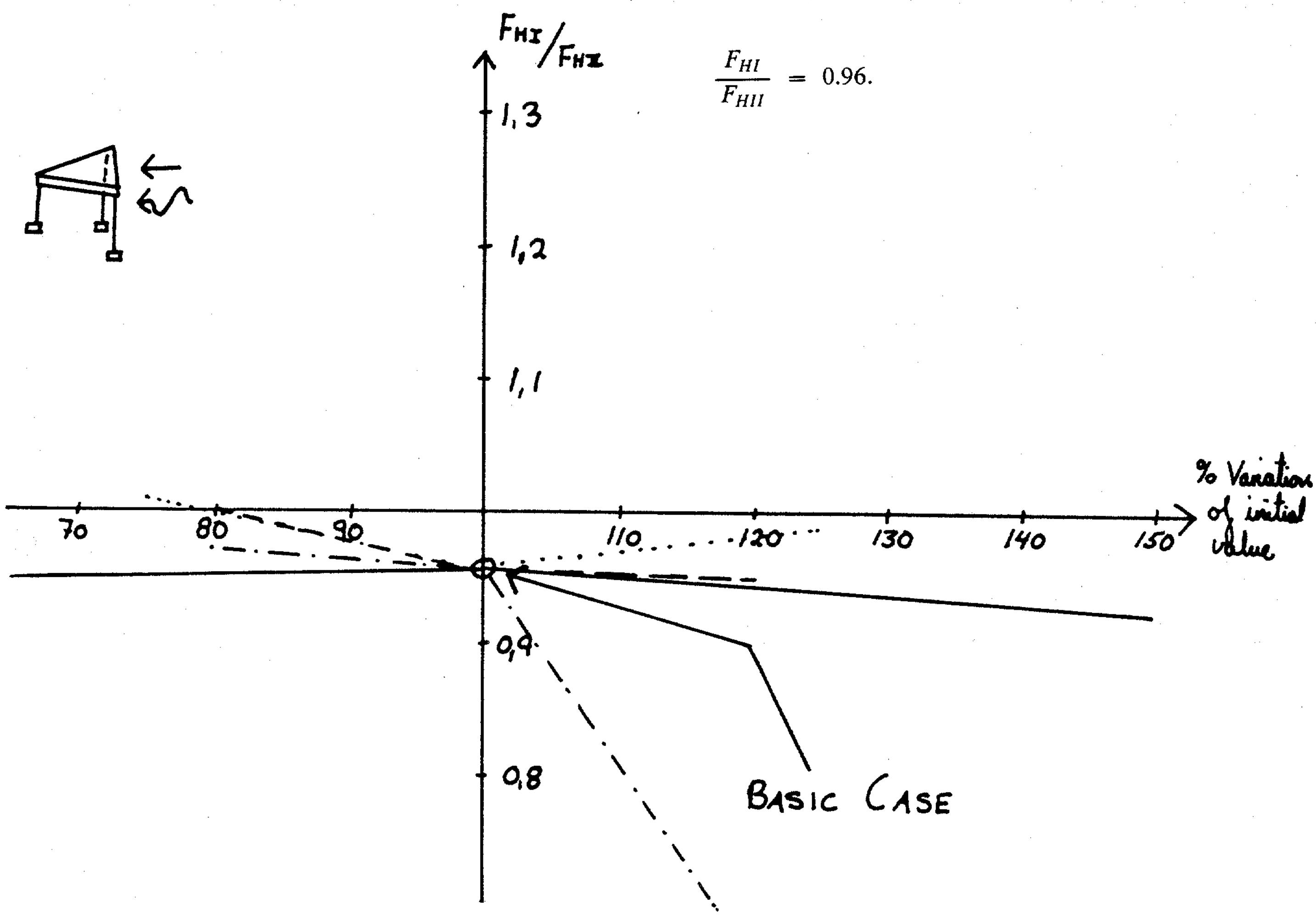
APPENDIX K:

Effect of variation in some important parameters for the distribution of horizontal forces for jackup platforms with independent legs.

The below figures show the relative effect of parameter variation. The results from the extreme points and value for basic case is connected with straight lines.

Basic Case: $F_H foreleg_{M=10} = 4290 kN$

$F_H aftleg_{M=10} = 4455 kN$



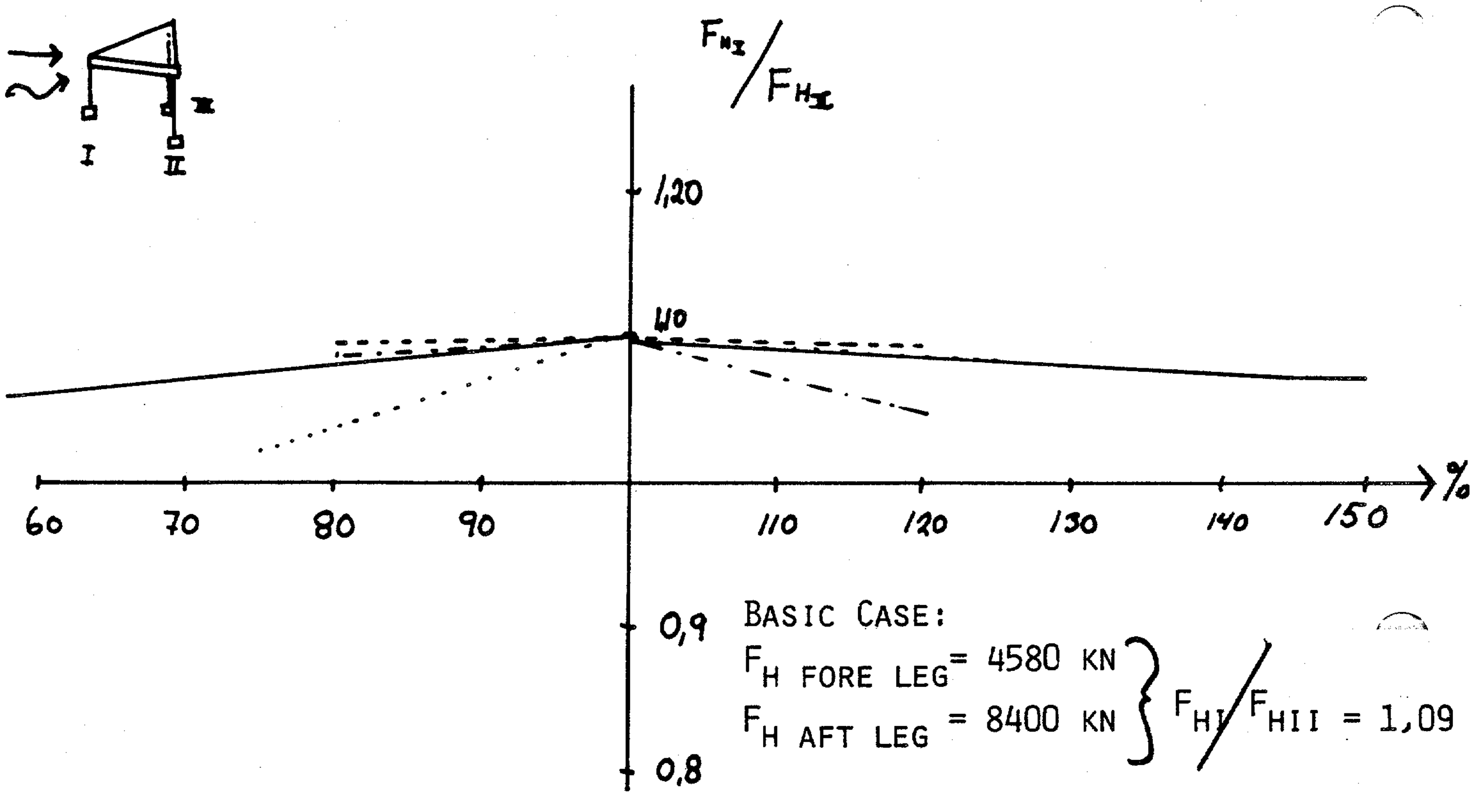
$$\frac{F_{HI}}{F_{HII}} = 0.96.$$

WAVE HEIGHT = – – – – – – –

CURRENT = ...

DRAG DIAMETER = ·–·–·–·–

PRELOAD = ————————



BASIC CASE:

F_H FORE LEG = 4580 KN

F_H AFT LEG = 8400 KN

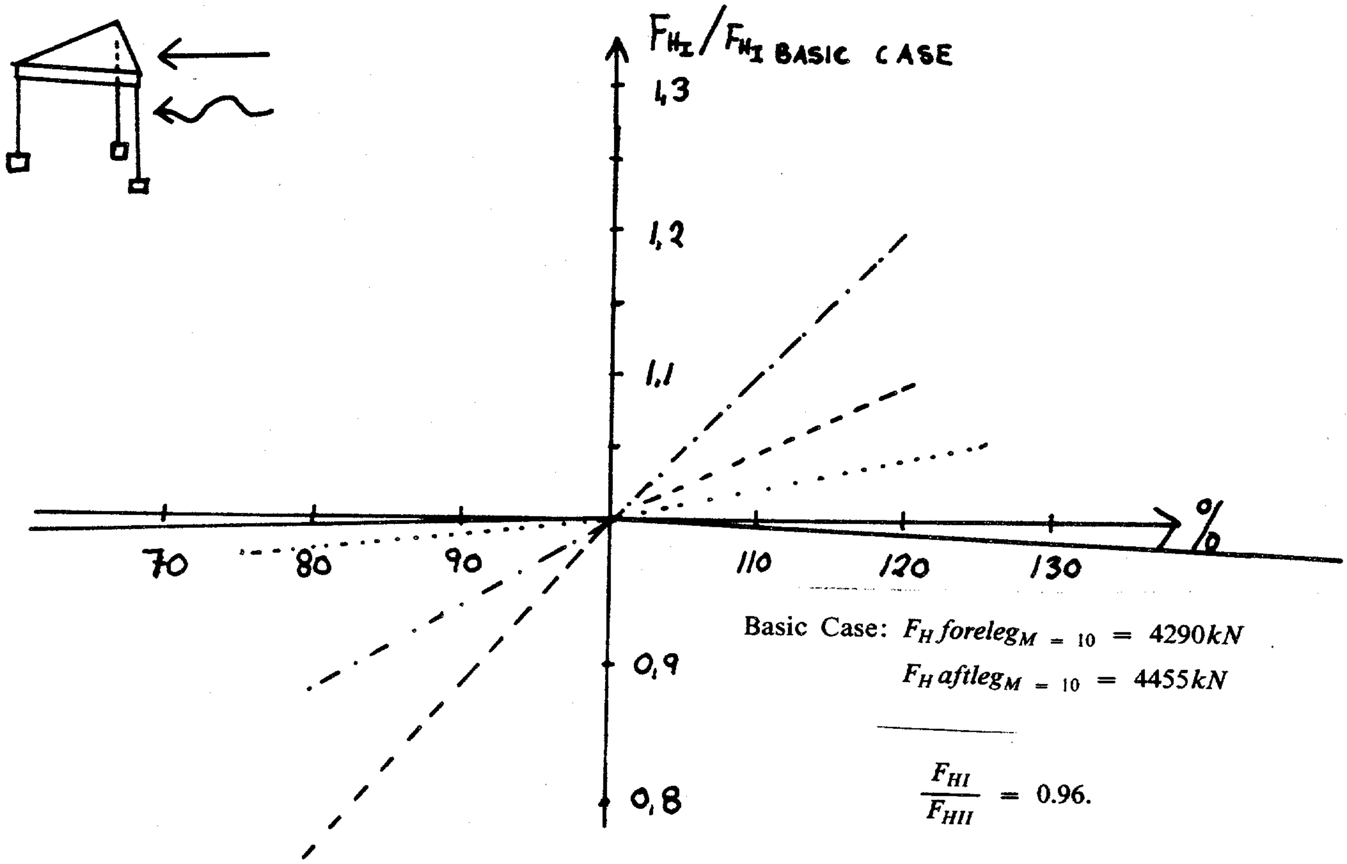
F_{HI}/F_{HII} = 1,09

WAVE HEIGHT = – – – – – – –

CURRENT = · · · · · · · · · · · ·

DRAG DIAMETER = – · – · – · – · –

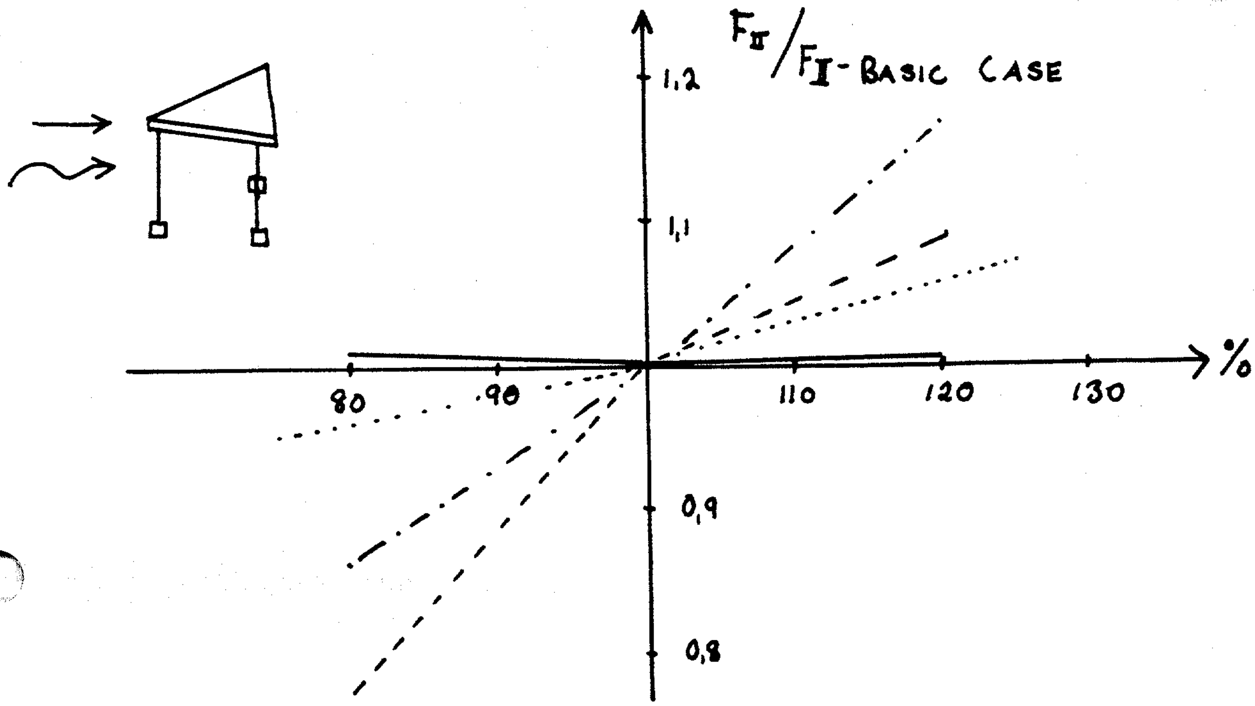
PRELOAD = ————————



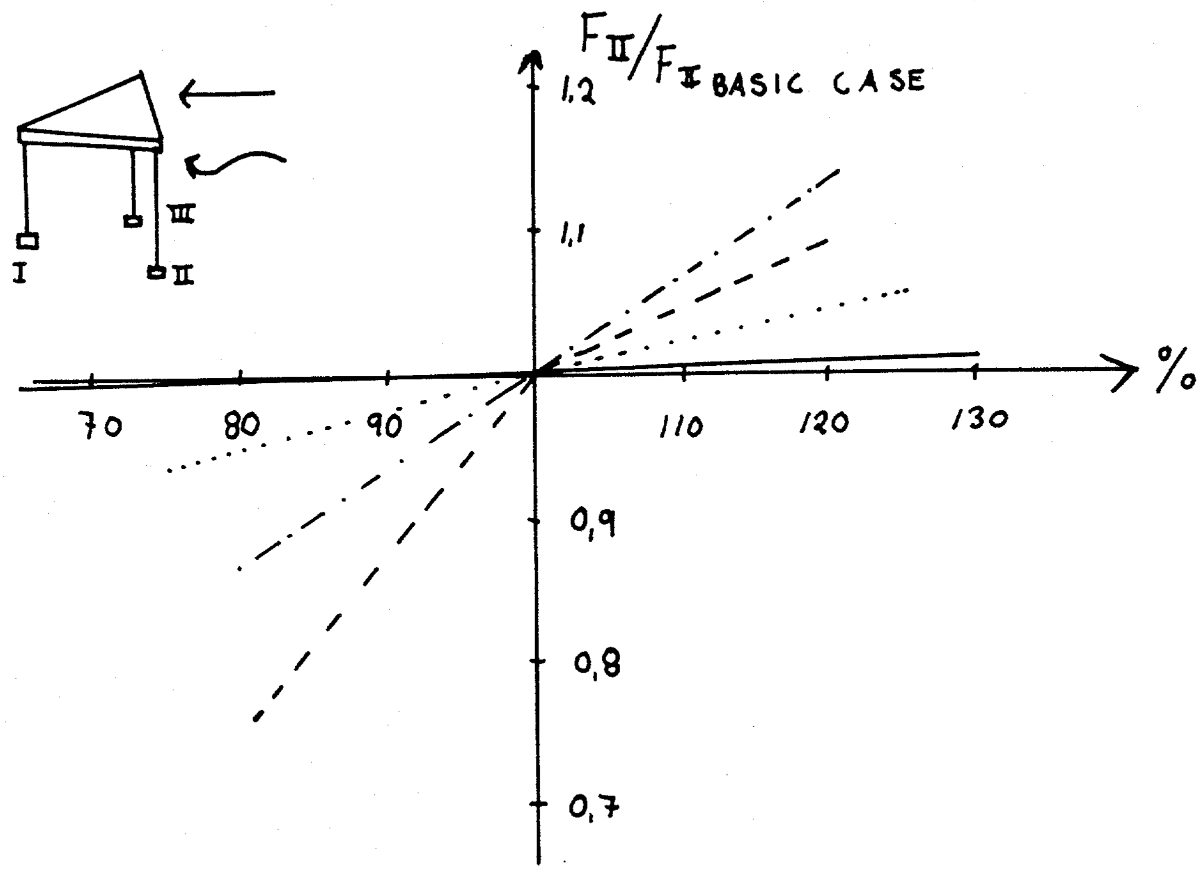
Basic Case: $F_H\,foreleg_{M\,=\,10} = 4290kN$

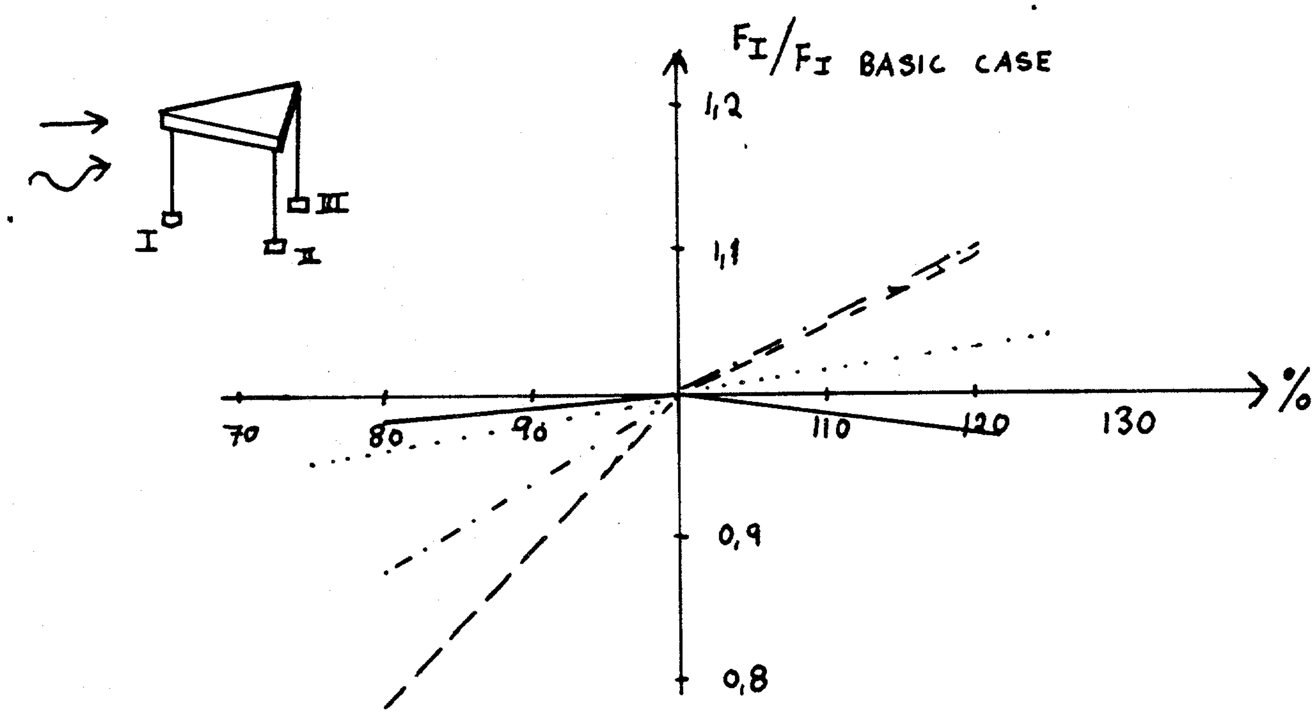
$F_H\,aftleg_{M\,=\,10} = 4455kN$

$$\frac{F_{HI}}{F_{HII}} = 0.96.$$

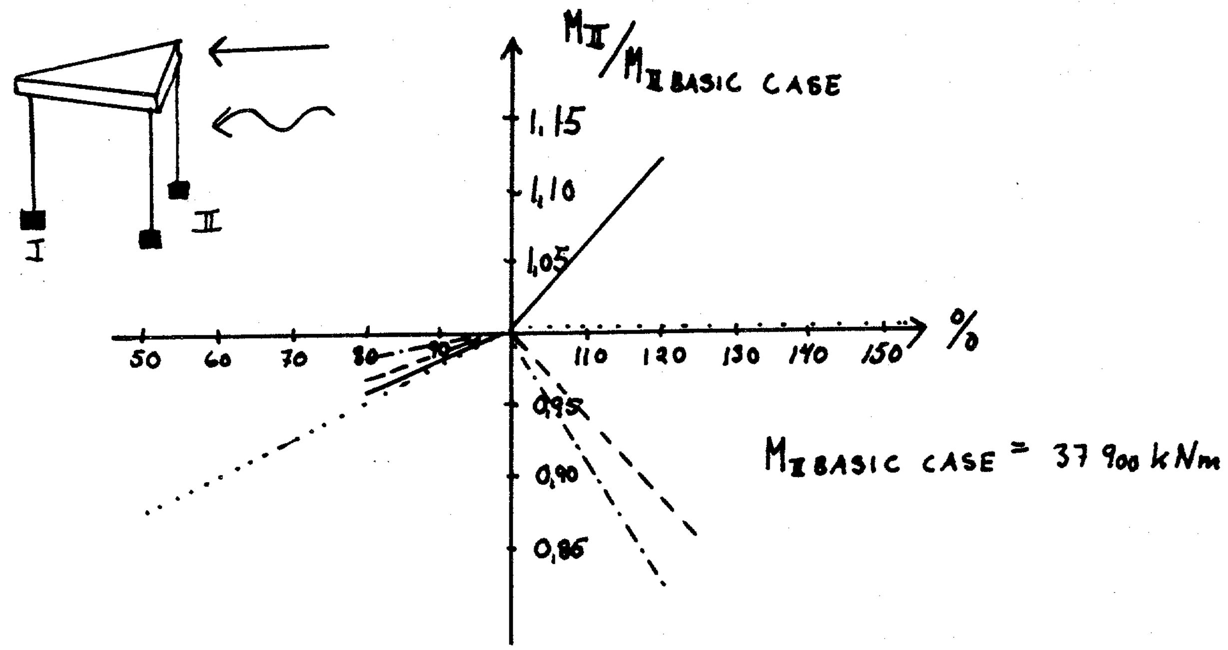


WAVE HEIGHT = — — — — — — —
CURRENT = · · · · · · · · · · · ·
DRAG DIAMETER = ·— · — · — · — · —
PRELOAD = ————————

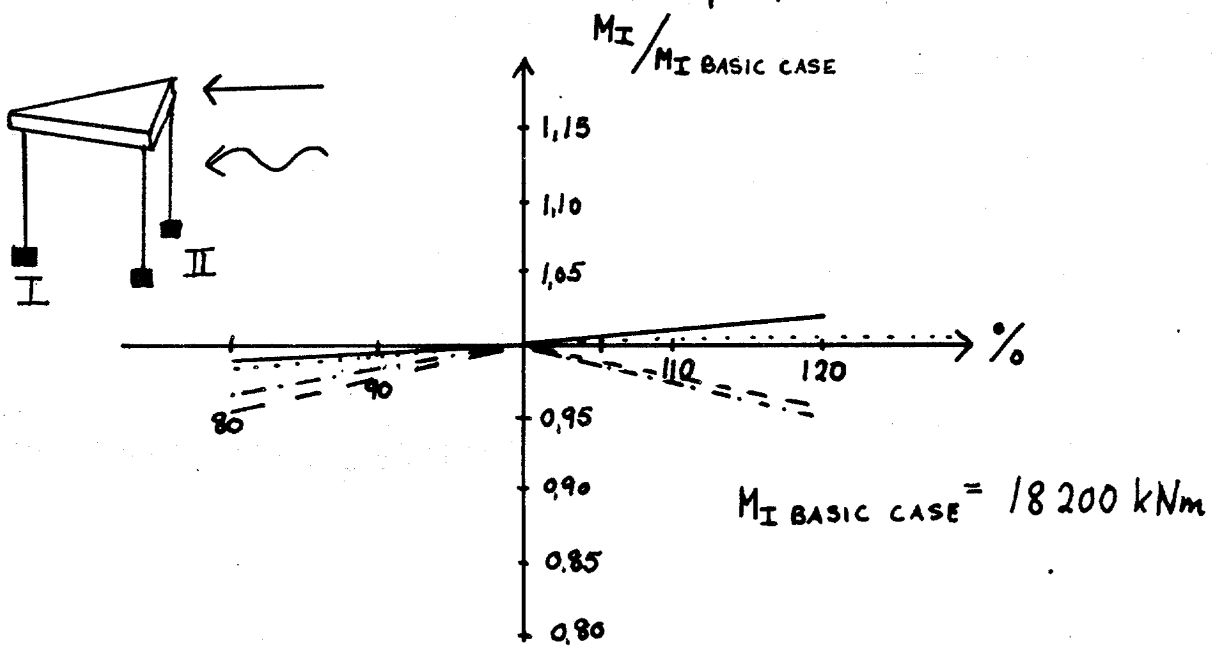




WAVE HEIGHT = — — — — — — —
CURRENT = ...
DRAG DIAMETER = .—.—.—.—
PRELOAD = ————————



MODEL NO. 1, Deep penetration

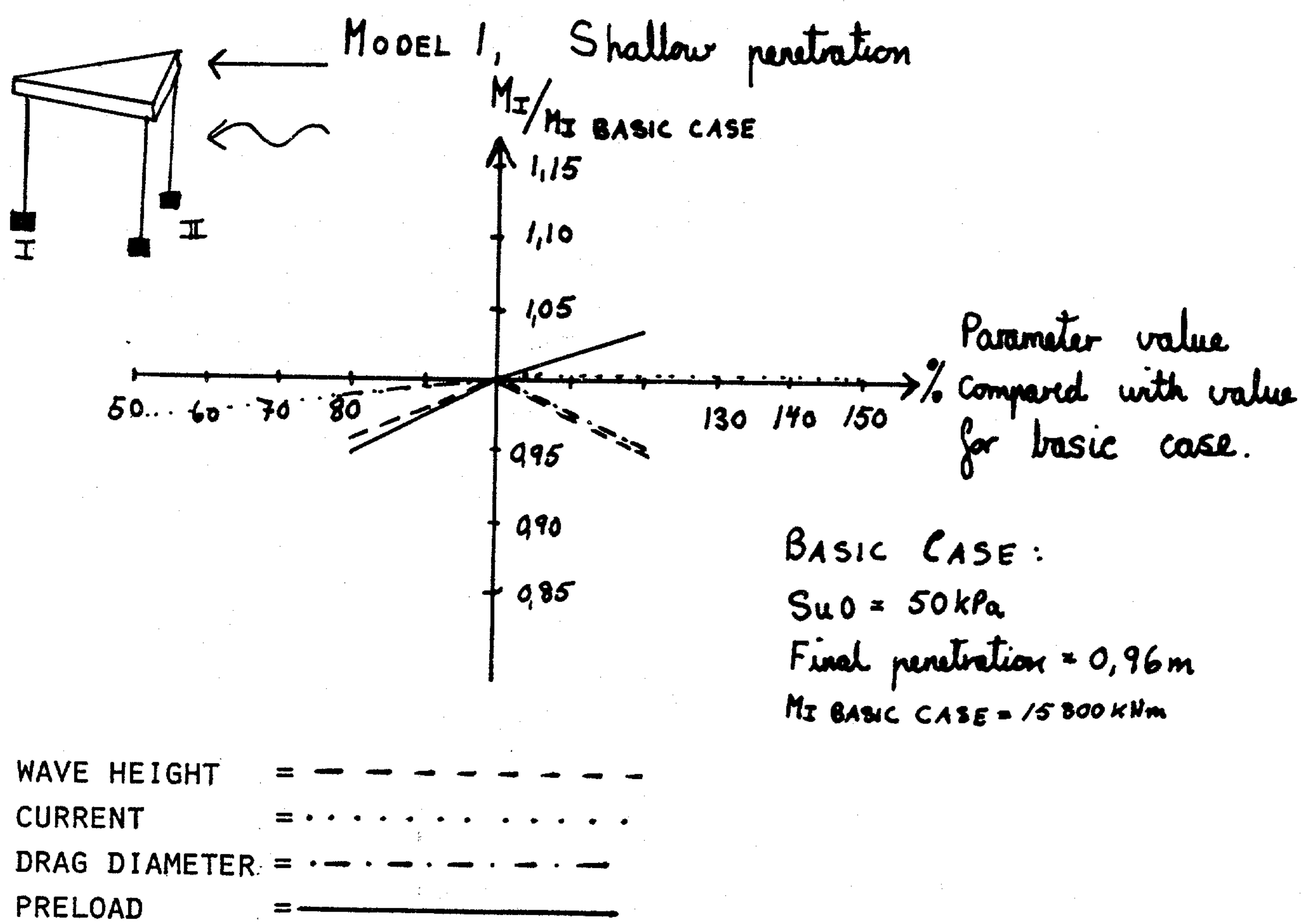


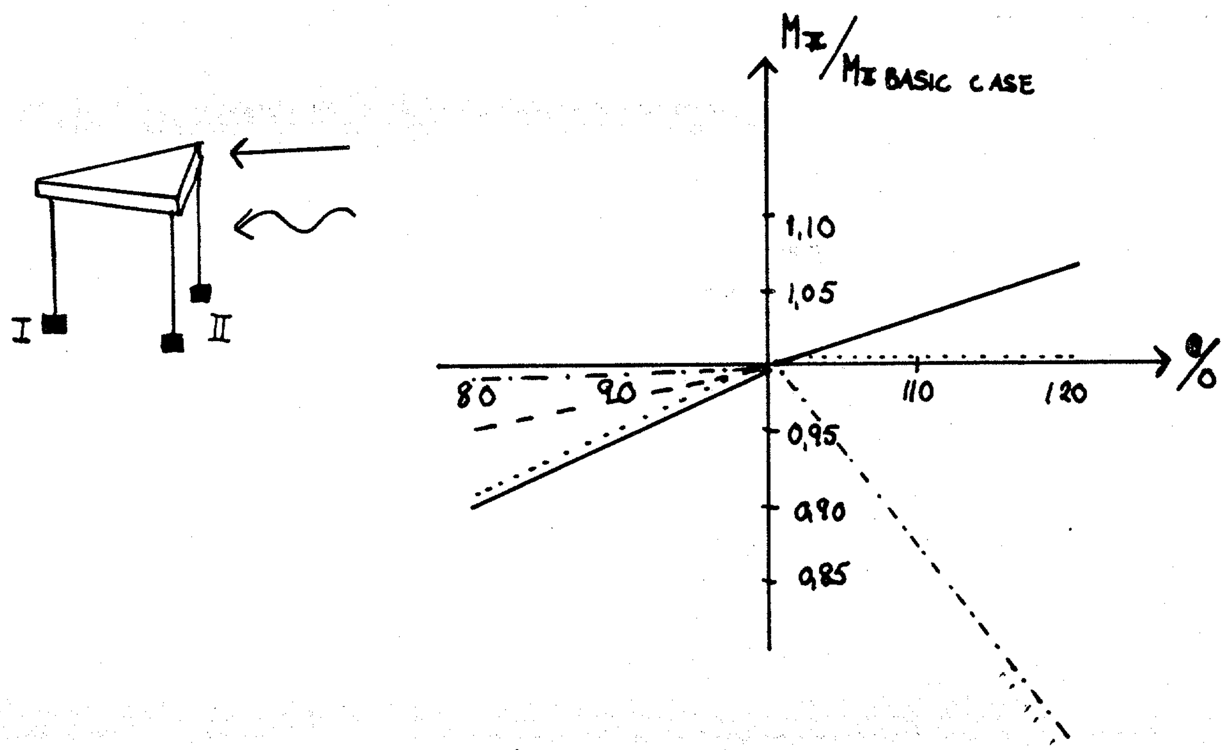
WAVE HEIGHT = – – – – – – – –
CURRENT = ...
DRAG DIAMETER = · — · — · — · —
PRELOAD = ————————

APPENDIX L:

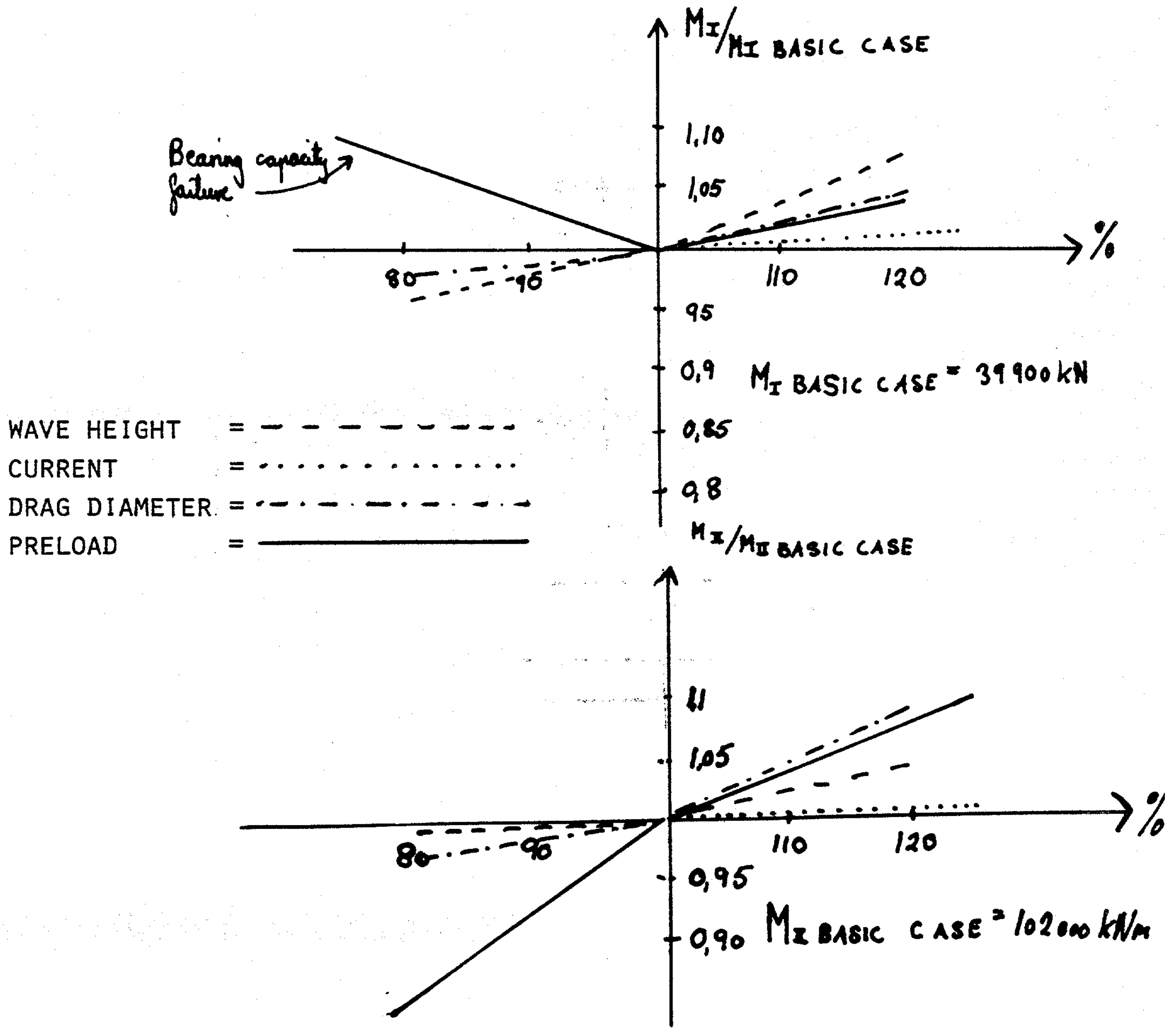
Effect of variation in some important parameters for moment capacity and moment distribution between legs.

The below figures show the relative effect of parameter variation. The results from the extreme points and value for basic case are connected by straight lines.

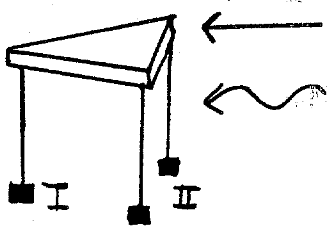


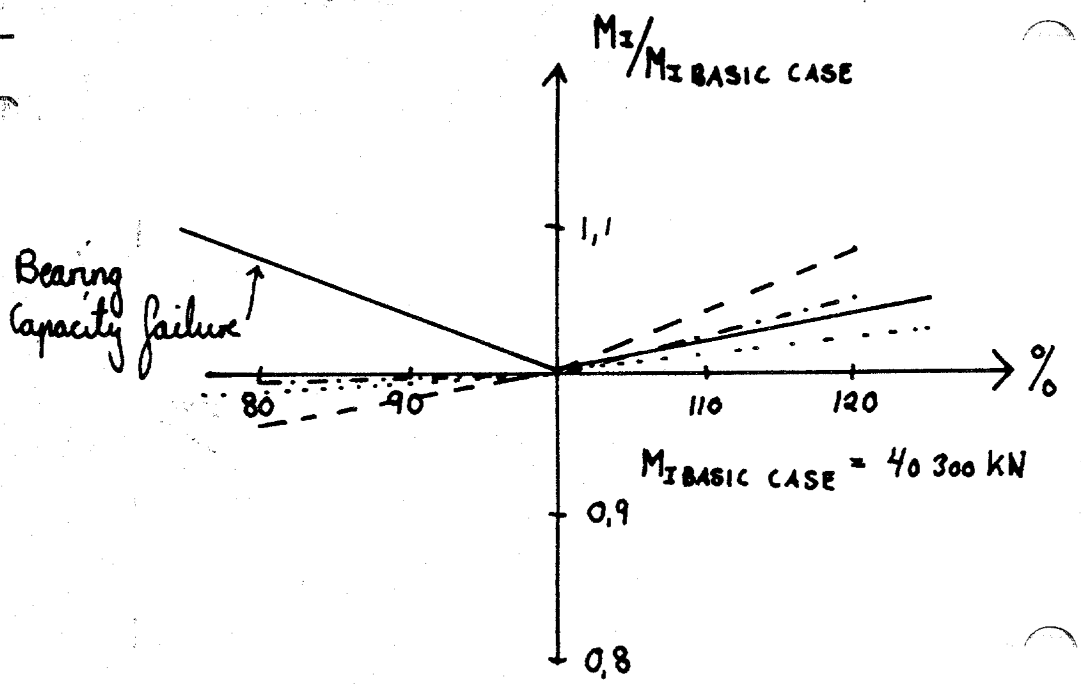


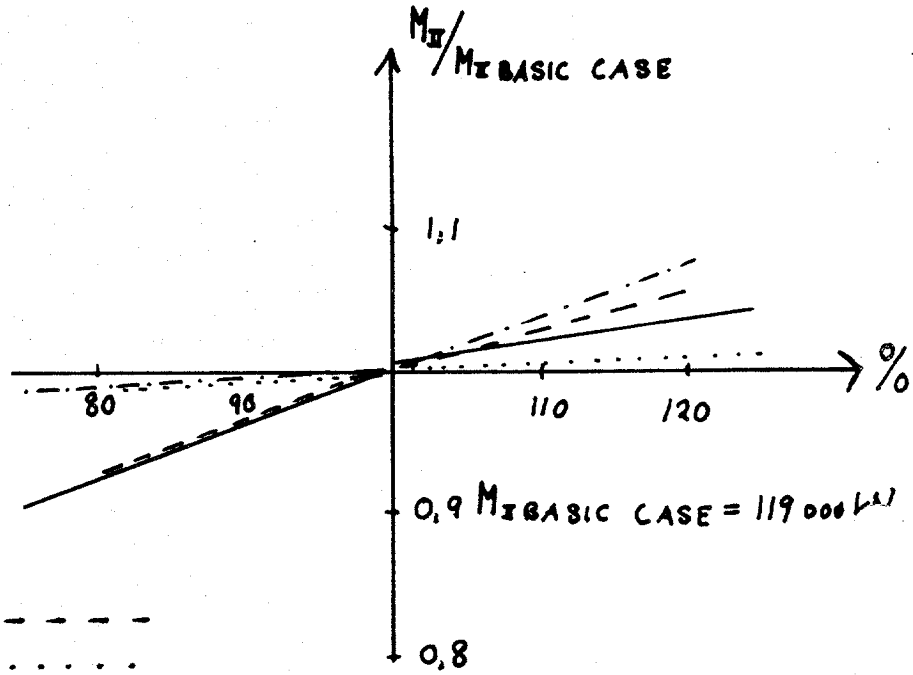
Model no. 2, Shallow penetration.



MODEL NO. 2 Deep penetration.

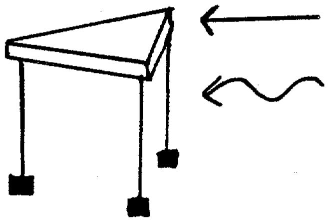


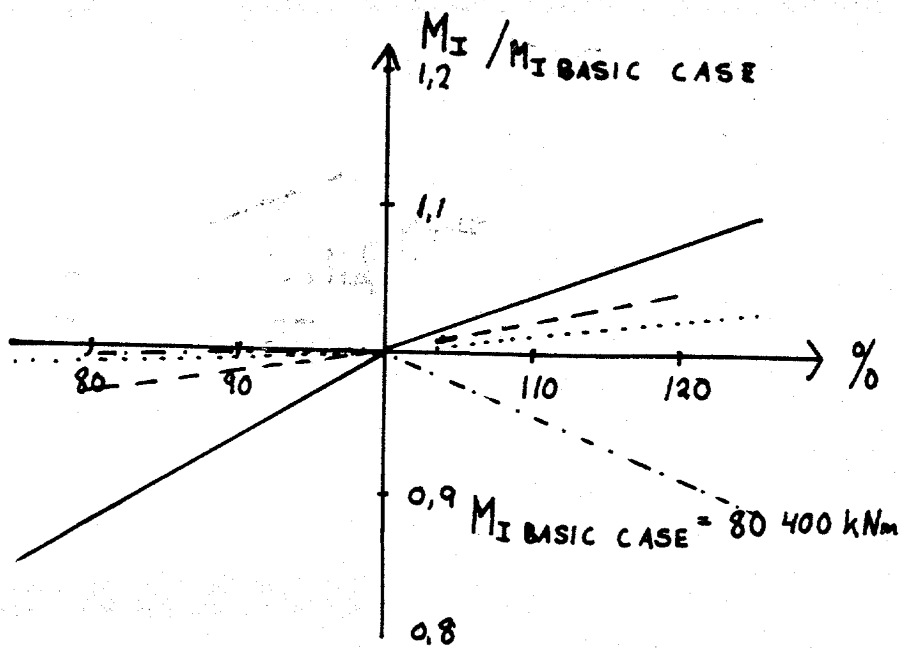




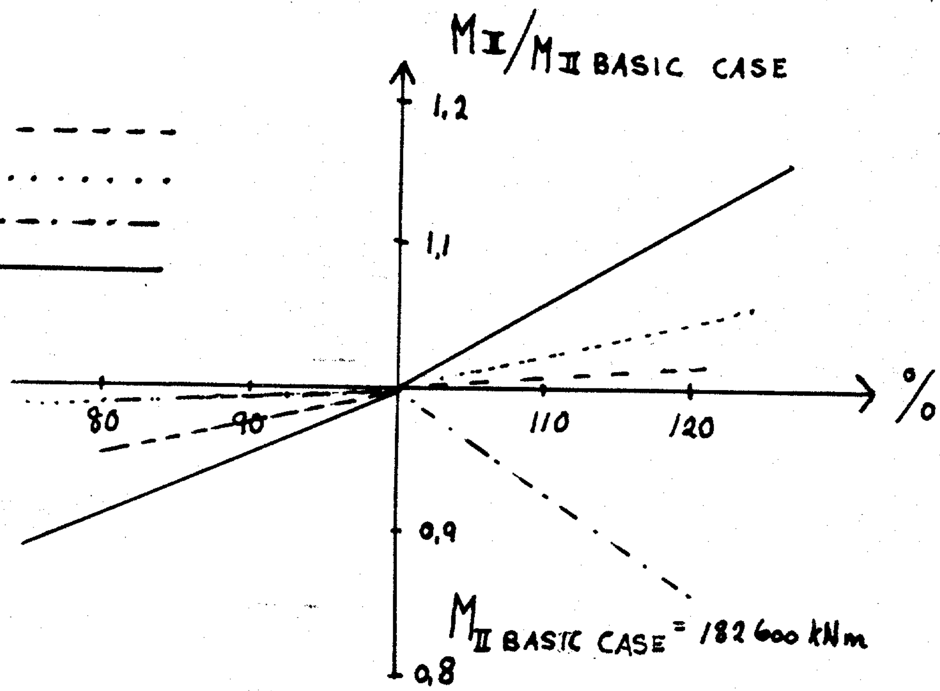
WAVE HEIGHT = — — — — — — — —
CURRENT = ...
DRAG DIAMETER = · — · — · — · —
PRELOAD = ————————

MODEL NO. 3. Shallow penetration

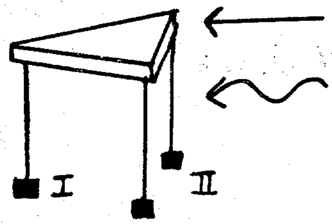


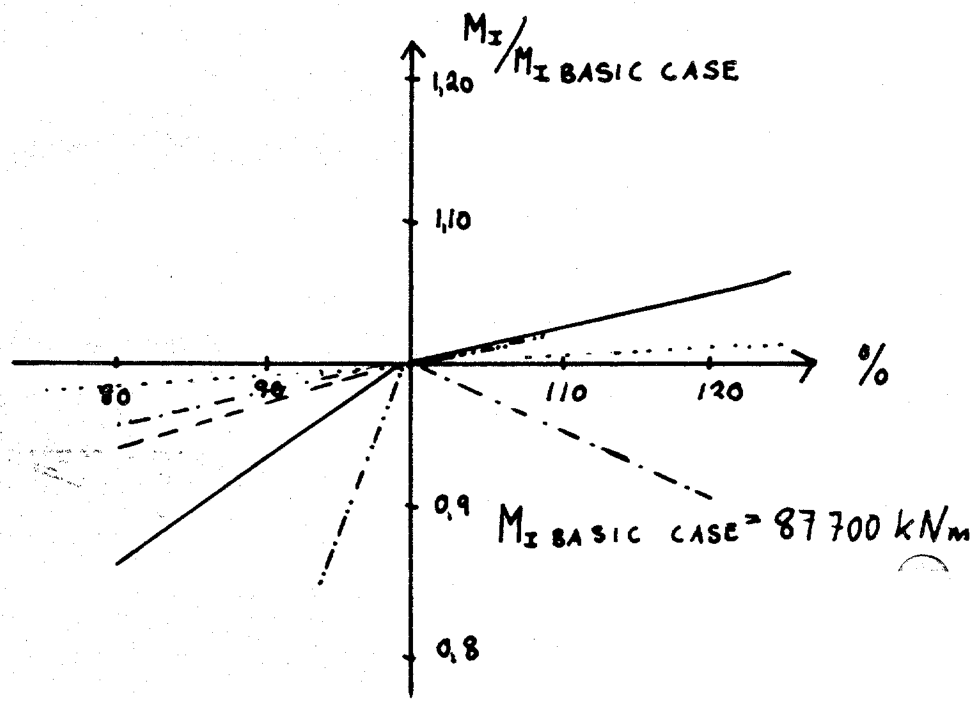


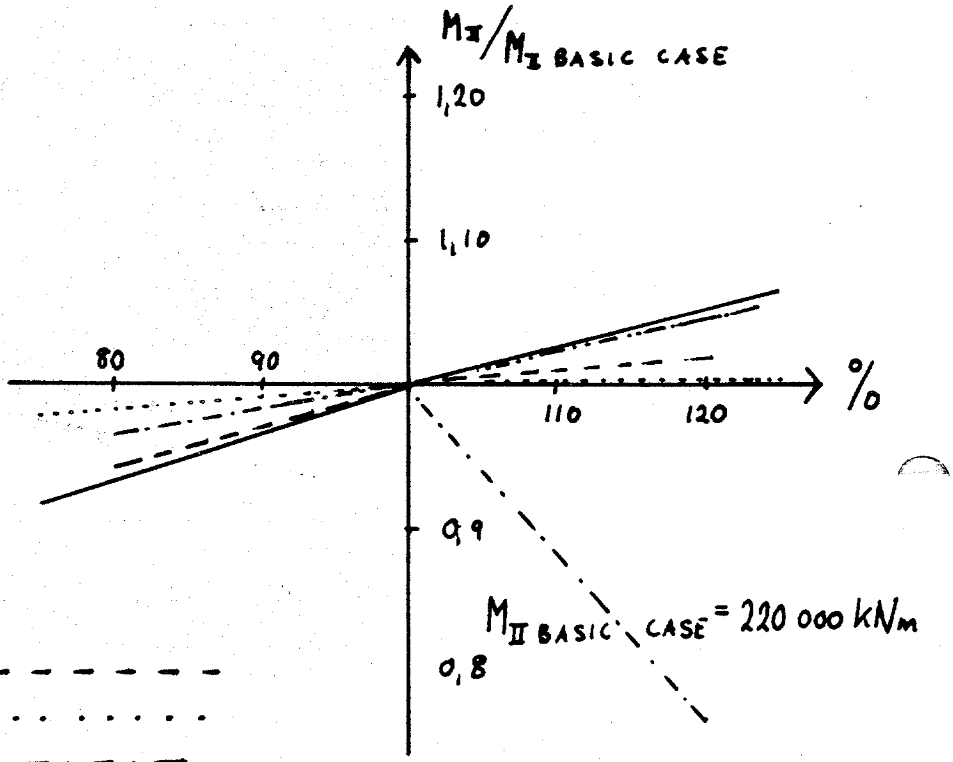
WAVE HEIGHT = - - - - - - - -
CURRENT = ...
DRAG DIAMETER = · — · — · — · —
PRELOAD = ————————



MODEL NO. 3 Deep penetration







WAVE HEIGHT = — — — — — — — —

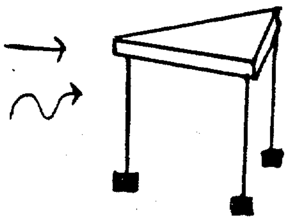
CURRENT = · · · · · · · · · · · ·

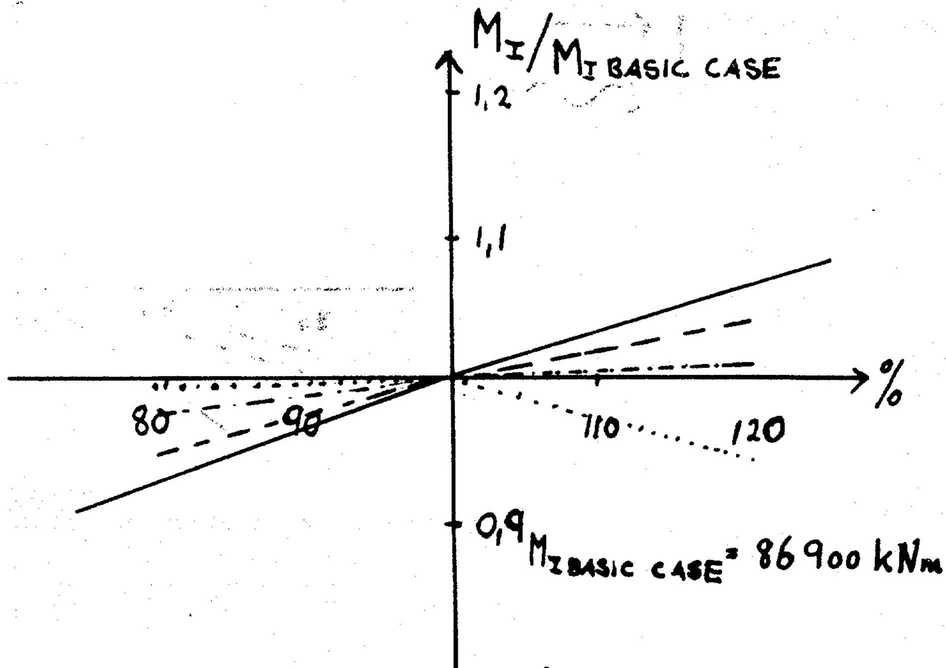
DRAG DIAMETER = ·—·—·—·—

PRELOAD = ————————

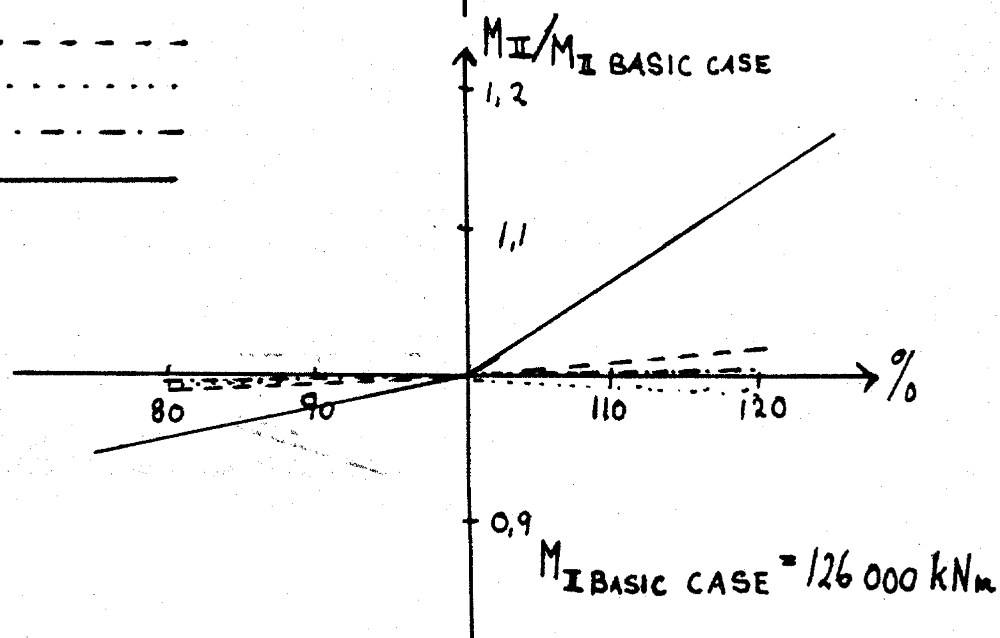
TOTAL SPUDCAN DIAMETER = —··—··—··—··—

MODEL NO. 3, shallow penetration





WAVE HEIGHT =
CURRENT =
DRAG DIAMETER =
PRELOAD =



APPENDIX M:

Effect of moment capacity on total vertical load.

This appendix presents the results from the parametric study concerning effect of variation in total spudcan diameter and preload capacity on embedment moment and total vertical load.

Table No. 1 gives an overview of the overturning moments that the different models are subjected to. Table No. 2 shows the vertical force and moment occuring for variation in total diameter and preload. Then finally Table No. 3 shows the reduction in vertical force because of fixity of the spudcans.

TABLE No. 1: Overturning moment from waves and wind.

MODEL NO.	PENETRATION	WIND	WAVE	TOTAL (MNm)
1	Shallow	103	96	199
1	Deep	117	126	243
2	Shallow	267	311	578
2	Deep	285	349	634
3	Shallow	531	755	1286
3	Deep	574	854	1428

TABLE No. 2: Vertical force and moment for the fore spudcan on a three-legged jackup platform subjected to environmental loads heading in from aft direction.

Model no. 1

Parameter (variation)	Basic Case	DTOT (+9%)	DTOT (-9%)	Preload (+20%)	Preload (-2(
Shallow penetration:					
F_{vI} (kN)	28200	28200	28200	28100	28300
M_I (kNm)	15800	15800	15500	16200	15100
Deep penetration:					
F_{vI} (kN)	29100	28600	29700	29100	29100
M_I (kNm)	18200	18400	17400	18500	17800

Model No. 2

Parameter (variation)	Basic Case	DTOT (+8%)	DTOT (-8%)	Preload (+50%)	Preload (-
Shallow penetration:					
F_{vI} (kN)	47300	47200	47800	47000	47400
M_I (kNm)	39900	40200	36200	43900	46900
Deep penetration:					
F_{vI} (kN)	48400	47600	49100	48600	47900
M_I (kNm)	40300	42900	38200	44200	48900

Model No. 3

Parameter (variation)	Basic Case	DTOT (+8%)	DTOT (-8%)	Preload (+50%)	Preload 0%
Shallow penetration:					
F_{vI} (kN)	69100	69000	68900	67600	70400
M_I (kNm)	80400	81900	82300	96000	55700
Deep penetration:					
F_{vI} (kN)	71100	70200	72300	71000	71500
M_I (kNm)	87700	88300	84200	98000	63600

TABLE No. 3: Overview of moment taken by the spudcans and corresponding reduction in total vertical load.

	M_{tot} by spudcans (MNm)	% of Tot.overt.mom.	F_v saved (%)
MODEL No. 1			
Shallow penetr.	53,7	27	6
Deep penetration	69,7	29	7
MODEL No. 2			
Shallow penetr.	141,9	25	8
Deep penetration	159,3	25	9
MODEL No. 3			
Shallow penetr.	262,4	20	8
Deep penetration	307,7	22	9

As Table No. 2 shows that the effect of improved moment capacity (by increased total spudcan diameter or preload) on reduction in vertical force is insignificant (less than 3% for the actual parameter variation). This is because the positive contribution of the increased preload weight or increased foundation diameter is reduced by negative effects. In the soil profile we have chosen in our parametric study, the positive and the negative effects were of the same magnitude.

For maximum overturning moment in the basic cases used in the parametric study (See Table No. 3), the vertical force from the most stressed leg to the soil is reduced 6 to 9% compared with a rig supported by pinned footings. The reduction is an effect of the moment capacity of the spudcans and it increases with deeper water and larger penetration depth.

To illustrate the effect of additional preload weight jackup model No. 3 located in soft clay may be considered for the case of deeply penetrated spudcans:

For 50% more preload weight (ballast) the penetration depth increases from 12.4m to 15.3m i.e. 23%, and thereby the total overturning moment is increased 3%. The moment taken by the spudcans increases by 18% while the vertical force in the legs are the same. Because the spudcans have penetrated deeper and the soil strength is linearly increasing with depth, the safety against bearing capacity failure is improved by 17% and the safety against sliding is increased by 13%. This means the displacements of both the footing and hull is reduced because of higher safety level (increased ultimate capacity) and thereby stiffer soil.

If the preload weight is reduced the safety level will also be reduced. This can lead to increasing penetration for the most stressed leg during survival condition, increasing overturning moment from hull displacement, increasing penetration for the fore leg and in some cases tilt of the entire structure.

An increased diameter does only affect the vertical load if the exposed foundation area is increased, which for our models means clay with undrained shear strength less than 50kPa. Wider spudcans will result in shallower penetration, increased footing moment (~ 1%) and a small reduction in vertical force (~ 1-2%). Spudcans with smaller diameter (~ 8%) produce a deeper penetration, a reduced embedment moment (~ 1%) and a slightly increase in vertical force (~ 1-2%). Variation in footing diameter only does not affect the safety against bearing capacity failure and therefore not the vertical displacements.

The safety against sliding is slightly increased by 2% for an 8% increase in diameter and decreased in the same magnitude for an 8% reduction of total diameter. If the diameter is too small the result may be too much penetration (too short legs) or pull-out problems.