CISM COURSES AND LECTURES NO. 283
INTERNATIONAL CENTRE FOR MECHANICAL SCIENCES

CASE HISTORIES IN OFFSHORE ENGINEERING

EDITED BY

G. MAIER

SPRINGER-VERLAG
WIEN GMBH

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While this book was in preparation the international scientific community had to mourn the untimely departure of CISM Rector Antoni Sawczuk. With his clear awareness of the potentialities of sound interactions and crossfertilization between mechanics and engineering, professor Sawczuk had expressed a particularly strong interest in the CISM courses in offshore engineering. Let this volume be one of the homages to the unfading memory of this outstanding scientist and engineer.

PREFACE

In little more than one decade, offshore engineering has developed from a relatively marginal area of unusual, highly specialized industrial activities, to a flourishing field in the main stream of today's technologies, with remarkable economical, social and political implications. All along this growth the involvement of applied mechanics has been crucial.

An offshore platform, together with its foundation, is required to withstand in service place a number of severe loading conditions (primarily, but not exclusively, wave forces) and other environmental effects such as corrosion. Moreover it must be constructed elsewhere, transported and installed, and all these phases further expand the variety of actions or interactions to be faced by the designer. Design must be closely related to the construction process and both are influenced by the peculiarities of the specific marine environment. Substantially the same remarks apply also to any major offshore pipeline. Ocean engineering is therefore interdisciplinary by its very nature and entails applications of various applied sciences, "in primis" of various branches of mechanics. As the resources of oceans and seas (primarily but not only oil and gas) are being exploited in deeper and deeper waters and more and more hostile environments, unusual or new problems arise, setting a real challenge to industry and to research and teaching institutions as well.

Awareness of this scenario gave origin to a series of CISM short courses on "Modern problems in offshore engineering" covering the main mechanics-related topics, namely: fluid loadings; structural mechanics problems; soil mechanics and foundation engineering problems; fracture and fatigue; corrosion and material properties; reinforced concrete; safety and monitoring; case histories.

The lecturers have been well-known experts from universities, industries and certification agencies: A. Agostoni, M. Baker, A. Berti, P. Bettess, C.A. Brebbia, P. Bristoll, J.B. Burland, R. Butterfield, R. Dahlberg, R. Eatock-Taylor, N. Ellis, Y. Eprim, V. Giardinieri, E. Gnone, K. Hoeg, P. Holmes, C. Kirk, T. Kvalstad, D. Lalli, J. Leonard, S. Maddox, R. Matteelli, T. Moan, R. Olson, A.C. Palmer, P. Pedeferri, J. Radon, G. Sebastiani, I.M. Smith, S. Venzi, L.C. Zaleski-Zamenhof, O.C. Zienkiewicz.

Assembling and ordering in books the large amount of technical expertise presented in these courses would have been a useful but probably too ambitious task. However, CISM thought it suitable and not renounceable to offer at least the edited text of lectures on

"Case Histories" included in the fifth (1983) course to an audience broader than the attendees of the courses. This decision, which led to the present volume, was motivated not only by the fact that a monograph on this subject appeared to be not available in the technical literature. In fact, engineering objectives and realizations are the unifying factors and the culminating phase of all the relating scientific investigations and analyses. A knowledge of them is particularly important in an inhomogeneous, multifaceted technological area such as offshore engineering. Here Newton's saying "exempla docent non minus quam praecepta" (examples are no less instructive than theories, methods and rules) sounds particularly appropriate.

Lessons from successful realizations and from meaningful failures are equally instructive. Those examined in this book are presented concisely, with inevitable differences in style, standpoint and focused aspects. They are not intended to cover the complete range of categories of offshore structures.This would be a laborious, hardly possible task in view of the large variety due to inventiveness and creativity stimulated by drastic novelties in engineering situations and not (or not yet) inhibited by consolidated practice and regulations. Nevertheless, it is hoped that this book may appeal to most people interested in offshore engineering (from the practising engineers to researchers in related fields). In fact its aim is to provide an up-to-date conspectus, through typical "exempla", of the present offshore engineering and of some future prospects in it.

I feel indebted to the CISM Rectors and Secretary General for having asked me to act as coordinator of the series of CISM courses on offshore engineering and as editor of this book. I want to thank here all the lecturers, in particular those who contributed to this monograph, for their valuable cooperation and the participants for many stimulating discussions. The enthusiasm and effectiveness of lecturers, the intrinsic fascination of the subjects, the friendly though austere atmosphere of CISM, made this series of CISM courses a memorable experience for me and, I trust, for many attendees. I hope that part of all this will be transmitted to the readers, along with the rich first-hand information contained in this book.

Giulio Maier

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THE PROGRESSIVE STRUCTURAL FAILURE OF THE ALEXANDER L. KIELLAND PLATFORM

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ABSTRACT

The "Alexander L. Kielland", a semi-submersible rig operating in the North Sea, capsized in March 1980. The causes of the accident were shown to be an initial weld defect which accelerated a fatigue failure of a brace. In rapid succession the remaining adjacent five braces connecting a column to the rest of the platform failed by overloading, resulting in loss of the column. This event implied loss of buoyancy, which caused heeling, progressive flooding and capsizing. The structural failure is discussed in this paper on the basis of studies of the fracture surfaces and materials properties and of fatigue and ultimate strength analyses with the emphasis on fatigue analyses, considering the S/N - as well as the fracture mechanics approach.

INTRODUCTION

"Alexander L. Kielland", a semisubmersible rig of the "Pentagone" type, capsized in the North Sea on March 27th 1980. Of 212 men on board, 89 were rescued. 123 men were killed in the accident.

As shown in Figure 1, the platform consisted of five pontoons (columns) which made up the major buoyancy elements. The columns and the deck on the top were supported by braces. According to the designers, diagonal and lower horizontal braces were not fitted between columns B and C, and C and D, because such braces would hinder the operations of supply ships and don't contribute significantly to a reduced stress level in other braces. While the upper and diagonal braces were designed to be watertight, all lower level horizontal braces were free flooding. Closed braces would have resulted in increased loads due to buoyancy, which would have necessitated the use of thicker plates in the affected areas.

Fig. 2 shows the starboard side of the platform with the location of the hydrophone, a positioning instrument, on brace D-6. The hydrophone support was located in a cut-out and welded to the brace by a double fillet weld, as shown in Fig. 2.

The rig, originally built as a drilling rig, was re-equipped and had been used as an accomodation platform in the North Sea during its entire period of operating starting in 1976. At the instant of the accident, the rig was moored adjacent to a jacket production platform at the Edda field. The weather was moderately poor, with wind velocities 16 to 20 m/s and significant wave-heights 6 to 8 m.

Shortly after "the Alexander L. Kielland" disaster a Commission of Inquiry was appointed through a Royal Decree to investigate the accident.

From the day of the accident it was clear to everybody that column D had separated from the rest of the structure by failure of six braces and that the rig capsized approximately 20 minutes later. To further clearify the physical causes of the accident the Commission initiated investigations of the broken column and the platform. The investigations undertaken included, among other things, chemical, mechanical and metallographic analyses, and fracture surface studies. In addition the Commission asked different specialists to carry out hydrodynamic, strength and stability calculations, and an analysis of the anchoring system.

The Commission took testimony from 117 witnesses and had at its disposal other transcripts, as well as documents and pertinent information from police hearings, drawings of the rig, letters of approval, certificates, and survey reports which related to the "Alexander L. Kielland" and the accident.

The Commission filed its report[1] one year after the accident. A summary of the initiating structural failure, the subsequent loss of buoyancy and stability; and the evacuation and rescue operations may be found in the paper[2].

In the present paper a detailed account of the structural failure leading to the loss of the column D will be given.

FAILURE INVESTIGATION

The loss of column D occured due to fractures of all the braces connecting this column to the rest of the structure. The locations of these fractures are shown in Fig. 2.

Possible sources of excessive loads acting on the structure and inadequate strength were investigated as potential causes of the structural failure. The basis for this work was among other things design documents, operation manuals, operation records, for instance of the environmental conditions at the various platform sites, witness statements and other documents, relating to possible abnormal loads (collisions, explosions, abnormal distribution of ballast, ..), measured scantlings on the platform, global appearance of fractures, macroscopic and microsopic investigations of the fracture surfaces, tests of the material strength of all failed members, and response and strength analyses. Experiences regarding failures and accidents in other plat-

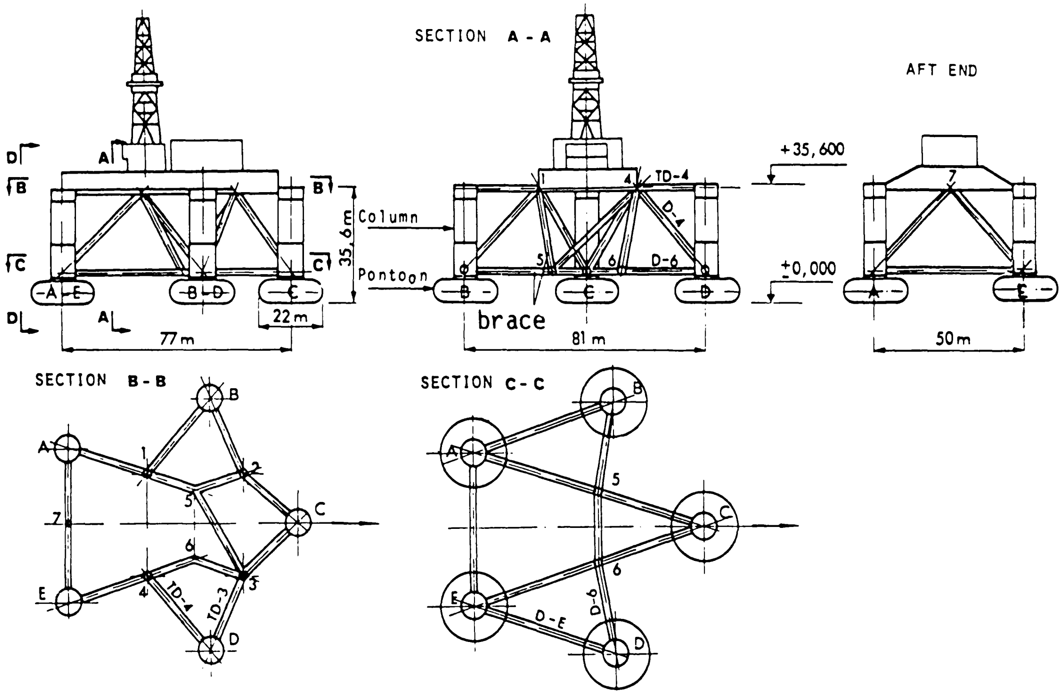


Fig. 1. Main arrangement of the "Alexander L. Kielland".

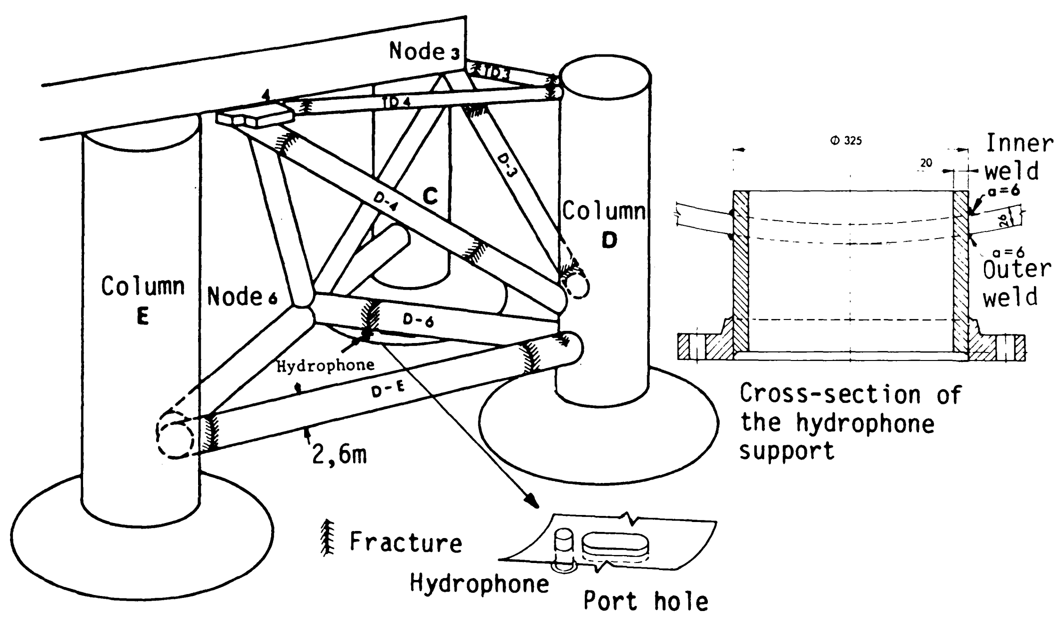


Fig. 2. Locations of fractures of the braces connected to the column D.

forms were also analysed. In this connection it is noted that the D-leg was accessible already a few days after the accident. Later the platform in an inverted condition and other pieces of failed braces became available for inspections, etc.

The Commission concluded that the structural failure had developed along the following course:

a) Failure of the D6 brace, due to a fatigue crack initiated in the fillet welds connecting the hydrophone support to the brace. The initiation and early growth of the crack had been strongly enhanced because of pre-existing cracks in the fillet welds between the hydrophone support and the brace. After initiation the fatigue crack had grown through the cross-section of the brace from both sides of the hydrophone support. Already the first examination of the brace stub on column D a few days after the accident, revealed the classical macroscopic signs of fatigue failure: A sharp, clean fracture with very little plastic deformation and a clear beach mark pattern emerging from the hydrophone penetration.

b) Final fracture of the D6 brace, mainly as brittle fracture.

c) Fracture of remaining braces connecting the column D to the rest of the structure, mainly as ductile overload failure. Stages b) and c) followed each other within seconds.
The braces with two fractures (see Fig. 2) most likely had the first fracture at the location near the column D. The second failure was due to the hydrodynamic loads on the cantilever remaining or an impact load by a possible contact of the cantilever with the seafloor during the capsizing.

The subsequent sections of this paper deal with the following aspects of the failure investigations:

- fracture surfaces
- material properties
- global load effects in the platform
- nominal stresses in the hydrophone support of brace D6
- the fatigue failure of brace D6
- the progressive failure of braces

The treatment is primarily confined to the physical aspects of the failure. However, the influence of human aspects relating to the design, fabrication and operation is also touched upon.

The lessons taught about design principles by this accident are also briefly outlined.

FRACTURE SURFACES

The fatigue fracture of brace D6.

Detailed measurements confirmed the impression gained at the preliminary examination of the fracture of D6, namely that very little plastic deformation was involved. Changes in global geometry were minor as indicated by diameter measurements; the largest deviation from the average outside diameter of 2.6 m was 37 mm or 1.4%. Reduction of area (lateral contraction) around the circumference of the fracture was also relatively minor except in the area of final rupture.

The fatigue fracture showed two distinct initiation points, one (referred to as point I), at the outer fillet weld of the hydrophone support; the other (II) at the inner weld (see sketch in Figs. 3 and 5). For about 60 to 100 mm from these points the surface was smooth and flat with a distinct beach mark pattern.

Farther away bands of a coarser, fibrous surface texture were observed indicating that the crack had propagated alternately by fatigue and tearing. From about 200 to 300 mm the surface topography was characteristic of ductile tearing, but distinct stop lines were visible. At distances 500 to 600 mm from the hydrophone tube the first signs of shear lip formation were present. The last stop marks were observed at 3120 mm and 1730 mm from points I and II, respectively.

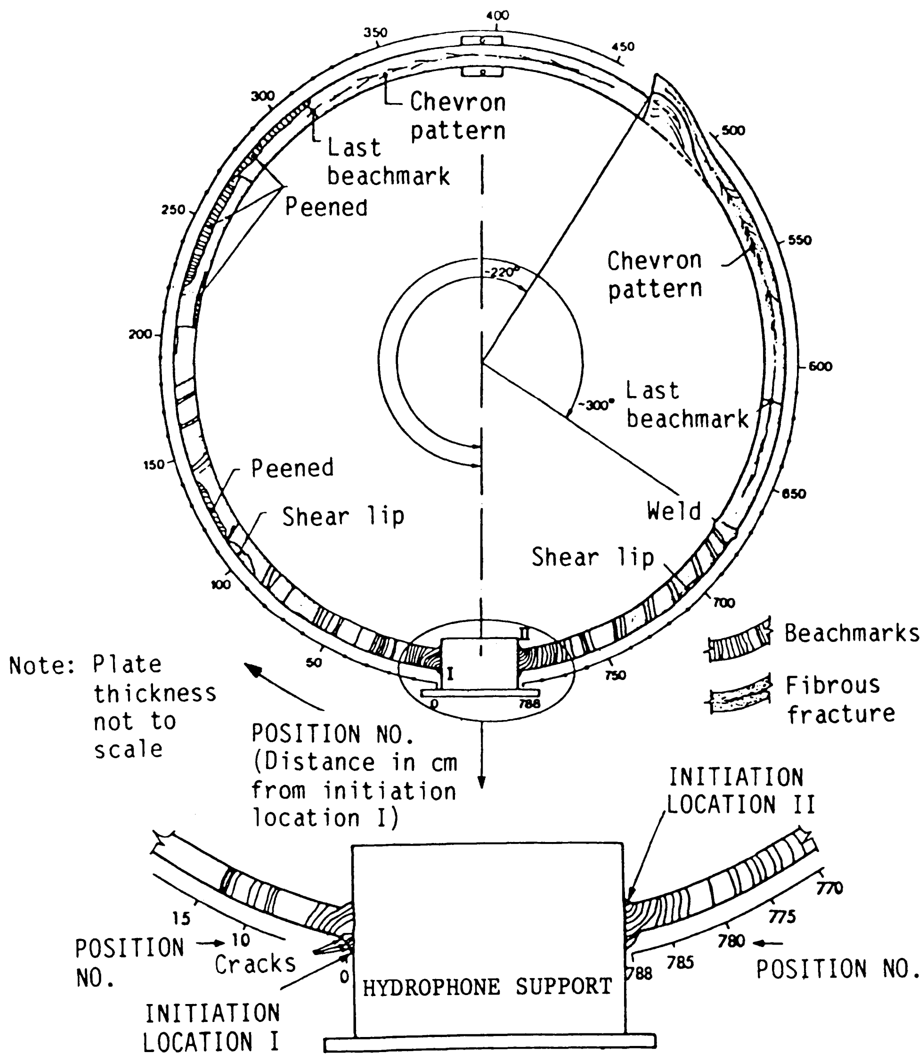


Fig. 3. Sketch of the fracture surface on brace D6.

Final fracture thus involved about 1/3 of the circumference. From the last stop marks to the lip that indicates final rupture a rather indistinct pattern resembling chevron or herringbone markings were observed.

Fractographic studies by electron microscope revealed patches of striations near the initiation points. However, the random nature of the wave loading made an attempt of predicting the macroscopic crack growth rate based on these observations, unsuccessful.

Fracture of fillet welds around the hydrophone support.

While the fatigue failure of brace D6 triggered the structural collapse of "Alexander L. Kielland", an examination of the hydrophone support indicated that parts of the fractures around the circumference of the hydrophone support were much older than the D6 fracture. This conclusion was based on observations of extensive corrosion attacks on parts of the fracture surface. The double fillet weld around the hydrophone support was cracked for more than 3/4 of the circumference. The type and extent of cracking is shown schematically in Fig.4.

Some areas of the fracture surface showed indications of lamellar tearing, mostly 1 mm below the surface of the tube plate in the heat affected zone. These fractures could have occurred during the welding operation or shortly thereafter. Such cracks are consistent with the poor through thickness properties of the material in the hydrophone support.

A thick layer of corrosion products and traces of biological activity on the fracture surfaces in the quadrants I and II (pos. 0 to 26 and 53 to 77, in Fig.4), indicates that fracturing in these areas must have taken place much earlier than in the rest of the weld. The age of the fractures on the inside fillet weld between pos.59 and 66 was substantiated by the discovery of paint on the fracture surfaces as shown in Fig. 6, which proved that this weld was cracked over a length of at least 70 mm during the fabrication.

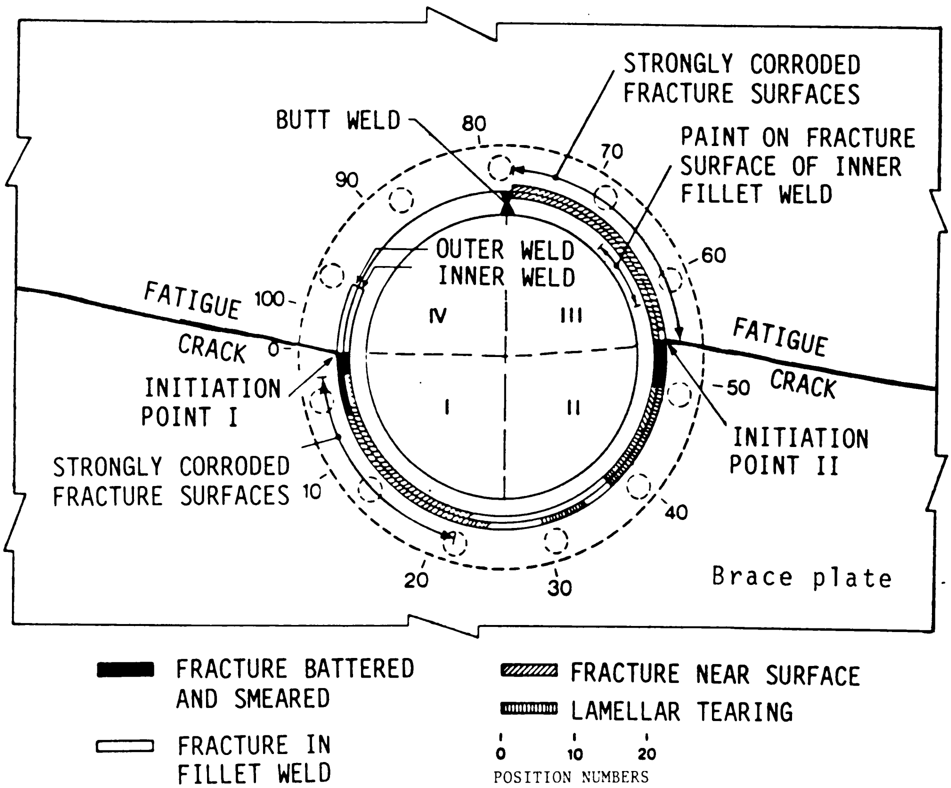


Fig. 4. Observations on the fracture surfaces between the hydrophone support and brace D6.

The observations mentioned above together with the local appearance of the circumferential cracks of the hydrophone support at the locations for intitiation of the main cracks in brace D6, indicate that the latter cracks most likely started to grow when the fillet welds were fractured in the quadrants I and III.

The double fillet weld was partly intact in most of the quadrant IV. While the weld generally satisfy the specified minimum weld throat thickness of 6 mm, the weld penetration into the hydrophone tube material was inadequate. The low penetration depth increases the probability of defects; additionally the susceptibility to lamellar tearing is increased. The shape of the weld beads was unsatisfactiory with weld flank angles ut to 90 degrees.

Fig. 5 Initiation point for the fatigue cracks in the brace D6.

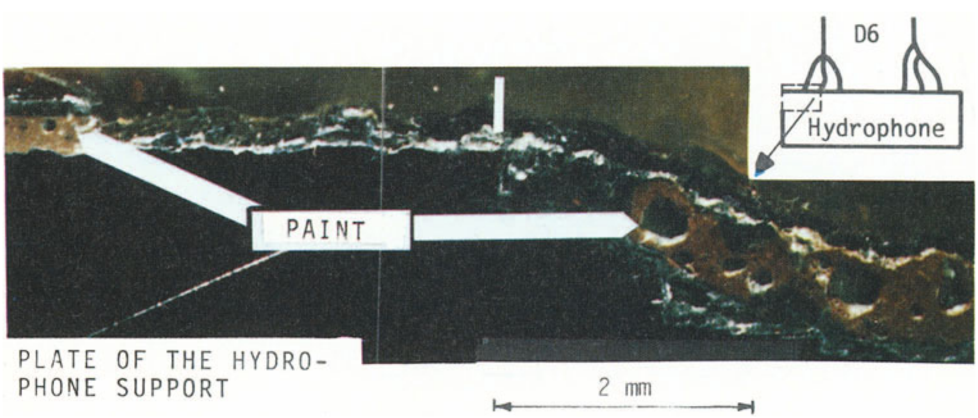


Fig. 6 Cross-section of the hydrophone support plate at pos.65 (See Fig. 4). Paint residues on the fracture surface.

The fracture surfaces near the initiation points of the fatigue crack in brace D6 were heavily corroded and were strongly deformed by rubbing and battering.

Overload fractures.

Except for the fatigue failure of brace D6 all other fractures in the braces connected to column D were ductile overload fractures with a fibrous surface appearance. Most of these fractures were relatively flat with small or non-existing shear lips. The amount of deformation (lateral contraction) was from 5 to 15% except in areas where the material apparently had been embrittled by local plastic compression prior to tensile overloading; in these areas little or no contraction could be measured.

MATERIAL PROPERTIES

The material used in the braces and hydrophone support was a carbon-manganese structural steel with specified minimum yield strength of 355 MPa. After the accident extensive materials testing was performed, with specimens taken from the braces and the hydrophone support. Essentially, the testing confirmed that the material properties were within specification regarding chemical composition, in-plate tensile properties, Charpy notch toughness, etc.

However, through thickness ductility, which was not specified, was poor. The area reduction measured in the through thickness testing for the brace D6 was found to vary between 6 and 12%; for the hydrophone support between 1 and 7%. The reason for this was apparently slag (mainly manganese sulfide) in a banded ferrite and pearlite microstructure.

Codes commonly require an area reduction of 15 to 30% to avoid lamellar tearing. In the design specifications for the "Alexander L.Kielland" the hydrophone supports were not considered to be part of the primary structure and thus no material specification given.

The ultimate strength in the through thickness direction of the hydrophone support was 398 MPa and hence below the in-plane tensile strength (490-608 MPa) specified for the brace material.

The low strength and ductility of the hydrophone tube material in combination with the poor welding caused partial cracking of the fillet welds shortly after construction, before the platform was put into service. The high stresses due to external forces on the platform contributed to a further cracking early in the service life of the platform.

GLOBAL LOAD EFFECTS

Three dimensional frame model of the platform.

Fig. 7 the shows space frame model used in the global analysis. Each brace and column was divided into 5 to 10 beam elements in order to account for variations in the plate thickness and stiffener arrangement along the braces and columns. The structural joints which are stiffened by bulkheads were modelled as completely rigid elements. It is seen in Fig. 7 that the deck structure including the deck dirders was modelled by relatively few beams. However, the stiffness properties of these beams were chosen to reflect the overall stiffness of the deck structure.

The model, being a free body, had to be fixed in space by boundary conditions. This was obtaiend by fixing the points 1, 2, and 3 in the deck. Reaction forces due to lack of equilibrium would thus have a negligible influence on the computed stresses in the lower braces.

The nominal stresses are calculated according to beam theory.

The computer program NV337, C-version, in the program system SESAM 69[3] was used for the structural analysis.

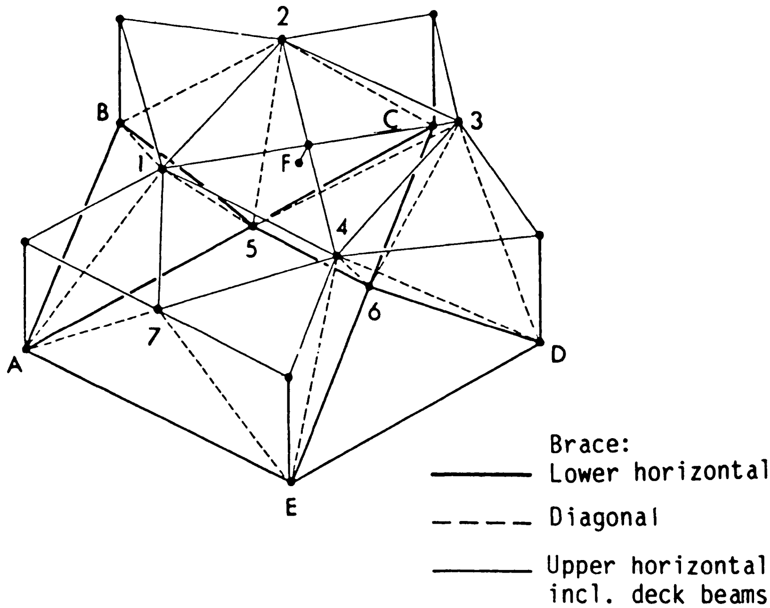


Fig. 7. Space frame model of the platform.

Load effects due to functional loading in still water.

Still water stresses were computed for the mean condition of the intact platform during its operational life. Effects of variations in deck load, ballasting conditions and draught were also investigated using the same model. Selected results are displayed in Table 1.

Load effects due to wave loading.

The dominating environmental load for the Pentagone structure is the wave loading. Wind and current loading primarily result in forces which are transformed to the platform via the mooring system, and the effect of these forces is relatively small.

The structural response (forces, moments, stresses) of the platform subjected to a harmonic wave was found by first determining the loads by applying a rigid body model of the platform and the computer program SEMSYST-D[4]. Dynamic effects due to elastic deformation of the structure were thus disregarded. These effects are negligible under normal conditions for an intact platform but could be important during a redistribution of forces due to a sudden failure of a strutural member, as will be shown in a subsequent section.

The excitation forces considered were inertia forces, drag forces, and forces due to variation in buoyancy. The excitation forces and the hydrodynamic mass for the braces were found from Morison's equation with mass- and drag coefficients equal to 2.0 and 1.0, respectively. The coefficients for the columns and the pontoons were computed according to diffraction analysis. The nonlinear drag force was linearized for each regular wave at the wave height H corresponding to a wave steepness of about 1/10.

The loading on the platform due to harmonic waves was computed for two deep-draught/load conditions, four wave directions, and 19 wave frequencies.

The forces found in this rigid body force/motion analysis were then applied to the abovementioned frame model to yield structural response values.

Both a deterministic and a stochastic wave model were used in this study. In the deterministic model the sea state was represented by an equivalent regular long-crested wave of given height and wave length. Selected results obtained by this method are displayed in Table 1.

The stochastic response in a short-term period may be obtained by a time - or frequency domain approach. In the latter the response characteristics are represented by transfer functions and the sea-state by a wave spectrum, e.g. characterized by the significant wave height H_S, and the mean zero-crossing period T_Z. The stochastic long-term response is achieved by weighting the responses for all short-term sea-states according to their probability of occurence over the long-term period.

Table 1. Maximum beam stresses for functional and wave loading in selected braces (See Fig. 1). The wave loading was calculated from a regular, longcrested wave with angular frequency 0.65-0.90 s^{-1}, wave steepness 1/10 and the most unfavourable wave direction of 0^0, 45^0, 90^0 and 135^0.

Deck load 10.5 MN: Mean operational condition with even ballast and 20.7 m deep-draught (as on March 27, 1980)

Deck load 21 MN: Design operational condition with even ballast and 18.0 m deep-draught.

Maximal total stresses (MPa)[1]

Brace	Deck load 10.5 MN			Deck load 21 MN		
	Functional	Wave	Total	Functional	Wave	Total
D6	75	109	184	57	151	208
DE	34	79	113	10	130	140
D4[2])	-76	-47	-123	-90	-53	-143
D3[2])	-69	-28	-97	-82	-33	-115

[1]) The stresses for each brace are given for the location where the maximum total stress in the brace occurs

[2]) Oblique braces

The stochastic long-term model was applied in the determination of response in brace D6 as described in the subsequent section.

Stress level due to functional and wave loading.

Table 1 shows the maximum total stress level for the braces connected to the lower end of the D-column, calculated for the mean operational condition and for the design condition.

NOMINAL STRESSES AT THE HYDROPHONE SUPPORT OF BRACE D6

Functional loads.

The nominal stress at the hydrophone support in brace D6 was tensile for all load cases and of the order 70-90 MPa.

Transfer function for stress due to waves.

The structural response of the platform in regular waves is most commonly expressed by the transfer function for the actual response quantities. Fig. 8 shows the computed transfer function for the nominal stress at the hydrophone support in the D6 brace as determined by the harmonic wave load model. The dragforces are linearized by assuming a wave steepness of 1/10, however, with the limitation that the maximum wave height is 15 m. The high response values in the frequency range 0.7 to 0.9 s^{-1} corresponding to wave lengths in the range 80-130 m, should be noted. Also, it appears that the wave direction $\phi = 90^0$ is the most critical.

Expected maximum of stress due to waves at the time of the accident.

The expected maximum stress due to waves was found by applying the deterministic and stochastic short-term methods.

In the deterministic approach a regular long-crested wave with a frequency of 0.8 s^{-1}, and with a steepness in the range 1/15 to 1/10, was considered. The wave direction was taken to be at a right angle to the platform ($\phi = 90^0$), corresponding to the situation at the time of accident. With these assumptions the maximum stress amplitude was found to be in the range 68 to 100 MPa, depending on wave steepness.

The expected maximum value of the stress was computed with the stochastic model based on the actual wave spectrum at the time of accident and a storm duration of 1 to 3 hours. The expected maximum stress amplitude at the hydrophone support was in the range 73 to 78 MPa.

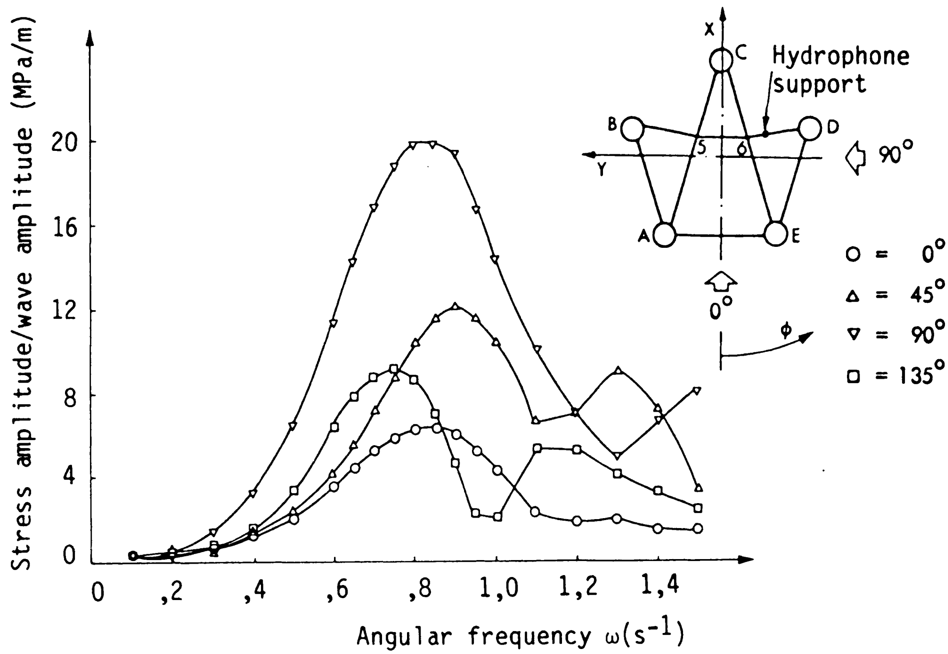


Fig. 8 Transfer function for the nominal stress at the hydrophone support in brace D6.

Long-term distribution of the stress amplitude.

The long-term distribution of the stress amplitude at the hydrophone support was calculated based on long-term wave data (H_s, T_z) for the Ekofisk field, where "Alexander L. Kielland" primarily operated; the transfer function in Fig. 8; and the assumption of the JONSWAP wave spectrum. Fig. 9 displays resulting distributions, reflecting the scatter in available wave data (hindcasting, and visual and instrumental observations).

The maximum stress amplitude with a return peiod of 100 years was found to be in the range of 110 to 150 MPa.

In a deterministic analysis with a regular long-crested wave, the corresponding maximum stress was found to be in the range 110 to 140 MPa. A beam wave ($\phi=90^0$) with an angular frequency of 0.8 s^{-1} and a steepness of 1/7 to 1/10 was considered. The modelling of wave steepness is important in the deterministic analysis. Most probably a steepness of 1/10 is a better assumption than 1/7, even for extreme wave conditions. Due to the shape of the transfer function, waves of frequency 0.7 to 0.9 s^{-1} will cause the highest stresses at the hydrophone support. Greater wave heights at lower frequencies or longer wavelengths will give smaller stresses. This feature of the global response actually makes the brace relatively more prone to fatigue than it would have been if the maximum response occured at smaller frequencies.

Both the stochastic and the deterministic methods employed in the analysis of load effects due to waves, are subjected to uncertainties. Those in the stochastic model are associated with data for the long-term distribution of the sea-states and the employed spectrum for a short-term sea-states. The small difference between assuming unidirectional and bidirectional long-term waves indicates that the uncertainty relating to the directionality is relatively small. The deterministic model based on a regular wave represents a significnt simplification of the physical reality, since ocean waves never appear as a regular long-crested wave. In addition, uncertainties are present in the choice of the wave height corresponding to a given wave frequency.

The stress amplitude with a 100 years return period is taken to be 110 to 150 MPa in the following.

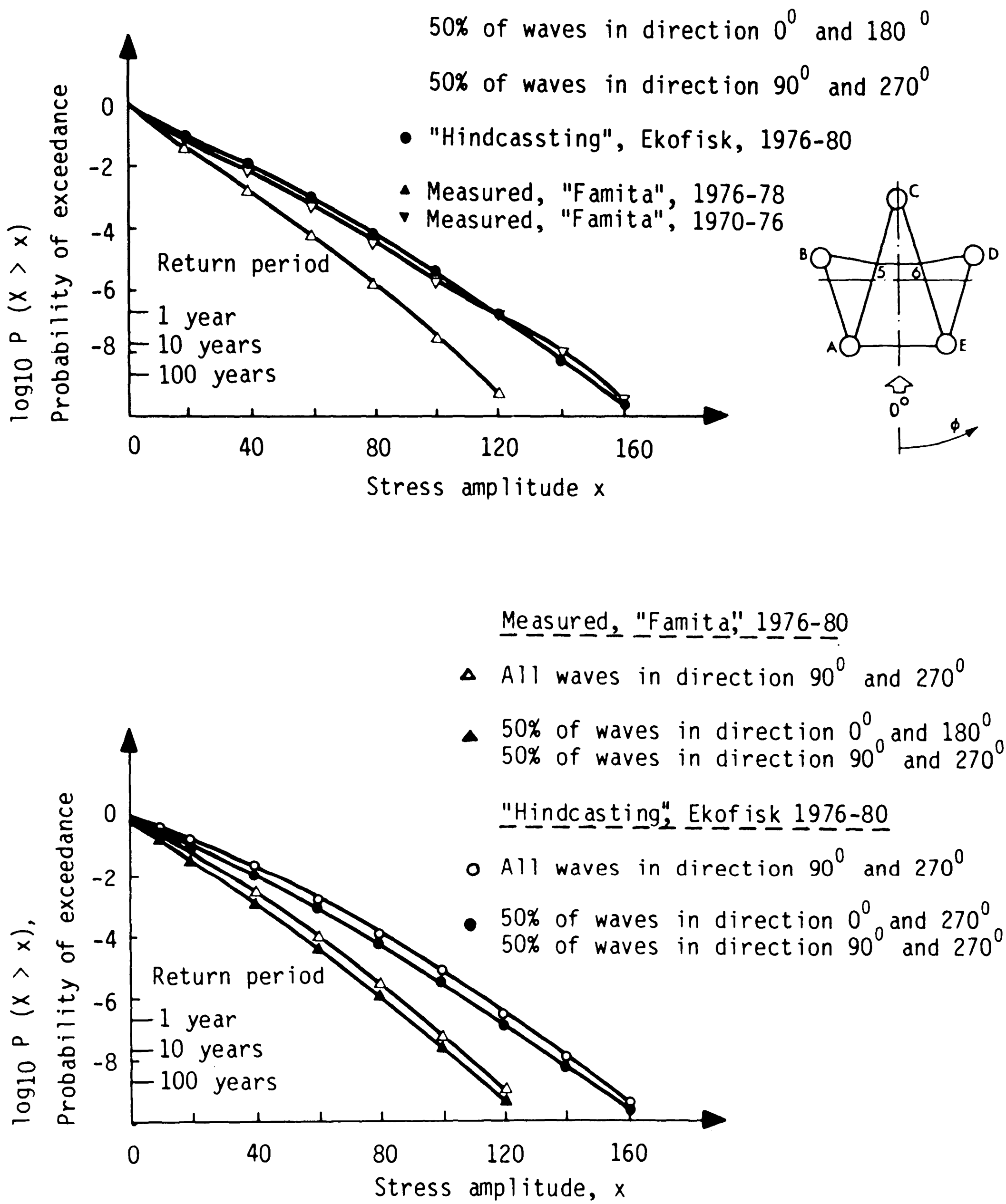


Fig. 9. Long-term cumulative distribution of the nominal stress amplitude at the hydrophone support in brace D6.

Load effects due to the mooring forces.

The stresses caused by the mooring system are not included in the results presented in this paper. An upper limit for these stresses was computed from a load condition where both anchor lines connected to column D were loaded up to their breaking strength, i.e. 3,0 MN each. The loading from the anchor lines was assumed to go through the axis of column D and the reaction forces to be taken by the A and B columns. Based on these assumptions, the maximum stress caused by the anchor forces at the hydrophone support on column D was found to be 31 MPa, or, less than 10% of the yield stress of the steel in the brace. Most likely the additional stress due to the mooring forces was less than 2-4 % of the yield stress. The dynamic part was only a minor part of this, and could be neglected in the fatigue analysis.

LOCAL STRESSES AT THE HYDROPHONE SUPPORT

In the S/N formulation of fatigue analysis used in this paper, load effects are taken as the "local nominal stress" at the weld detail, i.e. accounting for notch effects due to a level of detail corresponding to the weld, but without including the effect of the weld shape itself, possible cracks etc.

The local stresses adjacent to the hydrophone support were calculated with a FEM model based on thick shell elements (linear stress) and application of the SESAM system[3]. The size of the elements close to the weld was approximately 1.5 times the plate thickness.

Due to possible doubts regarding the state of cracking of the hydrophone support weld during the initiation and growth stages of the main fatigue crack in the brace (cf. subsequent sections), the calculations were performed for the conditions shown in Fig. 10.

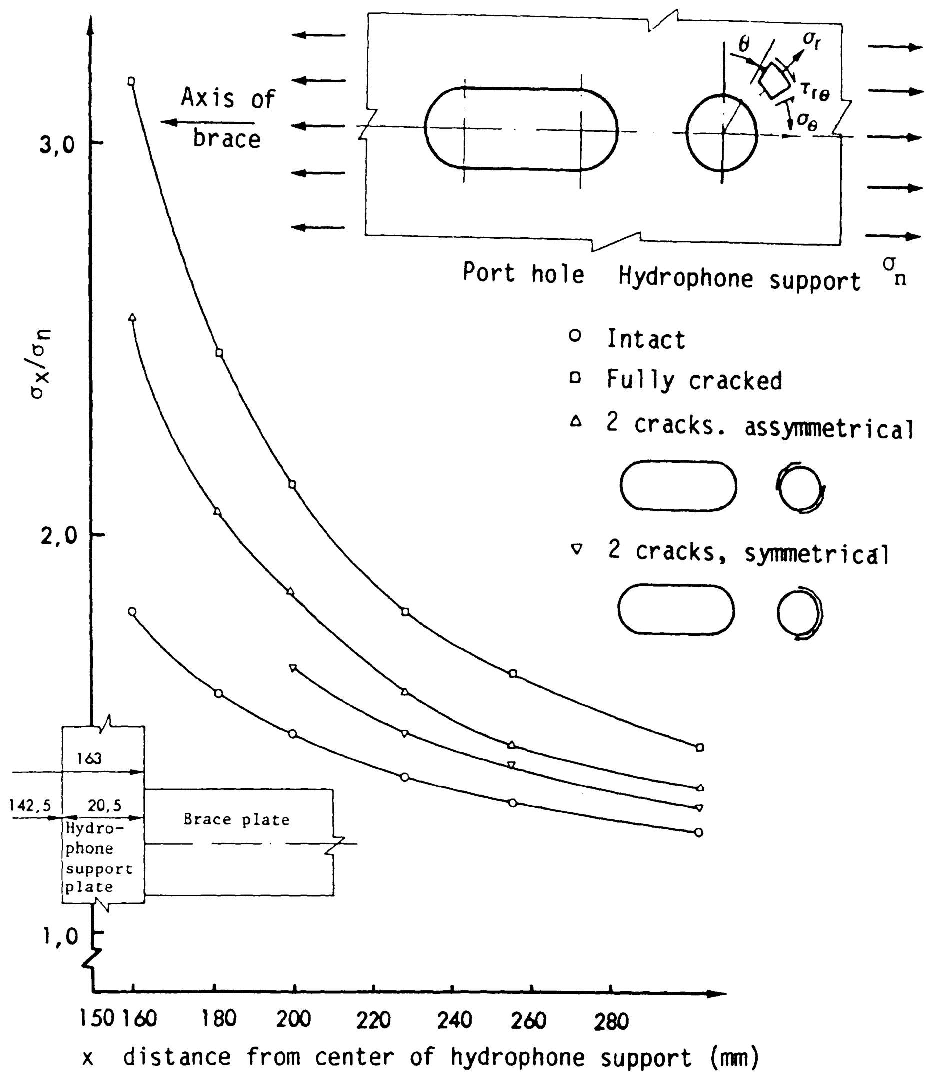


Fig. 10. Normal stresses in a section transverse to the axis of the brace and through the center of the hydrophone support, as calculated with a thick shell FEM model. Notch effects due to weld shape and cracks are disregarded.

As shown in Fig. 10 and Table 2 the stress concentration factor varies from about 1.7 for an intact weld to 3.1 for a fully cracked weld. If the stiffness of the weld, which was neglected in this model, had been accounted for, the former stress concentration would have been even less. For the latter case, however, this calculation is fairly accurate. For the cases of a partially cracked weld, the effect of local stress concentrations at the crack tips was neglected.

A qualitative comparison can be made between the two cases - intact vs. fully cracked weld - based on the analysis with the thick shell model. The results are shown in Table 2. As shown, the equivalent stress in the case of a fully cracked weld is greatest at $\theta = 0^0$, whereas for an intact weld the maximum stress is in the region $\theta = 15 - 30^0$.

Table 2. Stresses vs. nominal stress, σ_n in the brace at various positions θ (see fig. 10) along the hydrophone support weld on the inner side of the brace plating.

Pos. θ	Stress component[1) σ_θ/σ_n		σ_r/σ_n		$\tau_{r\theta}/\sigma_n$		Von Mises stress σ_e/σ_n	
	Intact	Cracked	Intact	Cracked	Intact	Cracked	Intact	Cracked
0^0	1.6	3.1	0.6	-0.1	-0.1	0.1	1.4	3.1
15^0	1.5	3.0	0.6	0.1	0.4	0.1	1.5	3.0
30^0	1.1	2.0	0.4	0.1	0.7	0.1	1.5	1.9
45^0	0.6	0.8	0.3	0.0	0.8	0.1	1.4	0.8
60^0	0.2	-0.1	0.1	0.0	0.6	0.1	1.1	0.2
90^0	-0.2	-0.9	-0.1	-0.1	0.0	0.0	0.2	0.9

1) For the case of an intact weld, stresses were calculated at a position corresponding to the toe of the weld. For a (completely) cracked weld stresses were calculated at the edge of the cut-out.

Also, a three-dimensional FEM model was applied for the case with intact fillet welds, using solid elements in the vicinity of the hydrophone support, and thick-shell elements for the remaining of the brace. The model used did represent the stiffness of the weld but not the notch effect due to its shape. The computed tri-axial stress components were combined as a von Mises equivalent stress. The resulting stress concentration for the inner fillet weld varied between 1.35 and 2.4, with the highest stress concentration for the sector $\theta = 30^0$ to 45^0.

THE FATIGUE FAILURE OF BRACE D6

Models for fatigue life prediction.

Fatigue design of offshore structures is generally based on an S/N formulation of the fatigue capacity[6]. In special cases, particularly related to the assessment of the significance of defects, fracture mechanics methods may be useful[7].

The S/N curve approach, as is well known, is based on constant amplitude tests of relatively small specimens[8]. For the later stages, however, the relative decrease in specimen cross-section in combination with load controlled test set-ups causes an accelleration of crack growth untypical of large-scale structural detail. Thus, the end-of-test criterion in S/N testing will lead to underestimation of the fatigue endurance. By fracture mechanics analysis, a more realistic assessment of the growth rates in the later stages of crack development may be obtained.

In the subsequent analysis of fatigue endurance, the S/N approach is used in a general assessment of the fatigue design, and both approaches are used in a reanalysis of the fatigue failure of the brace D6.

Analysis based on S/N-curves

In the fatigue analysis based on S/N curves, the following was assumed,

- Weibull distributed long-term nominal stress ranges
- stress concentration factors in terms of von Mises equivalent stress, using the crack opening mode stress components only, as calculated by FEM
- fatigue capacity as determined from tests in air (median S/N curve)[8] represented by a log-linear diagram, with no fatigue limit
- cumulative damage by the Miner-Palmgren rule

The load effect was defined by the cumulative distribution of stress ranges for a particular weld detail, in this case taken to be a Weibull distribution:

$$F(\Delta\sigma) = P(\Delta\Sigma \leq \Delta\sigma) = 1-\exp\{-(\Delta\sigma/\lambda)^{\beta}\} \qquad (1)$$

$\Delta\sigma$, $\Delta\Sigma$ - stress range

β, λ - parameters of the Weibull distribution, determined by fitting of calculated long-term distributions of stress ranges

β is a shape parameter. For braces in a Pentagone rig operating in North Sea environments, typical values of β are in the range 1.0 - 1.3. λ is related to the stress level, and may be calculated from the stress range $\Delta\sigma_0$ with a return period of 100 years as:

$$\lambda = \Delta\sigma_0(\ln N_0)^{-1/\beta} \qquad (2)$$

where $N_0 \simeq 5.10^8$ is the number of load cycles in 100 years.

For the nominal stresses in the braces of "Alexander L. Kielland" operating in the Ekofisk field, appropriate values of $\Delta\sigma_0$ are in the range 100-300 MPa. For weld details, stress concentrations must be taken into account, transforming $\Delta\sigma_0$ into a "local nominal stress" $\Delta\sigma_0'$.[8]

The fatigue capacity was defined by the S/N curve:

$$N = Ks^{-d}(\Delta\sigma)^{-m} \tag{3}$$

N - number of cycles to fracture
K, m - constants defining level and slope of median S/N curve
d - number of standard deviations below median S/N curve
s - standard deviation of S/N data
$\Delta\sigma$ - stress range

Applying the Miner-Palmgren cumulative damage hypothesis, and neglecting fatigue limit effects, leads to the following closed form expression

$$D_0 = \sum_i \frac{n_i}{N_i} = \frac{N_0}{Ks^{-d}} \lambda^m \Gamma(m/\beta+1) \tag{4}$$

D_0 - damage summation index for 100 years operation
n_i - number of stress ranges between $\Delta\sigma_{i+½}$ and $\Delta\sigma_{i-½}$
N_i - S/N endurance at the level $\Delta\sigma_i$
$\Gamma(\ )$-gamma function

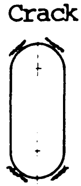
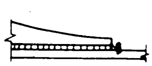
Calculated life t_e (years) of a structural detail is given by

$$t_e = 100/D_0 \tag{5}$$

Inserting for λ from Eq. (2) into Eq.(4), the fatigue life in this scheme is seen to depend on the two load effect parameters β and $\Delta\sigma_0$, and the fatigue capacity parameters K and m.[8]

Examples of calculated fatigue endurance of weld details typical for Pentagone rigs in the North Sea are shown in Table 3. The uncertainty related to the load effects are reflected in the estimates. In addition the variability of design/fabrication defects represents a significant uncertainty. It made the whole difference for brace D6! The fatigue endurance of the weld details of the hydrophone support will be discussed subsequently. The low fatigue endurances correspond fairly well to observations of rigs in-service in the North Sea.

Table 3. Calculated fatigue life for some weld details of the Pentagone structure.

Weld detail	Crack location	Type of stress 1)	Local nominal stress (MPa)	Weld class	Estimated median endurance (years)
Partial penetration weld at port hole in braces D6 and B5	Crack 	$\sigma_e(\sigma_r, \tau_{r\theta})$ at the toe	$\Delta\sigma_0$=280-390 β =1.0-1.2	F	3-16
Fillet weld at the end of stiffeners in joints 5 and 6 for braces A5, B5, D6, E6, 1-5, 2-5, 4-6, 5-6	Crack  Vert.proj.  Hor.proj.	Nominal stress in brace	$\Delta\sigma_0$=260-340 β=1.0-1.2	F2	3-16

1) See the definition of coordinate system in Fig.10

If a design S/N curve had been applied instead of the median curve, the calculated fatigue endurance would have been 1/3 the value in the table. Clearly, such endurance values are below current acceptance levels.

The fatigue endurance is clearly below current acceptance limits, and is explained by the fact that no fatigue calculations had been done during design.

S/N analysis of fatigue cracks at the hydrophone support in brace D6.

As shown in Table 4 there are several potential locations of fatigue cracks along the fillet welds at the hydrophone support. For each location stress components contributing to crack opening are considered when the equivalent (von Mises) stress or the corresponding stress concentration factor, is determined. In the particular case of partial cracking between the hydrophone support and the brace D6, with crack fronts at the positions $\theta=0^0$ and 180^0, the elastic stress is actually infinitely large at these positions. However, the results presented in Table 4 exclude the effect of the crack and accounts only for the stress concentration due to the "hole". The weld class designation for the case with partially cracked fillet welds is not straight-forward. According to the guidance given in design codes, a gas-cut edge produced by well controlled methods is a class C detail. Considering the uneven fracture surface caused by lamellar tearing, with ab initio existing cracks, the class D was somewhat arbitrarily applied. Median S/N curves were used. The calculated lower and upper bounds of endurances reflect the scatter in global load effects. In addition, comes uncertainties relating to the S/N curves applied (e.g. weld class designation, fabrication defect sizes).

For intact fillet welds the locations $\theta=0^0$ and 180^0 are most highly stressed. However, because the weld classes corresponding to $\theta=30\text{-}45^0$, $135\text{-}150^0$, etc. is poorer, these are the most likely locations of fatigue cracking. Note in this connection that the uncertainties relating to the load effects refer to the nominal stress at the

Table 4. Fatigue stresses, weld class designations, and fatigue endurances calculated for various locations, θ at the hydrophone support fillet welds. Load effect parameters were $\Delta\sigma_0$ = 220-300 MPa (nominal stress range in the brace with a return period of 100 years) and β = 1.0 - 1.2 (shape parameter of the Weibull distribution).

Initial condition of fillet weld	Position of crack initiation θ 1)	Type of stress 1) contributing to crack opening	Stress concentr. factor	Weld class	Estimated median endurance (years)
	0^0	σ_θ	1.6	D	4-25
Intact	30^0-45^0	$\sigma_e(\sigma_r, \tau_{r\theta})$ at toe	1.3	F	2-15
	30^0-45^0	$\tau_{r\theta}$ in weld	1.3	W	0.5-3
Partially cracked	0^0	σ_θ	2.5-3.1	D or worse	0.5-7 or less

1) See coordinate system in Fig. 10.

hydrophone support and are identical for all cases. The presence of abnormal fabrication defects may, however, invalidate this conclusion.

It is noted that the most probable location for fatigue cracking predicted theoretically for intact welds has been observed in experiments with ring-stiffened cut-outs in plates[9]. Hence, the fact that the cracks adjacent to the hydrophone support in the brace D6 developed at $\theta = 0^0$ and 180^0 is another indication that the fillet welds were partially fractured before the fatigue cracks developed.

The low fatigue endurance of the fillet welds (class W) in this case is due to the extraordinarily small throat thickness, with a nominal value of 6 mm, and a measured value in the range 4-9 mm. A poor root fusion decreased the effective throat section even more. The flooding port adjacent to the hydrophone support, however, was strengthened with a flange fixed with a full penetration weld. The reason for this apparent discrepancy in design was that, according to the designers, the port hole was considered as a structural detail, whereas the hydrophone support was regarded as appended equipment. Hence, no material specifications, weld design and procedures were enforced in the latter case. In particular, no ultimate strength and fatigue analyses were carried out.

Fracture mechanics analysis of the fatigue cracks in the brace D6

The fracture mechanics analysis of the brace D6 was performed under the following assumptions,

- full stress range was effective, due to the mean stress component of approximately 80 MPa tensile stress
  The fact that the fracture surface was hammered only close to the initiation points supports this assumption.
- crack growth rates, da/dn for a mode I crack opening, according to the Paris-Erdogan relation

$$\frac{da}{dn} = C(\Delta K)^m \qquad (6)$$

- cumulative damage according to the linear damage accumulation (analogous to the Miner-Palmgren hypothesis)
- the fatigue crack growth is divided into two phases:
  . Phase I, covering crack growth up to the size of a through thickness crack of the length of one plate thickness, assumed to correspond to the endurance criterion of S/N tests with similar type details.

  . Phase II, covering the subsequent growth of a through thickness crack to final failure, analyzed by fracture mechanics methods.

Fracture mechanics was also applied in an analysis of Phase I, thereby providing some overlap between the phases.

For Phase I the stress intensity function was calculated assuming the crack to be a circular corner crack (Fig. 11b). For a crack length a = 30 mm, a straight edge crack was assumed. In the transition region, the two functions were joined by smoothing. The in-plane stress was assumed to be linearly varying as illustrated in Fig. 11c. Hence the stress intensity functions as found for the in-plane bending case in the literature [10,11], was applied.

For Phase II the following form of the stress intensity function was assumed

$$K = \sigma_n(\pi a)^{\frac{1}{2}} F_W \cdot F_G \cdot F_C \tag{7}$$

σ_n - nominal stress in the brace

F_W - correction for finite circumference[10]

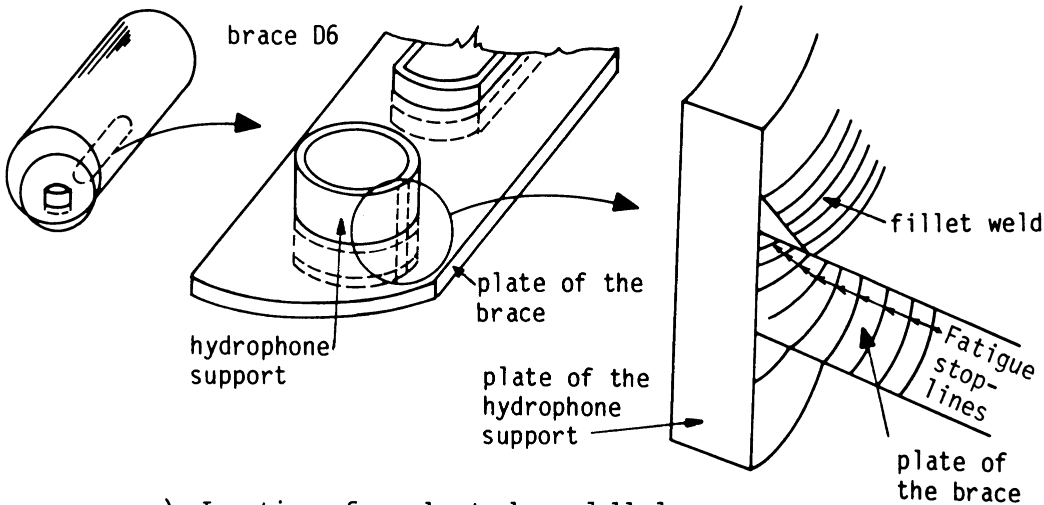
F_G - correction for stress concentration at the hole (containing correction for free surface[10]

F_G - correction for the cylindrical shape of the brace[12]

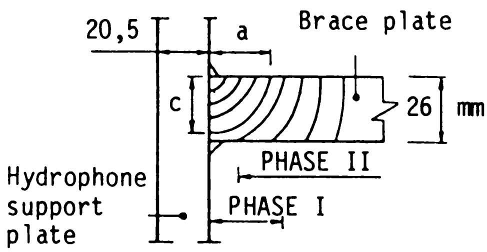
Fig.12 displays the resulting variation of the normalized stress intensity factor as a function of the surface crack length, a.

The nominal stress at the hydrophone support in brace D6 is assumed to follow the previously mentioned Weibull distribution with parameters: $\beta = 1.0 - 1.1$ and $\Delta\sigma_O$ = 240-300 MPa.

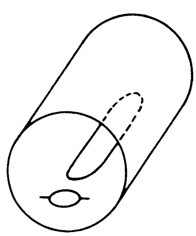
Fig. 13 shows possible crack growth histories, using different likely values for some of the parameters involved. The initial crack length, a_i is normally less than 1.0 mm and has an uncertain value, because no direct observation of a_i could be made in this case. The reason for this was peening of the actual locations due to contact between the crack surfaces. In this analysis, therefore, the a_i (and the time of crack initiation) was defined indirectly based on the end result - the crack length, a_f at the final fracture and the crack propagation properties of the fatigue crack in the 42-43 months effective service life of the platform. By inspection of the fracture surfaces, a_f was found to be about 3000 mm. As to crack propagation properties, the C in Eq.(6) was put equal to $1.7\ 10^{-13}$ and $3.0\ 10^{-13}$,



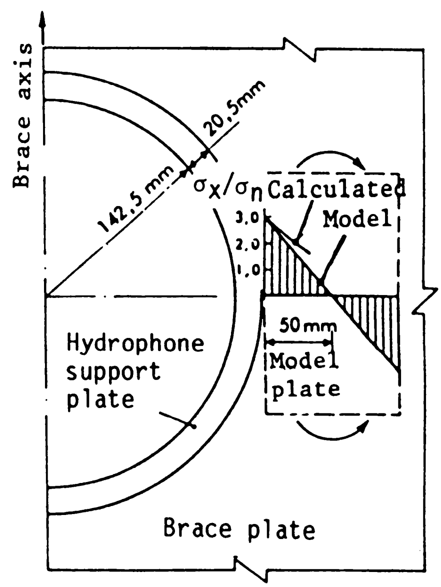
a) Location of crack to be modelled



b) Phase I and II



c) Model for Phase I (crack on one side of the hydrophone support)



d) Model for Phase II (cracks on two sides of the hydrophone support)

Fig.11 Fracture mechanics model for the fatigue crack growth in the brace D6.

corresponding to a median value for crack growth in air and a representative value for seawater, respectively [13]. The m-value was set equal to 3.0. The effect of a possible threshold value for the crack propagation (for a small) is indicated qualitatively in Fig. 13.

The uncertainties associated with the determination of the stress intensity factor as a function of a, shown in Fig. 12, should be observed. These uncertainties are expected to be greatest for small crack lengths. For increasing crack lengths the validity of the linear fracture mechanics approach will decrease. The reason is that already for cracks with a length of the order of 30 mm , peak loads will cause a crack-tip plastic zone comparable to the plate thickness. This plasticity implies a higher crack growth rate than predicted by the linear theory.

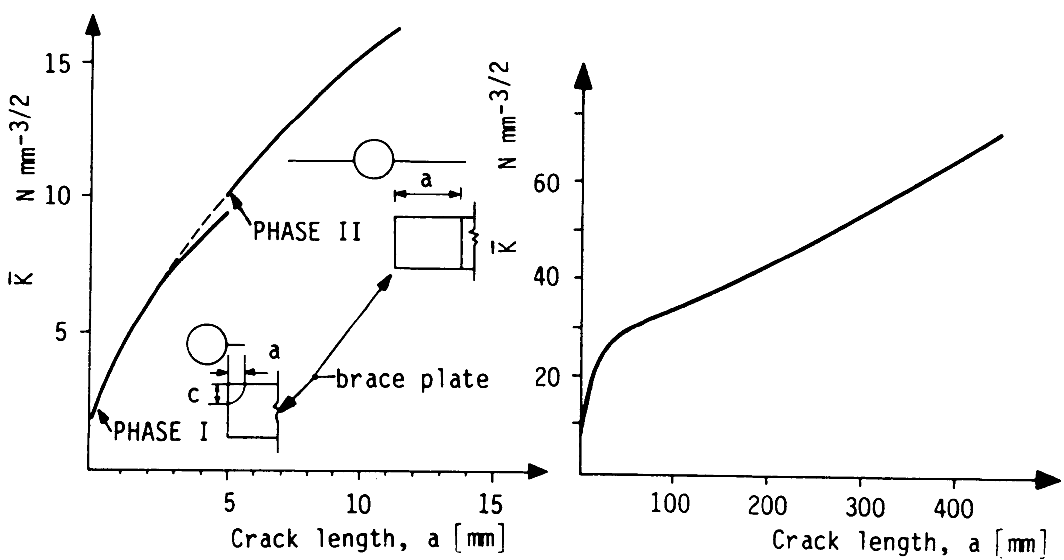
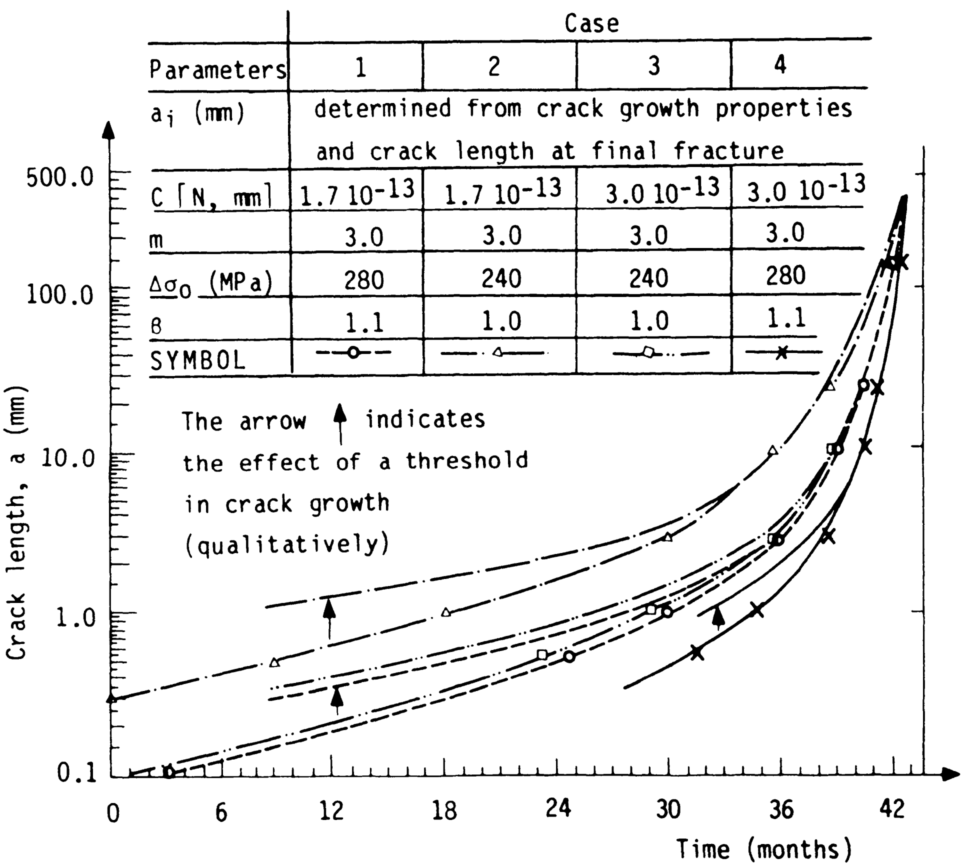


Fig. 12 Stress intensity factor range $\Delta K = \Delta\sigma \cdot \bar{K}$ where $\Delta\sigma$ is the nominal stress range at the hydrophone support in the brace D6.

Despite the inherent uncertainties the fracture mechanics analysis yields useful information. The main point of the results, a point which is quite independent of the choice of input parameters, is that the crack growth had accellerated very quickly with increasing crack length. The remaining endurance of the brace D6 after a through thickness crack had appeared, at most could be in the range of 6-12 months. Due to shortcomings of the models, the actual remaining life most probably was significantly lower. This gives an indication of the inspection alertness required for the safe-life operation of the structure.



Parameters	Case 1	Case 2	Case 3	Case 4
a_i (mm)	determined from crack growth properties and crack length at final fracture			
C [N, mm]	$1.7\ 10^{-13}$	$1.7\ 10^{-13}$	$3.0\ 10^{-13}$	$3.0\ 10^{-13}$
m	3.0	3.0	3.0	3.0
$\Delta\sigma_0$ (MPa)	280	240	240	280
β	1.1	1.0	1.0	1.1
SYMBOL	—o—	—·△—	—□··—	—×—

Fig. 13 Possible crack growth histories in brace D6 over its service life.

Detection of cracks by inspection.

The structure of "Alexander L. Kielland" had been inspected during fabrication and every year in operation according to the requirements of the classification society.

The inspection in the yard in principle should concern the complete structure, i.e. at least a visual examination of all weldments should be accomplished. The fact that this inspection failed to detect for instance the fairly significant cracks at the hydrophone support on brace D6, may be attributed to lack of attention - by the inspectors, but also by those planning the inspections, who failed to point out the importance of the actual area - and to inadequate thoroughness of the the required inspections.

The annual field inspections usually lasted for a few days and had a negligible effect with respect to detecting cracks, and particularly so for submerged parts, which were not inspected. Only the major inspection every fourth year seemed to have a potential of detecting cracks, if looked for. But it is not even likely that the hydrophone support area would have been included in the major inspection of "Alexander L. Kielland" that was due, because there was no evidence regarding predicted crack proneness or cracks experienced before the "Alexander L. Kielland" accident to suggest attention to these welds compared to the many kilometers of other welds in the platform.

If the hydrophone support of the brace D6 had been inspected during operation, the cracks in quadrants I and III might not have been detected, even by, say, the use of MPI or dye penetrant inspection, because the fillet welds were particularly unfavourable. For instance, MPI is said[14] to reliably detect cracks over 100 mm long. The experience from large structures in laboratories is that (hairline) cracks at weld toes need to be even longer for a reliable visual detection. It is also noted that major cracks at the hydrophone support could not be detected during fabrication of the "Alexander L. Kielland" - under more favourable conditions.

Concerning the brace itself, the exponential growth of the cracks there, implied that the first crack penetrated the thickness at most a few months before the complete fracture of the brace. Hence, not even an ideal accomplishment in the last annual inspection in the autumn of 1979 could have revealed this crack.

However, if the lower horizontal braces had been airfilled, detection of the leakage of water in case of through thickness cracks might have given an adequate warning, which perhaps may have prevented the catastrophe.

Clearly, inspecting the many kilometers of weldments, partly with difficult access, in a semi-submersible of this kind, requires excessive resources, e.g. many manyears. Therefore, only a limited number of locations can be examined in practice. Furthermore, even if looked for, cracks need to be relatively large to be detected.

Hence, the overall safety of the platform should not depend on detection of small cracks. The platform should be designed to have adequate strength in a damaged condition, in terms for instance of cracks which cannot be reliably detected.

PROGRESSIVE COLLAPSE - LOSS OF COLUMN D

The course of events following the fracture of the brace D6 can be understood in terms of nominal stresses in the remaining braces connected to the column D.

Brace D6 broken - stresses in remaining members.

The global analysis of load effects with brace D6 broken is based on the same general assumptions as for the intact platform. Table 5 gives the stresses for functional loading in still-water for the braces connected to column D. The stresses are seen to be very high, even for this loading condition.

Assuming a reasonable design wave for the sea-state at the moment of the accident, the maximum total stress in braces DE , D4 and D3 (diagonal) were calculated to be 949, -858 and 1230 MPa, respectively. Only one load condition was considered, long-crested wave with direction $\phi = 90^0$, angular frequency $\omega = 0.70\ s^{-1}$ and wave steepness 1/10. As shown, any loading representing the actual situation, will give rise to member stresses far in excess of yield, even without taking into account dynamic effects.

Table 5. Maximum stresses for functional loading in still water with brace D6 broken. Deck load 10.5 M , even ballast and deep-draught 20.7 m (actual condition at the moment of the accident). Nominal yield stress σ_Y = 355 MPa.

Brace	Axial and max bending stresses [MPa] 1)					
	Brace end at column D			Brace end at rest of platform		
	Axial	Bending M_y	Bending M_z	Axial	Bending M_y	Bending M_z
D6	-	-	-	-	-	-
DE	96	269	133	96	277	234
D4	-95	234	103	-109	133	28
D3	46	385	113	46	200	142
D4 (upper horizontal)	-56	264	47	-56	145	125
D3 (upper horizontal)	17	420	44	17	197	145

1) M_y - max. stresses at upper/lower side of the brace

M_z - max. stresses at right/left side of the brace

Redistribution of forces in the braces after the total fracture in D6

In the analysis above, dynamic effects due to the elastic deformations of the platform were not included. These effects can usually be neglected in normal loading conditions of an intact floating platform since the lowest natural frequency of the platform far exceeds the excitation frequency of the wave loading. However, by fracture of a main structural component, the redistribution of forces may occur so quickly that inertia forces caused by elastic deformations should be taken into account. This was the case with "Alexander L. Kielland".

The dynamic amplification or load factor will in general be dependent upon the ratio between the loading time and the natural period of the system.

If the crack tip velocity had been greater than 100 m/s, the loading time would have been less than 0.02 s. The elastic response of the structure to the failure of the brace D6 corresponds approximately to the first mode shape, with a natural period of about 2.5s, which is large compared to the assumed loading time.

Computations using a finite element program[15] showed in this case that the dynamic load factor for axial and bending stresses in the most heavily stressed braces connected to the column D were in the range 1.4 to 2.6.

These findings substantiate the conclusions of the previous section regarding overload failure of braces after the fracture of the D6 brace.

CONCLUSIONS

Regarding the mechanisms of the structural failure of the "Alexander L. Kielland", the following was established[1]:

1. A hydrophone support of tubular shape welded into a circular cut-out in the D6 brace, was made by steel with poor through-thickness properties. Mainly due to this, and the fact that the fillet welds between the hydrophone support and the brace were poorly designed in relation to the local stresses, cracks developed in the plate material of, but also in the welds around the circumference of the hydrophone support. A significant part of the cracking had occurred while the structure still was in the yard. This initial cracking increased the stress concentration of the cut-out and most probably assisted in the nucleation of fatigue cracks.

2. Fatigue cracks nucleated and grew from two locations on the edge of the cut-out, at right angles to the applied stress in the brace. The cracks grew about 2/3 of the circumference of the brace, whereupon the brace failed by unstable fracture.

3. Within reasonable limits of uncertainties, the fatigue life of the D6 brace could be verified, using available fatigue analysis procedures.

4. Fracture mechanics analysis showed that the crack growth in the D6 brace had accellerated very quickly from the initial stage. With a through-thickness crack of about 30 mm length, the remaining life of the brace was less, probably significantly less than a year.

5. The main reason for the structural failure was inadequate local design and material specification of the through-thickness properties of the hydrophone support. Supposedly, at the planning stage, the hydrophone support was considered as outfit, not as a part of the load carrying structure. It is worth nothing that no explicit evaluation of the resistance against fatigue failure or lamellar tearing was made for the actual structural component, despite the fact that the rules of the classification society contained such requirements.

6. The controls of the design were inadequate and the survey during the construction and operation failed to unveil crack defects. The reason for the deficient survey was that the intentions of the regulations do not seem to have been fully carried out during the annual inspection. And to a larger extent, even an ideal survey, according to the existing rules and regulations, would not have satisfactorily controlled the actual crack defects. To some degree, especially in building surveys, this must be attributed to the lack of attention to the actual design detail by designers and those involved in checking the design.

7. After the final fracture of the brace D6, the load-carrying capacity of the remaining braces connected to the column D was highly insufficient. With reasonable assumptions of loads at the moment of the accident, calculated member stresses were far in excess of yield. Taking into account the dynamic amplification for the part of the stress being redistributed, calculated to be in the range 1.4 to 2.6, the stresses would be even greater.

8. Progressive collapse was not a structural design criterion in the planning of "Alexander L. Kielland". When the platform was approved, no code for mobile units included requirements about residual strength. Progressive failure criteria had, however, already been introduced in certain codes for civil engineering structures.

The lessons this accident primarily teach us about design principles may be stated as follows:

9. Fatigue starts microscopically, and from essentially elastic stresses. Any part of a structure which may act as a stress raiser, should therefore be carefully designed and considered in a fatigue design check. This is especially so for platforms made of high strength steels and subjected to a dynamic loading, because the use of high strength steel allows an increased stress range level but no increase of fatigue strength.

10. Fatigue design procedures depend on data which are inherently uncertain, like environmental data and fatigue capacity data. This, together with the limited scope of inspection programs and the lack of reliability of inspection procedures of large welded marine structures, suggest design for residual strength of platforms with cracks or other damages.

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STEEL TEMPLATE PLATFORM ON PILED FOUNDATION

A. Agostoni

Tecnomare S.p.A.

ABSTRACT

The paper deals with barge transported jacket structures. Describing the actual cases of Enchova Central and Rospo Mare, the main characteristics of such structures are given as well as the aspects related to their design and to the marine operations necessary for their installation.
The difference between "conventional" and "skirt piled" jacket, as well as between conventional launch-upending and self-upending maneouvres, is highlighted.

INTRODUCTION

Offshore platforms have been developed in a variety of sizes, shapes and degrees of complexity.
From platforms erected on piles only, used for early offshore operations in very shallow water, reaching greater and greater water depths, through different configurations, they developed to the present concepts: steel piled foundation, steel and concrete gravity platforms.
The steel piled platforms are by far the most used world wide and include those installed in the deepest water, such as COGNAC (Gulf of Mexico, operator Shell) in 312 m, MAGNUS (North Sea, operator BP) in 186 m, NAMORADO II (Brazil, operator Petrobras) in 170 m.

The steel piled structures are commonly named "Jacket" from their original function to brace the piles. In the conventional jackets the topside loads are directly taken by the piles and transferred to the soil. Piles are driven through the legs of the structure by means of steam hammers located over the sea level and the annuli between piles and jacket legs are filled with cement grout. Fabrication is carried out building the framed structure laid horizontally in yards with direct access to the sea.
Apart from light weight structures, which are simply lifted from the yard onto a barge and from the barge into the sea, the ordinary practice is to load, on skid ways, by means of winches or jacks, the jacket out of the yard onto the transportation barge. The common procedure used for installation is to let the jacket slide from the barge into the sea, following a trajectory which finally leads the jacket to float almost horizontally. The weight/buoyancy characteristics of the jacket are such as to provide an adequate margin between the sea bed surface and the lowest point of the structure during the launch. Specific buoyancy tanks are designed for this purpose.

From this naturally floating position an uprighting operation is carried out to move the structure in vertical position. Then the structure is flooded and set down in its final location.

As the water depths increased and the environments became more hostile, leading to heavier and heavier structures, an improved configuration arose in order to save steel weight: the skirt pile jacket. Here the topside loads are taken by the structure and then transferred to the piles, thouth grouted sleeves.

Another improvement is the self-upending launch. The launching procedure is designed to carry the jacket in vertical position at the end of the launch trajectory. The buoyancy/weight distribution of the structure is designed so as to have one equilibrium position, the vertical one.
Therefore the common upending manoeuvre is not necessary, shortening the time required for marine operations.
A further improvement of the skirt pile concept has been recently made possible by the development of the underwater hammer technology, which currently provides reliable means for driving skirt piles without any need for followers and guides.
As a consequence the amount of steel required is less and the pile driving operation is simpler and quicker.

Two actual cases of skirt pile jacket are reported in the following:

Enchova Central, installed in January 1982 in 116 m water depth offshore Brazil, operator Petrobràs; 32 piles driven to 90 m penetration.

Rospo Mare, installed in July 1981 in 77 m water depth. Much smaller than the previous one but worthwhile mentioning due to its self-upending launch. The jacket, launched from the barge, rotated, assuming the upright position at the end of the launch trajectory.

ENCHOVA CENTRAL

Basic Philosophy

The platform is part of the CAMPOS field in the Brazilian waters of the Atlantic Ocean (See Fig.1). It is designed to handle the production of other four platforms, to which it is connected via sealine.
The modular concept was adopted for the topsides, including drilling modules, production modules, living quarters and helideck and leading to a maximum operating payload of 22000 metric tons.
The jacket structure had to fulfill the following premises:

- *optimum structural weight*
- *piles driving through surface steam hammer*
- *construction in Brazil*
- *minimum duration for offshore operations.*

Therefore the skirt pile concept was selected.
In order to minimize the weight of individual preassembled pieces to be lifted at yard during fabrication, the foundation piles were arranged around the main eight columns. Then the pile groups are composed of a limited number of elements and consequently the skirt sleeve assemblies were of limited weight, below the lifting capacities available at the Brazilian yard.

Furthermore the skid supporting beams for the topside modules have been made integral with the jacket structure, thus avoiding the related sea transportation lifting and hook up offshore operations.
Finally the piles and the appurtenances have been preinstalled onshore at the maximum extent, designing the buoyancy tanks arrangement accordingly, thus further minimizing the duration of the marine operations.

The installation was conventional , i.e.:

- *jacket launched from the barge*
- *launch trajectory leading to a final position of the jacket almost horizontal, floating on a row*
- *upending manoeuvre; through water ballasting of selected members the jacket is rotated up to the vertical position*
- *setting down in the final location*

The launching sequences are shown in fig. 2, while the upending phases are in figs. 3 and 4.
The main characteristics are summarized in fig. 5.

Jacket Description

The structure is a tubular steel template composed of eight columns ending at the bottom with 4 skirt piles sleeves each and at the top with the framed structure of the modules support frame, including skid beams, integrally connected to the jacket.
The columns are braced laterally through diagonal and horizontal members to built the external longitudinal rows which are vertical and braced together through 4 transversal rows of which the external ones are battered.
The horizontal levels are placed at el. -114.4, -88.7, -63.0, -37.3, -11.6, +14.1, +21.9 with respect to the lowest astronomical tide level. The latter two form the module support frame.
The overall configuration is shown in fig. 6. The diameter of the tubular members ranges from 800 mm to 2000 mm of the legs, while the thicknesses vary from 20 mm to the 70 mm of the chords of the most loaded joints.
The steel quality is according to BS 4360 gr 50, with through the thickness properties for the joint cans.

The 32 piles, eight groups of four each, whose outside diameter is 66 inches and total fabricated length 125 m, were driven to 90 meters penetration, each providing a 3600 t ultimate bearing capacity. The annuli between piles and sleeves is filled with cement grout.

The structure was designed to accomodate 21 well conductors plus the following appurtenances: 2 boat landings, 12 risers, 17 caissons, 12 j-tubes.

The corrosion protection system is by sacrificial anodes. The 20 year design life of the platform required about 800 tons of aluminium alloy anodes.

Design Approach

The main steps followed in designing such a structure are shown in fig. 7.
On the basis of the Client Specifications a preliminary analysis was carried out, reviewing the design premises in terms of soil characteristics, environmental data, both for the platform in service life and for marine operations, topside layout and weights.

A basic configuration of the structure was defined taking account of its service life and of the marine operation preliminarily envisaged. Basic calculations were run to find out the behaviour of the structure. The results were reviewed and implemented in the structure configuration.

The detail engineering started therefore on the basic configuration which was checked against all the loading conditions the structure would undergo, such as fabrication and load out, transportation, lanching and installation, piling, in service storm and operating conditions, in service fatigue life.

Extensive model tests have been carried out at NSMB facilities, Holland, to analyse the jacket/barge behaviour during transportation and launch. The test results were well in agreement with the theoretical analyses performed in-house.
Fig. 8 shows some sequences of the launch during the model tests.

Construction

The fabrication was carried out at Sao Roque de Paraguaçao yard and took about 22 months.

The adopted assembly procedure foresaw relatively light individual preassembled pieces, tied into the structure in their final locations (see fig. 9). In fact no row of the structure was completed on the ground, then rolled and tied with the other ones.

The jacket was loaded onto the launching barge Micoperi M44 in late December 1981. The total load out weight was 22000 t and included seventeen caissons, twelve risers, twelve j-tubes, twelve piles preinstalled onshore.
There were fourteen additional buoyancy tanks, eight of 4 m outside diameter, four of 3 m O.D., two of 3.5 m O.D.. Fig. 10 shows the jacket at load out.
The structure was towed from Sao Roque harbour to the field in 20 days (see fig.11) and was successfully installed in January 1982 in 116 m water depth.

ROSPO MARE

The field is located in the Adriatic Sea (refer to fig. 12) and ELF is the operator.
Only the aspects related to the marine operations will be mentioned here, as they are the peculiarity of this structure.

Considerations on Marine Operations

The launching of a jacket is started only when a weather window is forecast to allow the completion of all the operations required to ensure the stability and safety of the jacket.
The uncertainties related to the upending manoeuvre sometimes affect the selection of the duration of such a weather window, leading to reject possible periods otherwise useful.

The result is a long waiting time and delays on installation and consequent impact on cost of the marine equipment.
The use of a self-upending procedure offers clear advantages with respect to the conventional launch-upending manoeuvres: it ensures that the jacket is ready for the setting down operation in a few minutes, without intervention of crane barge, divers, ballasting crew necessary for the time consuming upending phase, avoiding the above inconveniences.

The first self-upending launch

The Rospo Mare jacket is a relatively small structure, 81 m high, 34 m by 34 m at the bottom, 14 m by 14 m at the top.
It is a four column skirt pile jacket.
Its relevant characteristics are summarized in fig. 13.

The analyses of the marine operations have been performed on a structural configuration already defined on the basis of the in-service conditions only. Therefore, in order to achieve the characteristics required to allow a self-upending, the additional buoyancy was purposely designed.
The selected arrangement needed four tanks located near the top level of the jacket and disposed like a collar around the structure. The total additional buoyancy was 594 t.
The plot in fig. 14 shows the computed jacket positions during the launch.

The jacket fabrication took place at Ambes, France and its transportation to the field was performed in two phases.
The barge/jacket was first towed to Arbatax (Sardinia) to fulfill the operations relevant to the installation of the buoyancy tanks, then a second tow concluded with the jacket successfully lanched in 1981. As designed, it rotated assuming the upright position at the end of the launch trajectory.

The photo sequence relevant to the actual operation, shown in fig. 15, is well comparable to the plot of fig.14.

The Rospo Mare experience demonstrated the viability of such a procedure.


BRAZIL
BRASILIA
ATLANTIC
OCEAN
CAMPOS
RIO DE JANEIRO
GAROUPA
ENCHOVA

FIG. 1

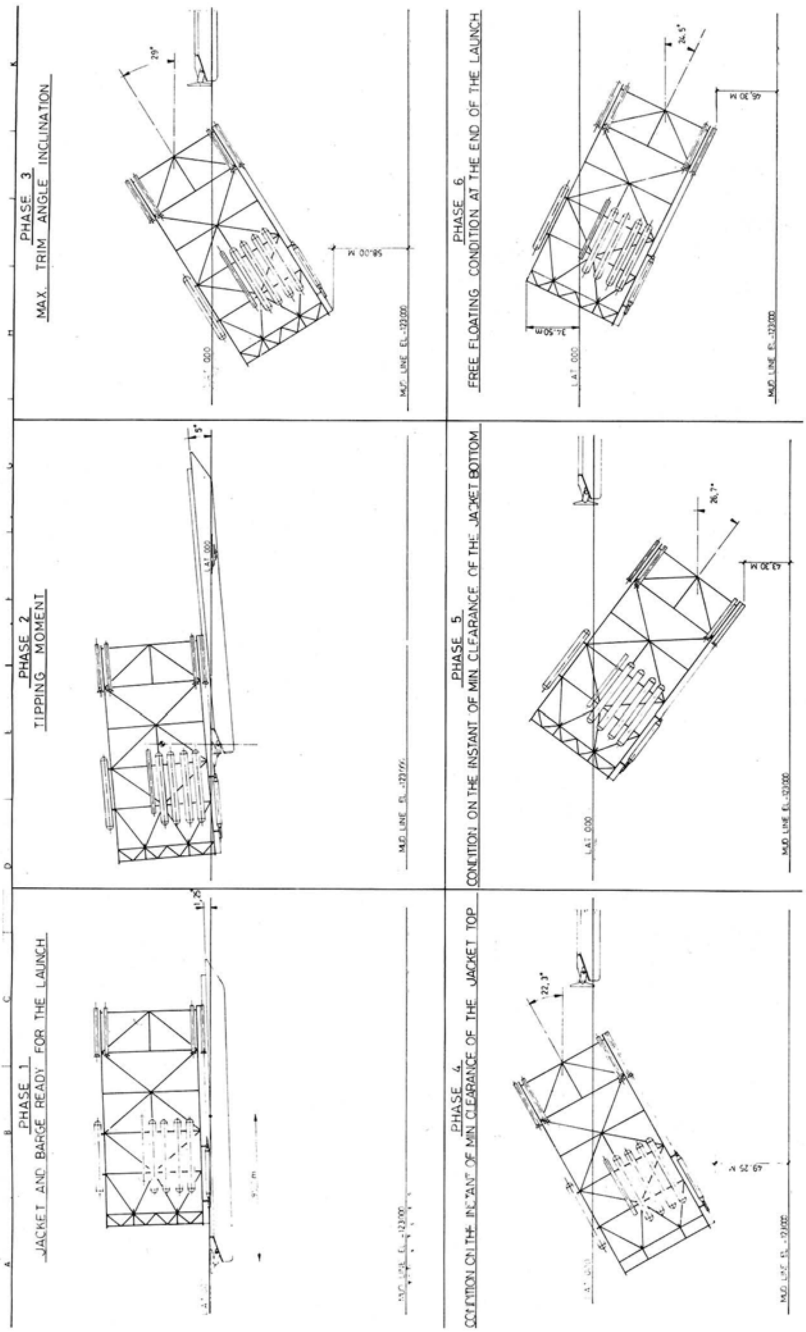

PHASE 1
JACKET AND BARGE READY FOR THE LAUNCH
PHASE 2
TIPPING MOMENT
PHASE 3
MAX. TRIM ANGLE INCLINATION
PHASE 4
CONDITION ON THE INSTANT OF MIN CLEARANCE OF THE JACKET TOP
PHASE 5
CONDITION ON THE INSTANT OF MIN CLEARANCE OF THE JACKET BOTTOM
PHASE 6
FREE FLOATING CONDITION AT THE END OF THE LAUNCH
58.00 M
29°
5°
22,3°
26,7°
24,5°
43.30 M
46.30 M
34.50 M
49.25 M
L.A.T. 000
MUD LINE EL -123.000

FIG. 2

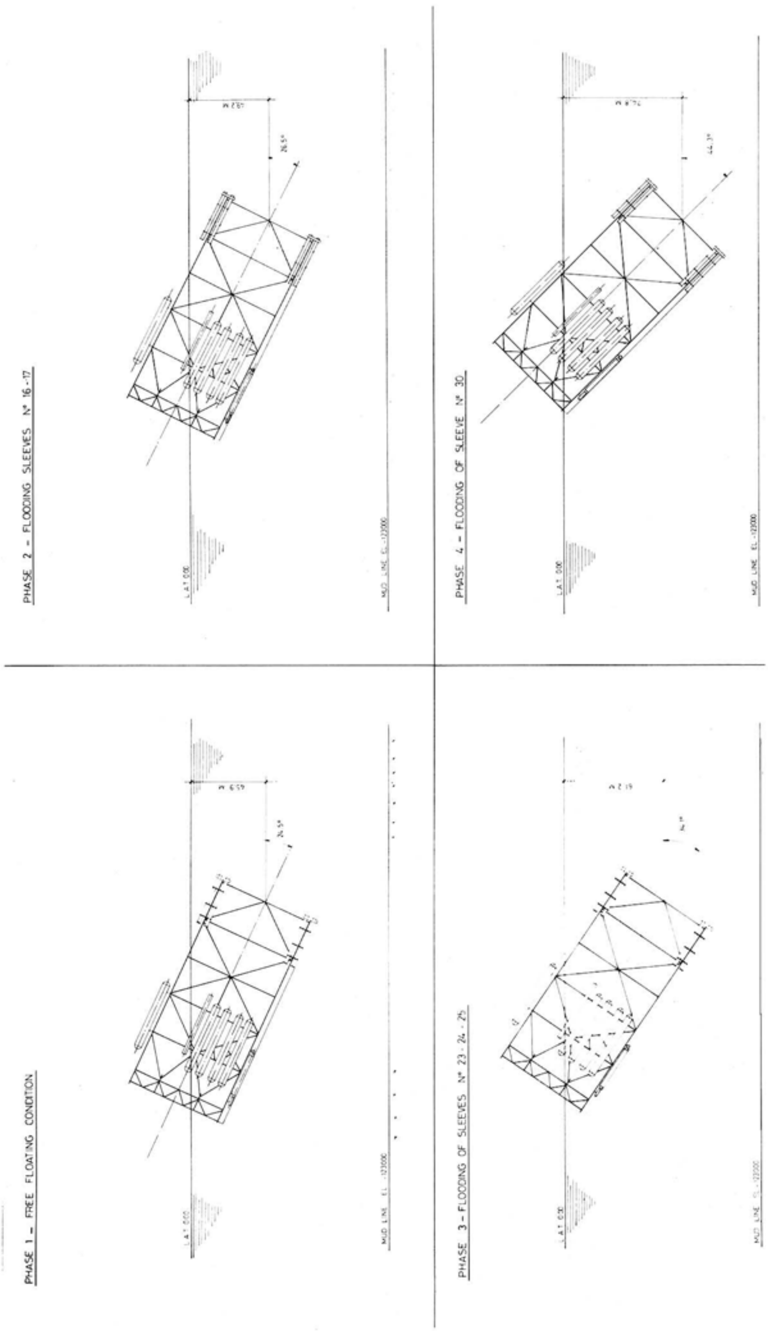

PHASE 1 _ FREE FLOATING CONDITION
PHASE 2 – FLOODING SLEEVES N° 16-17
PHASE 3 – FLOODING OF SLEEVES N° 23-24-25
PHASE 4 – FLOODING OF SLEEVE N° 30

FIG. 3

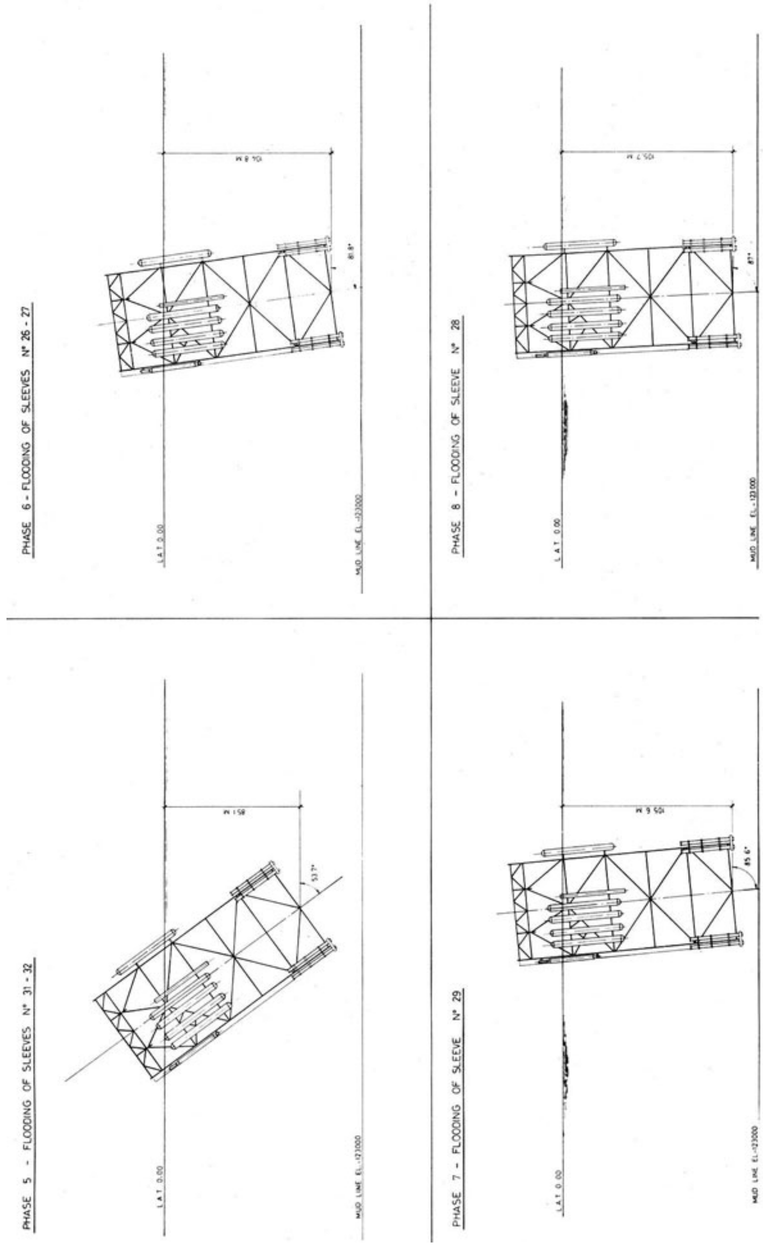

PHASE 6 - FLOODING OF SLEEVES N° 26 - 27
PHASE 8 - FLOODING OF SLEEVE N° 28
PHASE 5 - FLOODING OF SLEEVES N° 31 - 32
PHASE 7 - FLOODING OF SLEEVE N° 29

FIG. 4

ENCHOVA CENTRAL

Drilling, Production and Quarters Platform

Location	Offshore Brazil, Campos Basin
Distance from Shore	80 Km
Water Depth (L.A.T.)	116 m
Extreme Storm Wave	height 14.2 m, period 11.1s
Extreme Wind Speed (1 min.)	44 m/s
Jacket	
Height	138 m
Base dimensions	60.0 m x 79.9 m
Top dimensions	60.0 m x 72.4 m
No. of legs/main size	8/2 m
Weight	16600 t
Tow data	
Wind/Wave	40 Knots/7 m (significant height)
Length/Duration	800 nautical miles/20 days
Launch Barge	190 m x 50 m x 12 m/30000 t dw
Foundations	
No.of piles	32
Size of piles	66 inches O.D./125 m length
Max. Penetration	90 m
Total Weight	6700 t
Topsides	
Well Conductors	21
Size	72.6 m x 78 m
Operating Payload	22000 t

FIG. 5

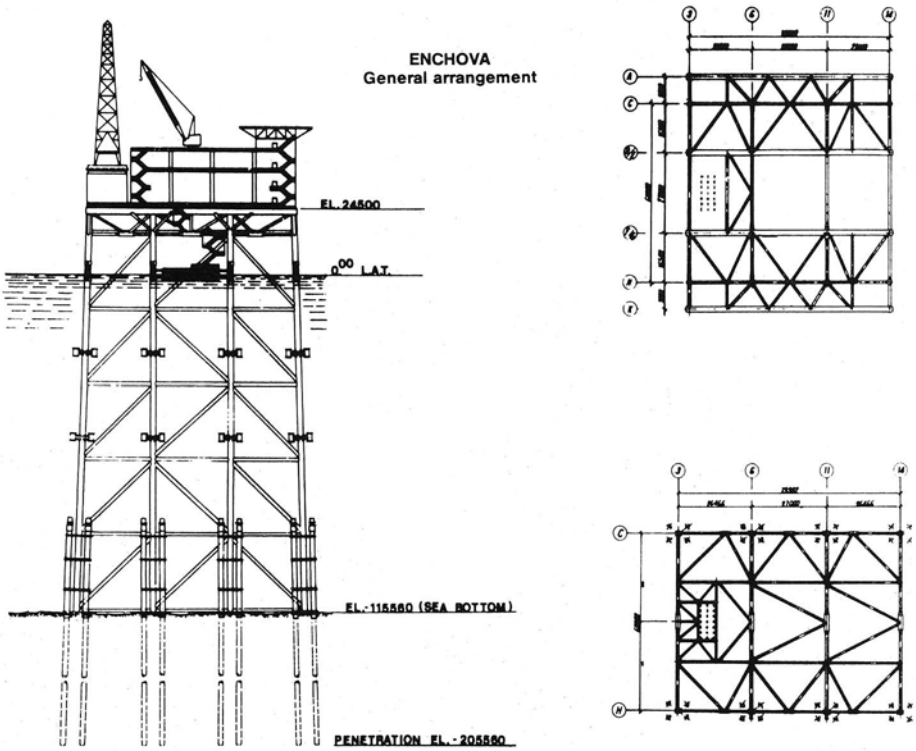

ENCHOVA
General arrangement
EL. 24500
0^{00} L.A.T.
EL.-115560 (SEA BOTTOM)
PENETRATION EL.-205560

FIG. 6

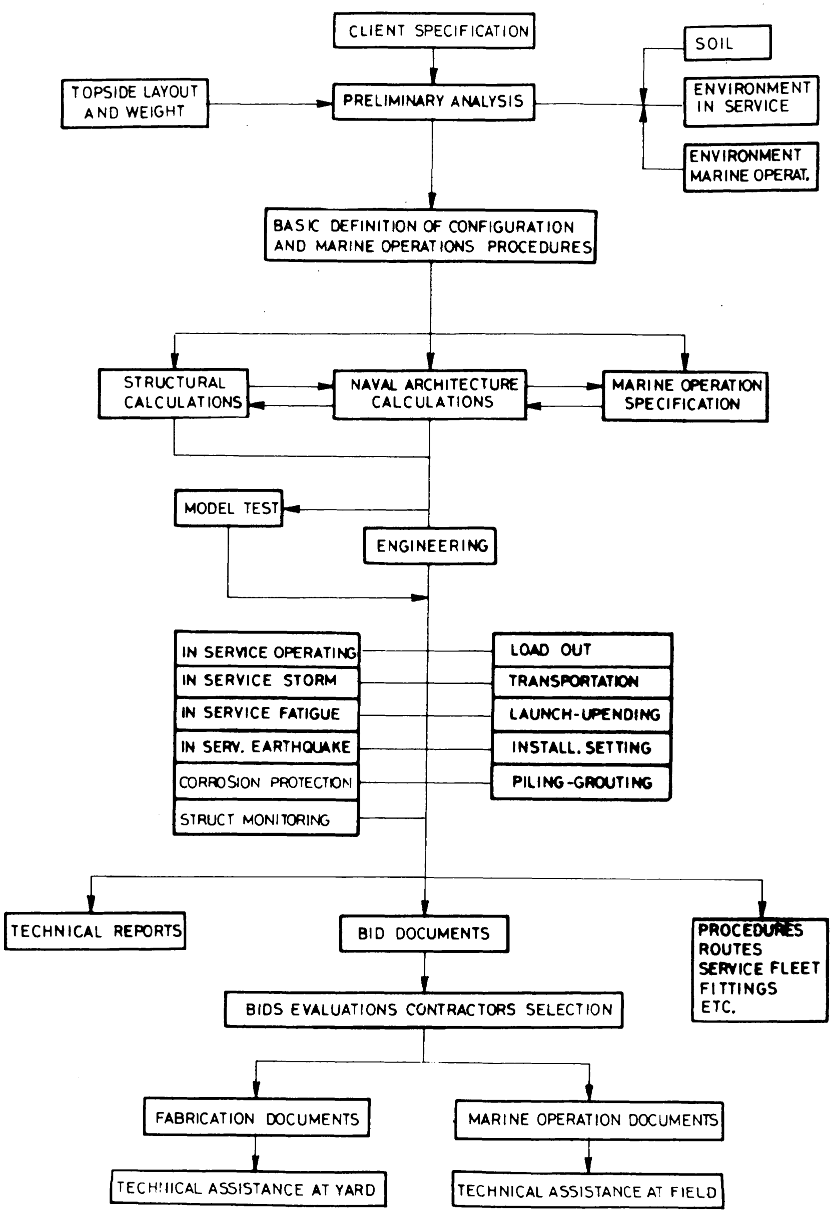

CLIENT SPECIFICATION
TOPSIDE LAYOUT AND WEIGHT
PRELIMINARY ANALYSIS
SOIL
ENVIRONMENT IN SERVICE
ENVIRONMENT MARINE OPERAT.
BASIC DEFINITION OF CONFIGURATION AND MARINE OPERATIONS PROCEDURES
STRUCTURAL CALCULATIONS
NAVAL ARCHITECTURE CALCULATIONS
MARINE OPERATION SPECIFICATION
MODEL TEST
ENGINEERING
IN SERVICE OPERATING
IN SERVICE STORM
IN SERVICE FATIGUE
IN SERV. EARTHQUAKE
CORROSION PROTECTION
STRUCT MONITORING
LOAD OUT
TRANSPORTATION
LAUNCH-UPENDING
INSTALL. SETTING
PILING-GROUTING
TECHNICAL REPORTS
BID DOCUMENTS
PROCEDURES ROUTES SERVICE FLEET FITTINGS ETC.
BIDS EVALUATIONS CONTRACTORS SELECTION
FABRICATION DOCUMENTS
MARINE OPERATION DOCUMENTS
TECHNICAL ASSISTANCE AT YARD
TECHNICAL ASSISTANCE AT FIELD

FIG. 7

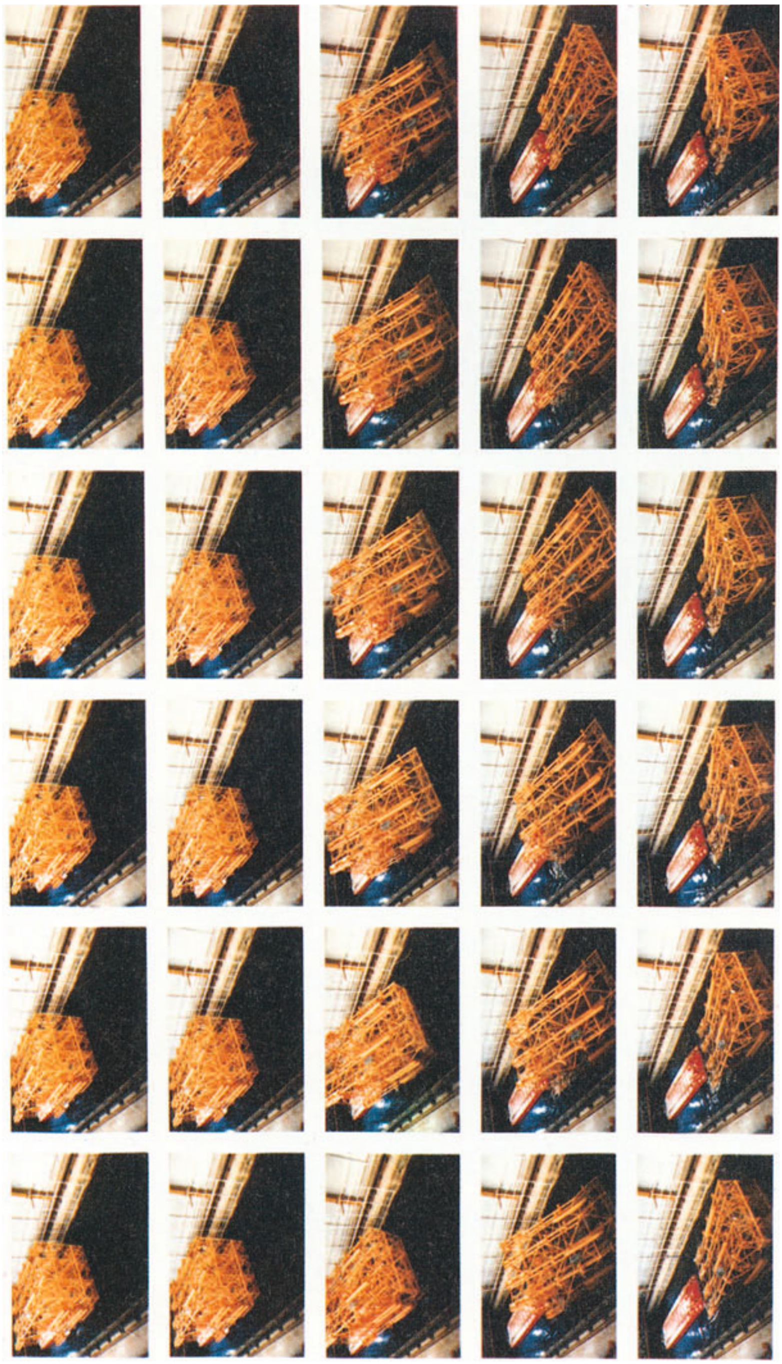


FIG. 8

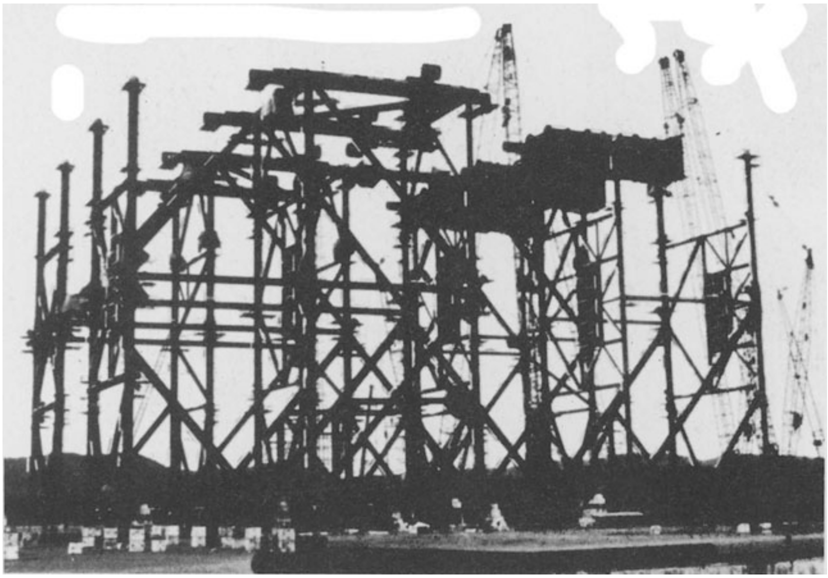


FIG. 9

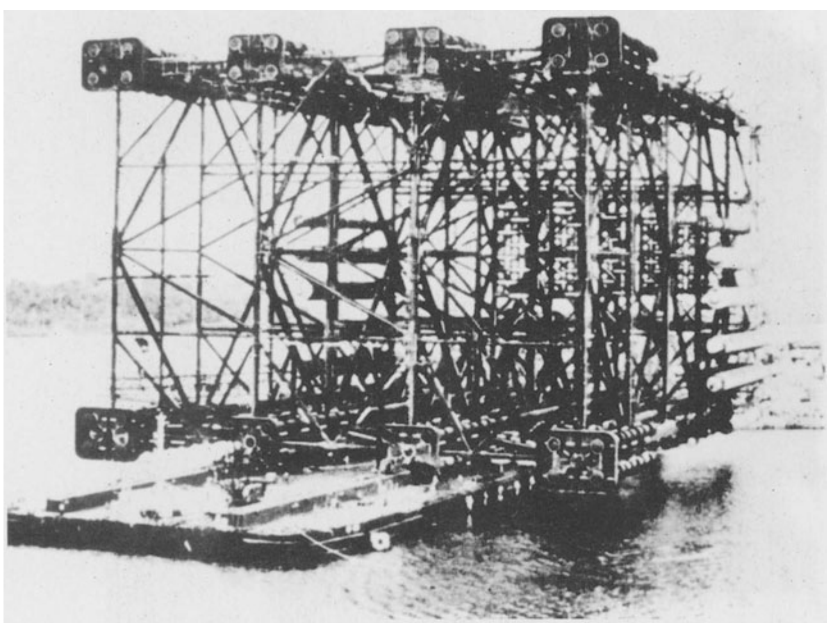


FIG. 10

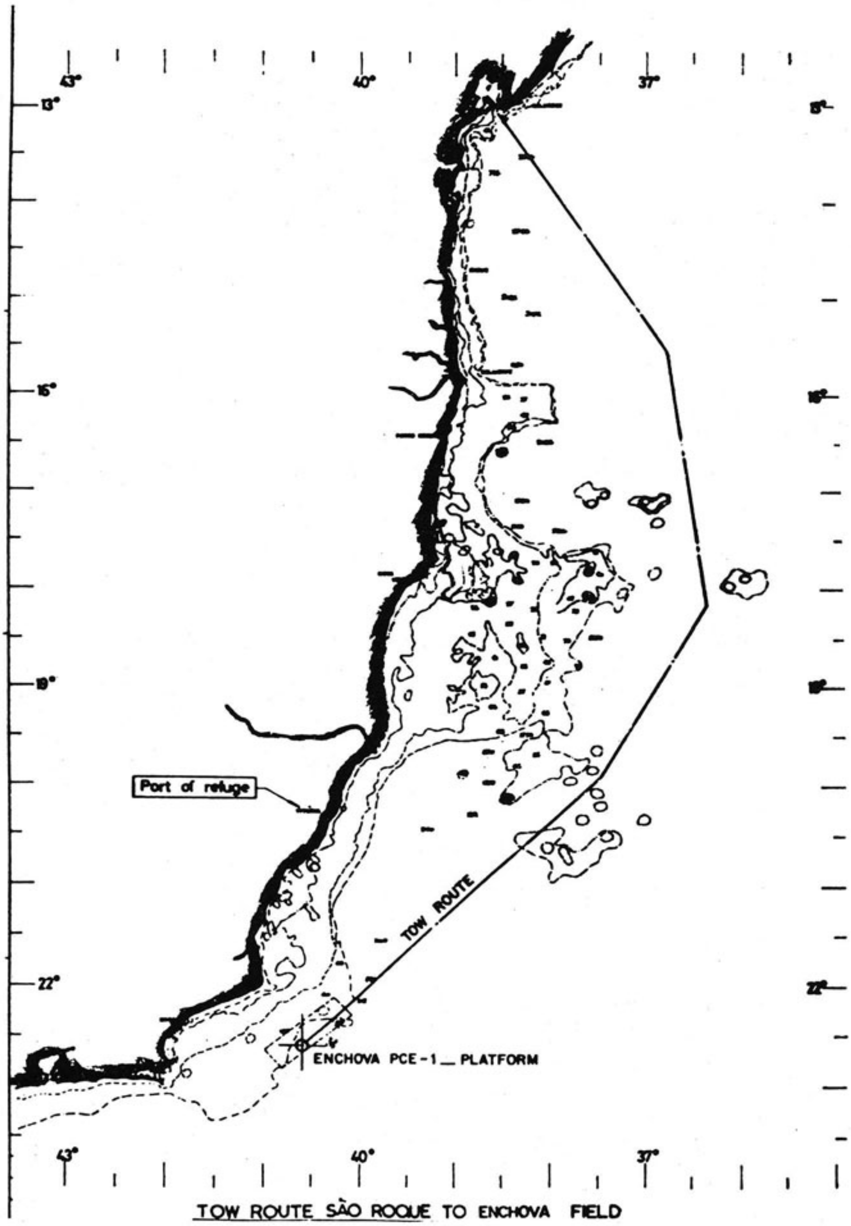

43°
40°
37°
13°
16°
19°
22°
Port of refuge
TOW ROUTE
ENCHOVA PCE-1 _ PLATFORM
TOW ROUTE SÃO ROQUE TO ENCHOVA FIELD

FIG. 11

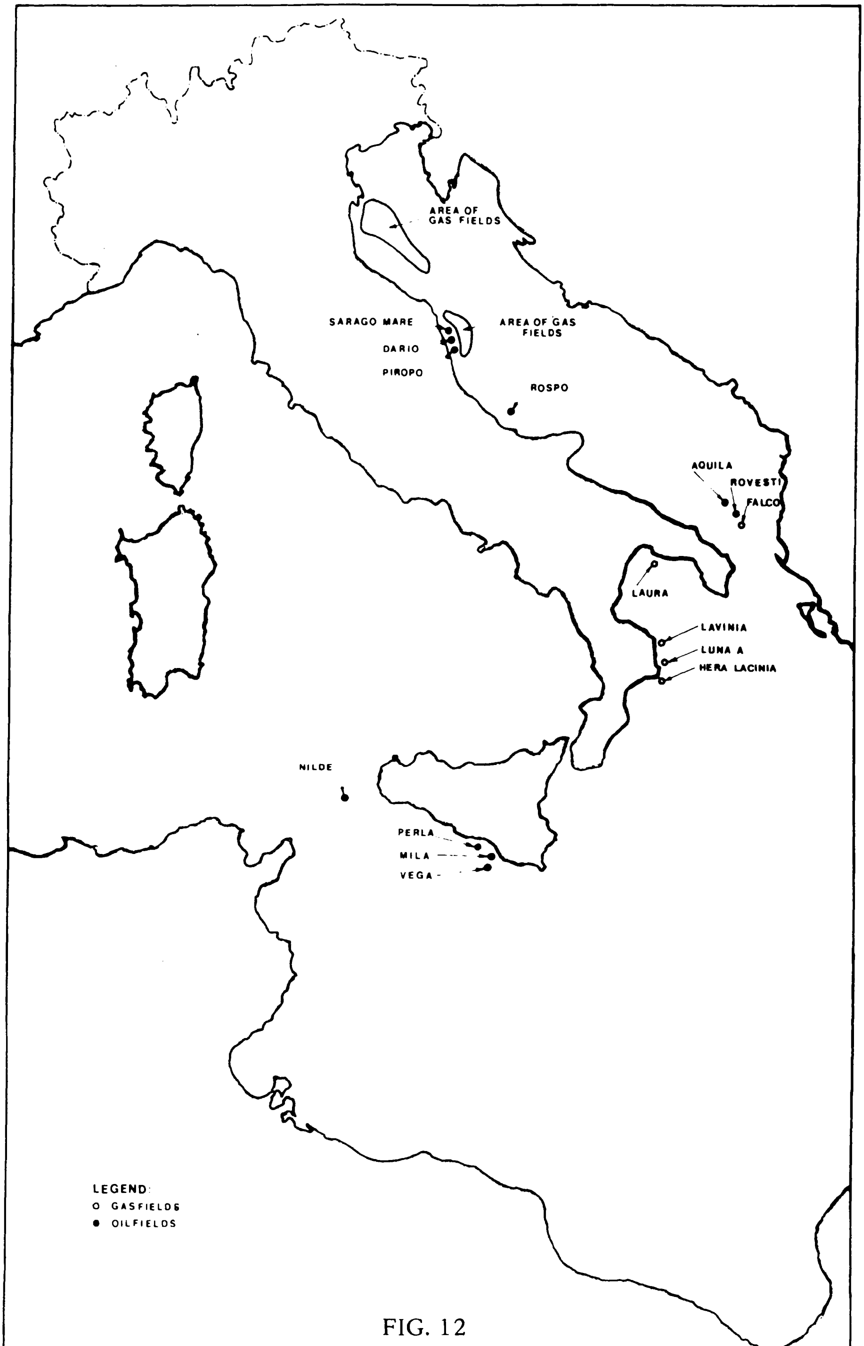

AREA OF
GAS FIELDS
SARAGO MARE
DARIO
PIROPO
AREA OF GAS
FIELDS
ROSPO
AQUILA
ROVESTI
FALCO
LAURA
LAVINIA
LUNA A
HERA LACINIA
NILDE
PERLA
MILA
VEGA
LEGEND:
O GASFIELDS
● OILFIELDS

FIG. 12

ROSPO MARE

Marine Operations Main Data

Field Location	Adriatic Sea
Water Depth	77 m
Jacket	
Height	81 m
Base dimensions	34m x 34 m
Top dimensions	14 m x 14 m
No. of legs	4
Weight at load out	1850 t
Buoyancy Tanks	
No./size	2/2.9 m diam. x 20.3 m length 2/2/9 m diam. x 23.5 m length
Launching barge	96 m x 28 m x 6.5 m 10500 t dw
Tow data	
length/duration	2600 nautical miles/20 days
maxwave height/period	5.4 m / 7.4 s
wind speed	64 Knots

FIG. 13

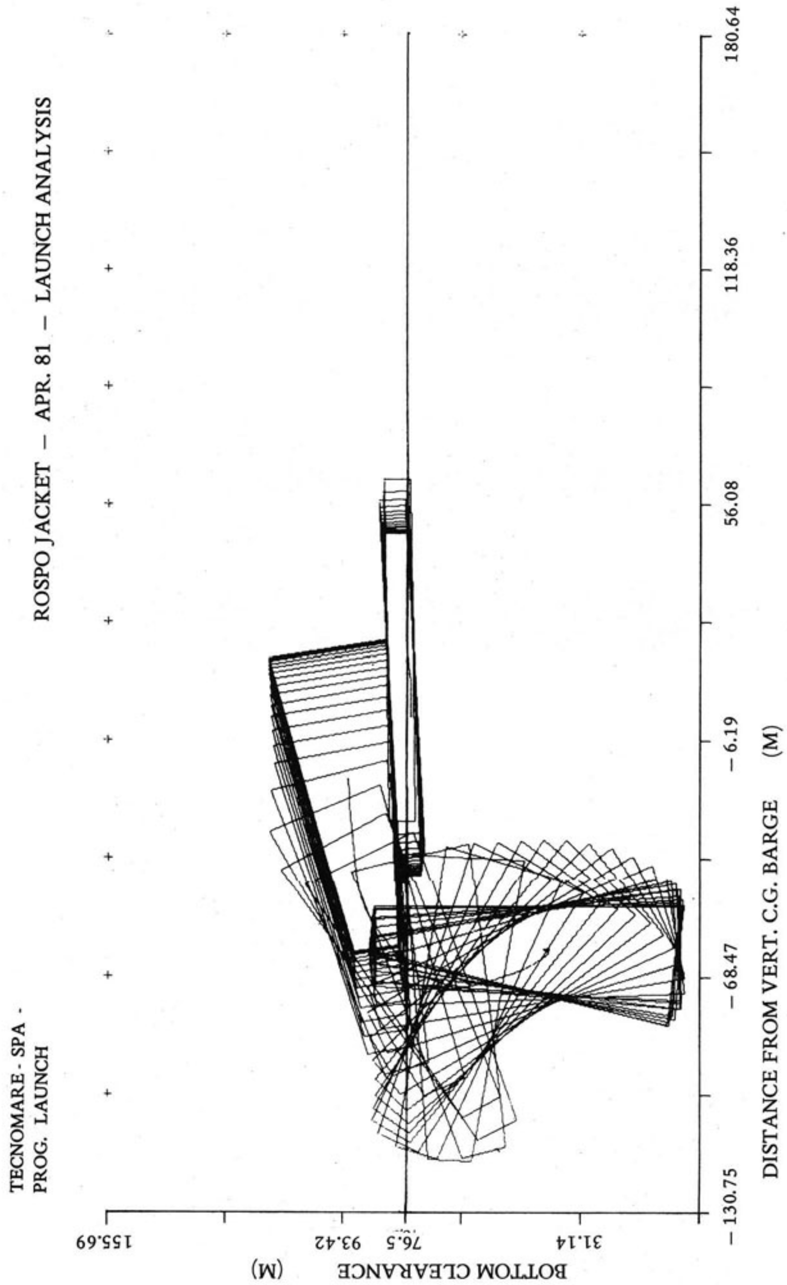

ROSPO JACKET – APR. 81 – LAUNCH ANALYSIS
TECNOMARE - SPA -
PROG. LAUNCH
BOTTOM CLEARANCE (M)
155.69
93.42
76.5
31.14
DISTANCE FROM VERT. C.G. BARGE (M)
– 130.75
– 68.47
– 6.19
56.08
118.36
180.64

FIG. 14

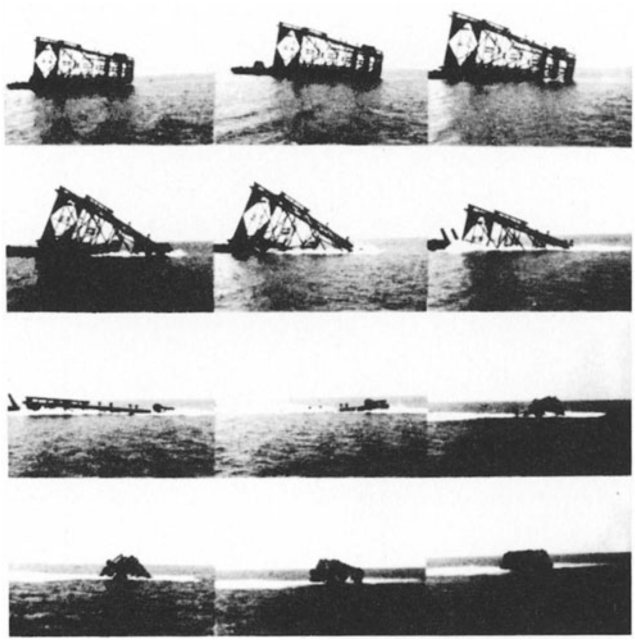


FIG. 15

MAUREEN
THE FIRST STEEL GRAVITY PLATFORM IN THE NORTH SEA

A. Agostoni and E. Gnone

Tecnomare S.p.A.

Tecnomare has designed, on behalf of Phillips Petroleum Company E & A, a Steel Gravity platform, with integrated oil storage, to be installed in the Maureen Field, in the British Sector of the North Sea. Tecnomare's interest in the Maureen Field development dates back to summer 1975. Since that time several proposals have been made concerning the structure to be used for the exploitation of the field.

In 1977 the Operator carried out detailed studies regarding two basic development concepts which were:

A) A workover and production platform installed over a subsea template with predrilled wells.

B) A drilling and production platform installed on a bare seabed.

Both concepts were to incorporate storage facilities and a tanker mooring and offshore loading system.

While the latter is a conventional field development, the former is an application of the "early production" philosophy.
A comprehensive and detailed study was undertaken, approaching many designers and fabricators and investigating all kinds of available solutions.
The conclusion has been that the early production concept was practicable and that the Tecnomare Steel Gravity (TSG) platform was the most suitable to accomplish it.

BASIC PHILOSOPHY

The early production concept aims at shortening the field development time and hastening the investment cost recovery, which is particularly important in order to render the exploitation of marginal fields profitable.[1] Moreover, in most cases, marginal fields require storage capacity and loading systems, as the long distance from the coast or from larger offshore facilities does not justify the use of submarine pipelines for transportation.
The above can be accomplished by:

- Drilling the production wells and fabricating the platform at the same time.
- Using a topside facility fully equipped onshore, thus minimizing the time lag from installation to production start-up.
- Providing the storage required.

The first point calls for a platform capable of being installed over a template of predrilled wells, the second for a platform capable of transporting an integrated topside deck, the third for a platform with integrated storage capacity.
The Maureen TSG platform has been installed during summer 1983 over the subsea template, through which the production wells have been drilled and capped. Production casings are being tied back to surface conventional wellheads.

All the critical problems concerning the installation of a platform over a template such as the required accuracy and the necessity to avoid damage to the template itself, are positively solved by the TSG platform, which presents the following main features:

- Self-contained ballasting and mooring system which allows the platform positioning and setting to be controlled by an operator console on the platform temporary deck.
- A particularly good floating dynamic behaviour which is even better than that of a semisubmersible of comparable displacement when a proper draft is reached.
- Foundation base divided into three separate circular pads, providing space and easy accessibility for the installation over existing underwater equipment.

The Maureen TSG platform has also transported to the field the drilling and production topside facility which consists of an integrated deck fully equipped on shore. The deck loaded onto a single barge was mated with the TSG substructure in the calm water of Kishorn. The TSG substructure basic configuration has been modified to accomodate the hideck and to allow the possibility of performing the mating operation.

The cylinders of the Maureen TSG platform ensure a total storage capacity of 650,000 bbls. In general, the available storage and the transportable payload are related: the larger the payload to be accomodated the larger the built-in storage available and vice-versa.
In the Maureen case the required storage provides the platform with the buoyancy and the naval stability needed to safely transport the payload.

MAUREEN TSG DESIGN DEVELOPMENT

The platform which has been successfully installed on 17 June 1983 is different from the one foreseen in 1978, which weighted 22750 t.

During the development of the design, substantial changes have occured in the design premises which have lead to the present structure.

The design parameters which mainly affect the final weight of the platform are: design rules, water depth, waves height and associated periods, fatigue wave spectrum, soil characteristics, payload during operative life, payload in tow, deck configuration, deck installation procedure, platform installation procedure.

The most significant changes and their consequences on the TSG weight are briefly discussed in the following.

Water depth, waves and associated periods.

The final water depth decreased from 99.06 m to 95.06 m.

The waves/periods changed from h = 25.7 m/T = 15 s to h = 27.00/T ranging from 13.5 to 17 s as extreme condition during the life of the platform.

The towing condition was 11.3 m maximum wave height. The TSG structure has been towed from Hunterston to Kishorn in November 1982 with a premise of 19.0 m max. wave height and T from 9 to 15 s associated periods. The whole platform, TSG and topsides has been towed to the Maureen field early summer 1983 with a premise of 14.0 m max. wave height and T from 8 to 14 s associated periods.

The combined effect of water depth reduction and wave increase have lead to a significant increase of the environmental forces.
In installed status the global horizontal force, including wind and currents, increased from 21,500 t to 30,500 t and the maximum of the vertical force on a single base passed from 28,000 t to 38,000 t.

The heavier conditions during tow caused an increase in forces and motions of the platform which affects both main structural members, like diagonals, and appurtenances, link walkways and piping supports.

Soil characteristics.

The design shear strength of the critical soil layer decreased from 144 to 101 KPa, requiring an increase in the necessary foundation surface area and skirt height. In addition the prevision of the existence of a layer of mud cuttings over the foundation soil required an internal circumferential skirt, radial ribs and a further increase of the height of the external circumferential skirt.

Payload during operative life.

The total payload passed from 18,100 to 28,500 t. This 5% increase directly influenced the overall weight of the structure.
It caused the increase in leg thickness and in the number of diagonals necessary to transfer the loads from the framed structure to the cylinders. The increase in the number of diagonals has also caused the increase in the dimensions of the joints and their complexity. In addition it has been necessary to add the vertical pilasters to the cylinders in order to transfer the overload to the foundation bases.

Payload in tow.

The towed payload passed from 11,700 t to 18,500 t. This 58% increase has generated an addition of two diagonal members to the top of one cylinder, a size increase in diagonal members in the upper part of the tower, an extension of the cofferdam in correspondence of the waterline during tow.

Deck configuration and installation procedure.

The initial deck configuration was conceived as a module support structure integrated with the hexagonal framed tower.

The decision to adopt the hideck configuration has determined:
- a modification of the upper part of the hexagonal tower in order to accomodate a deck on two support rows,
- the necessity to leave an open space on top of the tower and between the two support rows to allow the barge to enter inbet-

ween for for the mating operation,

- a lower efficiency in transferring the loads from the deck to the cylinders and the bases.

The result is an increase of leg diameter from 2.5 to 3.0 m, the addition of four diagonals to transfer the deck loads to the two suppressed legs of the original hexagonal tower, and a general increase in dimensions and complexity of joint in the upper part of the tower.

The necessity to transfer the platform (without any deck) from Hunterston to the deck mating site in an open sea tow, has required the installation of the temporary deck.

The deep water immersion of the platform for deck mating and the fact that the upper part of the tower is open for hideck mating has required the cantilevering of the temporary deck to the external upper part of the tower with a corresponding weight increase of the temporary deck itself.

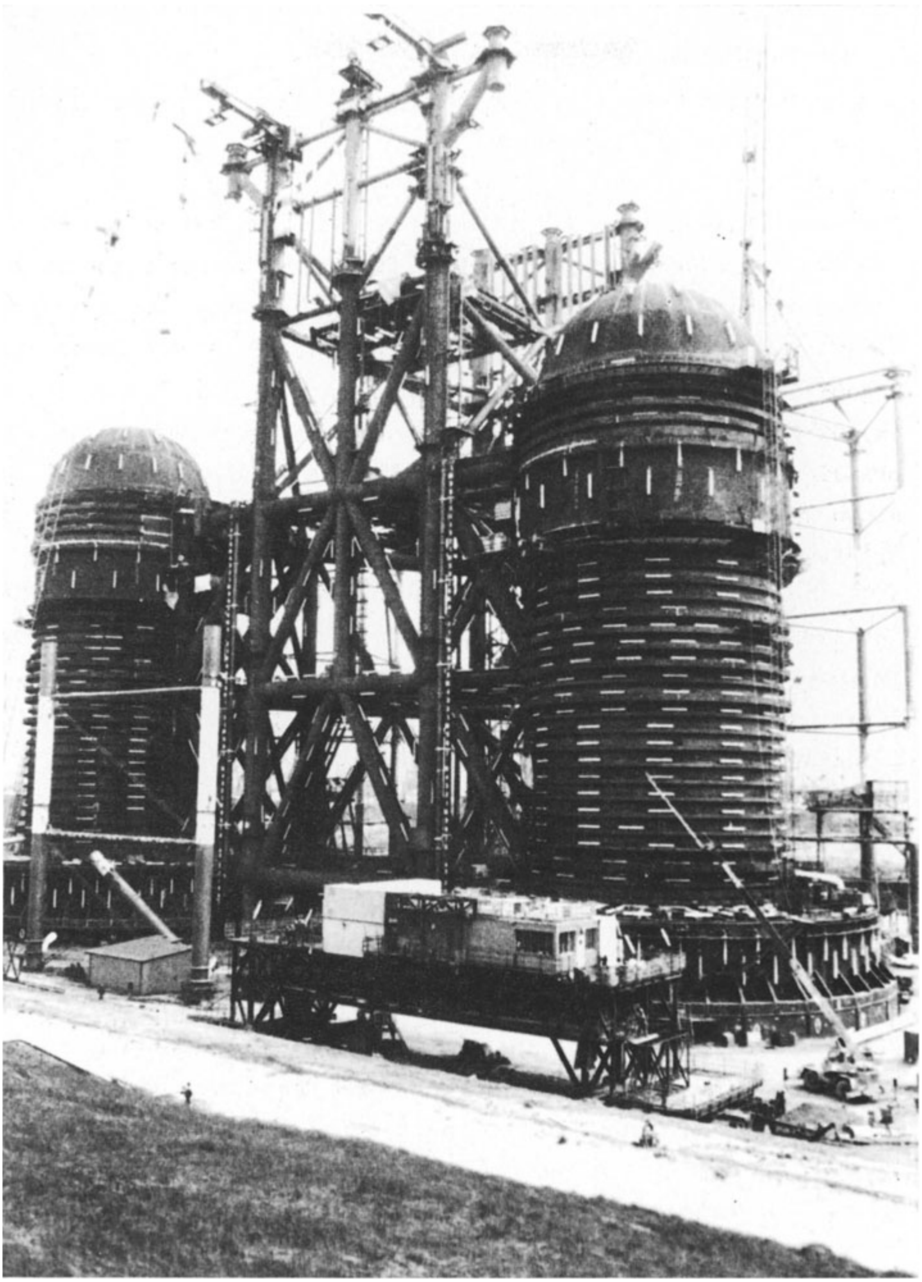


Figure 1 - Maureen TSG

PLATFORM DESCRIPTION

Figure 1 shows the Maureen TSG platform at the end of fabrication in AMC yard at Hunterston, Scotland. The design main premises, the principal geometrical dimensions and the weight breakdown, are shown in Figure 2. The items which characterize this platform are briefly described hereafter:

The framed structure is a tubular three-dimensional network composed of:

- two rows to match the deck superstructure legs and to allow its installation by a barge passing through the supports themselves
- one lower triangular structure connecting the framed structure to the storage/stabilizing cylinders and to the foundation bases.

The storage/stabilizing cylinders are circumferentially ring stiffened shell structures with the main rings corresponding to the connections to the framed structure. Each cylinder has two hemispherical cupolas (one at the top and one internal) and two vertical pilasters welded to its outside surface in order to improve the transfer of the laods from the framed structure.
The internal cupola has the aim of subdividing the cylinder into two parts in order to provide the necessary stability in case of damage during transportation.

STORAGE CAPACITY	650,000 bbls (9 days)	
MAX PAY-LOAD (installed)	28,500	t
MAX PAY-LOAD (in tow)	18,500	t
NUMBER OF CONDUCTORS	24	
DECK DIMENSIONS	76 x 79	m
WATER DEPTH (LAT)	95.6	m
DESIGN 100-YEAR WAVE HEIGHT	27	m
MAX SURFACE CURRENT	1.25	m/sec
STRUCTURE HEIGHT	120	m
STORAGE TANK HEIGHT	74	m
STORAGE TANK DIAMETER	26	m
BASE DIAMETER	47	m
BASE HEIGHT	6.8	m
BASE CENTRE SPACING	90	m
FRAMED STRUCTURE WEIGHT	12,450	t
STORAGE TANKS WEIGHT	12,650	t
BASES WEIGHT	9,700	t
TOTAL STRUCTURE STEEL	34,800	t
BALLAST WEIGHT	50,000	t
TEMPLATE WEIGHT	490	t

Figure 2 - Maureen TSG main characteristics

Moreover, a cofferdam, at the water plane, has been provided to protect the cylinder against accidental ship collisions.

The foundation bases: the cylinder shell extends inside each base as a continuous structure. In correspondence to the lower part of the base it becomes a ring foot whose function is to support the platform just after the installation. The base extends radially with plane frames tied together by rings.
Inside the bases the cylinder shell has both longitudinal and circumferential stiffeners, the lower outside part of each base is composed of a skirt 3.8 m high with longitudinal stiffeners.

The storage tank system is represented by the three tanks (cylinders) connected to the deck facilities through separate filling/emptying lines to allow, if necessary, independent operations for each cylinder or direct tanker loading. The system operates on the water displacement principle with free circulation through the internal cupolas.

The ballasting systems installed on the TSG platform are two: the water ballasting/deballasting and the solid ballast placement systems.
The first provides the capacity to raise and lower the platform during the different naval operations by emptying or filling with water the three cylinders. The solid ballast placing system fills the bases with solid pumpable material whose weight assures the necessary platform stability both when floating and when standing on bottom.

The mooring systems provided in order to accomplish all inshore and offshore marine operations are two: an inshore mooring system and one offshore at Maureen site.

The first has been used for solid ballasting and deck mating operations and is composed of anchors and one winch on the shore; the second controlled the installation of the TSG over the template and is composed of four mooring piles driven into the seabed.

Two towing systems were available, one was used during winter '82 for the float-out from the construction dry-dock and first tow to the inshore mooring location; the second was for the tow of the complete platform to the Maureen site.

The positioning system is provided to guarantee the position accuracy between the template and the platform necessary to perform the tie-back operation; this is composed of two subsystems:
- the primary mooring positioning system controlled via acoustic and video reference equipment,
- the close positioning system consisting of two docking-pipes / guide piles which work as mechanical rails for the platform during its final lowering.

The platform corrosion protection system has been foreseen as follows.
Extra thickness has been provided on the steel of all the members in the splash zone. External areas of the submerged zone are protected by sacrificial anodes. The inside surfaces of the storage tanks have been protected by application of an epoxy type coating associated with sacrificial anodes and extra thickness of the shell steel.

The monitoring and maintenance system allows the checking of the structural integrity of the platform and of its main components during their operating life. Furthermore this system has the

capability of accounting for the fatigue state of the platform with the aim of better planning the inspection program.
The items strictly related to the TSG concept are detailed in the following.

TEMPLATE

The template for the Maureen field is a rectangular framed structure supporting the guides for 24 slots.
The structure, 42 m long, 18.4 m wide and 4.3 m high is fixed to the sea bottom by four 42" O.D. foundation piles.
Figure 3 shows a picture of the template in the final phase of its fabrication.

Two large funnels are connected to the main structure in order to act as guides during the driving of the "guide-piles".
In this way it has been possible to achieve a very accurate relative position between the template and the guide-piles.
This is of primary importance for the final positioning of the platform which is guided during the installation by the guide-piles and must be positioned with a certain accuracy with respect to the template.
After the connection of the template to the foundation piles and the driving of the guide-piles, the two "wings" connecting the funnels to the main structure were cut by the divers. That was done in order to completely divorce the template from the guide-piles, thus avoiding the transfer of any load from the platform to the wells during the TSG installation.

The Maureen template was installed in June 1979 and drilling operations have been completed before the platform installation.

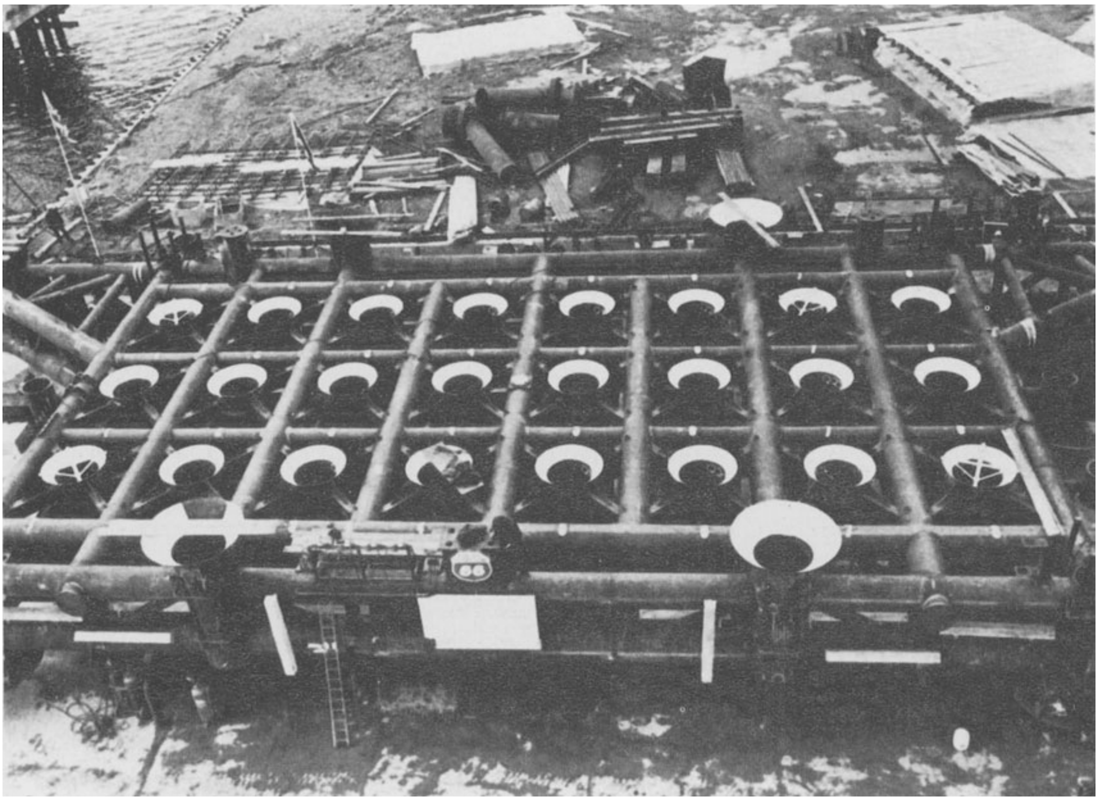


Figure 3 - Maureen Template

HYDRODYNAMIC ASPECTS

For this structure, which is composed of large bodies and framed elements, the calculation of wave loads has been carried out taking into account the following main aspects:

- the force acting on the large bodies must be evaluated by using the diffraction theory
- the presence of large bodies modifies the incident wave pattern.

These effects have been analyzed by using the in-house computer program CARDIF based on the three-dimensional sink-source technique.[2] Both towing and installed conditions are influenced in different ways.

The towing condition is the most affected by diffraction due to the effect of the short period wave on the merging cylinders.
The main results of the analysis are:

- the forces on each single cylinder are different from those obtained by using conventional methods; an actual mass coefficient (CM) has been derived
- a mutual influence between the cylinders is not negligible; the total force has a transverse component with respect to the wave direction which reaches 50% of the longitudinal one (Fig. 4).

The computerized procedure is based on the following steps:

1) Calculation of the wave inertia forces on large bodies by using the diffraction theory

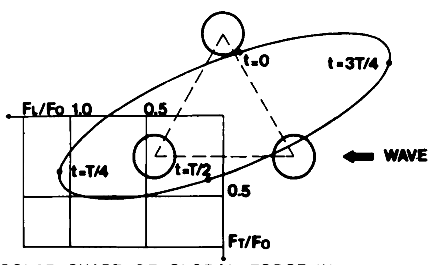
2) Evaluation of the modified (incident+diffracted) hydrodynamic field around the large bodies
3) Environmental load calculation on the entire structure by using step 1 and 2 results.[3]

The calculations have shown a sensible reduction of the global wave forces with respect to those acting on a single body platform of the same volume.

HORIZONTAL FORCE AMPLITUDE F/Fo

FLOATING CONDITION Ka=0.5

$F_o = \rho g \pi a^2 \frac{H}{2}$



POLAR SHAPE OF GLOBAL FORCE IN A WAVE PERIOD

FORCE COMPONENT	INTERACTION CONSIDERED	INTERACTION DISREGARDED
IN LINE F_L/F_o	1.07	0.94
TRANSVERSE F_T/F_o	0.52	-

Figure 4 - Hydrodynamic Loads

FOUNDATIONS

The design approach of the foundations of the Maureen platform is significantly different from that of a monolithic gravity structure.
For tripod configuration, in fact, a particular type of soil structure interaction occurs, and the mutual influence between the bases must be accounted for.
The foundation analyses have been affected by the complexity of the stratigraphy and by the presence of an unknown (in design phase) startum of cuttings due to the drilling operations which, in the "early production" concept, are carried out before the installation of the platform.

The base/foundation configuration includes the following features:

- Skirt and rib arrangement under base raft, to provide the required sliding resistance.
- A ring foot under each base in correspondence to the cylinder shell to reduce the effect of soil uneveness, to transfer directly to the cylinder the load during the installation and to provide a sufficient clearance under the base for grouting operations.
- An active drainage system to eliminate the excess of pore pressure in the closed sand chamber existing below each base.
- Underbase grouting has been foreseen to prevent the effects of soil uneveness due to the presence of drilling cuttings.

The foundation analysis has been carried out through the following main steps:

- Stability checks carried out taking into account the interaction between the bases.
- Sensitivity analysis of the structure/foundation to local uneveness, differential slope and soil property uneveness both in installation and operating phases.
- Elasto-plastic F.E. analysis of the whole base/soil with the aim of studying the soil-base interaction, the loading condition on the skirts and the stress pattern in the soil to be used in cyclic loading analysis. Figure 5 shows the distribution of horizontal force among leading skirt, base bottom and following skirt.
- Evaluation of the excess of pore pressure generation in sand during storm.
- Evaluation of the effect of cyclic loading in clay properties.

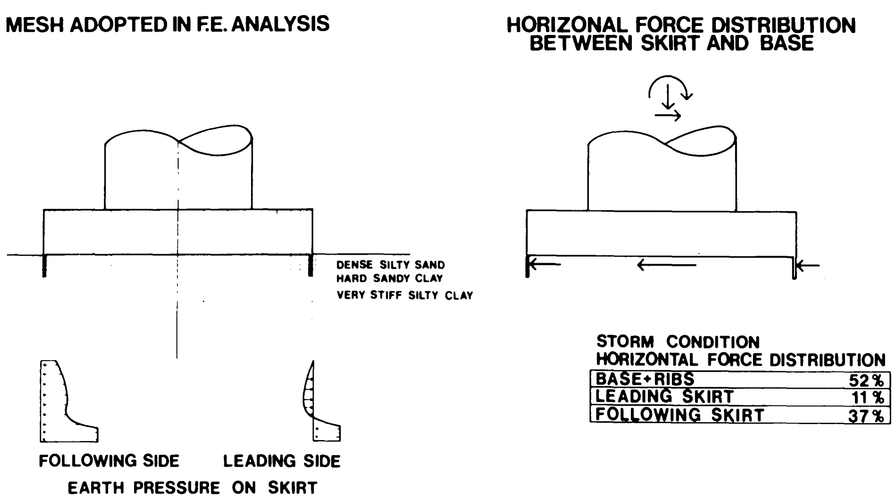


Figure 5 - Soil/Base Interaction

JOINT ANALYSIS

The most remarkable step of the design procedure for the dimensioning of the tubular joints of the Maureen platform has been the calculation of the Stress Concentration Factors (F) to be used in the fatigue analysis.
The geometry of the joints of large steel offshore structures like this is rather complicated and in general it differs from one joint to another. For these reasons to calculate the F with Finite Element analysis on actual joints would be extremely expensive and time consuming. On the other hand, due to the complexity of the geometries and to the presence of the stiffeners inside the chords, it is not possible to calculate the F by formulas like those generally used for the simpler joints.

For the Maureen platform the following procedure has been used:

- The complex geometry of the joints has been subdivided into simpler geometries which may be considered independent from each other. In this way the complete joints have been divided into a certain number of "K" "X" and "T" connections.
- A F.E. calculation has been performed on these simple unstiffened connections in order to check and calibrate the mathematical model used with the most widely accepted formulas
- An extensive parametric F.E. analysis has been performed on the stiffened simple connections.

The different parameters involved (diameter and thickness of the members, number and size of the ring stiffeners, number and size of the longitudinal stiffeners, etc.) have been varied within the ranges of interest for this particular application. As an example of the results of this parametric study Figure 6 shows the variation of the F with respect to same parameters for different loading conditions.

- The results of the above F.E. calculations have been used for the dimensioning of the joints of the platform.

From the above, the following main guidelines have been derived:

- No longitudinal stiffener has been used, as the calculation has shown that their influence of the F is very low, at least within the dimension ranges taken into consideration.
- The thickness of the chord has always been kept to the maximum possible value, as it has been demonstrated to be the most effective parameter in reducing F.
- The dimensions of the ring stiffeners have been assumed constant for the different joints, as it has been seen that the F are much more sensitive to the number than to the dimensions of the stiffeners.

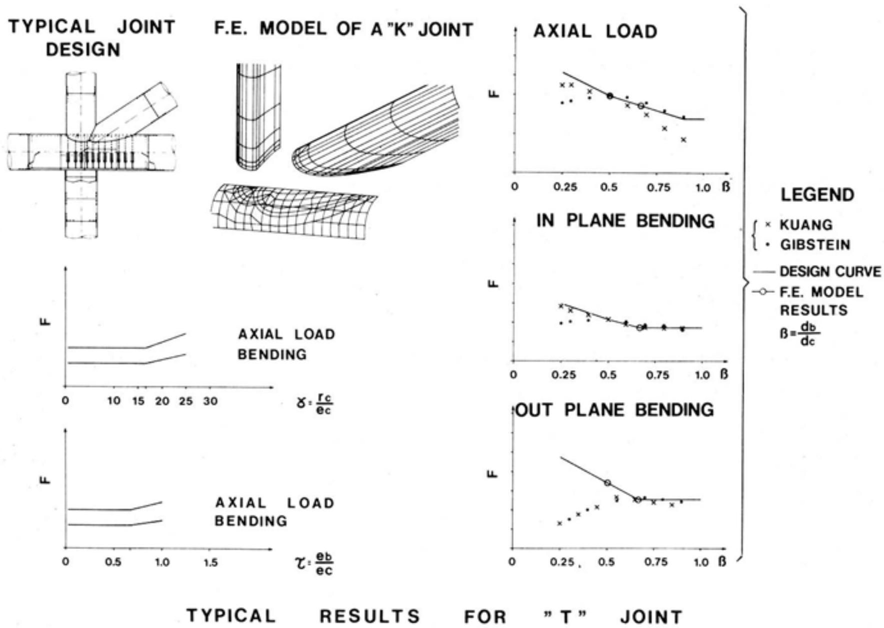


Figure 6 - Typical Results for "T" Joint

POSITIONIG AND INSTALLATION

The Maureen TSG platform has been equipped with proper mooring positioning and installation systems.
The mooring system is composed of:
- four anchor piles set at a proper distance from the template
- four winches running the anchor lines for the mooring operation.

The positioning system is composed of two subsystems: the close positioning subsystem (CPS) which is a mechanical system which permits the guidance of the platform during the final setting in order to achieve a position within the required accuracy (about 25 cm) necessary to perform the tie-back operations.
The instrumentation subsystem (PMIS) which is composed of the acoustic navigation system capable of guiding the platform during the towing and anchoring phases, and of a series of sensors installed on the platform to monitor and control the installation phases.

C.P.S.

The C.P.S. system is composed of two equal elements each including the following main items:

- Guide pile, docking pipe, upper support guide, lower support guide, docking pipe lowering winch, lower support instrumentation, docking instrumentation.
  Figure 7 shows the docking pipe arrangement.
- A guide pile is driven into the seabed and is provided with a head particularly shaped to allow the docking pipe engagement.
- The docking pipe is guided by two anular supports (lower and upper support guides) and is long enough to be lowered approximately 15 m to mate the guide pile with a maximum 1 m offset.
  It consists of two cylindrical sections of different diameters connected by a tapered conical transition shaft in such a way that, when it is in the extended position, there is a gap with respect to the guides which permits an easy mating with the guide pile. The conical transition, once engaged, guides the platform smoothly to the required position. A funnel arrangement at the docking pipe lower end is provided to facilitate the engagement of the guide pile head.
- The upper support guide is an anular guide in which the docking pipe upper section slides with a minimum clearance.
  It is connected through a proper rubber fender arrangement to the main frame of the structure in order to reduce the dynamic loads due to the wave motion.

- The lower support guide is similar to the upper one and is connected to the lower frame. The guide ring internal diameter is designed to fit with a close tolerance the docking pipe larger section during the last setting phase.
- The docking pipe lowering winch installed on the deck permits the engagement of the docking pipe with the guide head.
- The lower support instrumentation monitors the displacement of the guide ring and consequently the guide pile with respect to the platform. It gives all the information relating to the setting positions and the forces acting on the guide system.
- The docking instrumentation foresees the use of a TV camera installed on each docking pipe to monitor the actual position of the docking pipe with respect to the guide pile and acts as a control system with respect to the acoustic positioning system.

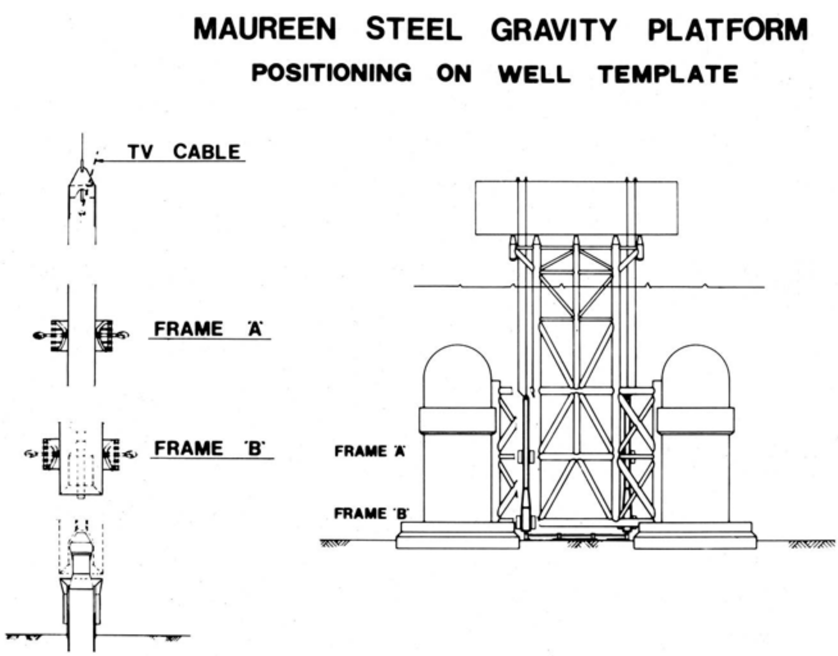


Figure 7 - Docking Pile Arrangement

P.M.I.S.

The positioning, monitoring and instrumentation systems consist of a series of sensors which are capable of monitoring the most significant parameters which permit the safe positioning and installation of the platform.
All data are collected and processed by a computer which is also used for the structural monitoring during the platform life.
Figure 8 shows the P.M.I.S. block diagram.

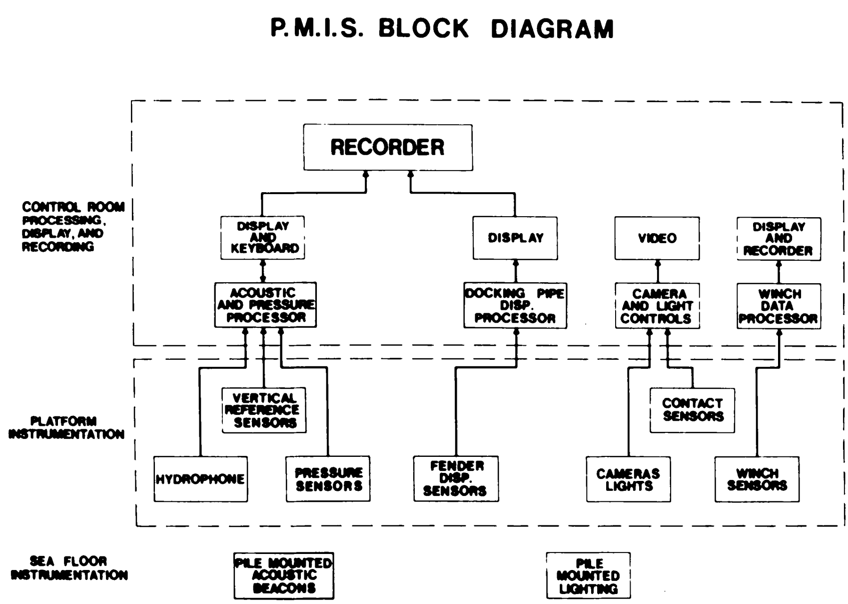


Figure 8 - P.M.I.S. Block Diagram

STORAGE PROCESS SYSTEM

The TSG platform oil storage system is integrated with the three stabilizing cylinders, which, in the Maureen case, provide a total storage capacity of 650,000 barrels of crude oil.
The storage system, based on the oil/water displacement principle, will be operated as a single unit with oil-water interfaces inside the three tanks, kept at the same level.
Provision is made, however, to isolate one or two of the three tanks during oil filling or emptying operations, in order to meet safety or operational requirements in particular situations.
Facilities are also provided in order to control the filling/emptying rates of each tank or to by-pass all the cylinders, thus allowing direct tanker loading of the produced oil.
A simplified flow-sheet of the oil storage system is shown in Figure 9.
The three tanks are connected to the production facilities through oil filling/emptying lines tied to the top of the cylinders. The displacement water line comes off the bottom of each tank. All pumping and control equipment are installed on the platform deck. The oil lines from the tanks pass into a manifold on the deck; from the manifold a line passes into an oil loading surge tank.
The purposes of this surge tank are to provide a flooded suction at the oil loading pumps at all times and to provide a point for gas relief in case high vapour pressure crude gets into the tanks.
Water is displaced from the bottom of the tanks as oil is added from the top. The flow of water into and out from the tanks is

controlled by a water surge tank. Water coming from the storage tanks is delivered to the surge tank and then to the oily water treatment unit, when the overflow level is reached. When the oil loading pumps begin to pump oil out of tanks the water level drops in the surge tank and a low level switch will open the cooling water return header for the tanks water filling.

The function of each storage tank is similar to that of a manometer with water in the leg leading to the water surge tank and oil in the leg leading to the loading surge tank.

Because of the difference in density between sea-water and crude oil, gas pressure has been added to the oil leg to keep the oil level at the desired point in the oil loading surge tank.

A proper level and temperature measurement system is associated with each storage tank in order to have a good operational control.

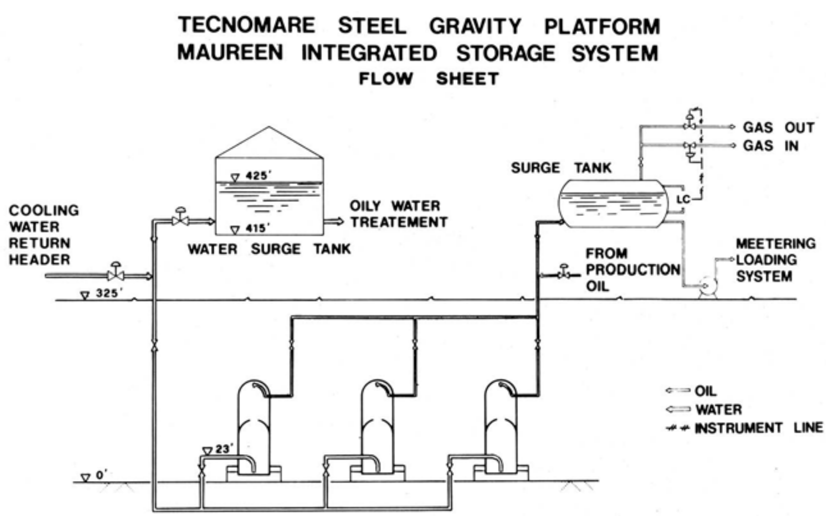


Figure 9 - Oil Storage System

STRUCTURAL MONITORING SYSTEM

For Maureen TSG platform an advanced permanent structural monitoring system has been provided. The main functions of this instrumentation are:

- To check that the performance of the structure foundation in field will be in agreement with the design assumptions and calculations.
- To use the collected data and the results of their analysis to get an optimized inspection planning and in principle to reduce the total amount of inspections.
- To perform occasional or periodical checks of macrointegrity of the structure.

The system includes the following sensors:

- wave and wind sensors
- settlement sensors
- inclinometers for overall tilting and for single inclination of each base measurement,
- accelerometers on the deck and on each base,
- strain gauges on three structural members,
- sand pore pressure sensors.

The location of the sensors is shown in Figure 10.
The integrity check of the structure during its working life is achieved by means of the analysis of the dynamic response modification and by accounting for the actual fatigue damage.

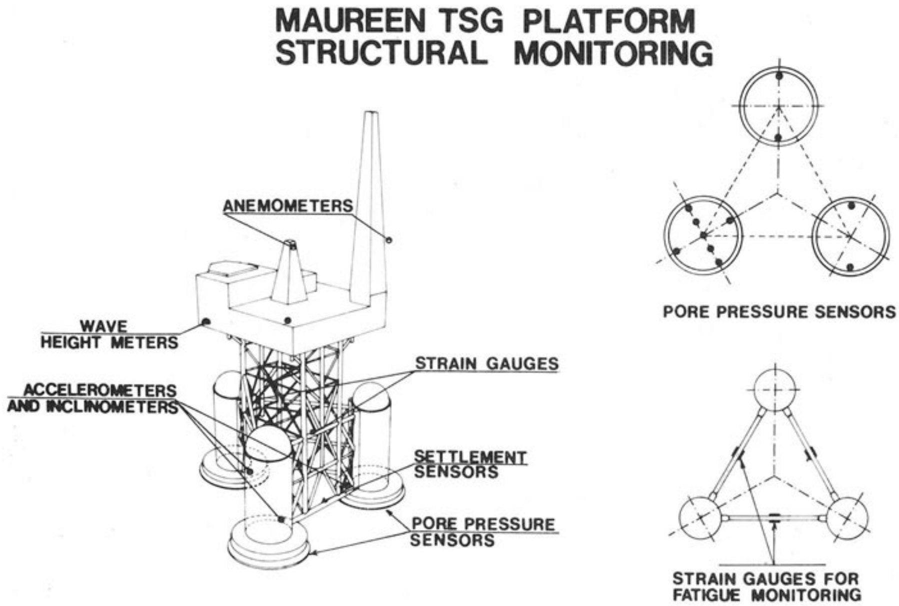


Figure 10 - Structural Monitoring

Dynamic response monitoring.

It includes four points (on the deck and on each base) fully instrumented by accelerometers. A procedure for dynamic response analysis has been developed in order to get:

- natural frequencies,
- principal normal modes of vibration,
- actual current motions of the instrumented points and of the entire platform.

An overall check of integrity of the main structure and its foundations is possible by interpreting the eventual modifications of the above parameters.

Fatigue built-up monitoring.

This is an original subsystem whose aim is to check in field the fatigue of the structure, by accounting for the actual fatigue damage.

The method is based on monitoring and recording the actual stress history in some members (three in Maureen case) by means of vibrating wire strain gauges.

The method for the actual fatigue damage build-up accounting in field consists of:

- Analysis of the stress history on the instrumented members in order to get a histogram of stress cycles in all members of the structure, by using element stress tables obtained in theoretical structural analysis.
- Periodical updating of the actual cumulated fatigue damage for each joint using the design S-N curve.

The instrumented members have been chosen as particularly representative to check by means of field data the theoretical transfer function between waves and stresses.

The procedure is described in ref.[4]

MODEL TEST ANALYSIS

An extensive model test program on a 1:50 scale model has been performed at the NSMB of Wageningen (Holland). A photo of the model is shown in Figure 11. Scope of the tests was to check the static and dynamic behaviour of the platform and its marine equipment during the different phases of the towing, anchoring, positioning and setting operations.

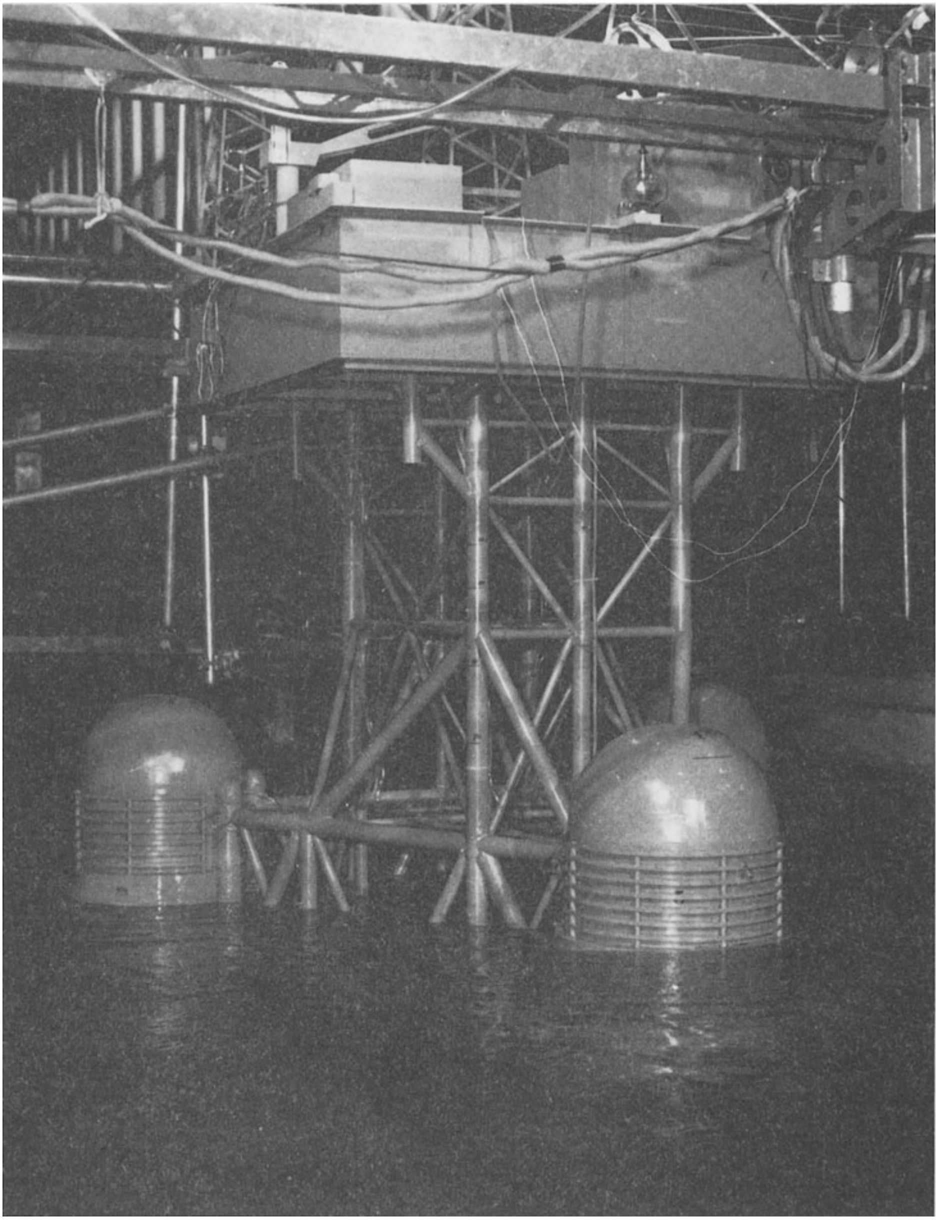


Figure 11 - Model for Basin Tests

CONSTRUCTION, TOW, POSITIONING AND INSTALLATION

In Figures 12 and 13 a general view of the main different phases, from the construction to the installation of the Maureen platform is given.

Construction.

The construction and assembly of the platform was performed at the Hunterston yard on the Clyde Firth in Scotland by AYRSHIRE MARINE CONSTRUCTORS.
The yard has a graving dock 165 x 150 m, 12 m deep where the platform was erected in vertical position (see Figure 1).

The fabrication of the three storage tanks started in late '79 and was was performed at site.
The erection of the interconnecting lattice structure began the following year when the nodes, fabricated by subcontractors became available.
These elements with thicknesses up to 120 mm were fabricated in U.K., Germany, Holland, Belgium and France. Welding procedures, further to hardness and thoughness requirement, had to pass C.O.D. requirements, stress relieving, 100% M.P.I. and U.T. examination and dimensional checks, in the final condition, were specified, to avoid erection problem at site.

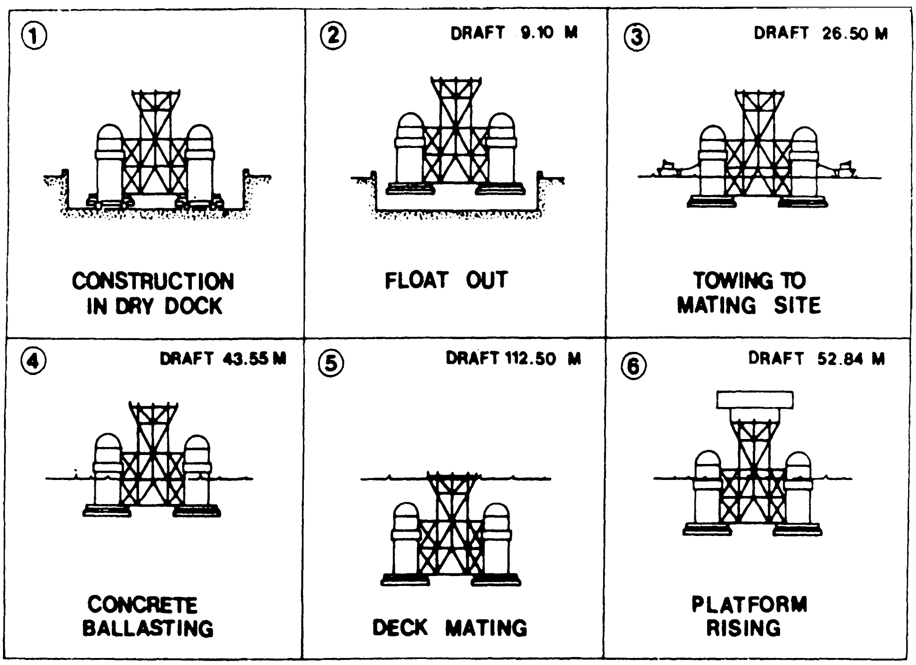


Figure 12 - Marine Operations: From Construction to Deck Mating

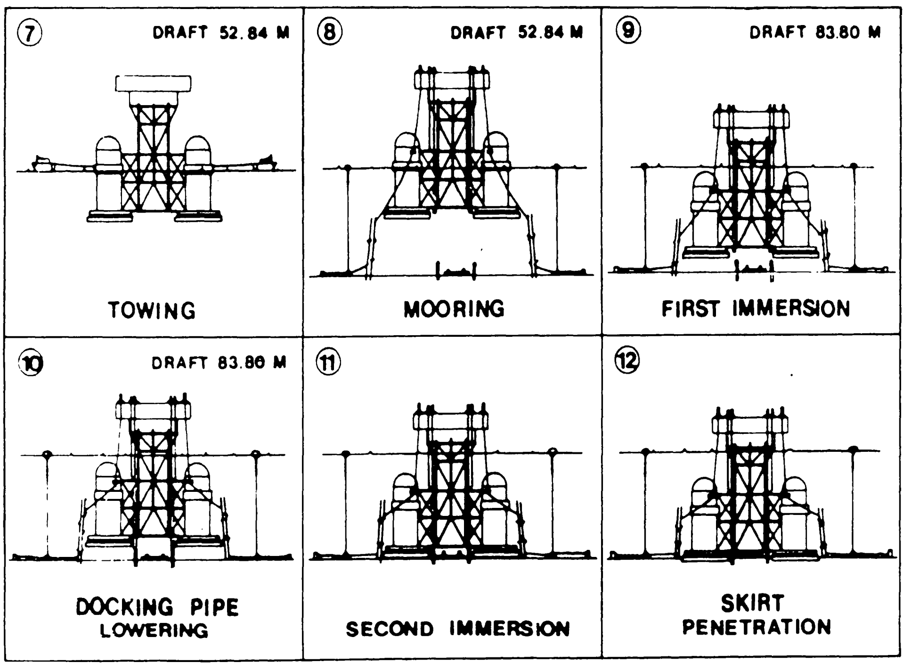


Figure 13 - Marine Operations: From Deck Mating to Installation

With a careful control of welding sequences (to avoid welding distortion) and control of electrodes, flux and their handling procedures to avoid cracks, the final result was that no problems arose at the yard, durig the lattice structures erection.
A main effort was devoted to the check of the avilable code requirements for dimensional tolerances of the structure and where too stringent (costly) requirements were found (as codes were often applicable to different sizes and thicknesses), an analysis was carried out to establish if a release was possible and to evaluate the procedures, the design implications and additional reports/inspections to be carried out. This subject involved: alignement of butt welds welding defects for pressure vessel and nodes out of alignement for crossing of welds, straightness of tubular members, local defects of vessels, etc.
This study required continuous contacts with the certifying authority and led to the definition of new requirements that resulted reasonable for subassembly and erection purposes.
The design effort for the designer included the "as built" dimensional situation of the nodes and established their fitness for use. Only twice, nodes were repaired as a consequence of stub mislocation (60 and 180 mm).

Outside fabrication was widely used with about 30 main subcontractors and over 150 different suppliers spread all over Europe; the subcontractors involved bases, pie sections, nodes, docking pile guides, temporary deck, conductor guide frames, etc., up to and including towing and mooring devices, ladders, walking, platforms, cutting bevelling and rolling of tanks plates, piping spools.
These subassembled pieces, in the range of weight up to 200 tons, once arrived to site were converted into larger assembly up to 300-350 tons before their final erection in the graving dock; in this stage, to save fabrication costs all possible miscellanea

were directly attached to each subassembly up to include ladders, walking platforms, instrument supports, cable conduits, piping spools, anodes, etc. (see Figure 14).

The largest subassembled piece was the temporary deck that after completion at the bottom of the yard, was lifted up to the top of the structure, as a single package in a few hours' operation using its own in-built installation and removal facilities (see Figure 15).

This fabrication procedure, enables to have a gravity type platform completely built onshore, without any special water depth requirement. A main task of the designer, to have the largest possible time and money saving, is to provide the fabricator with all details for all systems from the very beginning to allow their ground level installation for each assembly.

The structure was completed, tested and commissioned in the second half of October 1982, thus beating the 10th November '82 completion date set two years before, when all the implications of design premises changes were known and the yard labour problems were solved.

Dredging operations, to remove the dike, started on July 19, 1982 and the graving dock was flooded (together with the lower part of the structure) on August 30.

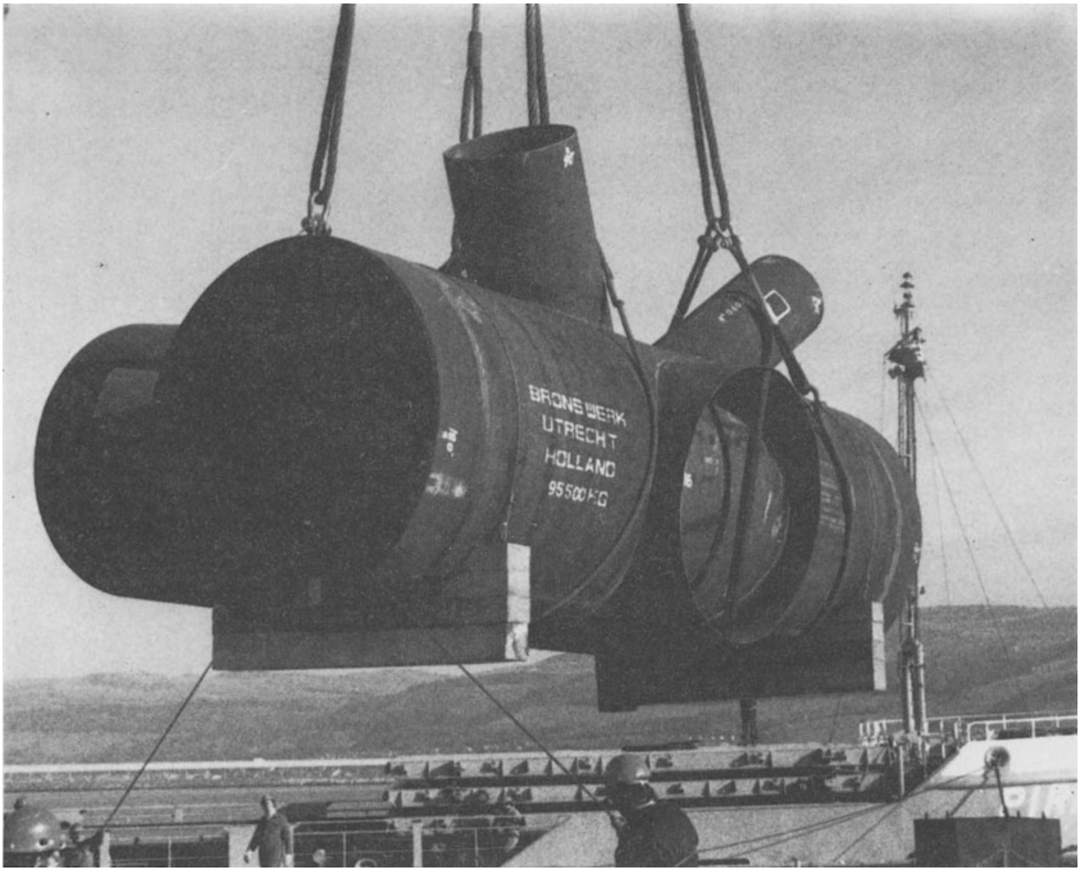


Figure 14 - Node Subassembly

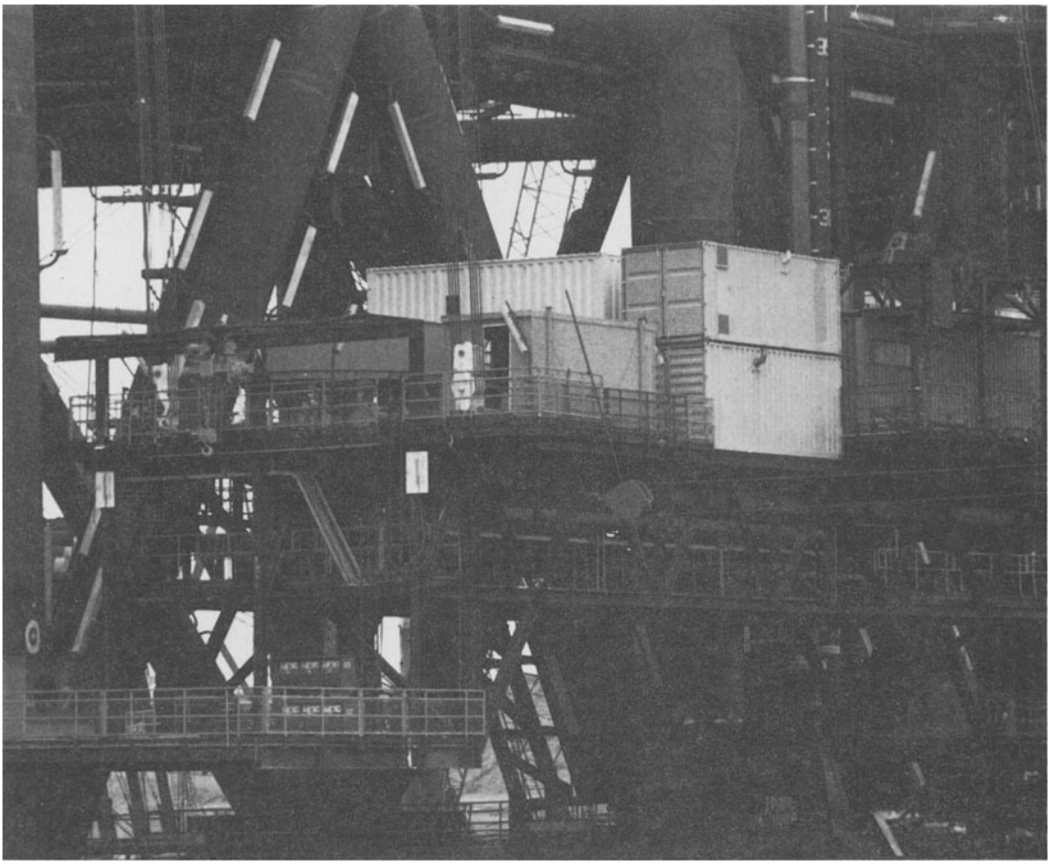


Figure 15 - Temporary Deck Lifting

FLOAT-OUT AND TOW TO THE MATING SITE (SEE FIGURES 16, 17 AND 18)

The TSG has been floated out on the 2nd of November 1982 in favourable conditions. The operations began during the night.
The TSG was deballasted and air was blown under the bases to obtain the TSG floating at a draft of 9.1 meters. In the morning, during the high tide, the structure was towed out of the graving dock.
The manoeuvre was performed by two 16,000 tugs, the structure being steered by six hydraulic winches operated from unique control console.
The minimum lateral clearance through the dock dike and in the channel previously dredged was 5 m. The manoeuvre was completed in about two hours. After the float-out the platform has been towed at the entrance of the Hunterston channel where other four tugs have been connected to the structure.
The skirt depressurization and the subsequent water ballasting of the bases was performed under tow in the Firth of Clyde at a towing speed of about one knot. The final tow draft of 25.6 m was reached before entering the North Channel.
At that draft the structure has been towed out of the North Channel, then in the open sea. The structure entered the sea of Hebrides on November 5th, and was towed through the little Minch (between the Island of Skye and the Hebrides) in the North Minch, then in the inner Sound of Raasay and finally to Loch Kishorn where she arrived on November 7th.

Figure 16 - TSG Float Up

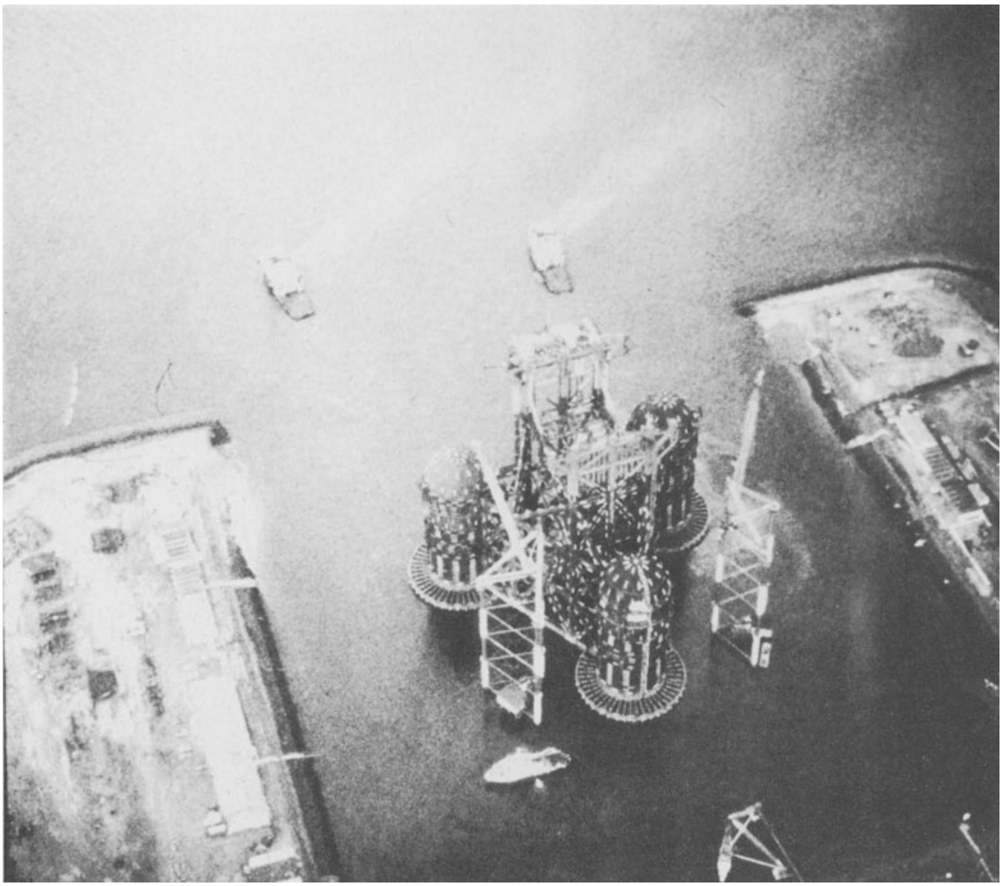


Figure 17 - TSG Floating

Figure 18 - TSG during First Tow

The total tow route length was 332 nautical miles and the mean tow speed 3.5 knots. During the tow a crew of 10 people was accomodated on the temporary deck.

Inshore operations (Figures 19, 20).

Once at the wet dock at Loch Kishorn the structure was connected by means of mooring wire rope to the pre-layed three leg mooring system.
The operation was performed in one day by a supply vessel.
Each chain leg (120-150 mm dia.) of the mooring system is abt. 800 m long, provided with four anchors on one end and a suitable buoy barge on the other end. Wire ropes and chains connect the barges to the structure. On one leg of the mooring system the anchors are replaced by an onshore winch able to tension the whole system.

In this condition the platform bases after water deballasting have been ballasted with about 50,000 t of solid material.
The platform, at the end of this operation, reached a draft of about 43.5 m.
In this condition, after water ballasting down to a 112.5 m draft, the platform was ready for deck mating. The barge carrying the deck was towed to the mating location and positioned within the TSG emerging legs. The barge moored to the structure has been positioned by its mooring system.
Once the deck was in the right position, the platform raised by means of its deballasting system and the mating was performed.
The platform was completely deballasted to its free floating condition at a draft of 52.84 m.
In this configuration deck-TSG hook-up and final testing and commissionign operations were performed before the 2nd tow.

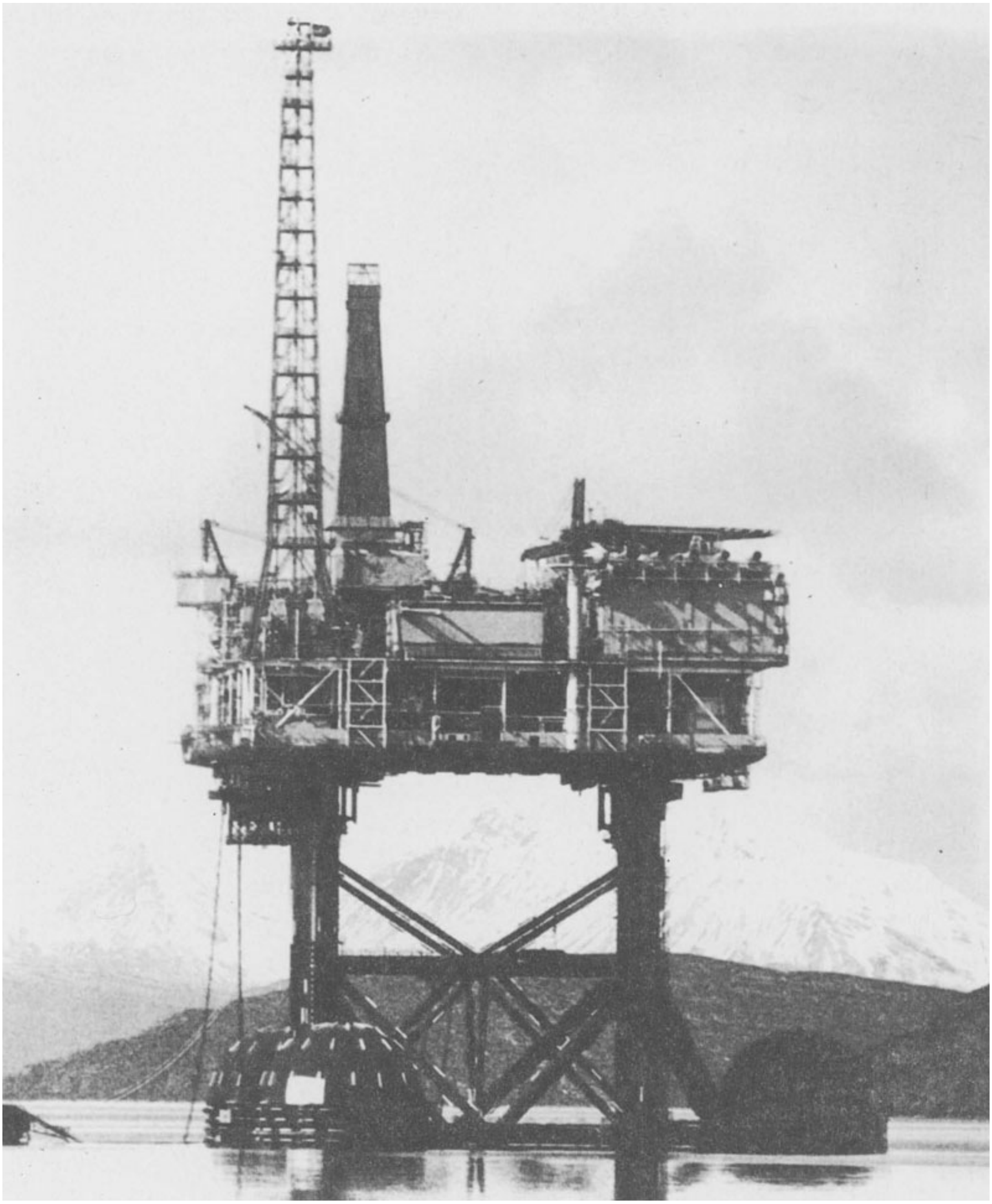


Figure 19 - TSG After Mating with the Hideck

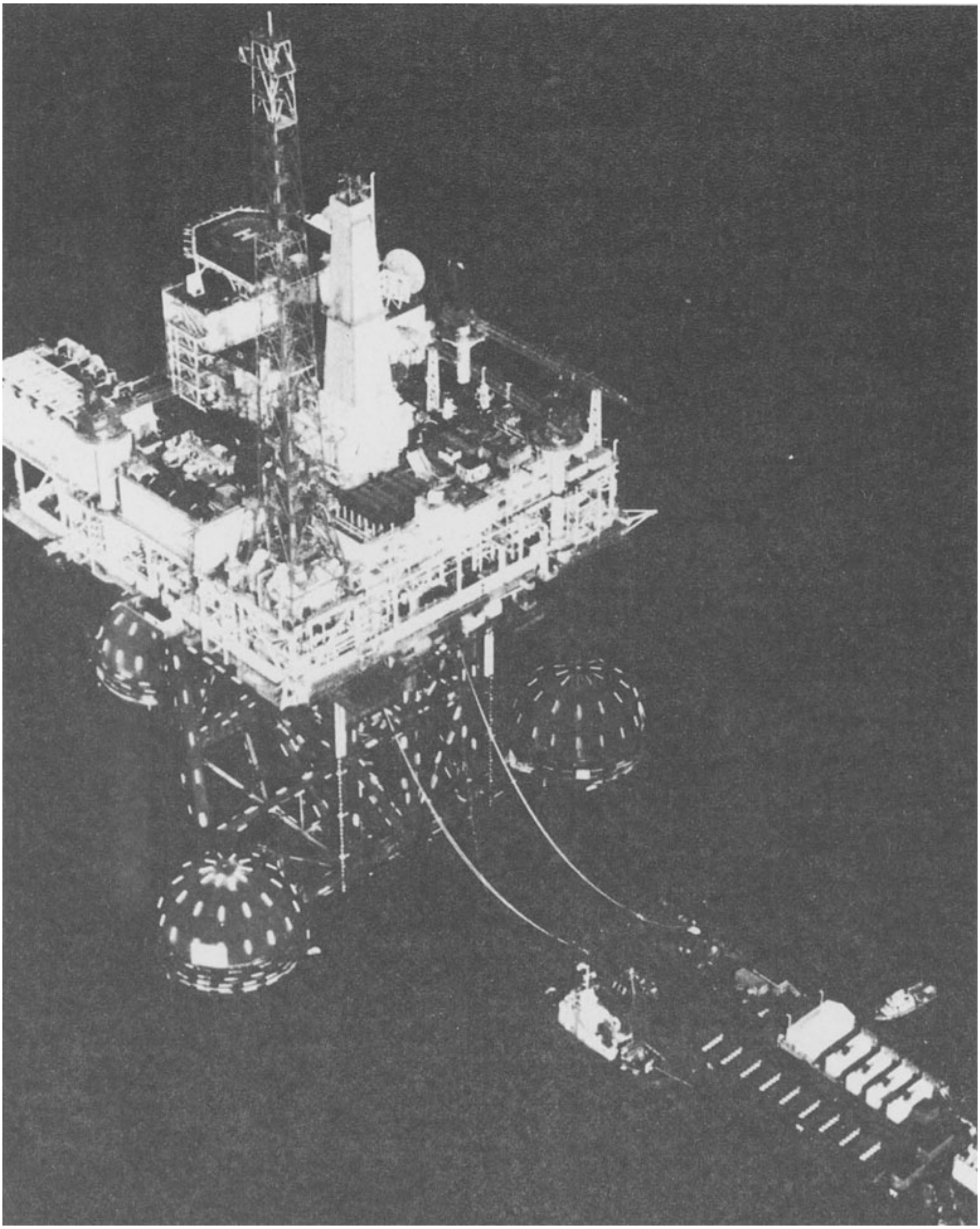


Figure 20 - Platform during Inshore Operations

Towing to Maureen (Figures 21 and 22).

During summer 1983 the platform, with the fully equipped deck, was towed to the Maureen Field by a towing fleet composed of 6 ocean-going tugs with a total ballast pull of 800 tonnes and an escort vessel.

Platform characteristics in towing conditions were:

- displacement	111.100 t
- draft	52.84 m
- metacentric height	11.34 m

A total mileage of 466 miles was covered in about 7 days at a mean towing speed of 2.5 knots.

The tow was affected by severe weather conditions for at least 5 days of towing; storms up to force 8 to 9 were experienced without problems, for the towing-operations had been designed to withstand the max summer storm for the North Sea, i.e. a significant wave H_s = 7.5 m and a wind speed V_w = 48 knots.

Installation (Figures 23, 24).

The operation sequence was:

- Mooring.
  On the arrival at field (once the platform is towed to location), an anchor handling vessel standing by on the location and the escort vessel starting dragging out the mooring lines.
  The operations took about 8 hours to complete the connection of the mooring lines; then the platform, held in position by the mooring system, was immersed to a draft of 83.8 m by sea water ballasting (First Immersion).

Figure 21 - Platform during Second Tow

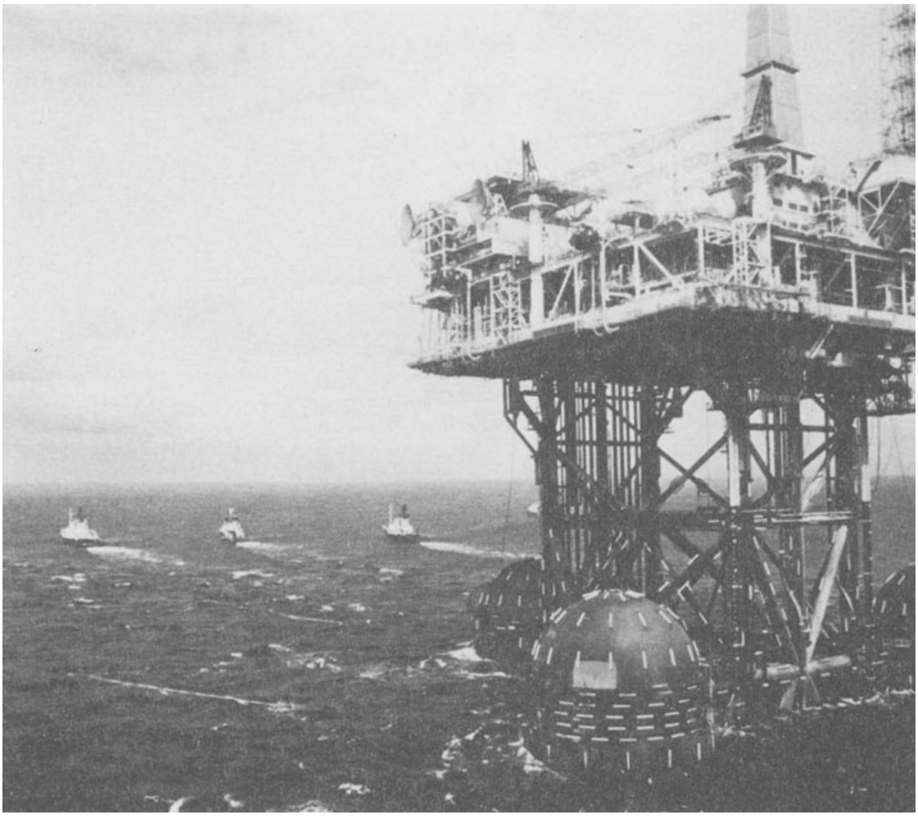


Figure 22 - Platform during Second Tow

Figure 23 - Platform Arrival at Maureen Field

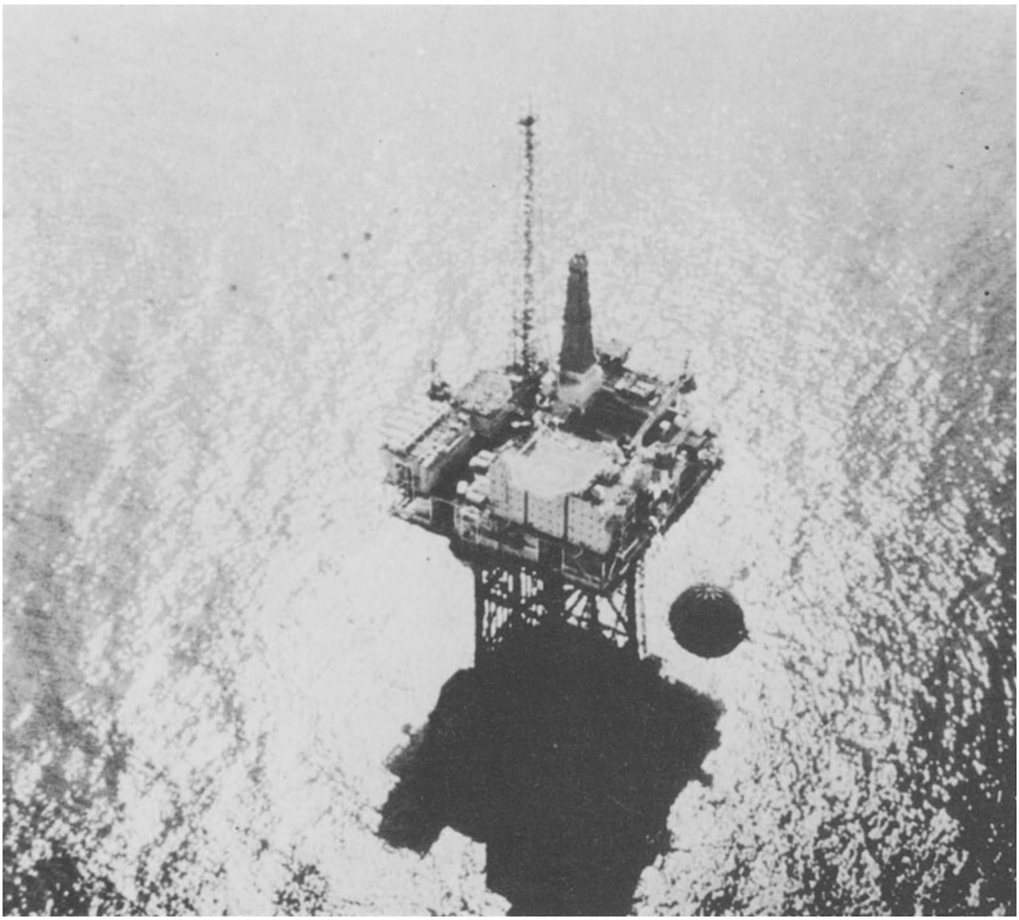


Figure 24 - Platform during Installation

At this draft the ballasting operation stopped in order to permit the TSG positioning in a 1 m range from the vertical of the template.
The operation was successfully performed by promptly adjusting the mooring system so that a final offset of about 4 winches from the position was achieved.

- Close positioning.
  The close positioning operations were performed as follows.

The engagement of the guide piles was performed by lowering the docking pipes by means of the pertinent winches.
Engagement of the docking pipes was very precise and smooth and eventually the TSG was lowered down to sea bottom by water ballasting. A self-stabilizing mechanical distributor fed by a submerged pump eased the immersion of the platform in the upright position. [5, 6]
The skirt touch-down of the platform occurred approximately within 2 inches from the target; a further water ballasting allowed the skirts to penetrate into the seabed.
Final touch-down was achieved after approximately 48 hours from start of the drag-out of the mooring lines.

CONCLUSIONS

The Maureen project has been for Tecnomare an important opportunity to prove the flexibility of the steel gravity platform concept.

The requirements called for by the early production scheme, that is drilling the production wells and fabricating the platform at the same time, using a topside facility fully equipped onshore and providing the required storage capcity, have been positively met with the design of the TSG platform described in this paper.

NOMENCLATURE

r = Cylinder radius
d_b = Brace outside diameter
d_c = Chord outside diameter
F_L = Longitudinal force component
F_T = Transverse force component
h_s = Significant wave height
L.A.T. = Lower astronomical tide
σ = Wave number
P.M. = Pierson Moskovitz
r_c = Chord radius
R.A.O.= Response Amplitude Operator
F = Stress Concentration Factor
t = Time
e_b = Brace Thickness
T = Wave period
e_c = Chord thickness
T_Z = Average zero up-crossing wave period
v_c = Current velocity
v_w = Wind velocity
V.D.U. = Video Display Unit

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THE NINIAN CONCRETE PLATFORM
IN THE NORTH SEA

L.C. Zaleski-Zamenhof

C.G. Doris

The case history of the Ninian Central Platform, as designed and built for Chevron U.K. by Howard Doris Ltd, is described in this paper ; the main characteristics of the platform being :

water depth at North Sea location	136 metres
concrete structure height	157 metres
concrete structure base diameter	140 metres
deck height above North Sea	31 metres
deck dimensions	55 metres x 79 metres
structural concrete weight	356000 tonnes
permanent pipework weight	5100 tonnes
deck and equipment at tow out	23800 tonnes

Furthermore, the new trends in development of the so-called second generation of concrete offshore structures, derived from the former one as represented by Ninian Central, are outlined.

PREFACE

The rapid development of offshore structures, to which, for some years now, we have been contributing, has brought about the emergence of structures of a new kind, which are very different from those found onshore, on account of their :

- siting,
- dimensions,
- the nature and size of the forces which they have to withstand,
- their method of construction and installation,
- the nature of their foundations.

Although the designation "offshore structures" can be applied to a structure fairly close to the shore, the economic necessity for the exploitation of marine ressources is causing offshore platforms to be placed further and further from the coast. The depth of water and the nature and size of the equipment to be installed determine the size and dimensions of the marine structure. It has already become something of a tradition to compare the size of these structures with the size of various famous buildings around the world (fig. 1).

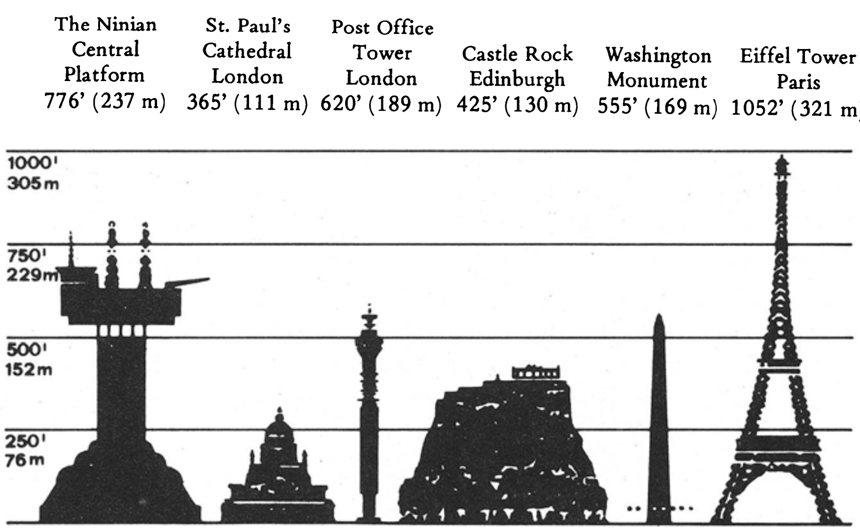

Fig. 1 : Scale of a marine structure

Over the world of off-shore platforms we may distinguish, independently of the services they are conceived for, a lot of types of structures, varying versus the nature (or simply absence or presence) of their foundation.

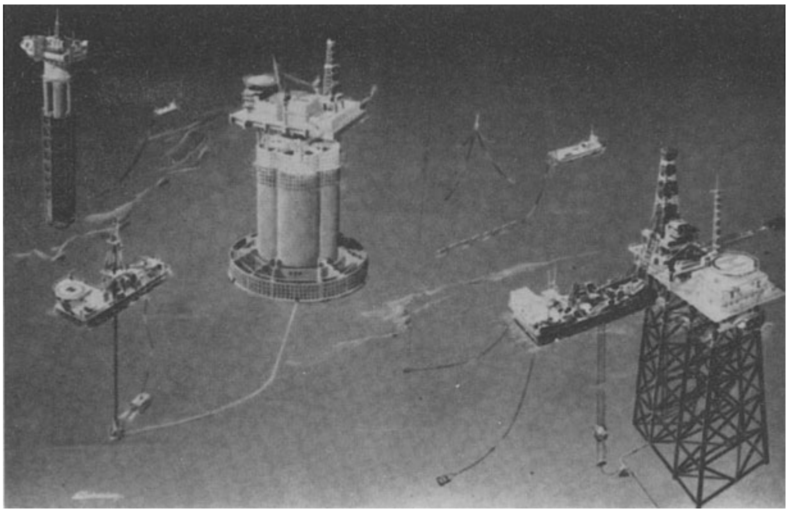

Fig. 2 : "Petroland" - a world of off-shore platforms

A following primary classification could be subsequently suggested :

1	Floating platforms
2	Fixed platforms

becoming, when some more detailed criteria applied :

1.1.	dynamically positioned
1.2.	conventionally moored
1.3.	tension legs platforms
2.1.	jack-ups (fixed removable)
2.2.	gravity structures
2.3.	piled structures

On such a "horizontal" classification let us try to superimpose a vertical one, when taking into account the nature of material, the off-shore structures are built of :

A	B	C
Steel Structures	Hybrid (composite) Structures	Concrete Structures

this table being necessarily not an exhaustive one.

The concrete gravity platform, representing the case history as dealt by the present paper with, is subsequently classified as a C.2.2. one.

The basic idea behind the concept of a gravity platform was to have enough weight at one's disposal, once the structure installed on the sea bed, to generate the self-righting moment as sufficient to compensate for the overturning one as generated by the waves.

The other characteristic of this sort of design is that it should be sufficiently buoyant to be able to be towed at an acceptable draught, preferably with the deck already in position, and accomodating the maximum amount of industrial equipment needed for its future service. If these conditions are fulfilled, the gravity platform, once in place, is a finished product, and is stable and operational right from the beginning.

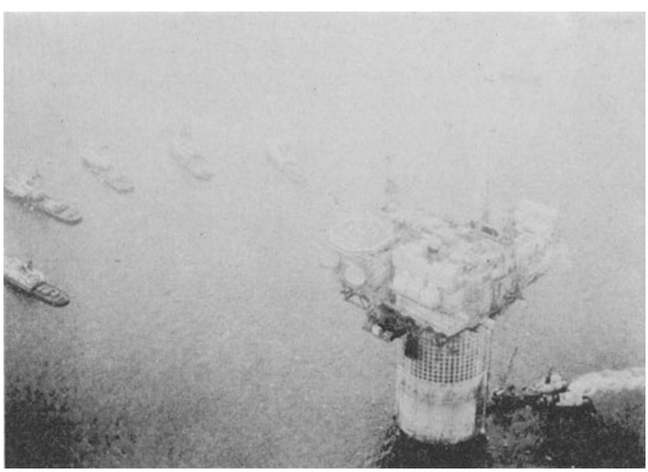

Fig. 3 : Ninian Platform under tow, supporting 23 400 tonnes payload

Offshore operations, which are always dependant upon the prevailing meteorologic conditions, are reduced to a strict minimum, particularly of course the driving of piles and the deck installation at sea, which are completely eliminated.

Further advantages of these structures deserve a mention :

- the buoyancy chambers required for the towing phase can later serve as oil storage tanks, if necessary, once the platform in service,
- the deck is generally supported by one or more towers which ensure buoyancy when under tow and which house the conductor tubes and risers, protecting them against collision or other accidents.

As for the materials used in the construction of such platforms, there is still an element of competition or, let better say : a complementarity, between steel, the traditional material, and pre-stressed concrete, the advantages of this latter one being summarized as follows :

- concrete buoyancy chambers are suited to resisting the high forces of compression resulting from external hydrostatic pressure while the platform is being towed and sunk into position, the risk of buckling being more easily avoided ;
- concrete towers present a particularly effective protection against collision,
- the concrete structure is practically unsusceptible to fatigue,
- the susceptibility of the material to corrosion is greatly reduced, therefore maintenance work during its operational life is minimal,
- concrete may be placed, without any harm, in direct contact with oil under storage,
- the concrete shell, on account of its thickness, is an effective means of thermal insulation for the oil under storage.

There are never advantages without disadvantages, however slight the latter may be ; the following are sometimes mentionned with respect to gravity platforms :

- gravity platforms are larger obstacles for the waves than the lattice work jackets,
- the gravity type foundation could present some problems when set on sea bed not previously prepared,
- there would be a risk of scour.

To the first of these objections, there can be pointed out that however great the horizontal force and overturning moment induced by the waves, experience shows that these can always be compensated by weight and the necessary self-righting moment. Furthermore, the following measures are applied to minimise the force of the waves :

- where the deck presents a small surface area, its support is reduced to a single tower with as small a diameter as possible, this depending of course on the equipment that has to be housed inside,

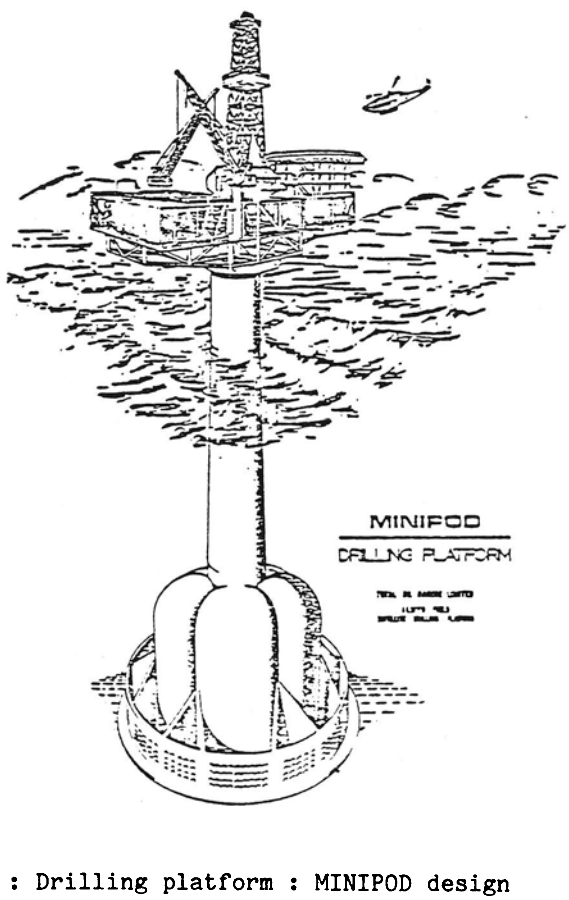

Fig. 4 : Drilling platform : MINIPOD design

- where the deck area is large, it is supported by several towers of small diameter, or by a central tower and columns mounted on a surrounding wall, or, why not, by an auxiliary "jacket", thus forming a kind of hybrid structure.

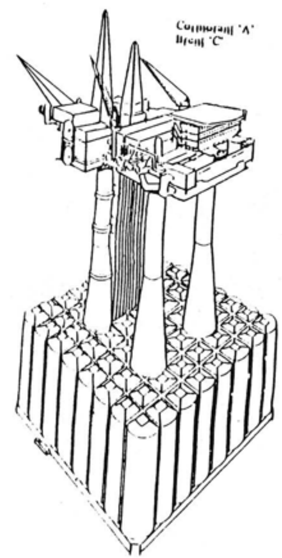

Fig. 5 Cormorant A and Brent C platform ; Sea-Tank Co design for SHELL Expro

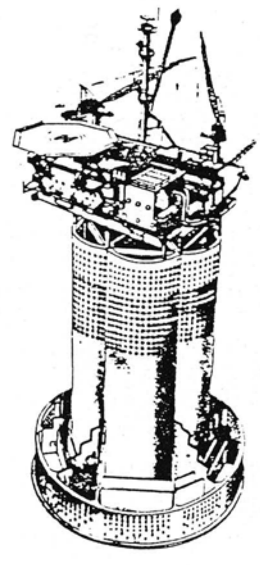

Fig. 6 MCP01 platform on Frigg field DORIS design for Total Oil Marine

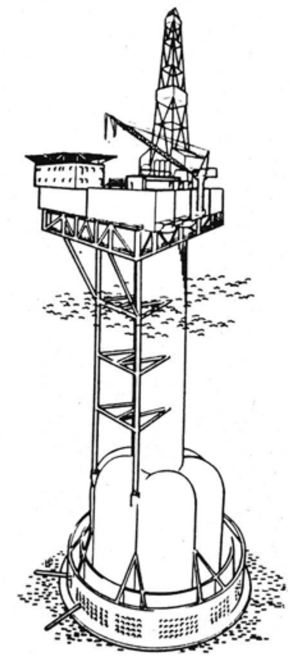

Fig. 7 The Doris hybrid structure design

In the case of the four large platforms, as designed by C.G. DORIS, and belonging to the so-called "first generation" of concrete gravity platforms, including also the NINIAN one, the option with a perforated surrounding wall has been selected, the perforations (a Jarlan's patent) producing there a substantial reduction of wave forces hitting the structure.

The perforated wall also acts as additional protection of the structure and allows a sizable number of conductor pipes to be housed inside (42 in the case on Ninian). In order to compensate for the differential displacement of the tower and the wall, the supports on the latter contain neoprene pads.

But, for poor soil conditions, for which the minimization of environmental forces on the structure becomes of a capital importance, the hybrid concept may appear especially appropriate. In the example of the so called TDHT, i.e. the Tecnomare-Doris Hybrid Tripod, both materials : steel and concrete seem to be located at their respectively best places :

- the steel lattice structure, supporting the deck, presents a reduced obstacle to the wave action :

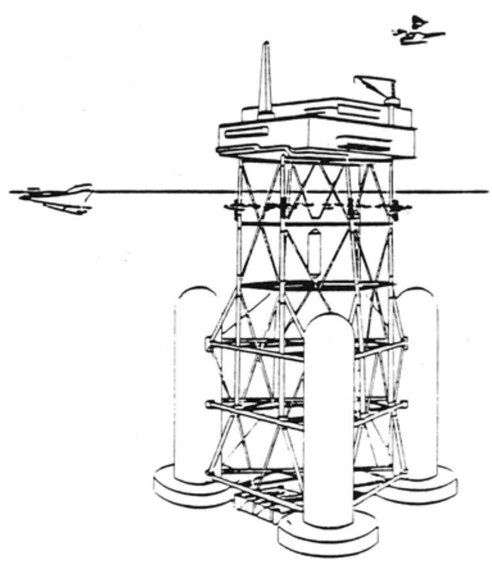

Fig. 8 : T D H T design

- the concrete buoyancy chambers present appropriate structure to resist high hydrostatic pressure,

- the concrete stiff foundation contributes to reduce forces in the steel lattice structure.

As far the problem of the foundation in general is concerned, this can effectively be dealt with, by sea bed surveys carried out beforehand, in order to find an acceptable site, and subsequent destruction of large obstacles (if there are any).

The base of the platform can be fitted with skirts, and hollows, if any, can be filled-in by grouting. Moreover, the presence of the skirts eliminates the risk of scour. Otherwise a perforated (Jarlan type) wall also serves as scour protection ; a special carpet (the filter X type for example) and rip-rap offer further alternatives.

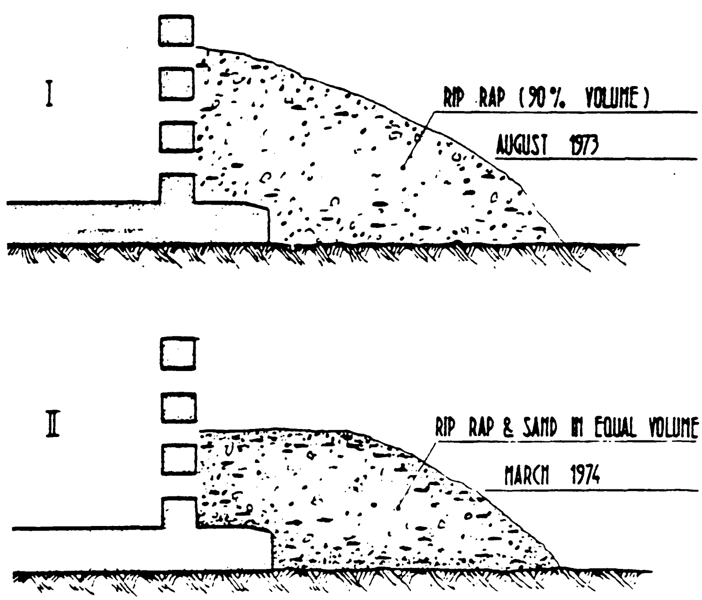

Fig. 9 : Rip-rap around Ekofisk platform as observed after its installation

As the subject of the present case history is concerned, the Ninian Central Platform is operated since 1978 by Chevron Petroleum (UK) Ltd. and their partners, for drilling and production of oil from the NINIAN Field in the Bristish sector of the North Sea.

The field is situated 180km to the east of the Shetland Isles in 136 metres of water.

The platform has been designed for the drilling and operation of 42 wells and is supporting a total operational deck load of 35 000 tonnes, of which 23 400 tones, including the deck itself, have been installed before tow-out to the field.

The displacement of the structure at towing has been 60 000 tones.

This platform, built by Howard Doris at Loch Kishorn, a construction yard on the west coast of Scotland, was at the time being the largest concrete offshore structure so far constructed. The platform measures 170 m high with a base diameter of 140 m and a total concrete volume of 140 00 m.

The prestressed concrete structure consists of a base slab, which is surrounded by a perforated anti-scour wall, inside of which are built a series of concentric lobated walls. The pyramid shape formed by three levels of dome roofs, and perforated Jarlan breakwater wall above the lobed structure, enhance the stability of the platform by lowering its centre of gravity. Steel skirts beneath the slab increase the resitance to sliding by penetrating into the foundation soil.

The design process of the Ninian Platform, as well as the construction methods, with a special emphasis on some prestressing details, and the heavy precast elements technique, will be presented in this paper. Finally, the design will be compared with the concept of platforms of the so-called second generation, as developed at present, when supported by the experience from the previous one.

DESIGN CONSIDERATIONS

The main function of the platform is to support a large deck area containing heavy equipment for oil drilling and production in a water depth of 136 metres.

The design and dimensioning of the platform are governed by the environmental loads, deck load and modules required at tow-out, stability and channel draught available during construction and towing, as well as by the seabed soil characteristics on site.

The main elements of the structure are shown in figure 10.

The deck structure of 79 x 55 m and modules are supported by two concentric cylindrical walls. The outer one, serving also as a breakwater wall, takes the majority of the vertical loading from the deck. It provides the supplementary support points over a large area, thus reducing the deck cantilever. The wall is conceived according Jarlan's perforated breakwater patent.

Between the breakwater wall (wall 2) and the central shaft (wall 1) the 42 drill slots are arranged in 4 rows, the wells having been completed by two drilling rigs on the deck. The outer wall protects the conductors against collision with floating vessels. The pipework connections to the sea bed, the risers and J-tubes, are also routed through this protected zone, as are all the service caissons, which descend from the deck for water retrieval and disposal.

The central shaft provides an open area of 14 metres diameter from the base of the structure on the sea bed up to the deck. Inside the shaft there is ample space for the ballasting pipework and other services to run vertically the height of the structure. In the original design this shaft housed the oil storage pumping and pipework for the one million barrel storage, as initially foreseen in the floatation chambers between walls 2 and 5. It also provided communication with tunnels on the base through which the risers could have led as in previous DORIS designed platforms. During construction the operator has decided that storage was no more required, as the platform had to be connected to the Ninian pipeline. The risers were to be subsequently located outside the floater. The central shaft was redesigned to provide a large storage capacity for diesel and fresh water (25 000 bbls of diesel and 40 000 bbls fresh water).

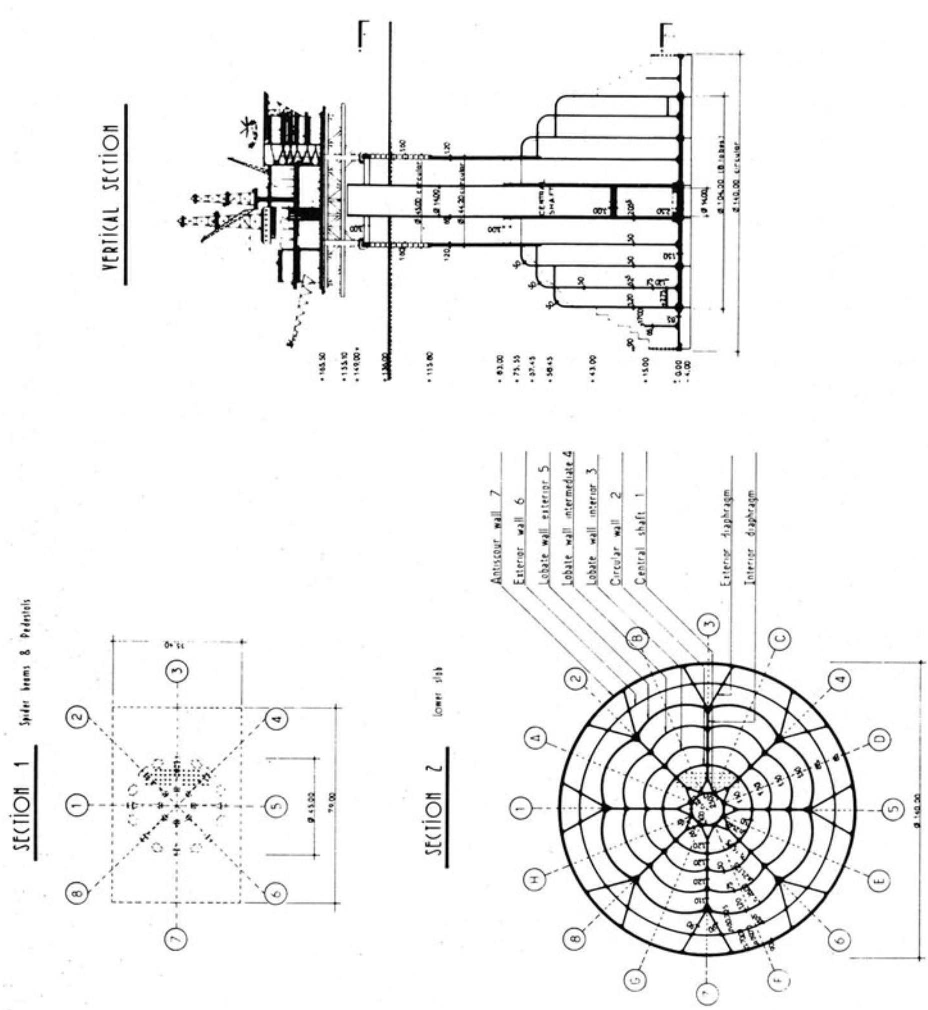

Fig. 10 : Ninian platform elevation and sections

The floatation unit provides, between the base slab and level + 75 m, the buoyancy necessary to float the structure with its deck and equipment from the construction site to the field. In order to navigate the route from the Inner Sound of Raasay (where the deck and equipment were installed) to open waters, the structure had to cross the East Shiant bank where the depth of water reaches a minimum of 88 metres L.A.T. The possible deep draft and stability of the structure have allowed to carry 23 400 tonnes of deck and equipment which could be subsequently installed and hooked up inshore.

The outer boundary of the floatation unit is formed by the lobate cylindrical wall 5 (diameter approx 104 m) which is stiffened by eight radial diaphragms. Internally the floatation unit is subdivided by other lobate walls of a similar design. These serve to segment the floatation unit allowing the ballasting system to perform in such a way that the differential pressures accross individual walls have been kept to an acceptable design level. This segmentation also allowed the platform to be water ballasted differentially to compensate for eccentric loadings on the deck during module placing. At the bottom of the floatation unit these walls serve as beams to reduce the span of the base slab sections. At the top they serve a similar function to reduce the span of the dome elements which form the upper closure of the floatation body.

The base slab transmits the loads from the structure to the sea bed, these loads being due to the structure self weight and to the imposed environmental loads. The base slab is 140 metres diameter, thus extending the 20 metres outside the floatation unit. The loads on this outer annular slab are picked up by two circular walls 6 and 7 and the external diaphragms. The peripheric wall is perforated to serve also as an antiscour protection. Since the clay soils at Ninian increase in strength with depth, the base of the structure is equipped with 4 metre long steel skirts, forced into the sea bed by the self weight of the structure during its installation. They are spaced on the underside of the raft sufficiently close together, such that potential shear planes are forced down into the lower, stronger strata. After installation any remaining space between the underside of the base slab and the soil is filled by grouting through pipes cast into the base slab.

The structure has been designed in accordance with the recommendations of the Fédération Internationale de la Précontrainte (F.I.P.) and the appropriate British Standards (C.P. 110 - 1972).

The environmental conditions, as governing the design, have been assumed as here below :

Water depth		136	m
100 year wave	height	31.2	m
	period	18	sec

Operating wave	height	17.0	m
	period	12	sec
Extreme wind	speed	42.6	m/sec
	gust time	1	minute
	speed	52	m/sec
	gust time	3	sec
Current	surface	1.0	m/sec
	sea bed	0.7	m/sec

thus generating, under so called design (100 year) conditions :

- a horizontal force of 102 000 tones
- an overturning moment of 4.18×10^6 tonemeters.

These forces have been calculated by a computer program, using Stokes 5th order wave theory for the diffraction of a wave. In addition, the methematical model was confirmed by tests on a 1/125 scale model carried out at the Bassin d'Essais de Carènes, Paris.

The concrete sections have been designed as reinforced members for bending, with prestressing used to counteract tensile membrane stresses and shear forces. The analysis of the structure was carried out using a 3-dimensional finite element program with 1/16 of the structure represented for uniform loading conditions (immersion on site) and 1/2 structure modelised for wave loading conditions. Careful study was made of wall intersection details and changes in member thickness to eliminate differential strains between elements.

The full hydrostatic pressure on the main flotation wall during immersion in 136 metres of water is reduced by the height of ballast water inside the wall compartments, which produces itself hydrostatic forces on the internal walls.

The stability of these walls is dependent on their geometry, constructional tolerances and the modulus of elasticity. Calculation for the latter is critical when considering long-term loading conditions such as submergence for installation of the deck and modules.

Throughout the construction period there was a constant check and analysis of the structural conditions existing at any one moment. This necessitated a complete understanding and close liaison between the design and construction teams which sometimes enabled different construction phases to proceed depending on manpower availability, craneage, materials supply and weather conditions. Thereby considerable programme time was gained with sufficient flexibility in the design arrangement.

Theoretical stability calculations were continuously verified with as-built information from the site in relation to wall thickness, geometry, ballast levels and equipment loadings.

The critical ballasting of the structure to a draught of 136 metres and the installation of the 6 300 tonne steel deck designed by Howard Doris was an operation which required close control from all design, construction and marine operation teams. Later, during installation of the modules, which weighed up to 1 000 tonnes, the structure remained at a draught of 136 metres and was able to accept o tilt of 2° before differential ballasting was applied to right the structure.

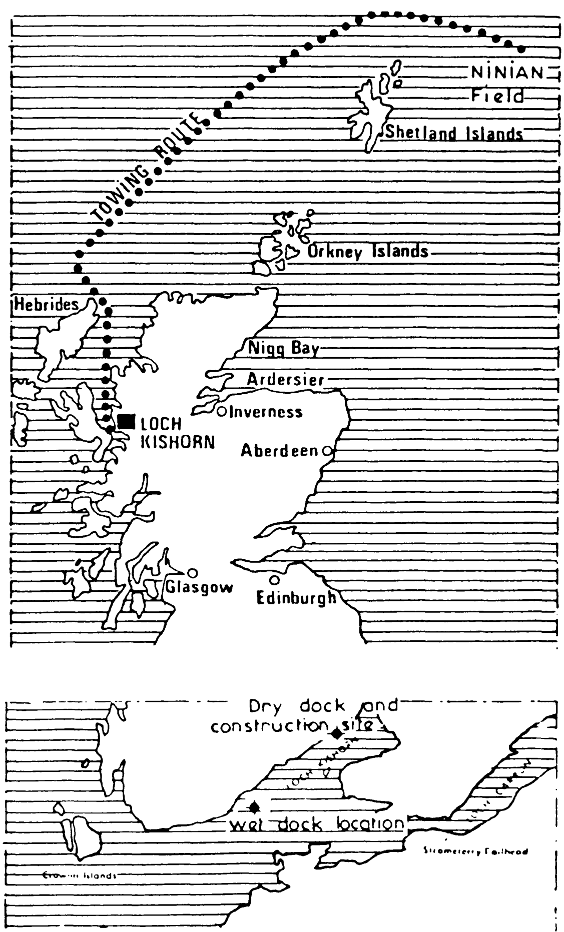

Fig. 11 : Site location

CONSTRUCTION SITE

The Howard Doris construction yard has been established on the west coast of Scotland at Loch Kishorn (figure 11).

This site was chosen for its sheltered location and access to deep water facilities for construction of floating concrete structures.

A dry dock 180 m x 250 m was excavated out of rock down to a level of -11.30 m. A total of 450 000 cubic metres of spoil was transported from the excavation to form storage areas and facilities surrounding the dock.

Initially, the dock was closed by a sheet pile wall driven through a rock fill bund. This was later removed after two prestressed concrete caisson dock gates, each 82 metres long, had been constructed. In this way it was possible to gain time on the construction programme by eliminating the bund removal operation from the critical path.

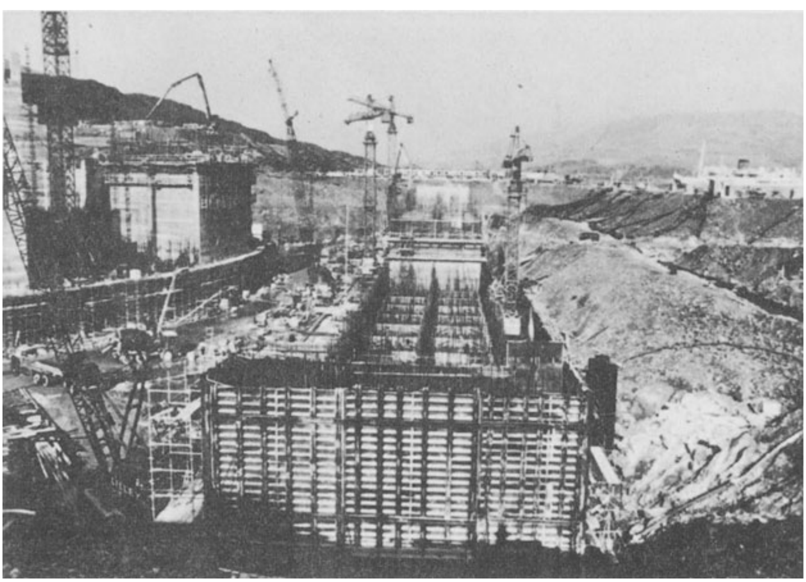

Fig. 12 : Platform base and dock gates under construction

CONCRETE MATERIALS

Considerable study went into the selection of a suitable mix design which would consistently provide 50 N/mm^2 concrete, the total quantity required for the platform and dock gates exceeding 140 000 m^3.

Since the specification limited the C_3A content in the mix, pozzolan in the form of high quality power station fly ash was added at the batching plant in the ratio of one to four parts of cement. A mix design was finally adopted containing 425 kg of ordinary Portland cement and 105 kg of pozzolan per m^3.

From tests it has been found that pozzolan contributed very little to the 28-day strengths but improved the long-term strength thereafter. Pozzolan was useful in reducing the heat of hydration in the mix and increasing pumpability.

The concrete strengths achieved have been consistently above the minimum required. A total of 2 100 sets of cubes have been crushed giving a typical average 28-day strength of 60 N/mm^2 with a standard deviation of 4 N/mm^2 for a 20 mm maximum aggregate size mix.

Some cubes have been crushed at 12 months and have confirmed that the use of an age factor of 20 % would be acceptable.

CONSTRUCTION PHASE IN THE DRY DOCK

Once the excavation of the dry dock was completed, a concrete floor slab and ground beams were cast to support the steel skirts which would eventually carry the full weight of the structure in the dry dock, 157 000 tonnes.

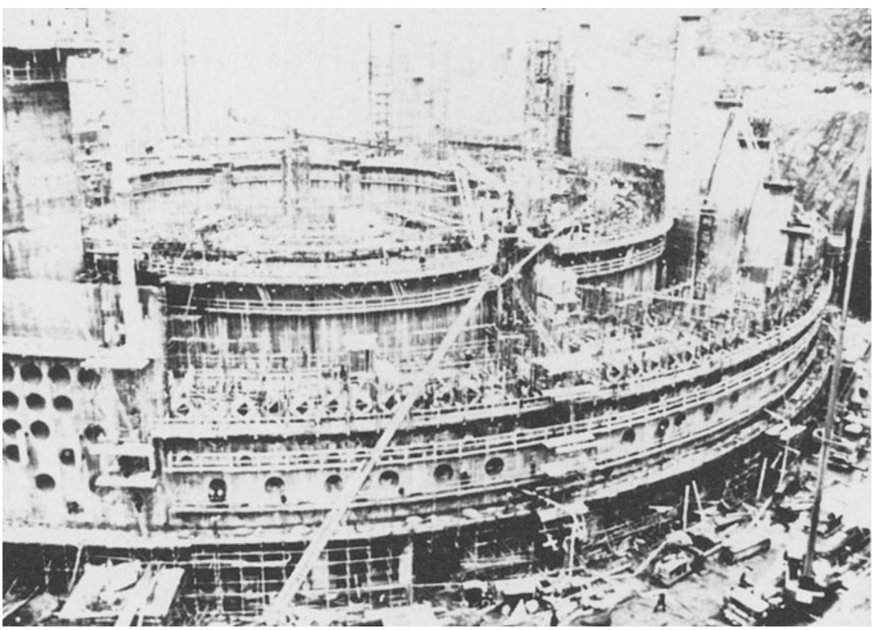

Fig. 13 : Construction in the dry dock

The disposition of 4-metre-high steel skirts followed the layout of the radial internal and external diaphragm walls with circumferential connections below the exterior anti-scour wall and lobed connections below walls 3 and 5. The areas enclosed by the skirts were compartmented to ensure stability during exit of the platform from the dry dock on a 3-metre air cushion, and to facilitate final base grouting operations when the platforms was installed in the North Sea.

The skirts forming a V-shape beneath the external diaphragm walls extend 20 cm below the circumferential skirts on the 140 metre diameter, and have been stiffened to take extra forces applied during initial penetration of the sea bed.

A total of 568 skirts weighing 4 300 tons were manufactured in the north of England and transported by ship direct to the Kishorn site where they were unloaded and immediately placed on prepared concrete foundations in the dry dock.

Individual panels were bolted together with friction grip bolts on site.

Access to the enclosed compartments between skirts was provided by removable door panels and circular holes for diver inspection. Final bedding of the skirts on the dry dock floor was achieved with a high strength epoxy mortar together with a bond breaking film between the steel surface and the mortar. This was an essential requirement to ensure an even lift-off of the platform when ready for tow-out.

To provide air-tight compartments for use of the air cushion system during exit from the dry dock all skirt joints and bolted connections were sealed with an appropriate mastic.

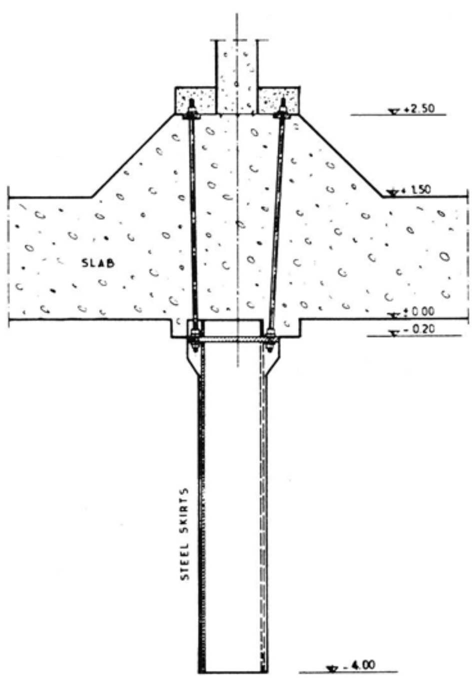

Fig. 14. : Skirt/slab Connection

Skirts were connected to the concrete base slab by a series of shear keys welded to the top plate of the skirt together with 40 mm diameter high tensile Macalloy prestressing bars.

The overall protection of the platform against corrosion favoured the use of sheradised prestressing bars without separate ducts passing through the concrete. Since the bars were to be placed before concreting the slab, it was necessary to select a suitable bond breaker to enable effective tensioning operations later. After several alternatives had been studied the use of Denso tape wrapping was approved.

8 320 Macalloy bars with an average length of 3 metres were therefore wrapped in Denso tape, covered with a polythene sheet and installed to project out of the haunches ot the slipformed walls at varying angles by means of tapered washers on top of the skirts.

After positioning of slab reinforcement and longitudinal prestressing ducts, the polythene cover was removed from the bars and the top anchorage plate installed with a capping device to ensure that the bar and plate were at right angles.

With two teams, each using a Macalloy Mk. 12 jack and pump, approximately 100 bars per day could be tensioned under ideal access and equipment handling conditions.

Corrosion protection of the anchorage was completed with a concrete capping beam for the top nuts and individual plastic caps filled with a bituminous mastic for the lower nuts.

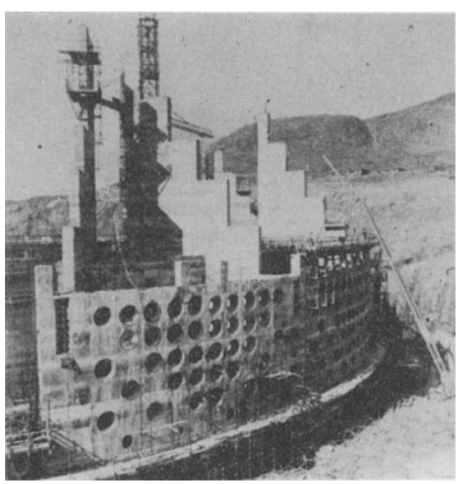

Fig. 15 : Antiscour Wall and External Diaphragms

The 140 metre diameter base slab with a thickness of 85 cm at the edge and 2.50 metres in the centre was constructed on timber shuttering supported on scaffolding 4 metres above dock floor.

The slab was divided into nine sections, one central and eight radial portions, each with a volume of concrete between 2 500 and 4 500 m^3. The radial sections were concreted sequentially without shrinkage gaps, each construction joint being formed with timber shuttering behind which was a rubber waterstop.

The final concreting phase for the slab consisted of one-eighth segment together with the central core ; altogether a total of 7 000 m^3. This operation required six days of continuous concreting. Distribution and placing of the retarded mix was by pipeline from the batching plant to two tower-mounted articulated booms which were moved and supplemented by skips from the tower cranes to keep the 50 metre long and 2.5 m deep face of fresh concrete alive.

As soon as one outer segment of the base slab had been concreted, the installation of the exterior wall slipform was commenced. This method allowed the re-use of slipforms for the exterior circular walls and diaphragms forming a V-shape on plan.

Recoverable steel void formers were used to form the tapered holes in wall 7, the outer circular wall of height 15 metres which was used to float out the structure from the dry dock and which serves as anti-scour protection for the platform, once installed in the North Sea.

The largest slipform was for the main floatation wall number 5. This consisted of eight lobed sections with an overall diameter of 104 metres. Guidance of this slipform had to be restricted to the node points where the lobed walls intersected with the external diaphragms. The slipform itself was supported from steel trusses which spanned the 35 metres between nodes.

All the major internal slipforms were concreted on a circular pattern using a two-arm central rotation distribution tower at each end of which was a hydraulically operated boom for placing the pumped concrete.

For wall 5, which was 1.2 metres thick, 520 m^3 of concrete were required for each metre lift.

Slipforming in the dry dock was up to a general level of 15 to 18 metres except for the external stepped diaphragms which were taken to their full height of 40 metres (figure 15).

A total of 3 160 tons of prestressing steel was required for the construction of the platform, of which approximately 40 % was installed in the dry dock construction phase. The PSC 12K15 system was adopted throughout, which gave a characteristic force of 290 tonnes per cable.

Horizontal cables were threaded into spirally wound 76 mm diameter ducts, delivered in 6-metre lengths and coupled with a standard 300 mm long exterior joint. Vertical cables were threaded into rigid tubes of the same diameter which were coupled with a 150 mm long spigot and socket joint.

All intermediate joints were sealed with tape and, in addition, the vertical duct joints were spot welded to prevent lifting during slipforming jacking operations.

The majority of vertical cables were of a U-form, allowing both anchorages to be situated at the top of the walls. The loop of the U was formed from a pre-bent rigid tube of internal diameter 85 mm. The minimum radius was 1.2 metres.

Prestressing operations were carefully studied in relation to the overall construction procedure and in particular the sequence of stressing was chosen to enable as much flexibility as possible in consideration of the numerous slipforming stages. Due to the complexity of wall layouts and access difficulties it was decided at an early stage to supply the site with a large number of jacks and pumps to avoid the constant use of crane time which even small movements of equipment required. A total of 22 K350 jacks and electric pumps together with three powered cable dispensers were available to deal with peak operational requirements.

Particular care was taken over the supervision and execution of all grouting operations to ensure that ducts were perfectly filled and cables completely protected against corrosion.

In the dry dock it was necessary to pump the grout over 300 metres before injection into some ducts which themselves were 150 metres long. A full series of tests on various mix designs was carried out to determine the optimum water/cement ratio in relation to pumping lengths, air temperature and additives. In general the ratio varied between 0.34 and 0.36 with the addition of a proprietary retarder and expansion agent based on aluminium powder.

Grouting of vertical strand cables up to 20 metres high was carried out in the dry dock construction phase using a standard method of injection confirmed by full-scale comparison tests on two U tubes fitted with a cable and anchorages. The test anchorages were cut open with a diamond saw after the grout had hardened to reveal complete filling behind the anchor plate.

WET DOCK CONSTRUCTION PHASE

Ten months after commencement of the base slab concreting operation the platform was ready for towing out of the dry dock on a 3-metre air cushion.

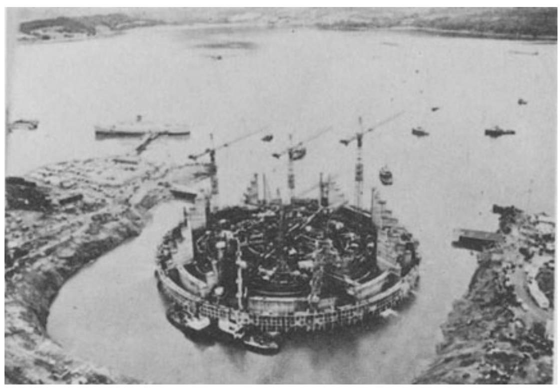

Fig. 16 : Tow-out from Dry Dock

After float out and towing 5 km to the wet dock site, the 140 metre diameter structure was anchored in 120 metres of water by three chains : two using DORIS mud anchors and the third on land with a tension adjustment.

The construction procedure afloat was designed to give maximum flexibility to the overall programme.

However, it was first of all necessary to define a level of construction such that :

- the transfer of water planes from wall 7 to wall 5 and the loss of air cushion would be structurally acceptable,

- the final order of construction would not affect the resistance of the part already built.

The optimum level was chosen at + 30 metres, from which the construction proceeded in three major phases :

Construction up to level + 83

The division of the main slipforms into three principal types was maintained with a flexibility above the optimum level of + 30 which enabled a complete interchangeability of the order of construction.

Construction above level 83

The upper level of the main flotation chamber was at 75.55, above which the structure simplified into two concentric circular walls.

The suppression of radial diaphragms and the flexibility in the order of slipforming led to the consideration of two independent slipforms with a light connection between them.

The optimisation of the construction programme and the presence of bi-conical holes in the breakwater wall above level + 115 promoted a study into the possibility of precasting this perforated wall in elements which represented one-quarter of a circle in plan. Also it was decided to limit the weight of the elements to 300 T, which would be within the load range of the available shear-leg crane. This crane could then be used for the installation of the lighter dome roof elements. Maximum flexibility was thereby obtained by using a crane barge which made precast element lifting independent of slipform operations and consequently not on the critical path.

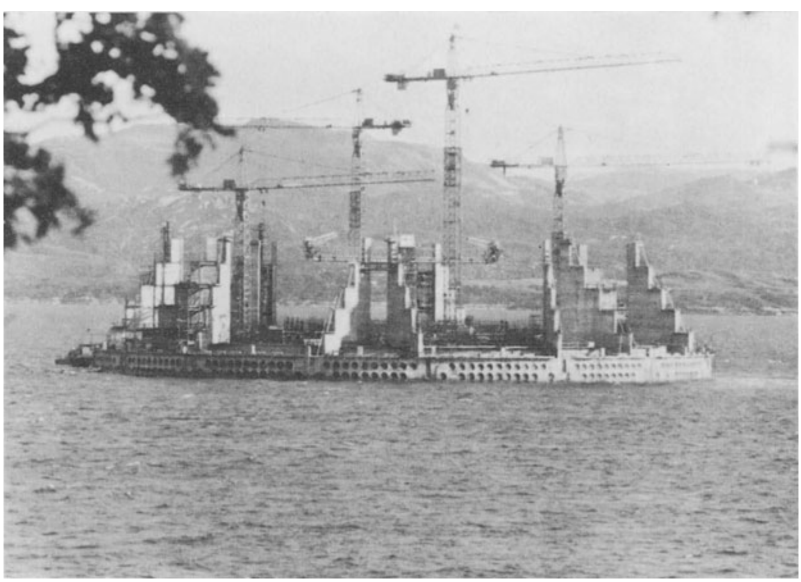

Fig. 17. : Construction in the Wet Dock

Construction above level + 115

Since the perforated breakwater wall was fabricated with heavy precast elements, the central shaft was slipformed alone to level + 155.

This relatively small concrete section allowed an average climbing rate for the slipform of 3 metres per day. At level + 147.00 there are 8 radial spider beams between walls 1 and 2. These served to stiffen the top of the structure.

During the wet dock construction phase, vertical U-cables up to 70 metres high were grouted from the top only, following a full-scale test on a mock-up constructed on scaffolding on top of the dry dock rock face. Strands were left protruding through the temporary anchorage sealings and retarded grout was funnel fed into each anchorage for a period up to 6 hours immediately after the primary injection phase. All bleed water was expelled and replaced by the retarded grout.

Whilst it is considered that satisfactory grouting, combined with high strength dense concrete, is providing adequate protection to the prestressing strands, an additional safeguard was incorporated in this platform by connecting all prestressing anchorages to the general cathodic protection system.

HEAVY PRECAST ELEMENTS TECHNIQUES

a) Domes

Between levels + 50.30 and + 75.55 the structure was closed by a series of toroïdal domes. In addition there was a flat slab system which both distributed the horizontal thrust and acted as an aditional line of defence in the event of a dome being accidentally damaged during construction.

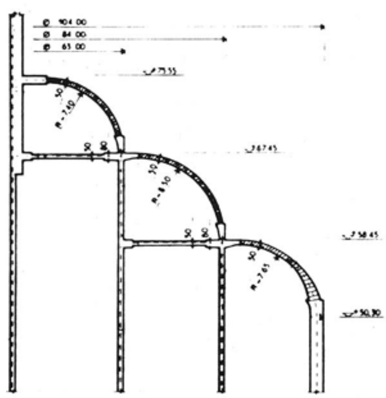

Fig. 18. : Dome roof layout

All of the domes and slabs were prefabricated by Anglian Building Products in their works at Lenwade, near Norwich. Consequently, it was necessary to limit the size of the elements so that they could be transported by road to Great Yarmouth, from where they were shipped to Kishorn. The road and sea transport of these elements was a major logistic problem, requiring careful planning and liaison with the shipping agent, and the Norfolk police. In addition to the logistics of the operation, the forces applied to the domes during shipping had to be carefully studied. The maximum weight of an element was 60 T. A total of 262 elements were fabricated, which included 12 spare elements (one of each type) in case an element was damaged during transport or handling.

Each precast unit had four steel legs on which it rested, spanning between two lobate walls. Cast into the top of the walls were a series of supports which could be adjusted vertically to distribute the weight of the unit uniformly between the four legs. To ensure that the dome and slab units would all fit in place without interfering with prestressing ducts and starter bars, strict tolerances were specified.

The domes were placed in position by derrick barges. The success and speed of the placing operation proved the advantages of a carefully engineered precast solution. The inner shutters for the in-situ concrete joints between precast units were attached before the units were lifted into place. Thus, the work required on the platform to complete the dome roofs was reduced to a minimum.

In order to reduce the size of the in-situ joints between units and to avoid projecting reinforcement it was necessary to use a total of 50 000 couplers.

The overall capacity of the coupler-reinforcing bar assembly was well known, however the deformation of such a unit at stresses up to 200 N/mm^2 was not fully documented. Also it was necessary to take into account the installation of the assembly, both the bar plus coupler in the prefabricated element and the second part of the threaded reinforcing bar installed on site. Because of this lack of direct information for couplers used in an identical situation the designers were concerned that a permanent deformation in the couplers could occur.

Since the allowable crack width was limited, it has been decided that the mean permanent deformation at a stress of 200 N/mm^2 should not exceed 0.03 mm for a series of tests and that the maximum permanent deformation for any one test should not exceed 0.05 mm.

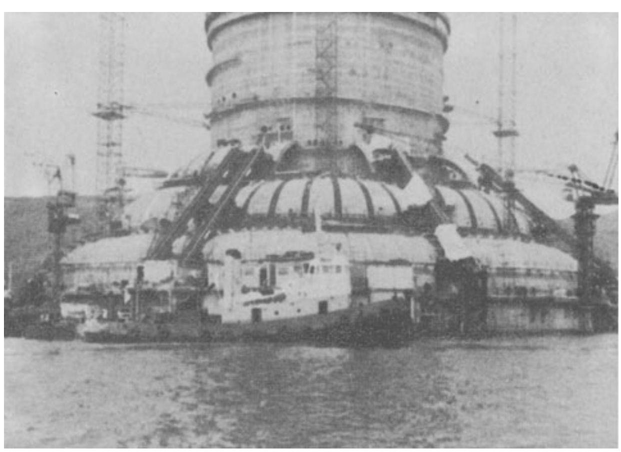

Fig. 19. : Dome roof construction

It was found that the permanent deformation could be reduced in two ways :

- by filling the voids between the threads ;
- by precompressing the threads.

The permanent deformation could be finally limited to about 0.01 mm.

b) Breakwater wall

The general form of the precast breakwater wall elements was represented by a 2.5 m high quadrant of a 45 metre diameter circle with conjugated horizontal joints and cast in situ vertical joints through which passed horizontal reinforcement and prestressing ducts.

The technique of installation and liaison of the elements had to consider the following important criteria :

- size of elements ;
- difficulty of installation ;
- necessity to avoid constraints on the installation due to restrictions with joint preparation ;
- tolerances in geometrical alignment.

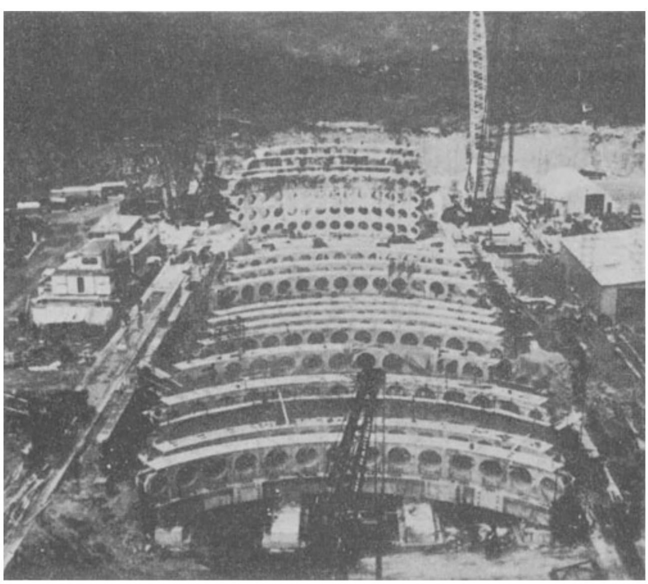

Fig. 20 : Precast elements for the breakwater wall

The finally adopted solution for avoiding these constraints, particularly on the interface joint, was to assemble the elements directly on top of one another with an initial "dry joint".

Consecutive elements were concreted one on top of the other to have a common conjugated surface of two 14 cm wide strips and a limited zone around the vertical prestressing ducts. In other words the upper surface of the lower element was flat and the lower surface of the upper element only came into contact along the 14 cm wide bands and around the prestressing ducts. The height of these conjugations was 10 mm.

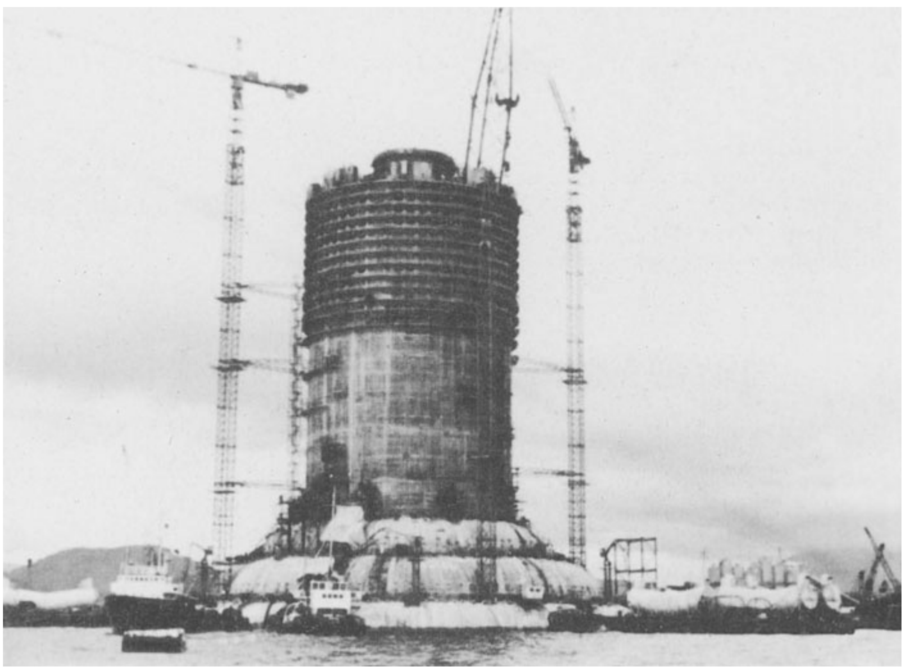

Fig. 21. : Installation of precast elements

After installation of the elements by HEBE 3 shear leg crane (figure 21) it was therefore necessary to fill the 10 mm gap with grout and at the same time to ensure complete sealing between the vertical ducts. The first operation was carried out after sealing the circumference of the joint with an epoxy mortar pumped into a pre-formed groove between the units. Filling of the 10 mm gap was carried out from three postions using a cement grout with an expansion agent.

Sealing of the joint between prestressing ducts was carried out using a mastic gasket pre-set into a V-groove in the lower element and held in place due to deformation under the upper element. A thin sealing of SIKAFLEX was placed on the lower element around the conjugations just before the upper element was lowered in place.

When all the precast elements had been installed, vertical U-shaped cables were threaded, tensioned and grouted. Simultaneously, the vertical in-situ joints were concreted and the horizontal cables completed.

STEEL DECK

The 6 300 tons steel deck was connected to the concrete structure by welding to a steel corbel ring which surrounded the central shaft, being anchored by vertical prestressing cables.

All horizontal forces were thus taken directly on the central shaft whilst the main vertical reactions from the deck and its equipment were carried through supports bearing directly on the 45 metre diameter breakwater wall.

The installation of the 79 metre by 55 metre deck in one piece took place in the Inner Sound of Raasay in November 1977 after the deck had been towed round the north of Scotland from Inverness.

The deck was transferred from its towing barge to two catamaran barges to enable it to float directly over the concrete structure which was submerged to a draught of 136 metres.

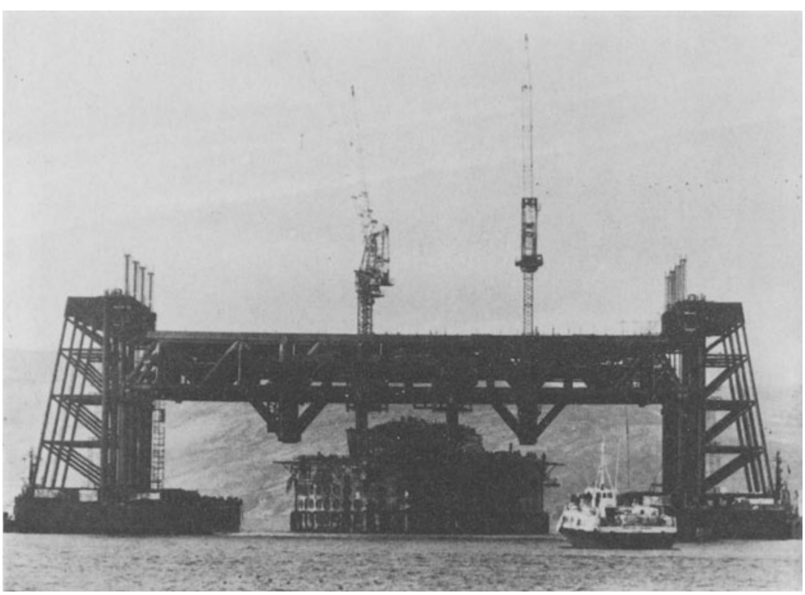

Fig. 22 : Deck mating operation

The operation of transfer of load from the catamaran barges to the concrete structure was carried out by deballasting the structure and using flat jacks below the main support columns.

The eight deck support columns were divided into two groups following their diameter ; four columns 72 inches diameter, and four columns 100 inches diameter. In the temporary load transfer stage it was decided to use three of the largest columns only to take the reactions from the deck.

After full transfer of load to each of the eight columns by means of inflating and deflating the flat jacks, the jacks were injected with grout and the space between each plinth support and the breakwater wall was concreted.

Module installation started immediately after welding the deck to the central corbel ring

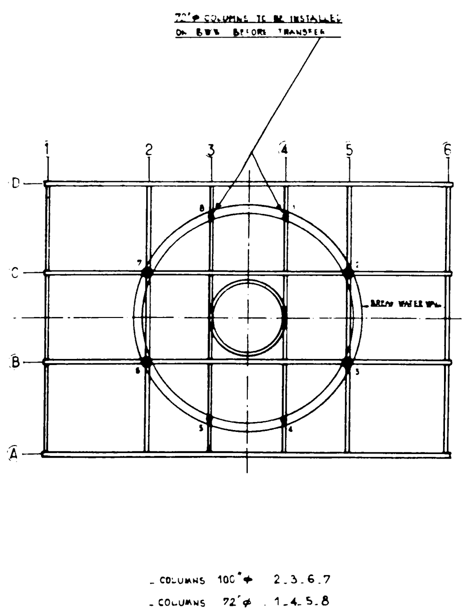

Fig. 23 : Layout of deck columns

Fig. 24 : Installation of modules

CONCLUSIONS

The Ninian Central Platform has been, at the time of its installation, the largest offshore structure so far constructed. It represents a bright specimen of concrete platforms as belonging to the so-called "first generation", which has started with the EKOFISK 1 tank, designed by DORIS for PHILLIPS Petroleum and installed in 1973, and which is covering at the time being fifteen North Sea realisations.

In the design and construction of the platform, as described here above, the following specific points should be insisted on :

- construction with numerous independent slipforms ;
- extensive use of precast elements from 60 t to 300 t ;
- use of large capacity cranes, both onshore and offshore ;
- use of a large number of reinforcement couplers ;
- grouting of long vertical U-loop strand prestressing cables ;
- instalation of a steel deck in one operation ;
- installation of all modules before tow-out.

As for other first generation platforms the construction has required two sites : a first one, onshore, which is a dry dock close to shore, levelled about 10 metres below sea water level, and permanently dry, and a second one, "inshore" for the floating construction phases, which is sheltered and is able to provide the necessary depth.

As nowadays Oil Companies are finding exploitable fields in new oceans of the world, and the conditions prevailing in those areas are not necessarily the same as on the fields as developed at present, the construction methods and appropriate structural concepts may change in subsequent way.

The dry dock is not really a problem but it is different for the deep site, which can be found only in some geographical circumstances which are particularly favourable. Such is the case in Norway and Scotland but not in most of the countries of the world. To get round the difficulty, the solution is to construct a flat caisson, which provides great floatability with a small draught. This consideration is representing a starting point for development of the so-called second generation of offshore platforms, fitting with the new geographical conditions.

The caisson of such structures, as designed at present, is toped with concrete columns supporting deck and with concrete buoyancy chambers. The columns are trunconically shaped at bottom and cylindrically shaped at the upper part with external diameter between 8 and 10 metres. One to four columns are provided according to deck arrangement and payload. Buoyancy chambers are cylindrically or trunconically shaped, their purpose being to ensure hydrostatic stability of the structure during immersion phases. Indeed, if the stability, measured by G.M. length is very large when caisson roof is above water level, or at the opposite, when the platform is nearly or completely immersed, however a critical phase exists at the time of the caisson roof submergence, for which G.M. is minimum as shown on the following sketch :

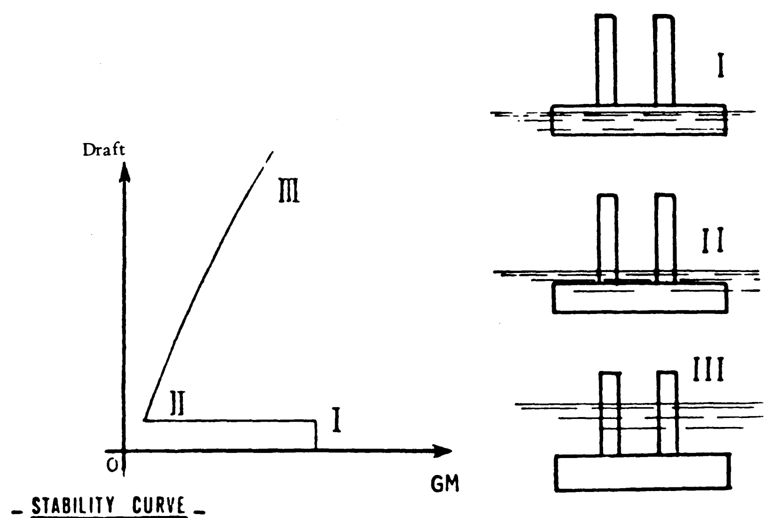

Fig. 25

In order to avoid instability at this stage of immersion, it is necessary to arrange buoyancy chambers at the four corners of the caisson, thus increasing shape stability due to floating area. The height of the buoyancy chambers is determined in order to have such shape stability until stability is assumed by weight stability in deep immersion configuration.

The caisson, square-shaped, is based on a pattern of vertical orthogonal walls which give it a great stiffness, and provide a rigid foundation base when the platform is installed. Concrete skirts below the raft slab may be arranged, providing, once having penetrated into sea bed, stability against sliding due to horizontal component of environmental forces.

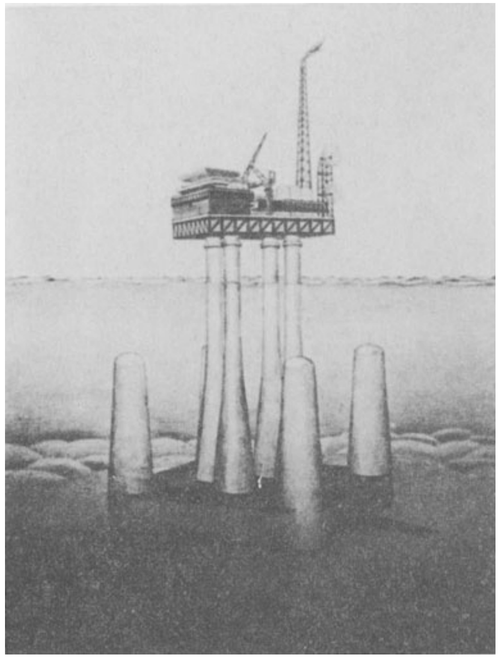

Fig. 26 : "C 80" (DORIS design)

Contrarily to first generation concrete platforms, the caisson roof on the outer vertical external walls too, is made of plane slabs instead of domes and vaults respectively. These flat elements are improving noticeably construction methods. Their use is allowed by the fact that in deep immersion configurations beyond some tens of metres depth, it is no more necessary to have the caisson partially filled with water in order to ensure platform floatation ; indeed this floatation is ensured by buoyancy chambers and columns. So the caisson may be completely filled with water and maintained in equipressure with sea water ; this avoids the dimensioning of the external element of the caisson in order to resist great hydrostatic pressure when immersion depth increases. Moreover, some caisson cells may be opened to sea at the upper part ; this arrangement allows, if necessary, easy pouring of solid ballast in order to improve marine stability when floating, and sea bottom stability when installed.

As most of concrete platforms, the one called "Concrete 80", has the advantage of allowing dry access to oil and gas risers, owing to watertight tunnels specially arranged in the caisson. This device allows welding of riser in atmospheric pressure, thus avoiding difficulties and risks of hyperbaric welding in marine environment.

"Concrete 80" platform may accommodate oil storage arranged in the four buoyancy chambers equipped with a piping network pumps and valves specially designed for this function.

On the other hand, if the storage function is not required, and if the stability of the platform on sea bottom is not sufficiently safe, i.e. in seismic area, it is possible to design removable buoyancy chambers that can be dismantled and removed when the platform is installed and eventually reused.

Another platform type of this "second generation" is the C 82, for which a single tower concept has been substituted to the multi-tower one.

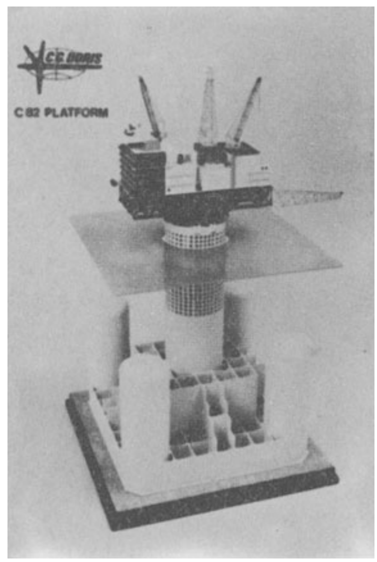

Fig. 27 : "C 82" (DORIS) design Fig. 28 : "COP" (Sea Tank Co) design

Once concrete shafts replaced by a steel lattice tower, the concept becomes hybrid ; a choice between the both ones depending on local conditions as well as on deck characteristics.

For a same deck payload and same functions, due to a best shaped design optimization, second generation concrete platforms save important quantities of material, compared to first generation ones. A comparative study, performed for 145 m water depth and a deck payload of 20 000 tons, shows a gain in material quantities, i.e. concrete and reinforcement, of an order of 30 %.

As regards construction method, it is noticeably improved by structural simplification and functions separation :

- the complete erection of the concrete substructures in dry dock may be performed in most of the countries bordering the sea ; indeed in such countries, it is always possible to dig a dry dock at convenient depths in order to build such structures ;

- the easy shapes of structural element, plane walls instead of vaults, lead to considerable reducing of construction costs ;

- a best arrangement of mechanical and process equipment has been performed due to experience gained in previous platform construction : mainly for columns in which now the minimum and only necessary piping is arranged. That allows a reducing in built-in items, which are difficult to position, and liable of causing delays in slipforming operations.

It has to be emphasized that the compliance of such a concept allows easy adaptation to deck payload increasing, even when these modifications are decided at a later stage of the substructure erection. Indeed in-plane caisson sizes are governed by floatation criteria for dry-dock tow-out and so are redundant for platform stability, when operating.

Consequently, to arrange on deck a larger payload than previously required, it is sufficient to increase the buoyancy chambers height which can be performed without other structural modification.

There should be mentioned finally, that new concepts of concrete gravity structures are proposed also at present for :

- the deep water conditions, ranging from 300 to 1 000 metres depth ;
- for ice infested waters in the arctic regions ;

the description of such designs being nevertheless outside the subject of this paper.

ACKNOWLEDGEMENTS

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A TENSION LEG FLOATING PLATFORM

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PREFACE

The tension leg platform is a new platform concept which is being developed for the Hutton Field in the U.K. sector of the North Sea. The Hutton TLP is a floating drilling and production platform attached by vertical tension legs to piled foundation templates on the sea bed. The deck supports all drilling and production facilities for ballasting, liquid consumables and tension leg anchorages. This paper describes the functional requirements, platform configuration and mooring system. Methods of analysis and structural design considerations including the influence of construction and marine operation requirements on the configuration are discussed.

INTRODUCTION

The Hutton TLP will be located in the northern part of the U.K. sector of the North Sea, about 90 miles north east of the Shetland Islands, in a water depth of approximately 148 metres.

The TLP is shown on Figure 1 and has a six-column configuration. The columns are connected at their base by rectangular pontoons while the column tops are connected by the boxed deck structure. The deck is a primary structural component, comprising a grillage of deep plate girders with structurally integral decks (main and weather decks) at the bottom and top of the 12m bulkheads. An important feature of the Hutton TLP is that most of the platform facilities are contained within the deck, although there are some modules and equipment mounted on the weather deck; and major items are accommodation, main power generator units and the drilling derrick. The platform geometry and the principal dimensions are shown in Figure 2.

The deck with all its integrated facilities and the hull are being fabricated separately and will be towed to a deep water inshore site where they will be joined together. After mating, the modules will be installed, the complex deck to hull joint will be welded and all the components will be hooked-up prior to towing the TLP to the Hutton Field for installation.

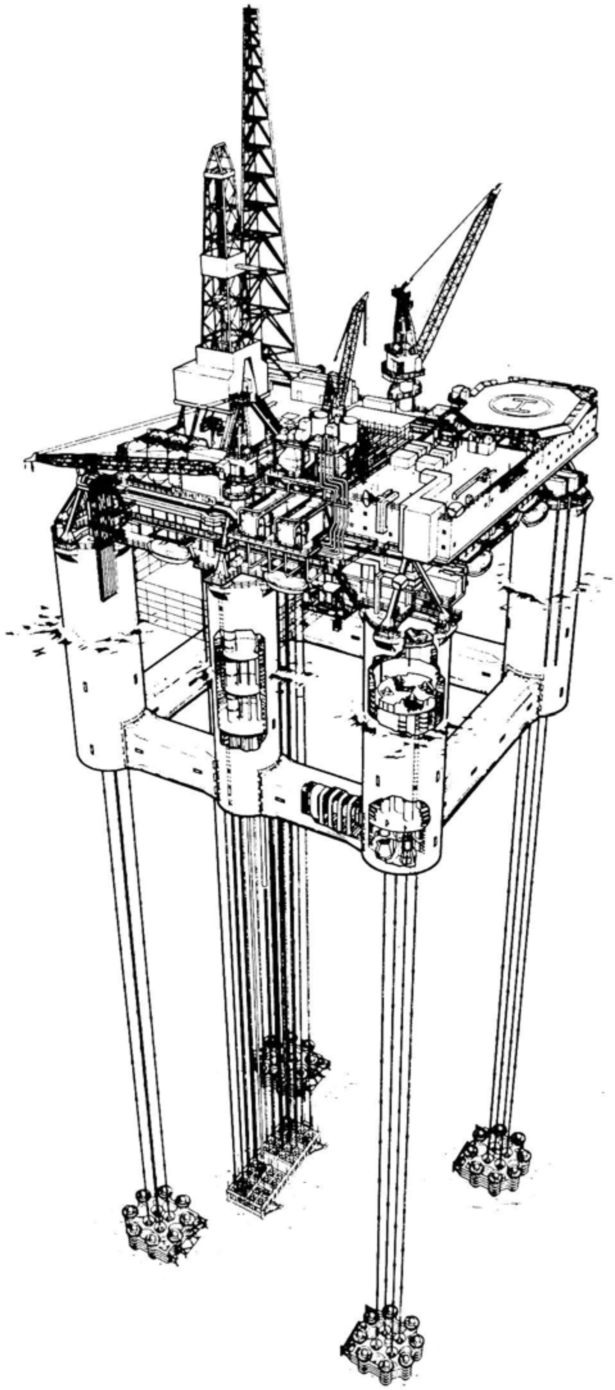

Fig. 1. The tension leg platform

GEOMETRY (All dimensions to moulded lines)			
LENGTH	-	Between column centres	78.00 M
	-	Overall	95.70 M
BREADTH	-	Between column centres	74.00 M
	-	Overall	91.70 M
HEIGHT	-	Keel to main deck	57.70 M
	-	Main deck to weather deck	11.25 M
DRAUGHT	-	Operating	32.00 M at LAT
FREEBOARD	-	To underside of main deck	24.50 M at LAT
WATER PLANE	-	Area	1324.00 M^2
COLUMNS	-	4 Corners	17.70 M Dia.
	-	2 Centre	14.50 M Dia.
PONTOONS	-	Height	10.80 M
	-	Width	8.00 M
	-	Corner radius	1.50 M
DISPLACEMENT	-	Approx.	61650 Tonnes
TOTAL WEIGHT	-	Including riser tension (Approx.)	48650 Tonnes

Fig.2

TLP GEOMETRY

The final design of the Hutton TLP has been influenced by many factors; some of the major ones are listed below:

1. Functional and operational requirements.
2. Topside weight and weight control.
3. Hydrodynamic behaviour.
4. Mooring system requirements.
5. Structural adequacy and fatigue life considerations.
6. Fabrication sub-division and sequence.
7. Marine construction activies.
8. Installation of the TLP at Hutton.

The influence of these factors are discussed in the paper.

PLATFORM FUNCTION

The platform supports all the necessary equipment and facilities for drilling and oil production operations, personnnel accommodation and ancillary services.

The drilling rig is rated for drilling to 18000 feet. It is integrated with the production facilities and is permanently installed for workover capabilities throughout the life of the field. The drilling derrick on its sub-structure is located on the platform weather deck and the other drilling equipment is located within the deck of the TLP.

Of the 32 well slots, 13 have been nominated for oil production, 11 for seawater injection and the remaining 8 are spare. Oil gathered from the 13 producing wells is processed through a single oil production train comprising 2 stages of separation and, at a maximum production rate of 110,000 barrels per day (BPD), is pumped to Sullom Voe through the Brent Pipeline System.

Hutton field crude has a low gas-oil-ratio and gas processing equipment is limited to producing a fuel gas for electricity generation. Any excess gas will be flared.

By far the largest quantity of fluid handled is seawater; the platform facilities have the ability to lift in excess of 200,000 BPD. Of this, 135,000 BPD are filtered, deaerated and then injected back into the reservoir for pressure maintenance. The balance is used for production and utility cooling services.

Power for the platform is provided by a combination of gas turbine driven generators; two rated at 15MW and one rated at 3.4 MW. The units can burn either fuel gas or diesel, or in the case of the larger units, both fuels simultaneously. Fuel gas supply is only in excess of demand during periods of peak production; hence the need for a dual fuel system.

A facility the size of the Hutton TLP will require up to of 239 offshore operations personnel. Accommodation is provided to house, feed and entertain this number of men. For reasons of safety the accommodation modules are located at the opposite end of the platform to drilling and oil production operations.

WEIGHT, BUOYANCY AND CENTRE OF GRAVITY

Platform weight and buoyancy are of fundamental importance in the design of the TLP as at all times the following simple equation of static equilibrium must be met:

Weight + leg tension + riser tension = Buoyancy

After many months of intensive work of investigating the static hydrodynamic performance of various hull configurations and detailed interactive weight estimating of the entire platform with its facilities, the hull configuration was fixed. The configuration, and hence platform bouyancy, was chosen at the end of an interactive process which had to balance all of the terms in the above equation. The weight budget was set and very close control of weight has been maintained throughout the detailed design and fabrication work. The approximate values of the weight, leg tension, riser tension and buoyancy are shown below:

Hull operating weight	26500)	47300 tonnes
Deck operating weight	20800)	
Tension leg pre-tension		13000 tonnes
Riser tension		1350 tonnes
Displacement		61650 tonnes

Also the centre of gravity of the platform is of considerable importance. The operating weight plan C of G should ideally be located in the centre of the platform as any significant deviation would result in a requirement for permanent ballast (and consequently permanent weight) to rectify it. The vertical centre of gravity is also an important parameter which controls the free floating stability during tow to the Hutton Field and influences hydrodynamic response of the TLP.

The magnitude of the tension leg pretension is considerably influenced by the hydrodynamic behaviour of the vessel which is discussed in the next section.

HYDRODYNAMIC BEHAVIOUR

The hydrodynamic behaviour (i.e. motions and responses) of the platform under the action of enviornmental loading is one of the most critical features of the design of the TLP. The extreme design environmental conditions used in the design and summarised in Figure 3. The TLP (shown in Figure 4) is a compliant structure which allows lateral movements of surge, sway and yaw and restrains heave, pitch and roll. The surge motion of the TLP is similar to that of an inverted pendulum where the excess buoyancy provides the restoring force, instead of gravity. The amount of excess buoyancy is designed such that the tension legs do not go slack nor are overstressed under the worst combination of payload and environmental loading. A more detailed description of the dynamic behaviour of the TLP is discussed in Ref. 3.

EXTREME DESIGN ENVIRONMENTAL CRITERIA	
Wind	
1 minute mean @ 10m elev.	44 m/sec
Wind gradient variation with elevation according to 1/8th power law	
Waves	
Regular "Design" Waves	
Height	30.3m
Period	14.6 - 18.5 sec
Irregular Waves	
Significant Height	16.6m
Average zero-crossing period	13.9 sec
Current	
5 minute mean at 10m depth	0.85m/sec
Waver Level	
Range between HDWL and LDWL	2.9m

Fig. 3

ENVIRONMENTAL DATA

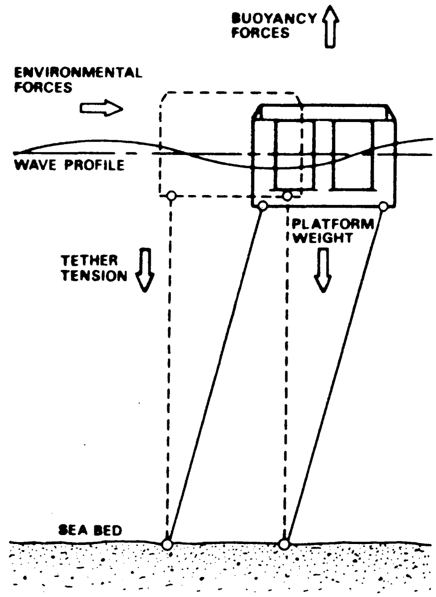

Fig. 4. Schematic representation of forces and movements and movements of TLP

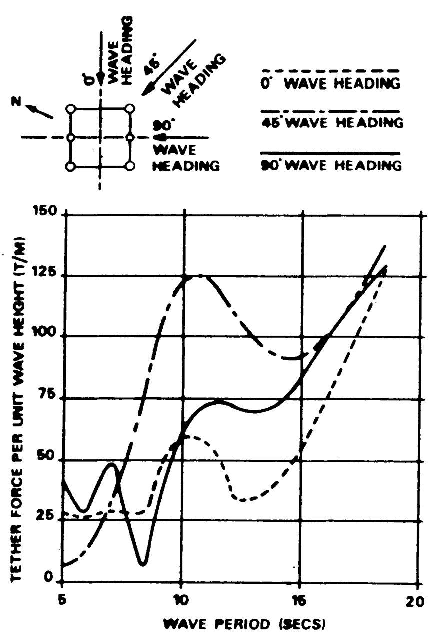

Fig. 5. Variation of tether forces with respect to wave incidence and period

The TLP is subjected to both horizontal and vertical forces arising from the environmental loads, including the inertia loading, which cause a steady and oscillatory lateral movement and variation in tether tension. The horizontal loads caused by wind, current and wave drift produce a mean offset of platform surge, sway and yaw. The restoring forces of the above loads are provided by the horizontal component of the inclined tether tension. The vertical load in the tension leg is primarily due to the pretension and to the restraint of heave and pitching motions.

Extensive model tests and theoretical calculations using a complex mathematical model were carried out to predict the responses and tether loads under various environmental conditions. Secondary effects arising from low or high frequency motions and tether ringing were also studied and are discussed in Ref. 3.

The natural periods for heave, pitch and roll are around 2 seconds while for surge/sway and yaw they are 50 and 42 seconds respectively. This means that the surge mode is effectively uncoupled from other modes of oscillations. The steady offset of the surge motion was found to be approximately 9.5 m while the oscillatory component was 15.5 metres for the extreme environmental condition. Wave heading and the wave height are also two other principal parameters which cause variation in the tether tension with respect to wave period and wave incidence due to unit wave height. It is observed that, between 9 to 12 second periods, where the wave lengths correspond to approximately twice the pontoon spacing, the transfer functions have relatively high values arising from the pitching moment. The trough between 12 and 15 second period corresponds to the cancellation of the buoyancy forces on the columns and the vertical inertial forces on the pontoon. The tether forces are usually higher in the direction of the 45° wave incidence since the restoring moment is effectively offered by half the number of tension legs compared with 0 and 90° wave incidence case.

The peaking of the transfer function at a lower wave period tends to control the fatigue life of the tension legs and the adjoining support structures since these elements will be subjected to more cyclic loading at a higher stress range.

VESSEL CONFIGURATION

The TLP vessel shown in Figure 1 comprises two main units, the Hull structure and the Deck Structure. The principal components of the hull structure are pontoons and corner and centre nodes and columns, while the deck consists of a grillage of bulkhead girders and structural deck pallets. The configuration of these items is discussed in the following section.

Corner Columns

Each corner column consists of a ring stiffened cylinder closed at top and bottom and sub-divided by watertight flats into six compartments as shown in Figure 6. The column top provides an encastre attachment between the deck and hull using a grillage of infill plates at four locations on each column; these are backed up by substantial vertical stiffening to transmit the deck load into the column shell plating. The column top (shown in Figure 12) supports all temporary facilities required for the deck/hull mating operation. The watertight flat at the column top is stiffened by a boxed section cruciform beam which supports a 125 tonne polar crane required for handling the mooring system equipment.

The mooring compartment, shown in Figure 6, houses all mooring equipment and is where the vertical component of the four tension legs is transferred to the column through the load block assembly. The tension forces are transmitted to the node directly by the tether shroud and also by the column shell through interaction of the watertight flat floor beams at various levels. The mooring flat is located within the splash zone of the column and is protected by a 2m wide by 12.5m deep double skin construction damage control ring. This protects the column from any accidental vessel impact, and limits the ingress of water. It also provides an alternative load path in the event of damage to the external shell plating.

The compartment immediately below the mooring flat is subdivided by vertical bulkheads to store potable water, drilling water, diesel oil

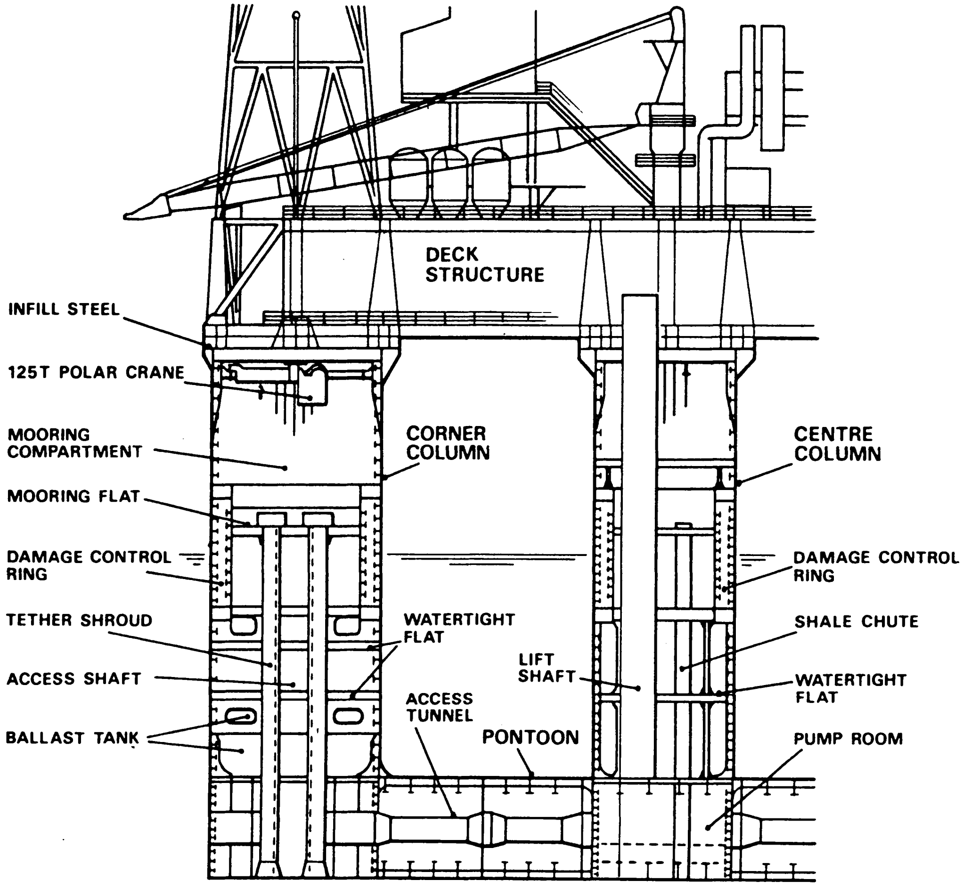

Fig. 6. Cross section through TLP columns

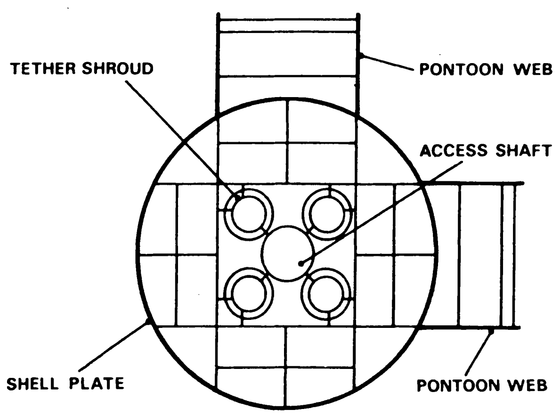

Fig 7. Column/pontoon node plan section

and for routing services from the mooring compartment to the lower tanks. The subdivision of these tanks is not identical for all four corner columns. A central shaft is provided in this area for access to various levels and to run services, thus avoiding a multitude of penetrations in the watertight flats.

There are five compartments between the mooring flat and the top of the node to control the effect of accidental flooding. The two lower compartments are also used as permanent and operational ballast tanks to rectify operational variations of centre of gravity of the platform. The bottom tank is kept either full or empty to avoid problems of sloshing. The upper ballast tank which may be partially filled has its floor beams above the flat to reduce the sloshing effect.

Centre Column

The centre columns, shown in Figure 6, are much simpler in construction than the corner columns since they do not have the complication of mooring system requirements. However, the centre columns share a greater proportion of deck weight and provide main access from the deck to the pump room in the centre node. The centre columns are divided into four compartments. The lower tanks are only ballasted in the mating or tow out condition where the TLP draught is required to be increased.

Pontoons

The pontoons are designed as rectangular box girders, the shell plating being orthogonally stiffened by longitudinal stiffener and web frames and watertight bulkheads (see Figure 8). The corners of the pontoons are rounded to reduce hydrodynamic drag forces. A central tunnel is provided through the pontoon to run services and for easy access from the corner column to the pump room or to the pontoon ballast tanks. Pontoon tanks are similar to centre column ballast tanks which control ingress of water in the event of accidental damage but they are not used during normal platform operation.

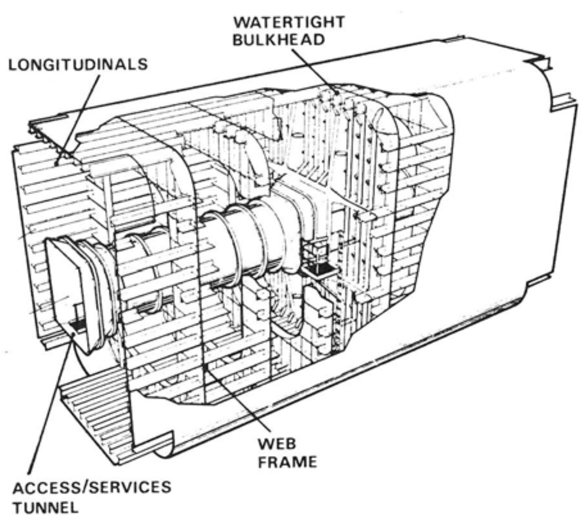

Fig. 8 Pontoon

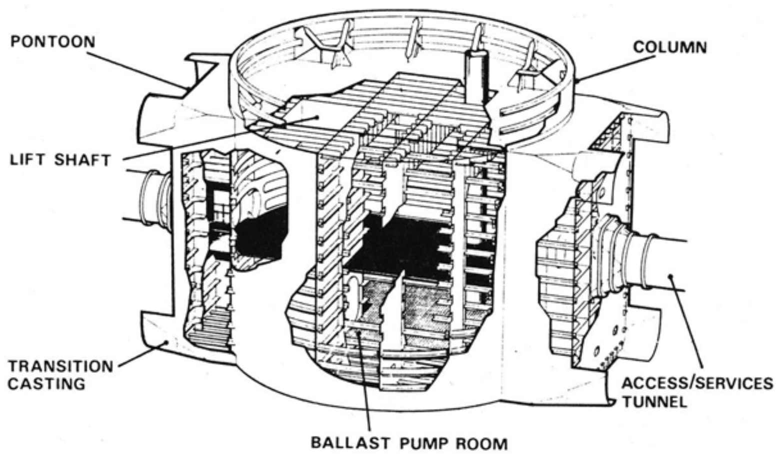

Fig. 9 Centre node

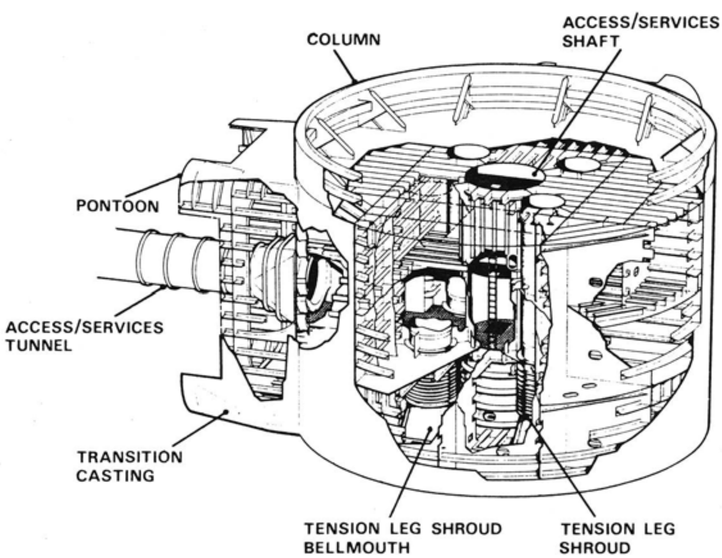

Fig. 10. Corner node

Corner Nodes

The corner node shown in Figure 10 connects the two longitudinal pontoons to the vertical column and resists all pontoon and forces in addition to the horizontal component of tether force. The layout of the corner node is controlled by many factors, such as, the width of the pontoon, the minimum construction clearance at the tether shroud bellmouth, accessibility in the space between the two intersecting pontoon web and shell plates (see Figure 7). The node has two intermediate flats to resist the horizontal reactions from the tension leg cross load bearing. The pontoon stiffeners are continuous into the node to provide a direct load path and avoid misalignment at the joint. The radiused corner of the pontoon are tapered to a spare configuration at the intersection of the shell to simplify construction details inside the node. A cast steel transition piece has been used at each corner of the pontoon to solve a complicated geometry problem and also to keep the critical welds away from the point of highest stress concentration. This casting reduces local stress concentrations and improves the fatigue life in this area.

Centre Nodes

The layout of centre nodes are much simpler since they do not have the complexity of the tether shrouds (see Figure 9). The centre node houses facilities equipment including the ballast pumps, sea water lift pumps and the transformer/rectifiers for the impressed current cathodic protection system. Access to the centre node is provided by a lift.

Deck Structure

The TLP deck is a large integrated box structure composed of 12 m deep bulkhead girders with main-deck, mezzanine and weather deck as shown in Figure 11. The bulkhead girders subdivide the deck into large spaces which house the platform facilities. The weather deck provides workspace for the topside facilities and supports large modules e.g. two accommodation modules, drilling derrick and substructure, helideck, flare tower, deck cranes etc. as shown in Figure 1. Some of these packages weigh up to 1000 tonnes each.

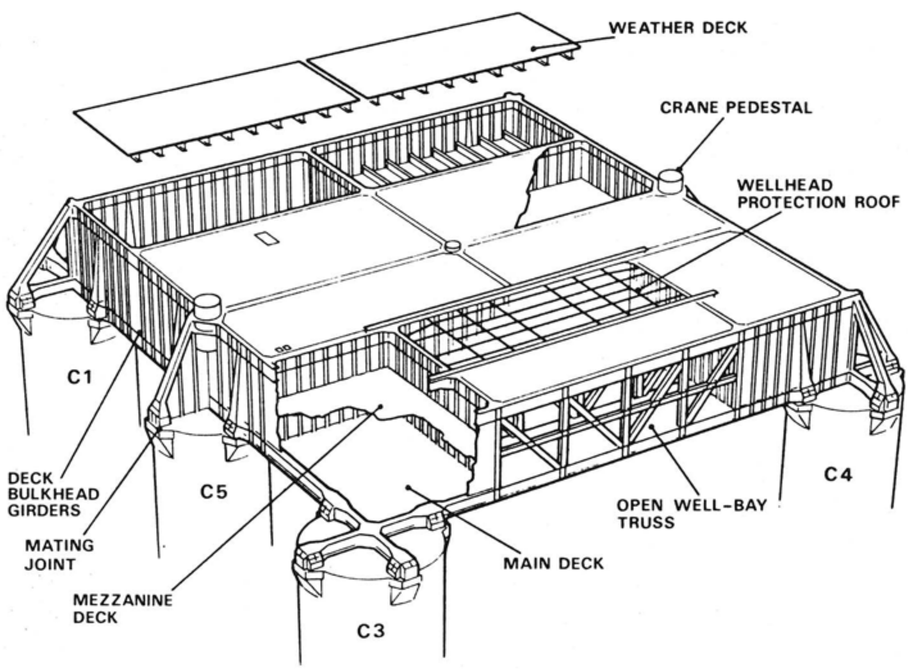

Fig. 11. Deck structure

The bulkhead girders are of plate girder construction having substantial horizontal and vertical stiffening except the two wellbay girders at the south end of the platform which are open truss girders to facilitate fire fighting. The girders in this area span 74 m and support the drilling derrick weight, (approximately 1,200 tonnes) and in addition support all riser tensions. The wellbay area has an impact resistant roof to prevent damage to the wellheads by accidental dropped objects.

The deck is supported by the hull at each column top and acts integrally with the hull to resist all environmental and other live loads. The deck structural arrangement incorporates many distinct features such as palletisation of deck, discrete girder construction and deck hull mating joints. These features were introduced to improve speed on construction and are discussed below.

Palletisation

Deck floor beams are palletised into large sub-assemblies and process equipment is installed on these pallets prior to assembly of the main structure. The deck pallets are lifted into position and connected to the main structure using a spliced detail to avoid any fabrication misalignment. The pallets act integrally with the deck structure and participate in supporting operational loads on the completed deck.

Discrete Girder Construction

The bulkhead girders are assembled as discrete girders. Wide flanges have been incorporated into both top and bottom chords to provide free standing stability during erection. The bulkhead girders have extensive horizontal and vertical stiffening to prevent buckling of the web and also assist in catering for the many penetrations through the girder web. The deck is assembled on top of a 4 m deep load out truss which is used to skid the deck from land onto the barge for transport to the mating site.

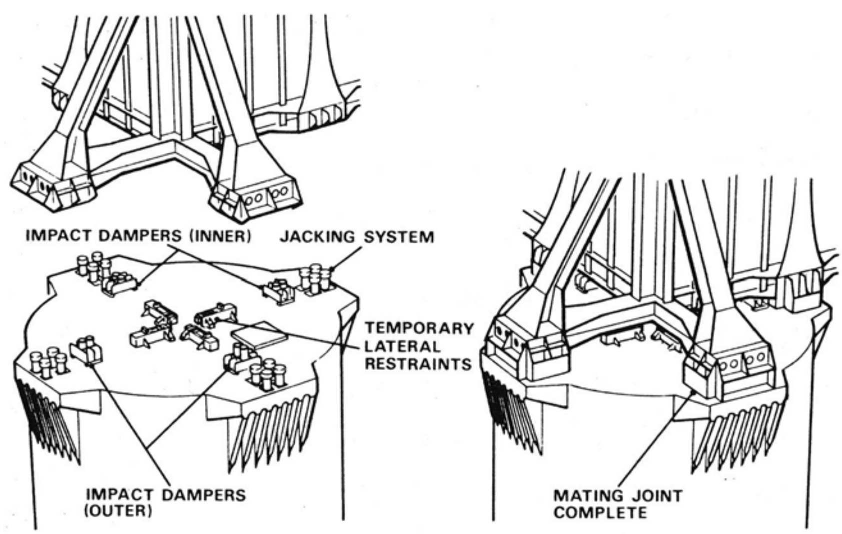

Fig. 12. Mating joint

Mating Joint

The deck to hull mating is a complicated marine construction problem since it involves connecting two large floating bodies in exact location without inducing undesirable impact or overstress in the deck or hull. The deck loads are transmitted to the column shell plate through the mating joint (see Figure 12). The mating joint is formed by a grillage of plates which line up with deck stiffening in one direction and align with the column top stiffening in the orthogonal direction.

The design of the joint (Figure 12) was influenced by the fabrication misalignments between deck and hull structures and the interface load during mating operation which has to be controlled throughout the operation.

The mating operation and the necessary special equipment are discussed later in the paper.

STRUCTURAL ANALYSIS AND DESIGN APPROACH

The analysis can be broadly sub-divided into static, dynamic and finite element analysis. Since the deck and hull are built separately, the analysis had to be carried out for different phases in the platform's life. i.e. deck structure, hull structure and combined TLP structure with their respective loading to arrive at the worst combination of stress. Various platform phases and load combinations are discussed in ref. 1 and 2. Since the design is based on limit state approach, various loads were applied as separate load cases to take account of their different partial safety factors. These loads are summarised as follows:

1. Structural dead load
2. Environmental dynamic loads i.e. wind, wave and current.
3. Platform functional load
4. Equipment
5. Tension leg reaction
6. Riser reaction
7. Module load
8. Buoyancy
9. Ballast
10. Temporary mooring loads

Global analysis was carried out on a stick model using the NASTRAN computer program to obtain member forces for all static load cases. The dynamic forces were evaluated by using a computer program VODAS. VODAS was correlated with extensive wave tank model testing to ensure correlation between analysis and model tests. VODAS output was compatible with the NASTRAN stress analysis program which provided element stresses at various locations in the structure (see Reference 1). Local finite element analysis was carried out on various sections of the structure with boundary conditions established by the global analysis. These include centre and corner nodes, mating joint at column top, deck structure, typical sections of column, pontoon and deck girder with penetrations. The results obtained from these analyses were combined to produce the worst design case for every element of the structure.

The largest and most complex finite element analysis model was one of the corner nodes which was modelled using 4000 elements and had over 100 individual load cases. A post-processor was developed to perform automatic load combinations and check plate panels on strength and serviceability criteria for both operational and extreme conditions. The fatigue life at various points of each structural element was also established using the above program.

Design for fatigue was one of the major problems encountered during the structural design of the TLP. Large areas around the column/ pontoon intersection and the mating joint at column top were found to be extremely sensitive to the fatigue loading. Various methods of improving fatigue life were adopted to minimise the weight increase. These include grinding, avoiding cope holes, grinding stiffeners at terminations, toe burr grinding at critical areas and also the use of cast steel transition pieces at the points of stress concentration.

Finite element analysis had to be heavily relied upon throughout the analysis and it was considered necessary to compare the F.E. analysis with acrylic model tests. A correlation study was carried out using a

1 to 30 scale acrylic model with 400 strain gauges to cover all critical areas. The correlation with the F.E. analysis was generally very good, however stick model results were not accurate at the points of high stress concentration.

MOORING SYSTEM

Design and Development

A most important feature of the TLP is the method of permanent mooring between the Hull and Foundations, the general arrangement of which is shown on Figure 13. The design of this system involved a wide spectrum of engineering disciplines in order to meet a novel combination of requirements.

The mooring system must be capable of assembly in and deployment from the TLP at the Hutton Field. Cyclic tension loads up to 9100 tons per corner column must be accommodated. Fatigue tolerance was required as approaching 100 million load reversals could be anticipated during the 20 year design life. Extremely high reliability was required as the operational integrity of the mooring is vital to the operation of the platform. The mooring system is also required to accommodate the platform motions which although restrained still contain variable offset, rotation and yaw. These requirements have all been incorporated into the Tension Leg System which has sixteen steel tubulars located in sets of four at each of the platform corner columns.

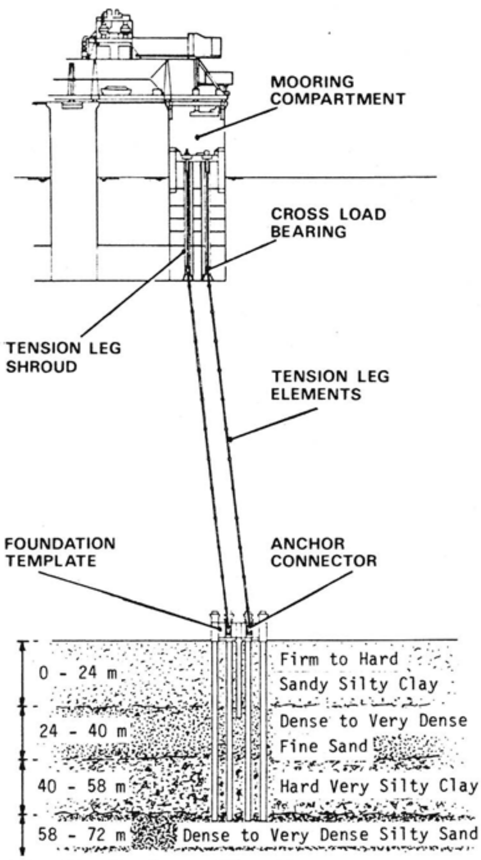

Fig. 13. Mooring configuration

The handling and deployment of the tension leg components together with hydrodynamic effects upon the installed leg dictated a minimum weight, minimum scantling solution. A high strength material was therefore required. Additionally, in order to provide predictable through life defect tolerance to either material flaws or superficial damage, significantly high fracture toughness was required.

This unique combination produced a material specification requiring a proof stress of 795 MPa minimum couples with a minimum COD (crack opening displacement) value of not less than 0.45 mm at $0^{o}C$. The consistent achievement of this specification on such large components required a great deal of manufacturing control. It was necessary for potential component manufacturers to further develop their expertise on $3\frac{1}{2}$% Nickel Chrome Molybdenum Valadium steel ($3\frac{1}{2}$ NCMV) in order to comply with the above requirements. The mechanical properties are achieved by close control of chemical composition and quality heat treatment. The steel is manufactured by the basic electric arc process, is secondary ladle refined and vacuum degassed. The resultant steel exhibits excellent resistance to fatigue, corrosion fatigue and stress corrosion cracking in the operational environment.

Mooring loads are transmitted to the hull at two locations. The vertical loads are transmitted by a Load Block assembly (see Figure 14) mounted on the floor of the mooring compartment. The load block comprises a pair of parallel beams which may be hydraulically separated to facilitate installation and upon which a screwed collar engaging the upper leg component (the tension adjusting element) is supported. Lateral loads are transmitted by the Cross Load Bearing (see Figure 15) which is a component of the leg positioned so as to bear upon the tether shroud at a predetermined location in the corner node which is structurally designed for this purpose. Connected between the tension adjusting element, the cross load bearing and the anchor connector are 14 tension leg elements.

At the lower end of the leg is the anchor connector (see Figure 16). This hydraulically actuated device can be remotely latched and unlatched. The anchor connector engages with a removable template insert and load is transmitted via this insert to a large forged abutment ring which is an integral part of the foundation template.

The Mooring System has also been discussed in Reference 4.

FOUNDATIONS

The foundations (shown in Figures 13 and 17) consist of four separate steel octagonal templates. Each tempalte is anchored to the seabed by eight tubular steel piles, 72 inches in diameter, stabbed

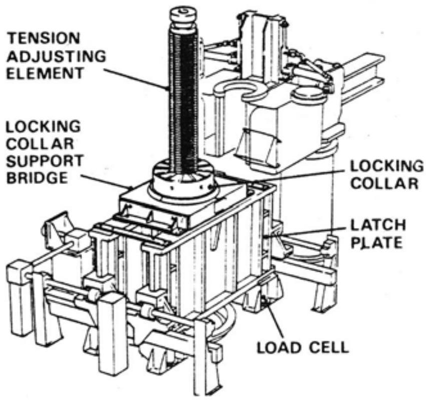

Fig. 14. Load block assembly and torque tool

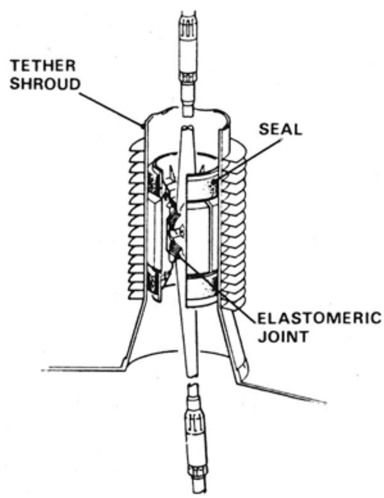

Fig. 15. Cross load bearing

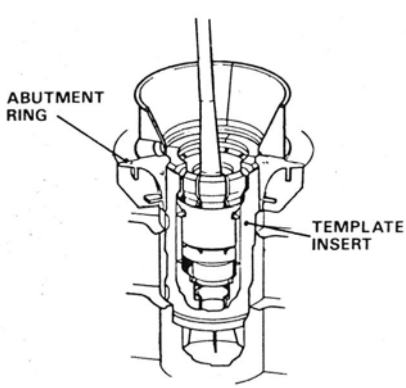

Fig. 16. Anchor connector inserted in foundation template

through the sleeves in the template and driven to a minimum penetration of 58 metres. The foundation templates are positioned on the seabed using a large temporary guidance frame. The pile diameter and the penetrations have been established after extensive site investigations and geotechnical testing and analysis. A typical subsoil description is shown in Figure 13.

The pile group has a pitch circle diameter of 16.5 metres which ensures the group has maximum efficiency in terms of axial tensile capacity.

An additional pile sleeve is located at the centre of each template through which a pin pile is driven to a penetration of 24.5 metres. The pin pile is used to stabilise the template during the main piling operation and to ensure that the required foundation positional

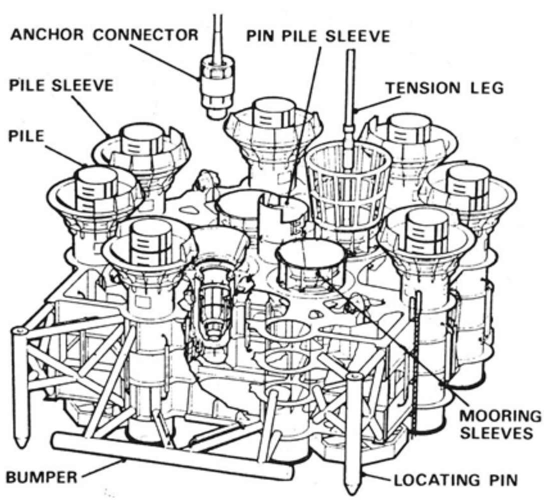

Fig. 17. Foundation template

tolerances are maintained. All the piles are connected to the template by injecting cement grout into the annulus between the pile and the sleeve. These grouted connections between the structural piles and template are an important part of the load transfer path. A series of specific tests were conducted on grouted connections to confirm the validity of the design that has been adopted.

Located around the pin pile, on a pitch circle diameter of 6.5 metres are four mooring sleeves, each of which incorporates, at the upper flange level, the forged abutment ring.

The mooring sleeves are connected to each other and to the pile sleeves by 4.5m deep plate girder. Additional stability and a more even distribution of load is further achieved by connecting adjacent pile sleeves to each other in a similar manner.

The loads generated by the platform and transferred to the foundations are large and tensile in nature comprising a sustained vertical component with superimposed cyclic horizontal and vertical components. The maximum combined sustained and cyclic load per foundation for the 100 year design storm is 9100 tonnes.

The primary objective of the pile group design was to ensure an adequate margin existed on the axial tensile capacity under the most severe long term load conditions. The effect of both lateral and axial cyclic loading on the pile group capacity was one of the foremost considerations in the foundations design.

The pile penetration and diameter was governed by the presence of a dense, thick sand layer approximately 58 metres below the mudline and the capacity of the largest underwater pile driving hammers presently available.

MARINE OPERATIONS

The marine operations may be split into three main phases: mating, vessel tow and installation.

Mating

The mating operation is a critical marine operation where the deck structure, weighing more than 14,000 tonnes, is placed accurately above the ballasted down 25,300 tonne hull and lifted by deballasting the hull in less than 24 hours.

The deck is transported on a transport barge with a freeboard of of 15 m and positioned above the ballasted hull. The mating operation involves first fixing the two free floating bodies in exact location and suppressing their relative motions without inducing undesirable impacts on the hull prior to transferring the deck load to the columns through 24 points of contact (6 columns and 4 support points per column). It is important that the loads at each column top are distributed uniformly at all stages of the mating operation; this is why a jacking system was incorporated in the design. The hull is deballasted from 44.5 m to 41.33 m to lift the deck from its transport barge. The mated unit is then further deballasted to a draft of 35 m for the welding of deck to hull infill steel and to complete hook-up operations. The mating operation is designed for an environmental condition of up to 1 m significant wave height of 6 sec. wave period.

These operations have required very careful engineering since the relative motions between hull and deck are to be suppressed at various stages and many special features were incorporated such as stab and guide, impact dampers, jacking system, temporary lateral restraint (see Figure 12). These items are discussed below.

A stab and cone arrangement was introduced between the deck girder and the top of one of the centre columns to restrain relative surge motions. The stab pin is 1930 mm long, composed of a 850 mm dia. fabricated tube with taper ends, and is capable of resisting a lateral impact load of 500 tonnes during mating. Following the stabbing operations and further deballasting the yaw motions are effectively restrained by a lateral guide on the other centre column.

Impact dampers are located on the top of the corner columns to restrain the deck/barge pitching motions. The dampers cushion the impact load and are capable of resisting up to 2000 tonnes impact load at each corner column. The inboard dampers are 700 tonnes capacity hydraulic dampers, while the outboard dampers are 300 tonnes capacity solid urethane block.

A jacking system is used at the 24 points of contact for equalisation of loads at the deck/hull intersection. 96 jacks are used, of which 56 inboard jacks are of 650 tonne capacity while 40 outboard jacks are of 350 tonne capacity. Four jacks are nested at each point of contact (see Figure 12). Each jack has a spherical bearing to accommodate rotation and a smooth low friction interface with underside of deck girder. In the case of failure of one jack in the nest of four, the jacking system is designed to cut-off the diagonally opposite jack to avoid eccentricity of loading. After the deck loads are transferred to the jacks the total column load may be adjusted to correlate with the theorectical value prior to locking off the jacks and subsequent completion of the infill steel.

A temporary lateral restraint has been incorporated at each column top to resist horizontal loads up to 1100 tonnes arising mostly from the environmental loading between deck and hull. These temporary lateral restraints are required to be released during module lifting operations to allow for the deck profile changes. They are locked during welding of the infill steel.

The infill plates between the column top and deck are introduced in two stages. In the first stage the jacks remain in position and hold the load while sufficient infill steel is welded between deck and hull hull infill steel and to complete hook-up operations. The mating and the remaining infill plates are welded to complete the joint and the deck and hull would then act as an integral structure to resist all environmental and other live loads.

Following completion of the mating joint all TLP systems are hooked-up and, where possible, commissioned prior to the TLP being towed to the Hutton Field for installation.

Installation

The installation of the TLP on its operational location at the Hutton Field has required considerable installation engineering effort and the development of new handling equipment for the deployment of the 16 tension leg assemblies.

The prime objective of the platform installation is to satisfactorily connect the tension legs to the pre-installed foundation templates. The operations are undertaken in a prescribed manner to translate the vessel from the free floating to the TLP (heave suppressed) mode and to establish the necessary leg pretension by the discharge of onboard ballast.

Prior to installation the tension leg components and handling equipment are pre-outfitted in the mooring compartments.

In order to install the tension legs the platform is positioned over the foundation templates and restrained from lateral excursions imposed by the prevailing environment. The lateral restraint is achieved through the use of temporary moorings connected to tugs arranged in star formation.

Tension leg installation is weather sensitive and therefore the installation procedures have been developed to install the legs as quickly as possible commensurate with safe operations.

The tension leg assemblies are run in four rounds of one per corner. The first round is installed using the Tensioner/Motion Compensator which performs the anchor connector stab into the foundation template, draught.

The stab in operation is carried out with the aid of visual and acoustic displays of the anchor connector from ROV's and subsea mounted transponders.

Deployment for the second, third and fourth round of tension legs is carried out using the polar cranes since the vessel heave has been suppressed. After installation of all 16 legs a final deballasting operation is performed to establish the required leg pretension.

This operation completes the installation of the TLP. The pre-drilled wells will then be connected to the platform by tensioning the individual risers and the wells will be completed using the platform drilling rig. The oil sales riser will also be connected to the well template and hence to the pre-laid export pipe-line. Commissioning of the oil producing part of the platform system will complete all of the preparation for FIRST OIL.

CONCLUSION

The Hutton TLP is the first of a new generation of offshore platforms and will have helped to extend production capability into deeper waters. The engineering of the platform has enhanced knowledge and understanding of the development of prototype offshore systems in general and of tension leg platforms in particular.

Many of the components, sub-assemblies and systems of the platform are new or incorporate novel features or design methods. These newly developed aspects of the TLP together with the conceptual and detailed design and analysis, the extensive model testing and material testing programmes are the tangible benefits to the engineering community from the bold decision of the participating companies to proceed with the Hutton TLP.

ACKNOWLEDGEMENTS

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Conoco (U.K.) Limited (Operator); Britoil Limited; Gulf Oil Corporation; Amoco (U.K.) Exploration Company; Gas Council (Exploration) Limited; Mobil North Sea Limited; Amerada Petroleum Corporation of the United Kingdom Ltd.; Texas Eastern North Sea Inc.

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DESIGN AND ANALYSIS OF DEEP WATER MARINE RISER SYSTEMS FOR FLOATING PRODUCTION FACILITIES

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Battelle Petroleum Technology Center

Introduction

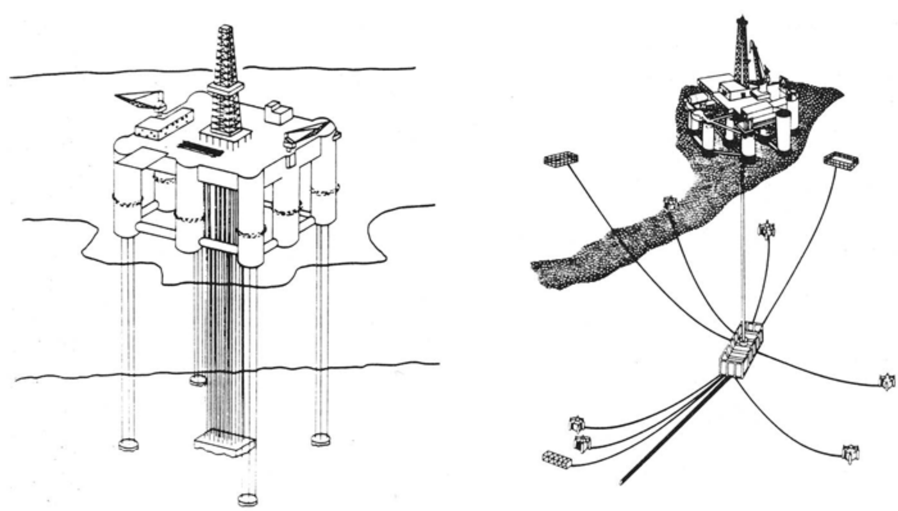
As the global offshore search for oil and gas reserves expands, deeper waters and more hostile environments will be encountered. However, as the search continues, the likelihood of finding very large reservoirs that justify the expense of fixed platform production facilities (conventional jackets or gravity-type structures) decreases. Furthermore, the water depths encountered, in some cases, exceed the bounds of fixed platform technology. To produce the marginal fields, or those at great depths, therefore, suggests the use of floating production facilities. From the standpoint of the marginal field, a floating production facility requires low capital outlay and provides an early return on investment. In addition, after the field is depleted, the floating production facility can be easily reused. From the standpoint of a deep water reservoir, a floating production facility may be the only technological and economic alternative for producing the field. In either case, the riser system for the floating production facility is a crucial element in the system and, consequently, the structural analysis of the riser system becomes a key in the design process.

The basic design goals for a floating production facility riser system are; a) maintain functional and/or structural integrity under an extreme loading condition, and b) show acceptable fatigue life under long term loading. In order to achieve these goals, detailed design and analysis must be performed on every component in the riser system; from threaded connectors, to hydraulic line stabs, to the tubulars, to guides between risers, to tensioner attaching lugs, etc. To do these detail designs, however, the analyst needs loading data, which can only be obtained from an overall global riser system motion and stress analysis. Although the detail component design work is important, it is, for the most part, application of well known design techniques. The overall riser motion analysis for a floating production facility, on the other hand, is not so routine and thus, is the focus of this chapter of the case studies.

In the sections that follow, the design and analysis of a riser system for a floating production facility will be discussed. Although the discussions could be immediately focused on a specific riser installation, a slightly broader treatment is given, since the details of the specific riser designs are generally proprietary and because the narrowness of the overall topic, risers, easily allows a general treatment for all floating production facility riser designs. With this overall philosophy, specific topics to be considered are basic floating production riser system characteristics, global riser system analysis fundamentals, and application of the first two topics to a floating production riser system design.

Floating Production Riser Systems

The riser system in a typical floating production facility consists of a number of individual risers, from 6 or 8 up to 40 in some cases, connecting the sea floor wellheads and a floating vessel of some type. Figure 1 shows a Tension Leg Platform type system and a conventionally moored (catenary anchored) semi-submersible type system. Regardless of the vessel and its mooring arrangement, the risers are lengths of pipe



a) TLP System b) Semi-submersible System

Figure 1. Typical Floating Production Systems

assembled into a long string with some type of connector (threaded, bolted, locking ring arrangement, etc.), and tensioned at the top. The individual riser strings may be independent, or there may be guides constraining motion between various strings at certain elevations. Aside from some minor design details, all floating production risers are structurally the same: A tensioned, near vertical pipe from the sea floor to the floating production vessel. Although all floating production facility risers are generically the same, differences in function do exist which must be considered in a riser analysis.

In terms of function, a floating production facility may have three different types of risers:

- Drilling riser
- Production riser
- Export riser.

Each functional type of riser is different geometrically (larger, smaller), and has different design requirements that must be considered in a riser system analysis.

Drilling Riser. The drilling riser for a floating production facility is generally a large diameter (20 inches) riser that is used only on a temporary basis until all wells are drilled. The basic functions of the drilling riser are:

- Protection of the drill pipe from wave and current loads

- Return path for drilling mud and cuttings
- Path to well bore for control of formation pressure
- Support for control lines to blow out preventer.

Typically, drilling risers are designed for a rather limited fatigue life (30 years) and are easily inspected and repaired, if needed. Included functionally with this category are the actual drilling risers and workover risers for maintenance of the well bore after the well has been completed.

Production Riser. Production risers are generally from 4 to 10 inches in diameter and bring the fluids produced by the well(s) (gas, oil and water) to the surface vessel for processing. Production risers generally are designed for a 100 year fatigue life, are often intended to remain in-place for the life of the field and, compared to drilling risers, operate at higher pressures. Included in the production riser category are gas lift risers, small diameter risers paired with a production riser to inject produced gas into the crude at the base of the well bore, used to reduce the specific gravity of the crude and enhance flow of the produced fluids to the surface.

Export Riser. Export risers, often called sales risers, are used to ship the produced oil and gas to market. Sizes range typically from 6 to 15 inches, with fairly high pressure ratings (in excess of 2000 psi). Also included in the general export riser functional category are water injection risers and gas injection risers used to pump produced water and gas back into the formation. As in the case of the production risers, export risers are generally designed for a 100 year life.

A floating production riser system consists of some combination of drilling/workover, production, and export risers, depending upon the specific floating production facility operating design. At the minimum, a floating production riser system will consist of one production riser and one export riser in fixed locations on the sea floor and vessel. At the other extreme, a floating production riser system will consist of two or three drilling/workover risers, 30 production risers and 8 export risers including water injection risers, all able to be located in any slot, at any time, on a 40 slot sea floor well template.

Riser Analysis Requirements. In spite of the diversity of possible overall configurations of floating production facility riser systems (sizes, functional types and locations of risers), the basic design requirements for the riser system are quite clear:

- Provide reliable flow paths for the various fluids between the subsea manifold and vessel
- Survive a specified extreme storm (50 or 100 year storm) intact without structural failure
- Exhibit a satisfactory fatigue life.

Specifically, the design requirements for the floating production riser system dictate the following analyses:

Subject to wind, wave, current and vessel motion, determine:

- The tensioner and buoyancy requirements
- The potential for interference between various risers, and between the risers and the vessel

- Design loads for all riser components
- Fatigue life of the riser tubulars.

To meet these requirements, sufficient hydrodynamic and structural analyses must be performed to establish the validity of the design, and to ensure that the design is economical.

Fundamentals of Riser Analysis

The application of the floating production facility riser analysis requirements to a system of drilling, production and export risers, as described in the previous section, is not as straight forward as it may appear. The designer, faced with a new riser system to analyze, must make decisions about the formulations and solution techniques that he will use to do his analysis. Because these decisions can have a profound impact on the cost of an analysis and the confidence in the results, the formulations and mathematics associated with riser analysis need to be explored before discussing an actual floating production facility riser design. To this end, the basic equations of motion are presented for marine risers, a discussion of the various types of dynamic analyses is given, approaches to modeling multiple tube riser systems are discussed, two fatigue design approaches are presented, and a brief discussion of the overall analysis methodology is given.

Equations of Motion of Risers. The small deflection equations of equilibrium of a tensioned or compressed member, ignoring shear effects and subject to a lateral distributed load, q, and a distributed vertical

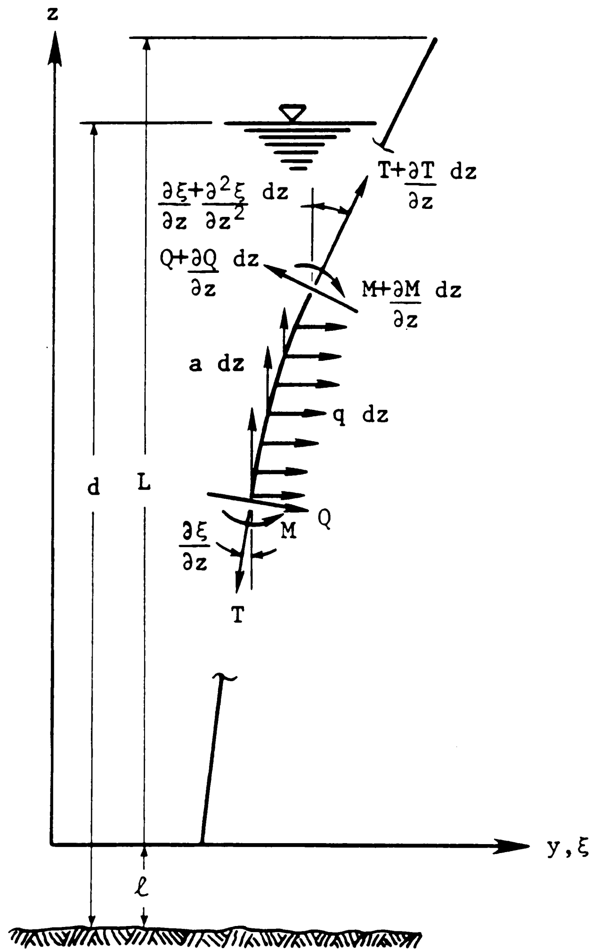


Figure 2. Riser Differential Element

load, a, as shown in Figure 2, are:

$$\frac{\partial}{\partial z}\left(T\,\frac{\partial \xi}{\partial z}\right) - \frac{\partial Q}{\partial z} + q = 0 \tag{1}$$

$$\frac{\partial T}{\partial z} + a = 0 \tag{2}$$

$$Q = \frac{\partial M_b}{\partial z} \tag{3}$$

where the bending moment, M_b is related to the transverse displacements, ξ, by

$$M_b = EI \frac{\partial^2 \xi}{\partial z^2} . \tag{4}$$

For a marine riser, the distributed lateral and axial loads consist of the

- Weight of the riser material
- Inertial loads due to acceleration of the riser tube and contents
- External and internal hydrostatic loads
- External hydrodynamic loads
- Applied concentrated loads.

Assuming Morison's formula for the hydrodynamic loads, and D'Alembert's principal to define inertial loads, the lateral distributed loads can be written as:

$$q = -(m_s + m_c)\ddot{\xi} + \frac{\partial}{\partial z}\left((p_o A_o - p_i A_i)\frac{\partial \xi}{\partial z}\right)$$

$$+ \frac{1}{2}\rho_w D C_D \left| (V + u - \dot{\xi} \right| (V + u - \dot{\xi})$$

$$+ \rho_w \frac{\pi}{4} D^2 \; (C_M \dot{u} - C_A \ddot{\xi}) + F_L \tag{5a}$$

$$0 < z < (d - \ell)$$

$$q = -(m_s + m_c) \; \ddot{\xi} - \frac{\partial}{\partial z} \; [p_i A_i] \; \frac{\partial \xi}{\partial z} + F_L \tag{5b}$$

$$(d - \ell) < z < L$$

where

$$p_o = -\rho_w \; g \; (z + \ell - d) \tag{6}$$

$$p_i = p_i^{top} - \rho_f \; g \; (z - L) \; . \tag{7}$$

Referring to Figure 3, the axial distributed load on a riser is given by

$$a = -w_s - w_c^k \; \delta(z-z^k) + p_o \; \Delta A_o^n \; \delta(z-z^n) + F_V \tag{8}$$

where

$$\Delta A_o^n = A_o^{n+} - A_o^{n-} \; . \tag{9}$$

Although a great number of variations such as flowing internal fluids, non-constant drag coefficients, changes in internal cross-section, etc. are possible, the above equations summarize the essential aspects of the riser distributed loadings.

Substituting equations (3), (4) and (5) into (1) and rearranging

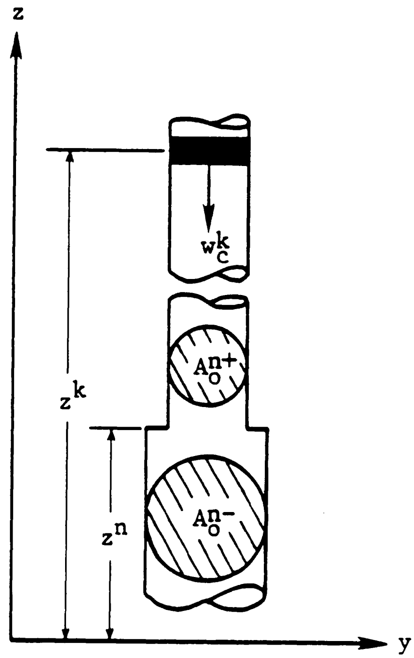


Figure 3. Lumped Weight and Section Change Parameters

slightly, the lateral equation of motion becomes:

$$\frac{\partial}{\partial z}\left((T + p_o A_o - p_i A_i)\frac{\partial \xi}{\partial z}\right) - \frac{\partial^2}{\partial z^2}\left(EI\frac{\partial^2 \xi}{\partial z^2}\right)$$

$$- \ddot{\xi}\,(m_s + m_c) + \frac{1}{2}\rho_w C_D D \left|(V + u - \dot{\xi})\right|(V + u - \dot{\xi})$$

$$+ \rho_w \frac{\pi}{4}(C_M D^2 \dot{u} - C_A D^2 \ddot{\xi}) + F_L = 0 \qquad (10)$$

where it is understood that above the water surface (still water level or wave surface) the external hydrostatic and hydrodynamic terms are zero (i.e., $\rho_w = 0$).

Substituting equation (8) into equation (2) and integrating, the riser wall tension becomes

$$T = T^{top} + p_i^{top} A_i^{top} - \int_z^L w_s \, dx$$

$$+ \int_z^L F_V \, dx + \int_z^L p_o \, \Delta A_o^n \, \delta(x-x^n) \, dx$$

$$- \int_z^L w_c^k \, \delta(x-x^k) \, dx \qquad (11)$$

where the last two terms represent step changes in the riser wall tension.

Equations (10) and (11) can be solved by any number of methods. Typically, a finite element or finite difference approximation is made resulting in a set of coupled linear algebraic equations for the motion.

Alternative Solution Schemes for Riser Systems. Given the basic equations of motion for a marine riser, equations (10) and (11), a suitable spatial integration algorithm (finite elements, finite differences, transfer matrices, etc.) and prescriptions for the water particle velocities and accelerations, the motions and hence, stresses in the riser system can be determined. In carrying out the analyses, the user can select any of four different methods of analysis:

- Static solution
- Wave slice pseudo-dynamic analysis
- Frequency domain dynamic analysis

- Time history dynamic analysis.

The assumptions for each of these solution methods are different, which in turn influences their relationship to a riser design study.

In static analysis, the terms $\ddot{\xi}$, $\dot{\xi}$, u, and $\dot{u}$ in equation (10) are all taken as zero. Essentially these assumptions yield a set of linear equations of the following form when the spatial integrator is used

$$[K]\{x\} = \{F(V,F_L,\xi,z)\} \ . \tag{12}$$

Obviously, the force vector {F} is a function of the displacements and axial position of a point on the riser. Ignoring the functional dependence of {F} on the lateral displacement, ξ, and assuming small deflections, equation (12) is a linear set of equations that can be solved without iteration for the displacement {x}.

The second mode of analysis, wave slice, assumes in equation (10) that only $\ddot{\xi}$ and $\dot{\xi}$ are zero. Effectively, this amounts to adding wave particle velocities and accelerations to the static loads, thus giving a better approximation to dynamic analysis than a static solution. The equilibrium equation for the wave slice mode is

$$[K]\{x\} = \{F(V,u,\dot{u},\xi,z)\} \ . \tag{13}$$

For a riser that does not undergo large lateral motions, equation (13) is a good approximation to a dynamic solution. As in the case of the static analysis, equation (13) represents a set of linear algebraic equations that can be solved without iteration for $\{x\}$, or in a very few iterations if the force vector dependence on ξ is allowed.

Dynamic analysis considers equation (10) in its full nonlinear form. When written out in matrix form, the equations are

$$[M]\{\ddot{x}\} + [K]\{x\} = \{F(V,u,\dot{u},\xi,\dot{\xi},\ddot{\xi},z)\} \ . \tag{14}$$

Equation (14) represents a set of coupled second order differential equations with a nonlinear forcing function that has a periodic solution with decaying initial transients. That the solution is periodic is a consequence of the periodic nature of the forcing function (waves), while the transient portion of the solution is the result of the initial conditions. In general, the solution that is sought in a riser analysis is the periodic solution and two approaches to finding this solution are generally applicable, frequency domain solution and time domain solution.

The frequency domain method seeks the periodic solution to equation (14) in closed form. Through a process of linearizing the problem (the relative velocity squared term of the viscous drag is the most powerful nonlinearity), an iterative solution to the dynamic response of the riser system can be obtained. Basically, to carry out this method of solution, it is assumed that the motion of the riser system can be described by a harmonic function (at the wave frequency) of the form

$$x = x_o + \hat{x}\cos(\omega t + \phi) \tag{15}$$

while wave particle velocities and accelerations are taken as

$$u = \hat{u}\cos(kx - \omega t)$$
$$\dot{u} = du/dt \tag{16}$$

which are the forms for linear (Airy) waves. Substitution of (15) and (16) into equation (14) and expansion of the nonlinear drag force in a Fourier series [1,2] yields a set of coupled algebraic equations for x_o, x and ϕ at each degree of freedom. The displacements and phase angles are functions of linearization coefficients that are in turn functions of the solution. The net result of this is that an iterative solution must be found for the two sets of equations

$$\left([K] - \omega^2[M]\right)\{\hat{x}\cos(\omega t + \phi)\} = \{F_1(u,x,B_1,z)\}$$
$$[K]\{x_o\} = \{F_2(V,B_2)\} \tag{17}$$

in which values of B must be assumed, a solution for x_o, x and ϕ found, B updated, etc. The details of the manipulations and evaluation of the B's to effect a solution are given in the references. Given solutions for x_o, x and ϕ at each degree of freedom, the solution for any time, t, can be found by use of equation (15).

The time domain method seeks the solution to equation (14) by directly integrating the coupled equations of motion in time. Utilizing assumed relationships between accelerations, velocities and present, future and past displacements over a time increment Δt, equation (14) is reduced to[3]

$$[K_{eff}]\{x^{t+\Delta t}\} = \{F_{eff}\} \tag{18}$$

where $[K_{eff}]$ and $\{F_{eff}\}$ are known functions of previous displacements. A solution for the displacement x at any time, t, is found by stepping through equation (18) in time increments of t until the desired time is reached. In general for riser analysis, sufficient time steps are run to achieve a steady state solution.

Given that there are four solution modes generally used in riser analyses, one must have some basis for selecting one or the other method. Table 1, in a rather rough way, ranks the solution modes in order of increasing accuracy and increasing computational cost: Time domain is the most accurate, but is also the most costly; static is the least expensive but gives a poor approximation to dynamic response. A well formulated frequency domain solution can produce solutions approaching the quality of a time domain solution but at a cost much closer to a static solution. Wave slice, on the other hand, does not have the quality of solution that a frequency domain solution has, but it is less expensive.

Multiple Riser System Analysis. The modeling of multiple tube riser systems can take one of two basic approaches: An analysis which considers each and every tube in the system or an analysis that models the riser system as a single "equivalent" riser. Each approach has its pros and cons and each serves a useful purpose in the analyst's repertoire of structural analysis techniques.

Solution	Matrix Size r: real c: complex	Load Vector Solutions	Cost Factor
Static	1r,n x n	1	1
Wave Slice	1r,n x n	1/wave slice	2
Frequency Domain	1r,n x n 1c,n x n or 1r,2n x 2n	1/iteration 1/iteration 1/iteration	8
Time Domain	1r,n x n	1/time step	100

Table 1. Comparison of Riser Solution Modes

The riser equations (10) and (11) can be written for each tube in a multi-tube system. Interaction between the risers can be accommodated via F_L and F_V using Dirac delta functions. An exploded section of a typical multi-tube riser is shown in Figure 4. The central riser,

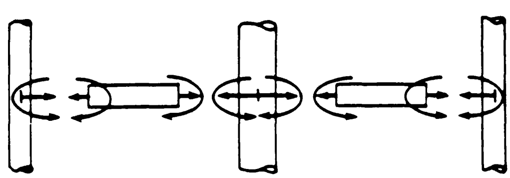


Figure 4. Exploded Section of a Multi-tube Riser

surrounded by a group of other risers, has support spiders which serve to guide the peripheral risers in the lateral directions.

Analysis of a multi-tube riser system, considering all of the risers, is fairly straight forward. Each riser in the system is modeled using its own structural and hydrodynamic properties. The structural interaction between risers can be handled by inserting "riser" elements between the riser tubes or by use of constraint equations. Typically, computer programs that are capable of performing this type of analysis, use a finite element discretization of the equations of motion. They can accommodate very diverse geometries and boundary conditions, and could be made general enough to account for the effects of hydrodynamic interaction (shadowing) between the risers. The price to be paid for the generality and flexibility of this approach, of course, is execution speed.

In the equivalent tube analysis technique, riser equations (10) and (11) are again written for each tube in the string. In this case, however, the separate equations for the tubes are reduced to a single equivalent equation. When the separate tubes are "bundled", the interactions between the risers become internal reactions and hence, are self-equilibrating. Considering that a riser behaves essentially as a tensioned string, it is reasonable to assume that at every level, all members of the bundle experience identical slopes, deflections and substantially the same wave and current loadings. Thus, for the "equivalent" riser system in which the subscript j denotes the j^{th} tube, the equations of motion become

$$\frac{\partial}{\partial z}\left(\sum_j (T + p_o A_o - p_i A_i)_j \frac{\partial \xi}{\partial z}\right) - \frac{\partial^2}{\partial z^2}\left(\sum_j (EI)_j \frac{\partial^2 \xi}{\partial z^2}\right)$$
$$+ \frac{1}{2} \rho_w \sum_j (C_D D)_j \left|(V + u - \dot{\xi}\right| (V + u - \dot{\xi})$$
$$+ \rho_w \frac{\pi}{4}\left(\sum_j (C_M D^2)_j \dot{u} - \sum_j (C_A D^2)_j \ddot{\xi}\right)$$
$$- \ddot{\xi} \sum_j (m_s + m_c)_j + \sum_j (F_L)_j = 0 \tag{19}$$

$$T_j = \sum_j T_j^{top} + \sum_j (p_i^{top} A_i^{top})_j + \sum_j \int_z^L (F_V)_j dx$$
$$- \sum_j \int_z^L (w_s)_j dx + \sum_j \int_z^L (w_c^k)_j \delta(x-x_j) dx . \tag{20}$$

The exact details of the equivalences are a function of the particular computer program input, but if the program has been designed to run equivalent bundled riser analyses, it will surely have a very large number of independently specifiable diameters (structural diameter, hydrostatic diameter, structural mass diameter, etc.) to allow for easy user input. If, on the other hand, the program was not specifically designed for this type of analysis, compromises will have to be made. Boundary conditions will have to be set so that the overall global behavior of the system is properly modeled.

Within the bounds of the assumptions of the equivalency, the displacements and slopes produced by the equivalent riser analysis will be the true values. All other parameters of interest in the analysis such as moments, shear forces, stresses, etc., will be equivalent values. If subscript e denotes the equivalent values and j the values for the j^{th} tube, the following should hold

$$M_j = \frac{(EI)_j}{(EI)_e} M_e \tag{21}$$

$$Q_j = \frac{(EI)_j}{(EI)_e} Q_e \; . \tag{22}$$

The tension forces are most easily calculated by use of equation (11) for each riser.

Equations (19) through (22) summarize the basic theoretical aspects of creating an "equivalent" riser model. In practice, the equations do not quite fulfill expectations. Except for rather benign cases in which all risers have identical top and bottom boundary conditions and intermediate elastic or ball joints at identical locations, the question of what the correct equivalent EI's are must be addressed. The choice of the equivalent EI when only one riser out of seven has a ball joint is not very clear. Furthermore, the use of equations (21) and (22) become questionable around the ball joint location. To resolve these difficulties, a slightly different approach to equivalent riser modeling must be adopted.

Assuming that equations (19) and (20) are valid for the global behavior of the riser system and that appropriate stiffnesses can be found for discontinuities such as ball joints in only one of several risers, a motion analysis can be performed. Rather than using equations (21) and (22) to recover individual riser moments and shears, an alternate approach is used: An assumption is made that the motion at guide locations in the equivalent model is the true motion of each of the risers. A model for each individual riser can be analyzed using the loads appropriate to that riser and the displacement boundary conditions at guide locations from the "equivalent" riser model. The moments, shears and stresses from this analysis are assumed to be the true values for the riser. Boundary conditions and discontinuities in the individual tubes are thus accounted for, while the overall global riser system response is preserved.

The question of what stiffness properties should be used when only one riser out of several has a ball joint still must be resolved. A very crude approach would be to use a small segment of riser at the ball joint location with the value of the equivalent EI equal to the summation of the EI's of the risers without the ball joint. A more sophisticated approach would be to find an equivalent EI on the basis of equal virtual work for an actual and equivalent segment of the riser system. In either case, the analyst can probably find a reasonable value. If there is doubt about the choice of the equivalent EI at a discontinuity, a sensitivity study can be performed.

In general, the success of using the above outlined equivalent riser analysis approach will depend upon the particular riser program being used and the analyst's skill in making the "equivalent" model. The individual riser models and the retrieval of individual riser stresses should present no particular problems.

Fatigue Analysis. The last topic to be considered in the basics of riser analysis is fatigue life prediction for riser systems. Reduced to its simplest terms, the activities/data needed to do a fatigue life prediction are:

- Wave scatter diagram
- A number of dynamic analyses at various wave height-period combinations
- An algorithm for computing fatigue damage.

Obviously, the wave scatter diagram is given in the design specifications. The dynamic analyses are needed to define mean stress and stress range for the various locations on the riser as a function of wave height and period.

The algorithm for computing fatigue damage is generally Miner's rule,

$$D = \Sigma \frac{n_i}{N_i} \tag{23}$$

where

D = fraction of life expended

n_i = number of occurrences of stress range S_i

N_i = number of occurrences of stress range S_i to fail the specimen.

The values of N_i generally come from S/N curves (Figure 5).

As an alternative to the Miner's rule - S/N curve approach to fatigue life prediction, a crack growth analysis, which assumes an initial flaw and grows the crack analytically to failure according to the

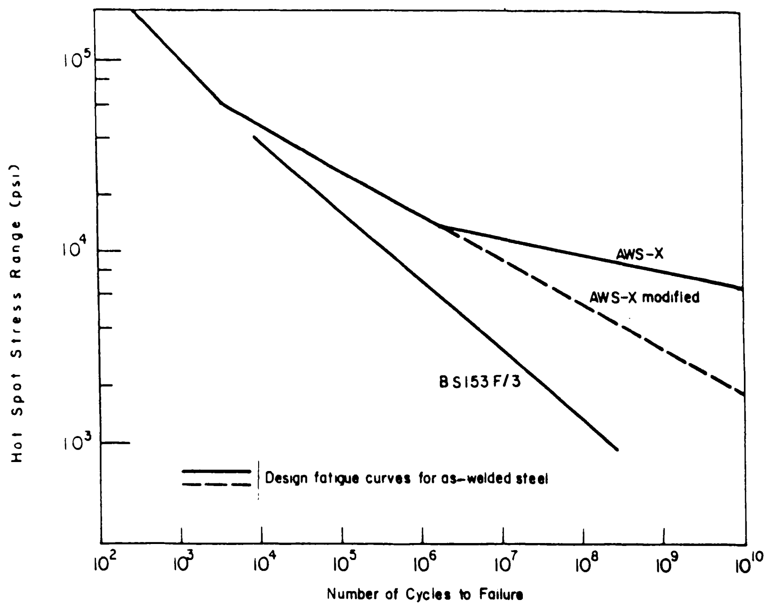


Figure 5. S/N Curves for Fatigue Analysis

principles of linear elastic fracture mechanics can be used:

$$a = \int_{N_o}^{N_f} \frac{da}{dN}\, dn + a_o \tag{24}$$

where

a = final crack size

a_o = initial crack size

N_f, N_o = final and initial cycle counts

da/dN = incremental crack growth rate.

Values of incremental crack growth rate are generally functions of the stress intensity factor range ΔK, which in turn is a function of crack size, material thickness, material properties and applied stress range. Incremental crack growth rate is generally plotted on log-log scales as shown in Figure 6. (ΔK is a measure of the stress intensity factor range at the crack tip - not the same as the stress concentration factor K_t). In either case, Miner's rule or crack growth, the basic riser data requirement is the same: A large number of dynamic analyses with various wave height-wave period combinations.

Riser System Analysis Methodology. The previous sections covered, in rather brief terms, the basic analysis techniques available and/or needed for doing a riser analysis. To carry out an actual floating production riser system analysis, the analyst must select options from

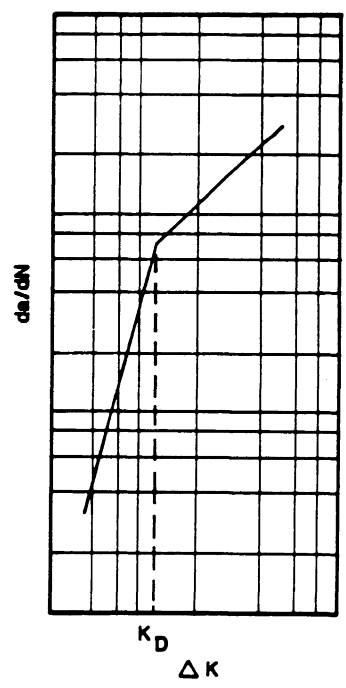


Figure 6. Crack Growth Rate Versus Stress Intensity Factor Range

this menu that will allow him to meet his particular design objectives within time and cost constraints.

As indicated in the introduction, the floating production facility riser analysis, as described herein, is concerned with the global riser system response, rather than the detailed analysis of the individual riser system components such as connectors, hoses, stabs and tensioning assemblies. In this context, the riser system analysis may include:

- Determination of tensioner and buoyancy requirements
- Definition of the potential for interference of the risers and vessel
- Establishment of preliminary riser system operating criteria
- Determination of design loads for all riser system components
- Providing load data for detailed component fatigue analysis
- Evaluation of the fatigue life of the riser tubulars.

Relative to these objectives, the riser system analysis need only be concerned with the environment - waves and current conditions; the vessel's station keeping response; the tensioner - capacity and dynamic response; the upper marine riser package - weight, tensioner attachment, location and guiding; riser tubulars - arrangement, buoyancy, connector weights, guides, section properties and internal fluids; and the lower marine riser package - weight and guideline attachments. Items such as the stabs on the lower marine riser package, the threads in the riser joint connectors, the lugs that fasten the tensioners to the riser, etc.

are peripheral to the global riser analysis and are only related to the analysis in that loads to design these components come from this analysis.

From the standpoint of how to meet the objectives of a floating production facility riser analysis, it is clear that time domain, multi-tube analysis will provide the most accurate data. Unfortunately, to meet even just a few of the objectives stated above using this type of analysis would be prohibitively expensive. As a consequence, the design and analysis of a marine riser system for a floating production facility is a series of trade-offs on two levels: The analytical approach and the actual system parameters. It is the job of the engineer to balance the technical accuracy of the analysis, the cost of the analysis and overall system constraints to achieve a safe, economical riser system design.

Riser Design Example

As an illustration of the riser analysis techniques and design methodology discussed above, some of the highlights of the design analysis of a multiple tube production riser system will be presented. Although the results presented can not cover all of the analyses done (over 300 individual computer runs were made), the highlights of the system description, analysis goals, typical results from the computer runs, and overall conclusions of the analysis will be presented, to illustrate what the design and analysis of a marine riser system for a floating production facility entails.

Floating Production Facility System Description. The floating production facility considered as the design example in this chapter is the result of an economic study that indicated that, relative to fixed jackets or gravity type structures, a floating production facility utilizing as much proven equipment as possible should have a lower initial cost and a faster return on investment for the water depth at the reservoir site. Figure 7 shows the general arrangement of the facility. Satelite production wells, drilled before the production platform was put on site, produce through "wet" trees and subsea flowlines back to a manifold collecting station. The production of each well is brought to the floating platform by a separate flowline riser for each well. On the vessel, the production of each well is logged, the gas, water and crude are separated, and the combined crude output of all

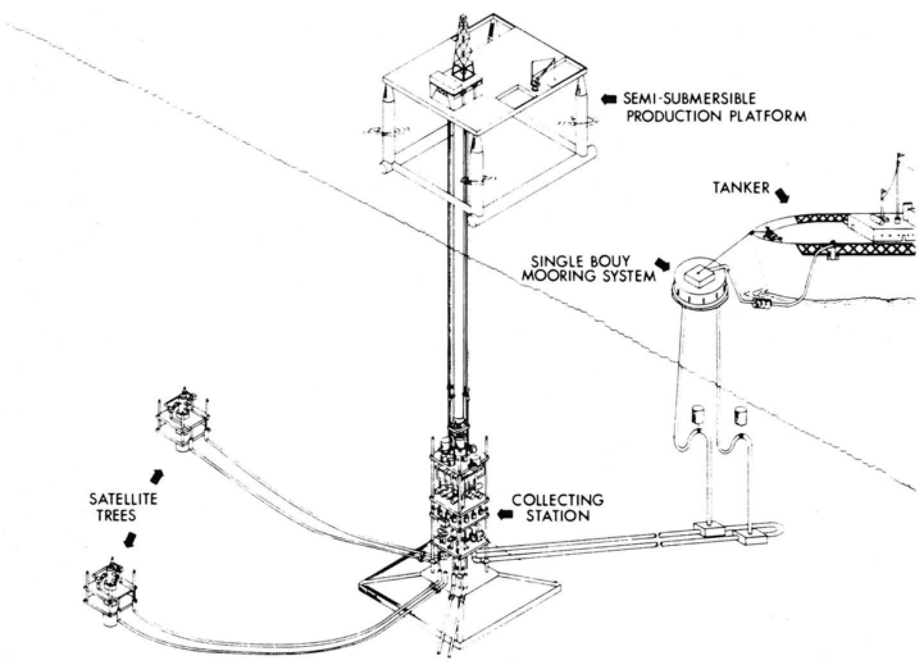


Figure 7. Floating Production Facility Design Example
General Arrangement

of the wells is shipped out to a tanker via a large export riser, subsea pipeline and moored buoy loading system. Although this general description of the facility gives an overall perspective of the system, to perform the riser analysis, more detailed information is required for the environment, the vessel, and the risers.

For the particular site of the floating production facility, a complete environmental report was generated. Included in the report were wind, wave, current, tide, precipitation, and temperature predictions based on actual measurements and data interpolated and/or extrapolated from sites nearby. Although the report is rather voluminous, based on criteria for gas separator operations, riser handling, etc., the environmental data needed to do the riser system analyses reduces to that shown in Table 2.

In terms of the vessel, two basic items must be known: a) The station keeping response of the mooring system, and b) The dynamic motion characteristics of the vessel in wind and waves. The first vessel parameter determines the static offsets from directly above the mainfold

Water Depth:	250 feet
Wave Height:	15 feet to 50 feet
Wave Periods:	6 seconds to 16 seconds
Current:	Typically none, 1 knot at surface, maximum

Table 2. Environmental Data for Riser Analysis

that must be considered, while the second determines the forced motion excitation at the top of the riser system. In the case being considered, the vessel was a converted semi-submersible drilling vessel, 200 feet by 225 feet, anchored with a 12 line catenary mooring system. Based on the environmental report, mooring analyses, and dynamic vessel motion simulations, typical vessel response data as shown in Figure 8 were generated. The static offset is given as the maximum likely to occur in operating conditions, while the dynamic vessel response is given in terms of vessel response amplitude operators (RAO's) and phase angles relating

a) Static Offset: maximum 10 feet

b) Vessel Dynamic Motion:

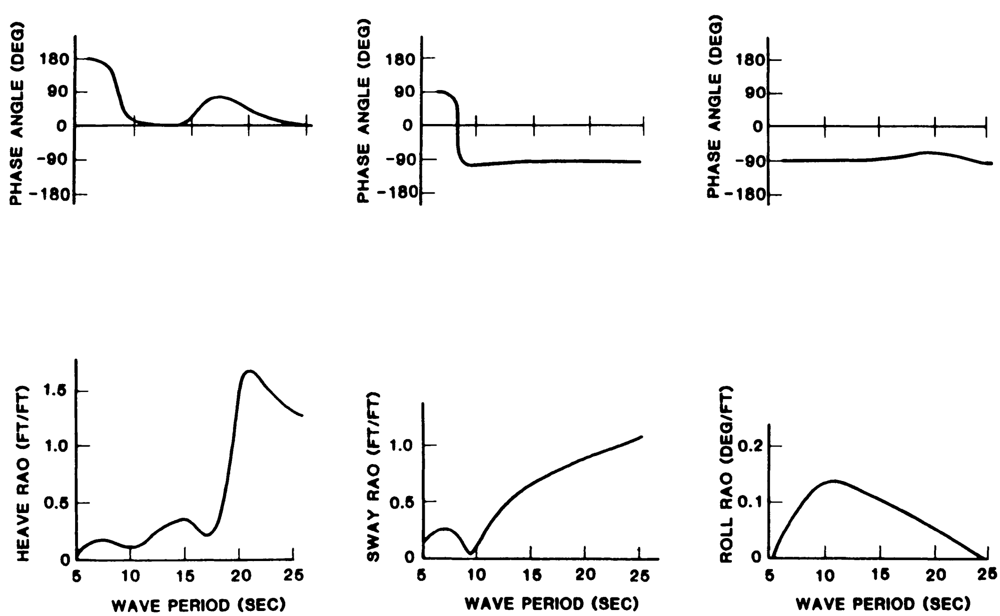


Figure 8. Typical Semi-submersible Vessel Repsonse
(Beam Seas)

wave amplitude to vessel response amplitude at various wave periods for various wave incidence angles. The only other vessel data directly relevant to the riser analysis is the clearance between the lower pontoon hull bracing, 28 feet, at 73 feet below the water surface.

The basic geometry of the riser system is shown in Figure 9. Essentially there is a large diameter riser with six flowlines around the periphery. The flowlines are constrained by guides to move with the central riser at various locations, with the risers at the 12, 3, 6 and 9 o'clock positions in Figure 9 constrained at every other location starting from the 160 foot elevation, with the remaining two risers constrained at the remaining locations. In terms of general configuration, each riser is tensioned separately, the flowlines have a cantilever connection at the manifold while the export riser has a ball joint, and any riser can be individually run or retrieved.

The export riser is made up of 45 foot joints of 10 3/4 inch diameter by 1/2 inch wall thickness pipe with threaded connectors between joints. The export riser is rated for low pressure (<300 psi) and carries the support guides for the flowlines.

The flowlines, all identical, are 45 foot joints of pipe, 4 1/2 inch diameter by 17/32 inch wall thickness pipe with threaded connectors. To carry the higher stresses at the built-in ends on the manifold, the lowest joint of each flowline is 6 5/8 inch diameter by 23/32 inch thick. All flowlines are designed for a 4000 psi pressure rating.

Analysis Goals. In the first part of the chapter, the design and analysis of floating production riser systems was discussed in general terms which included a rather comprehensive list of what the goals of a

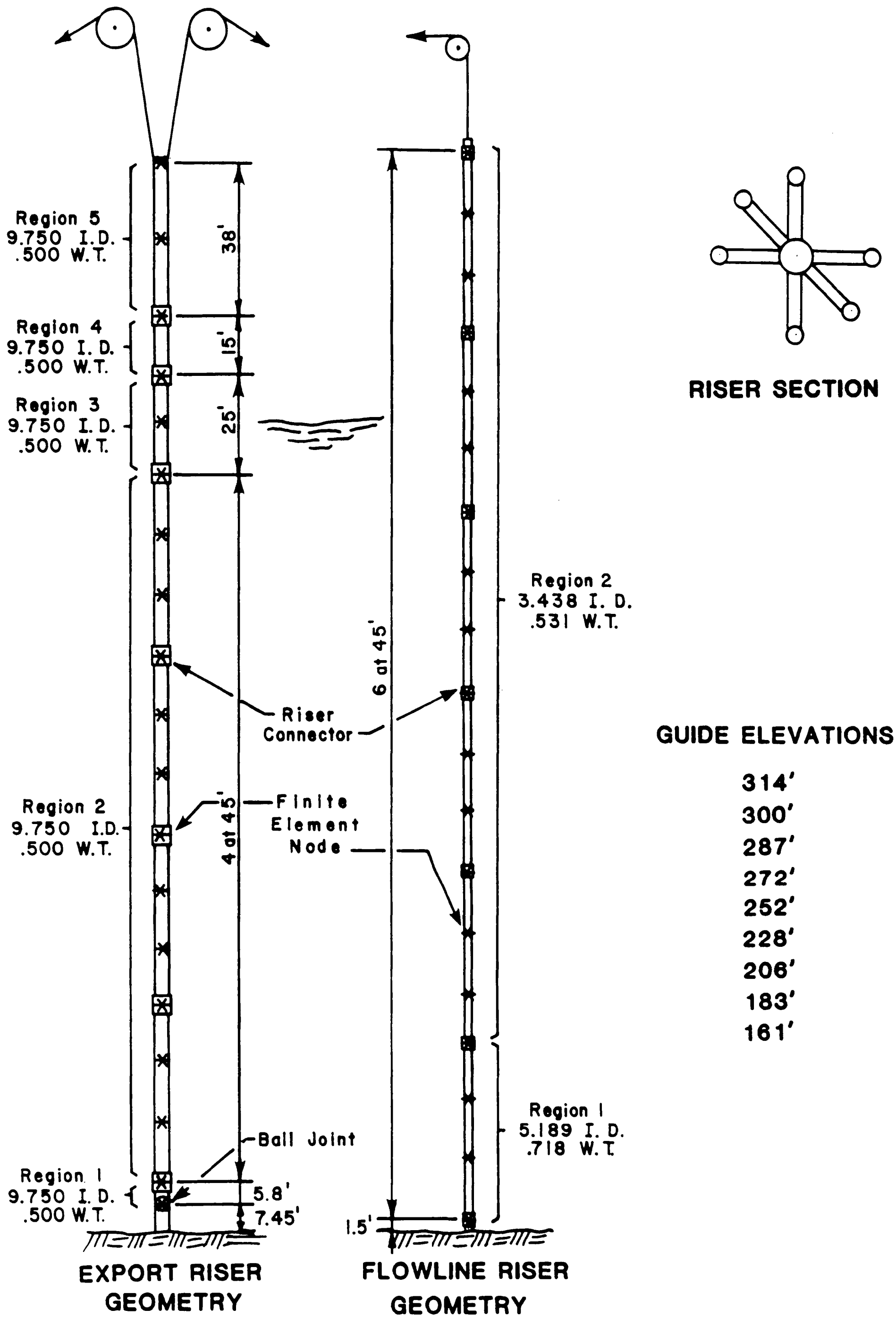


Figure 9. Basic Geometry for the Riser System

particular riser system analysis may be. Relative to the case being discussed in this example, the goals were quite simple:

- Determine a tensioning schedule for the export and flowline risers that optimizes vessel tensioning capacity and riser stress level
- Identify conditions that may cause interference between the riser and the vessel
- Establish operating criteria for conditions that would lead to overstressing of the riser.

Analysis Results. To fulfill these design goals, a sequence of three types of analyses were performed - dynamic analysis of individual risers, equivalent dynamic riser analysis of the bundled system, and multi-tube dynamic analysis to verify the results of the equivalent riser analysis. Within each of these basic categories, a number of parameters were varied - tension, wave height, wave period and wave angle of incidence.

The first sequence of computer runs made were time history dynamic analyses of the export riser by itself and a flowline by itself. The rationale for doing these runs was two-fold: a) It is very likely that the export or flowline risers could be installed without the other (ie. the effect of guides between risers would be missing), and b) Basic optimizing of tensions for each type of riser can be studied at a reasonably low cost with this type of run. As indicated previously, a large number of computer runs were made in which tension, wave period and height, and angle of incidence of the waves were varied. Figures 10 through 15 show the results of some of the computer runs.

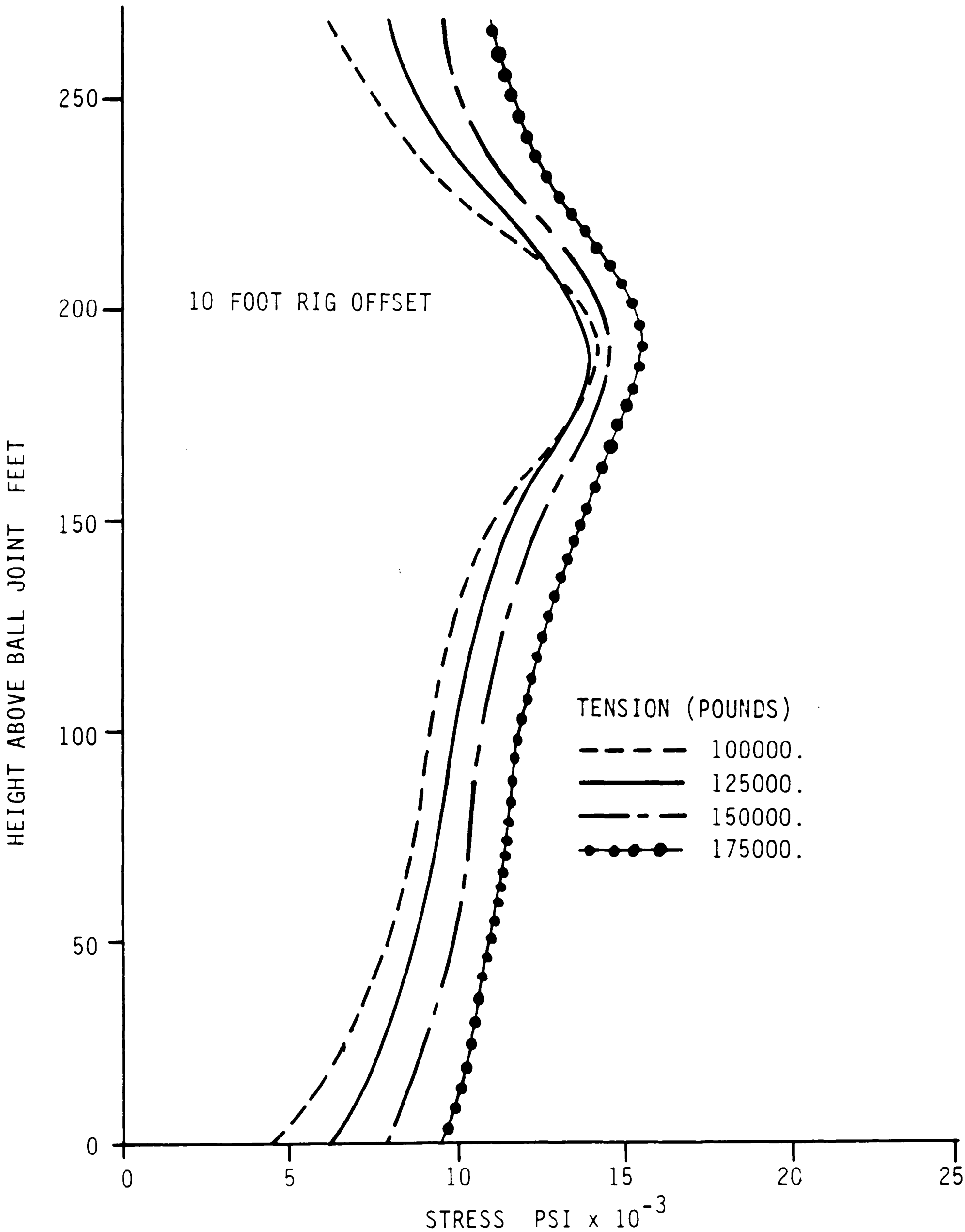


Figure 10. Stress Distribution Along the Export Riser for Various Top Tensions (25 foot waves, 10 second period, beam seas)

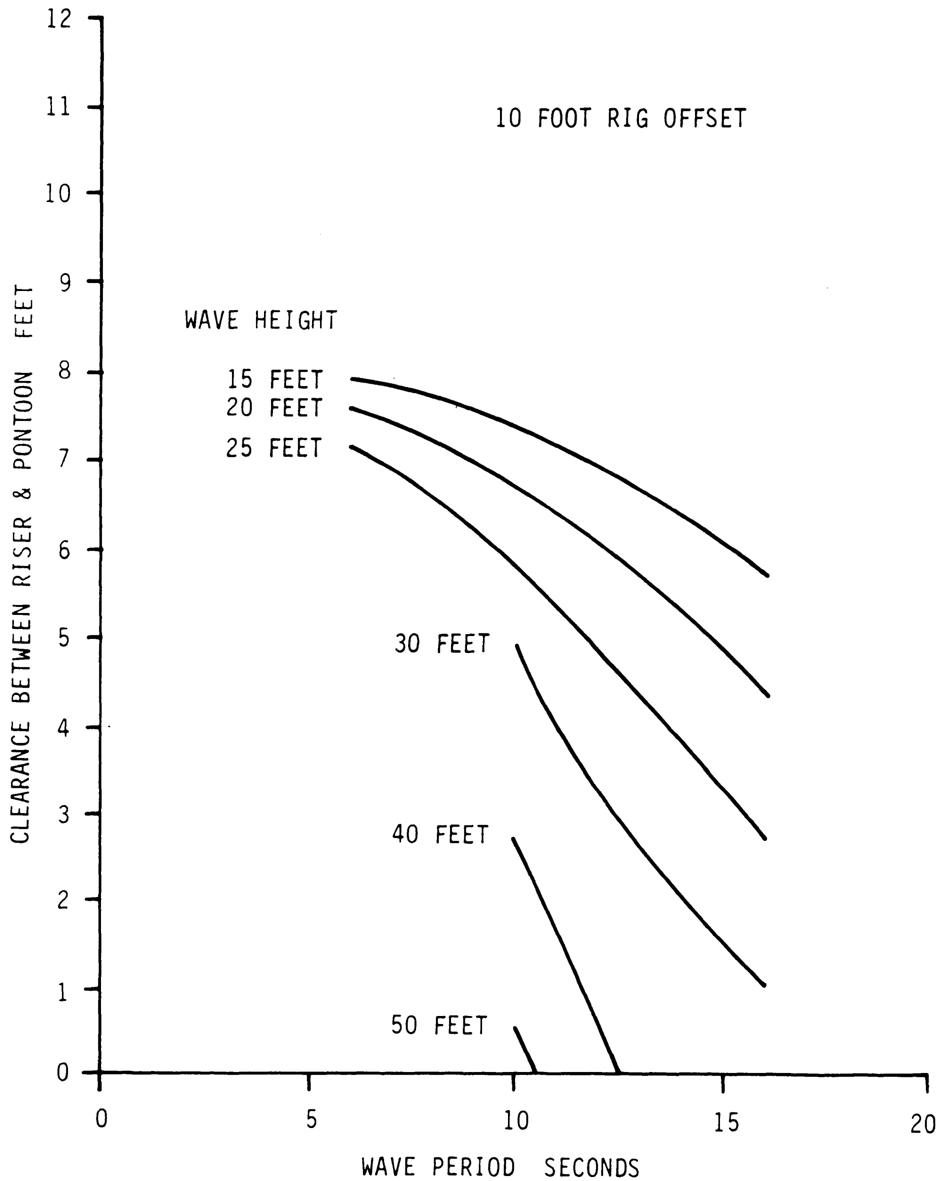


Figure 11. Export Riser Pontoon Clearance
for Various Wave Heights
(100 kip tension, beam seas)

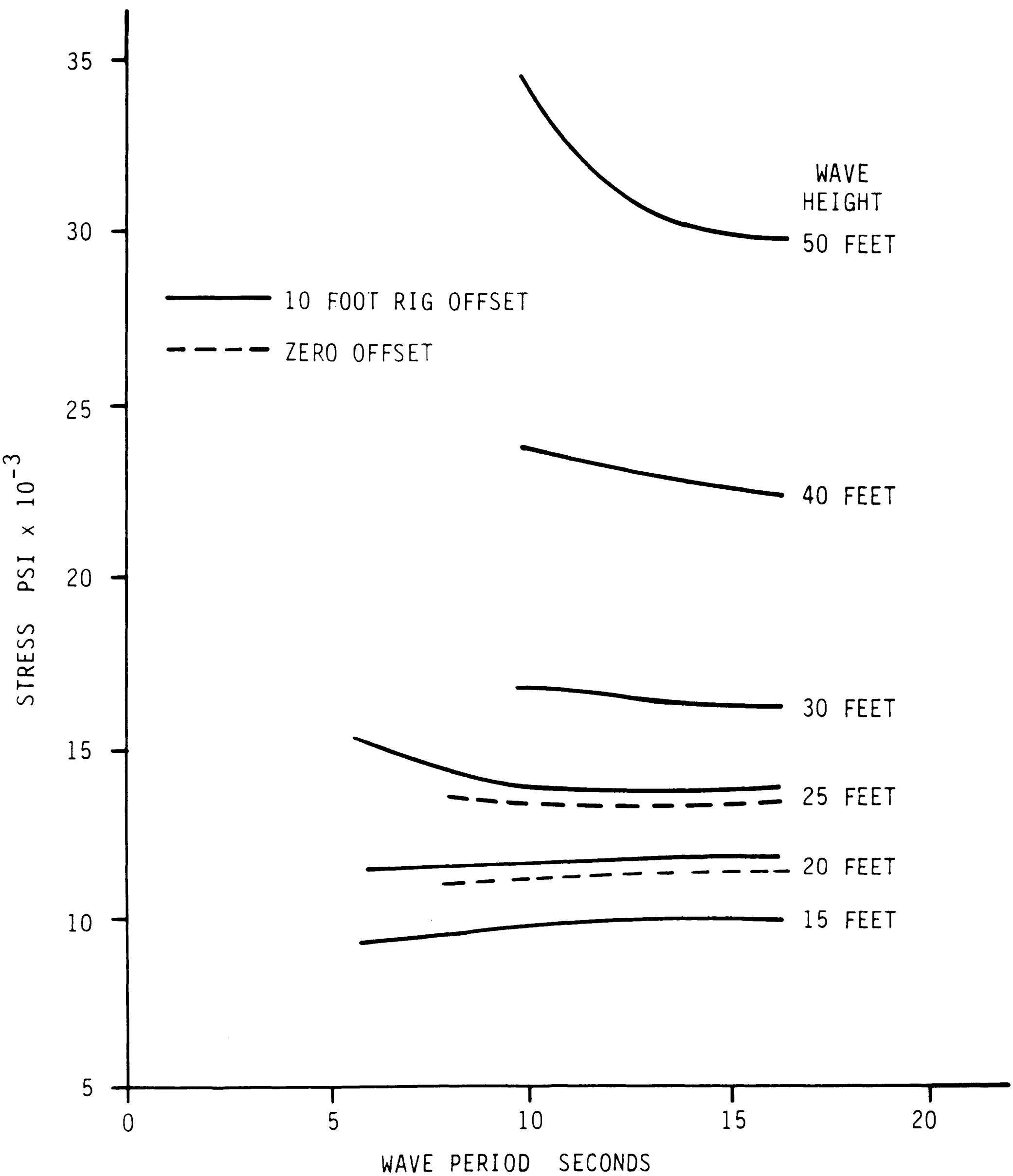


Figure 12. Maximum Export Riser Stress
for Various Waves
(125 kip tension, beam seas)

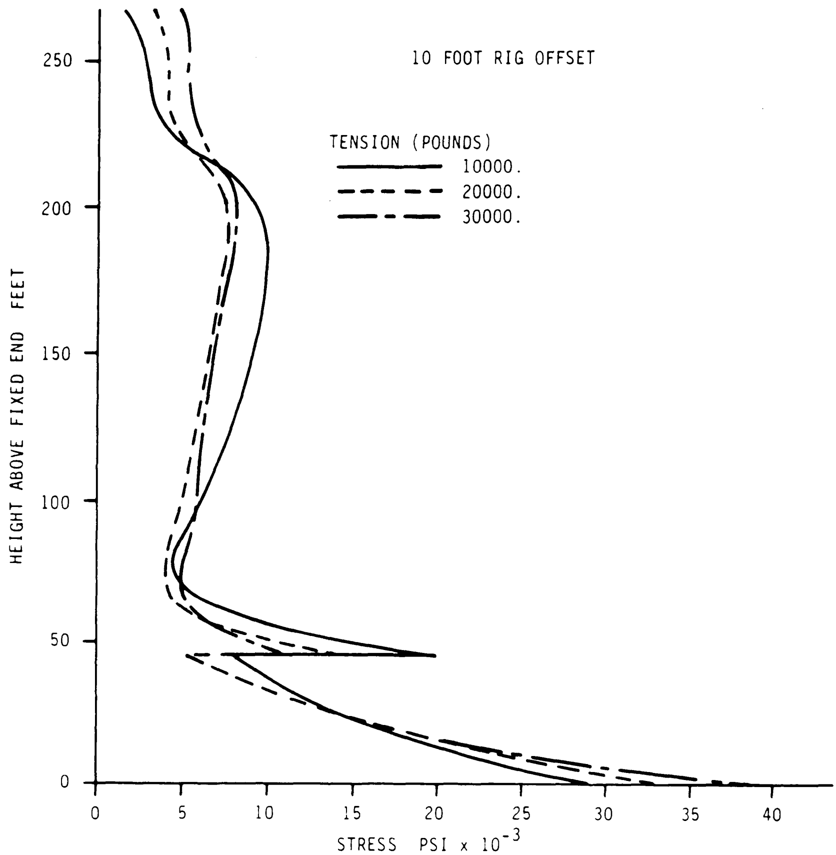


Figure 13. Stress Distribution Along the Flowline Riser for Various Top Tensions (25 foot waves, 16 second period, beam seas)

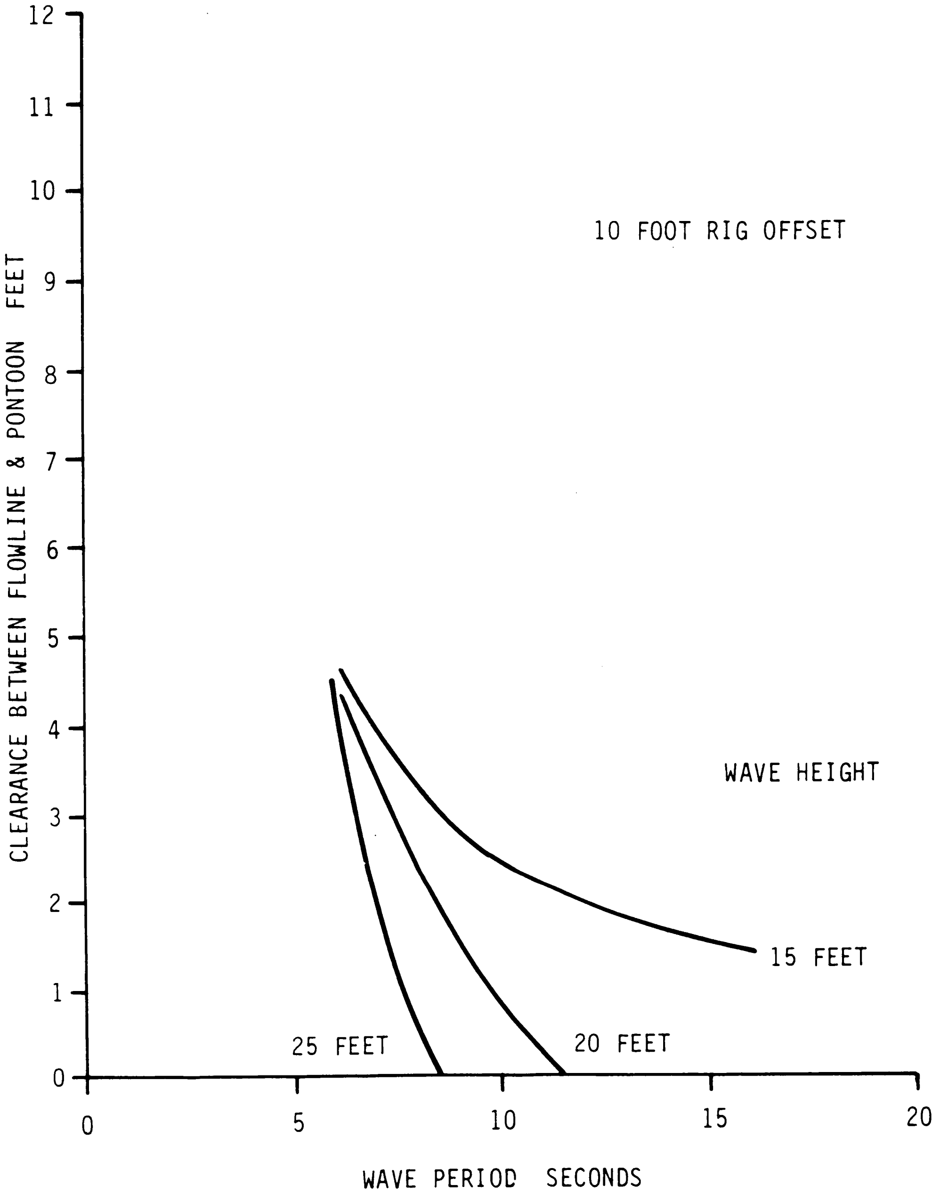


Figure 14. Flowline Risers Pontoon Clearance for Various Wave Heights (10 kip tension, beam seas)

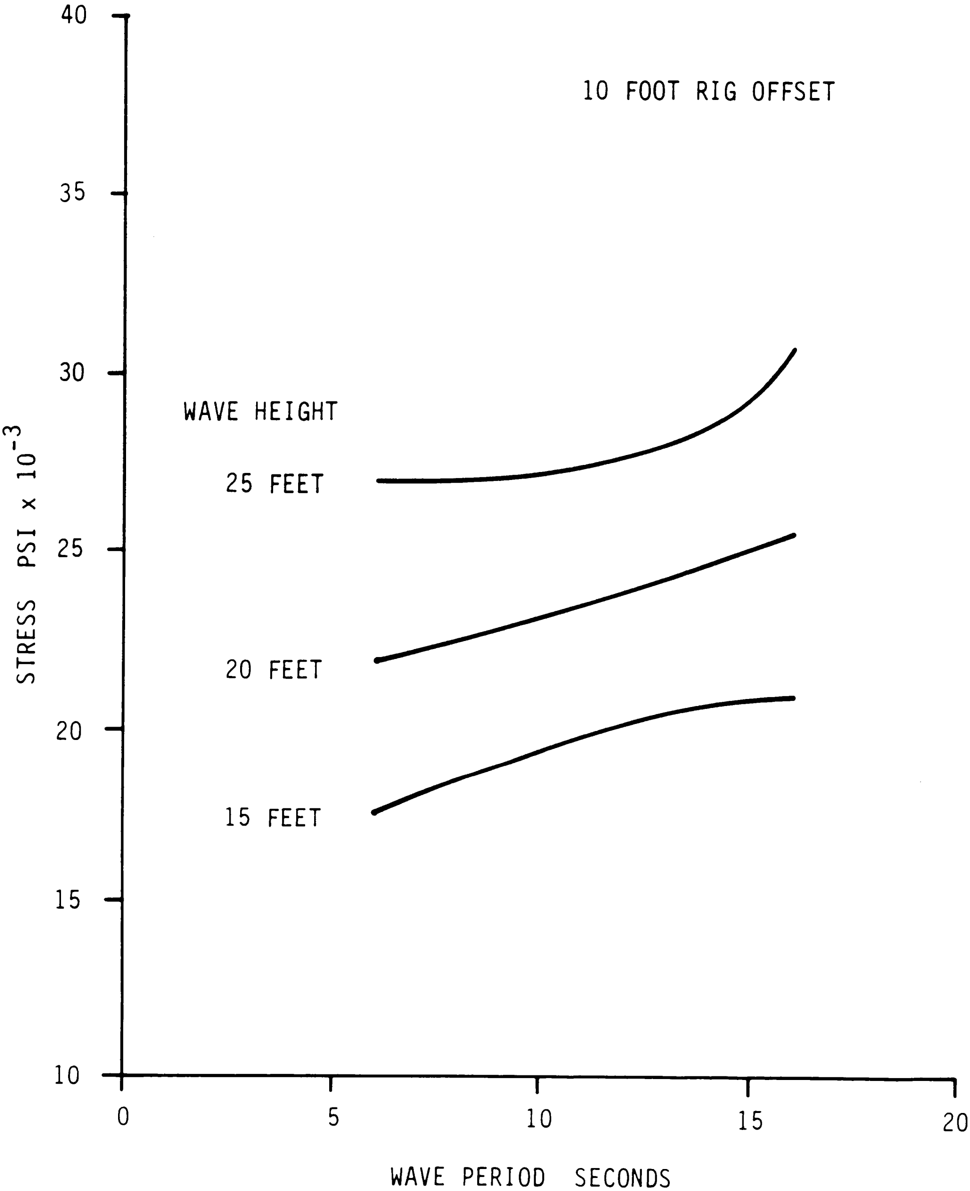


Figure 15. Maximum Flowline Riser Stress
for Various Waves
(10 kip tension, beam seas)

In terms of the analysis goals for the system, the results shown in Figures 10 to 15 illustrate a number of points. Because a riser depends upon tension to achieve lateral stiffness, it is important that top tension be sufficient to prevent excessive lateral motions. Unfortunately there is a trade-off that must be made: High tensions reduce lateral motion and thus bending stress, but cause greater axial stress and place a greater demand on the vessel. Low tensions, on the other hand, are easily accommodated by the vessel tensioners and produce low axial stress, but result in high bending stress. The designers job is to find the optimum tension value - ie. the tension which gives the minimum stress within the capacity of the tensioning system. The second point illustrated by the results shown in Figures 10 to 15, is that one must do a fairly large number of runs to establish operating criteria.

The basic conclusions from the individual riser analysis study, relative to the design goals, were as follows:

- The optimum tension for the individual risers is 125 kips for the export riser and 20 kips for the flowlines
- Interference between the risers and pontoons is likely in 20 to 25 foot seas with wave periods above 8 seconds if flowline tension is allowed to fall below 20 kips
- The riser system must be retrieved (disconnected and pulled up) if seas are expected to be in the range of 50 feet.

Given the basic tensioning schedule and operating constraints developed in the individual riser analyses, a series of time history, dynamic, bundled equivalent riser analyses were made to determine the response of the system when all risers are installed. Figures 16 and 17 show typical results for this type of analysis. In the figures, the discontinuities in stress are the result of the effects of the guide constraints on the system. Although Figures 16 and 17 cannot be directly compared with Figures 10 and 13 because of differences in loading conditions, several general conclusions can be drawn:

- Although the individual riser analyses allow rapid, low cost tension optimization, the behavior of the "system" is substantially different
- The guides dominate the bundled riser results.

In spite of the apparent lack of agreement between the individual riser analyses and the bundled riser equivalent results, subsequent computer runs demonstrated that the tension schedule used, 125 kips for the export riser and 20 kips for the flowline riser, was appropriate.

As a final step in the analysis of the floating production facility riser system, a check was made on the adequacy of using the bundled equivalent technique for modeling the overall behavior of the complete riser system. Figures 18 and 19 show a comparison of the bundled equivalent analysis and a full multi-tube analysis. Obviously, the exact quantitative values of stress are not identical, but the general trends are substantially correct. Because of the difference in cost between the bundled equivalent and multi-tube analyses (1:1.6), the remainder of the system design was carried out using the equivalent analysis technique, with a small number of multi-tube check cases to verify its validity.

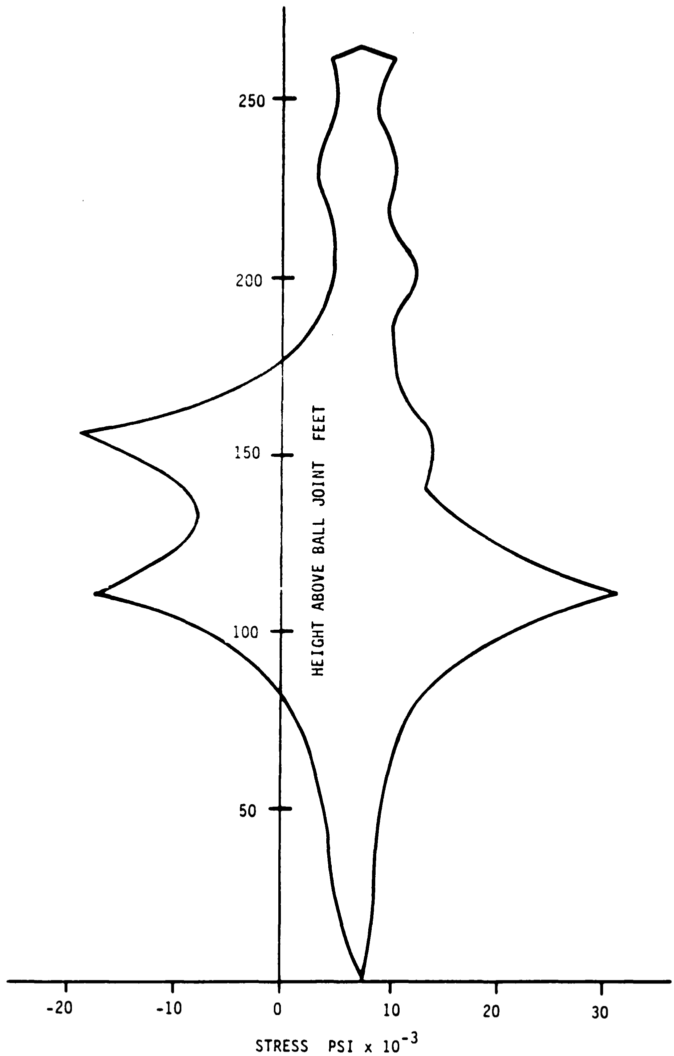


Figure 16. Equivalent Model Export Riser Dynamic Stresses
(25 foot wave, 16 second period, beam seas)

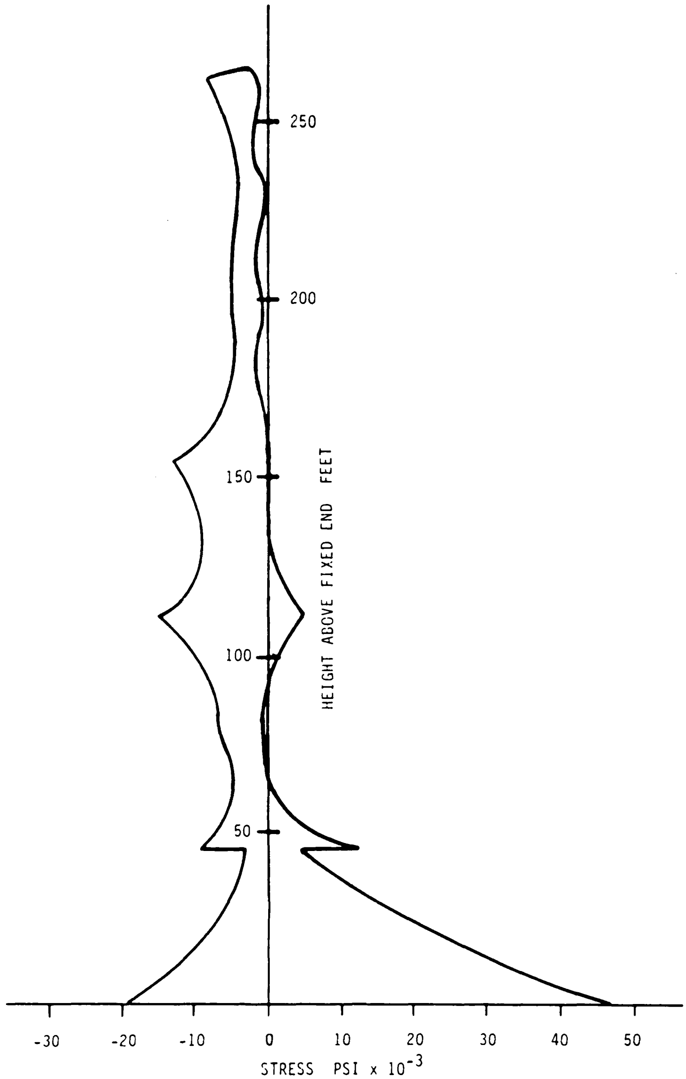


Figure 17. Equivalent Model Flowline Riser Dynamic Stresses
(25 foot wave, 16 second period, beam seas)

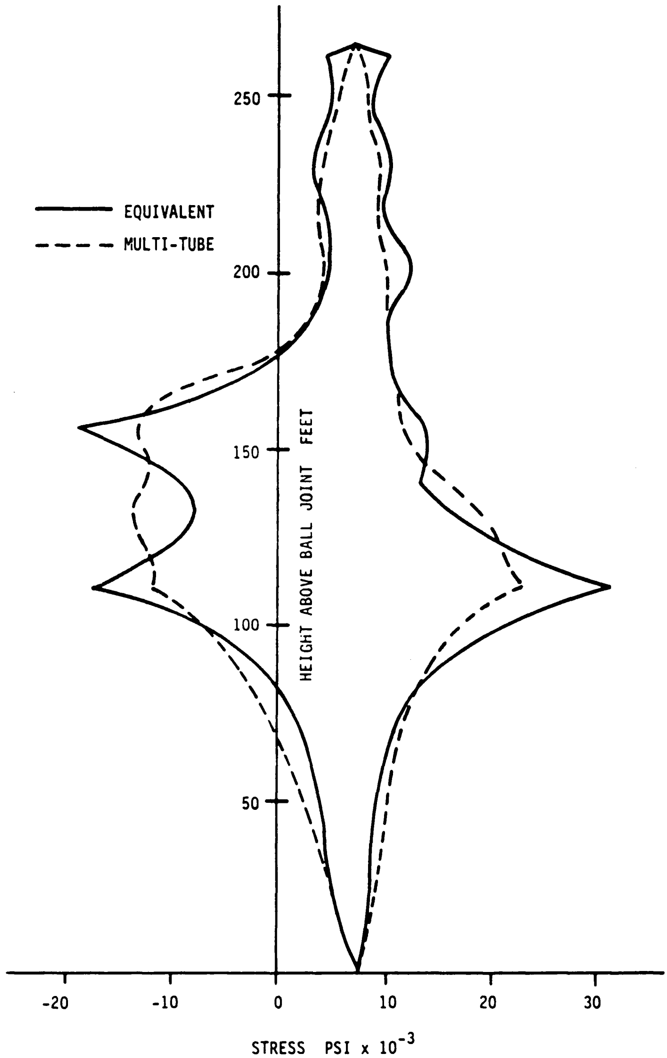


Figure 18. Comparison of Export Riser Dynamic Stresses
(25 foot wave, 16 second period, beam seas)

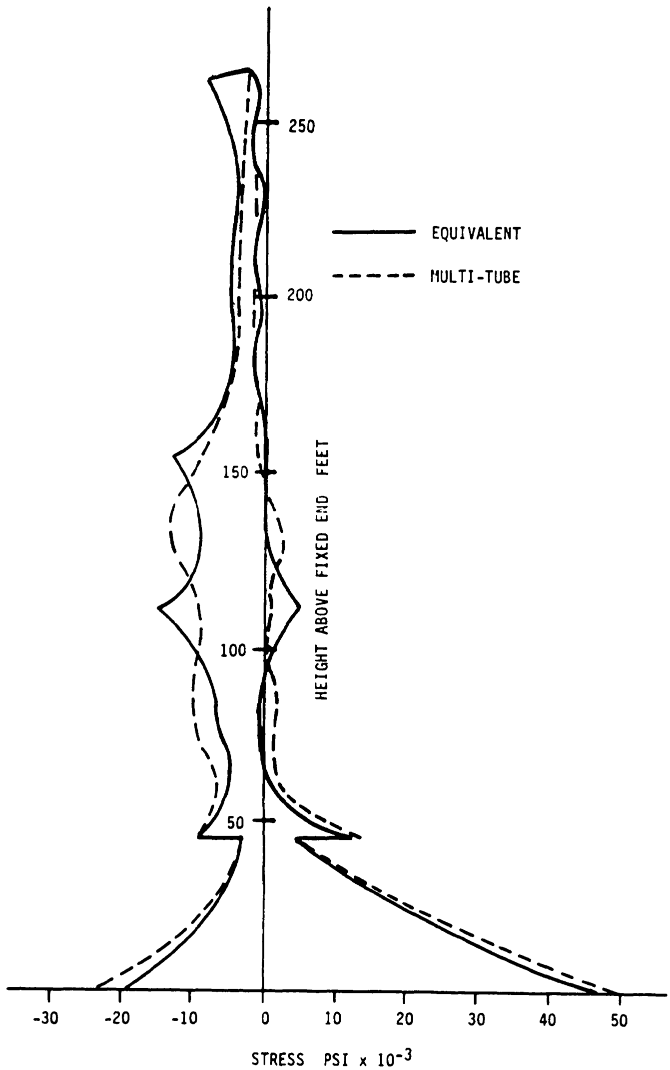


Figure 19. Comparison of Flowline Riser Dynamic Stresses
(25 foot wave, 16 second period, beam seas)

Design Conclusions. As a result of the design analyses conducted on the riser system, a number of recommendations were advanced, relative to the global riser system performance: The use of high strength steel will be required, the location of the guides and ball joint on the export riser could possibly be moved to reduce stresses, and there are weather conditions that dictate that the riser be disconnected and pulled up to avoid damaging it. In spite of the general negative tone of these overall conclusions, however, the system was judged to be an economically viable scheme for producing the reservoir because of the low capital investment and rapid payback.

Although the discussions directly related to the global riser analysis end at this point, relative to the design goals of the complete floating production facility, a whole new activity begins - design of the individual components. From the bending moments, shear and axial forces coming out of the global riser analysis, the connectors between riser joints, the guides, the lower manifold connectors, the tensioner arrangement, all must be designed. Although proper design of these components is important to the success of the design, the critical element in ensuring the integrity of a floating production facility riser system is the global riser analysis.

Conclusions

The design of a marine riser system for a floating production facility can be very complex and costly. Within budget and time constraints, the riser system designer must make decisions as to how to proceed in his analysis and how to make the most effective use of his analysis tools. This chapter has presented some basic information on riser systems for floating production facilities, a brief discussion on the fundamentals of riser analysis, and some typical results of a floating production facility riser system analysis. Because the general topic is very focused, risers, the treatment within the chapter has been deliberately broad, ie. applicable to all floating production riser systems. Although the riser design example did not cover all possible design cases (interference between risers without guides, an extensive fatigue analysis, etc.), it did illustrate some typical results and methods for performing a riser analysis.

If any one, overall conclusion can be drawn from the material presented, it is that analysis of floating production riser systems can be difficult and expensive. To do these analyses one must have preliminary data (environmental, vessel motion, system configuration), good analytical tools, and a good plan for achieving the riser system design objectives. As in all engineering, the designer must balance time and cost considerations with the technical merits of a particular solution, realizing that the basic global riser system design results are the input data for the individual component design and analyses.

Nomenclature

a	Vertical distributed load
A	Area
B_i	Linearization coefficients
C_A	Hydrodynamic added mass coefficient
C_D	Hydrodynamic drag coefficient
C_M	Hydrodynamic inertia coefficient
d	Water depth
D	Outside diameter
E	Modulus of elasticity
F_L	Concentrated lateral force
F_V	Concentrated vertical force
$\{F\}$	Load vector
g	Gravitational acceleration
I	Second moment of area
$[K]$	Stiffness matrix
ℓ	Height of bottom end of riser from sea floor
L	Total riser length
M_b	Bending moment
$[M]$	Mass matrix
m_c	Mass per unit length of riser contents
m_s	Mass per unit length of riser steel
P	Pressure
q	Lateral distributed load
Q	Shear force
t	Time
T	Tension
u	Wave particle velocity
$\dot{u}$	Wave particle acceleration
V	Current velocity
w_s	Weight per unit length of riser
w_c	Concentrated weight
x	Variable of integration
$\{x\}$	Displacement vector
z	Vertical coordinate
$\delta(x)$	Unit step function $= 0, \; x < 0$; $= 1, \; x > 0$
ρ_f	Mass density of riser internal fluid
ρ_w	Mass density of sea water
ξ	Lateral displacement
ω	Wave frequency

Subscripts

e	Equivalent
i	Inner
o	Outer

Superscripts

k	Value at location k
n	Value at location n
top	Value at top of riser

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SOME ASPECTS OF THE TECHNOLOGY RELATING TO SUBMARINE PIPELINE CROSSING OF UNEVEN SEABED AREAS

A. Berti, R. Bruschi and R. Matteelli

Snam Progetti

ABSTRACT

The Transmediterranean Pipeline can be considered the most advanced submarine pipeline project that has been undertaken up to the present date.

The project required the use of the most advanced technologies and equipment available at the time, as well as the development of new technologies whose successful application opened up new horizons for submarine pipeline crossings in deep water and across seabeds with an uneven morphology. //1, 2, 3//

At the start of this paper it is intended to refer briefly to the size of the project and, in particular, to the technological developments arising directly from it.

Subsequently, some of the applicational problems which occurred during the construction phase of the project will be discussed.

In particular:

– The problems encountered in the construction of the winding stretches of the pipeline are described. The winding nature of these stretches was directly related to the particularly uneven seabed and, consequently, it was necessary to pass through narrow corridors that were not aligned with the route of the pipeline.

– The problems encountered in the phase immediately after pipeline installation are described. It was necessary to decide if, when and how to intervene on the free spans which had inevitably formed as a result of crossing uneven seabeds.

1. THE TRANSMEDITERRANEAN PIPELINE: A NEW AGE FOR SUBMARINE PIPELINES IN DEEP WATERS CROSSING UNEVEN SEABEDS

Fig. 1.1 shows the route of the pipelines across the Strait of Messina and the Sicilian Channel and the corresponding altimetric profile. The first pipeline, although not very long, is complicated by the considerable depth, the very strong tidal currents and the irregularities of the seabed. The second section, ten times as long, touches a maximum depth of 608 metres.

The lack of experience, even in worldwide terms, in the fields of design, construction and maintenance of pipelines at depths of over 150 metres, crossing uneven seabed areas, required new techniques to be developed by the Companies involved in the project. //1//

Tab. 1 shows the main data relating to the project, with particular emphasis on the laying bed preparation works and the remedial work performed on the elastic equilibrium configuration assumed by the pipeline on uneven sections of the sea bed.

The importance of the experience acquired in the crossing of uneven sea beds cannot be understated: the technology successfully employed for the Transmediterranean Pipeline opened up new ways of solving the problem of submarine pipeline crossings of uneven areas-such crossings had, up to then, been considered technically and economically unfeasible.

A brief indication will now be given of the vast research programme required to develop the new technologies proposed.

This programme was gradually developed over a period of several years and its main milestones are as follows:

- the design and construction, in 1974, of a test pipeline 10" 3/4 O.D. linking Sicily to Calabria;
- a laying test in the Sicily Channel, in 1976, during which two pipelines, respectively 12" and 16" O.D., 3.2 km. and 3.5 km. long, were installed and reached a maximum depth of 560 m.;
- a series of tests for the analysis of vortex induced oscillations, performed in the Lagoon of Venice between October 1978 and April 1979:a 67 m. long free span of 20" O.D. pipe was positioned in a channel of the Lagoon where the tidal current reached the typical values of the

synchronization between vortex shedding and the elastic response of the free span. //2//

This introduction to the Transmediterranean Pipeline is concluded by summarizing those aspects which underwent major developments and most benefited in the course of such a new and important project.

With respect to deep water submarine pipeline technology, the major developments were obtained in relation to the following:

- survey techniques;
- route selection techniques;
- laying corridor preparation techniques;
- deep water installation techniques;
- techniques for intervention on laid pipes;
- inspection, maintenance and repair techniques.

With respect to analytical modelling for forecasting real pipeline behaviour, in view of the scale and importance of the project, those models previously available were completely revised, particularly in relation to the following:

- collection and analysis of bathymetrical, lithological and meteomarine data for route selection;
- the analysis, for each lay barge available on the market, of the installation phase under various environmental conditions and with geometrical restraints for the route to be followed;
- analysis of the elastic configuration of the pipe resting on the sea bottom.

Similarly, the back-up hardware for the above models progressed in parallel and the latest development is directed towards C.A.D.D. systems.

2. THE PROBLEM OF LAYING IN CURVED ROUTES WHEN APPROACHING PREFIXED PASSAGES ON UNEVEN SEA BED

a) Introduction

The crossing of extensively uneven areas implies that the pipeline has to cross either compulsory passages in narrow natural corridors, or passages made by dredging and/or levelling operations using explosive charges.
These corridors may be found far away from the pipeline alignement and so they must be approached by means of very wide radii bends on the bottom plane.
For this reason the laybarge has to follow a suitable route to ensure that the pipeline follows the required route as closely as possible Fig. 2.1, to enter the narrow corridor with the required tolerance.
Field experience, aquired from the Sicily channel crossing, showed-up the critical nature of this operation and, in particular, the problem of stability for the on-bottom curve configuration of the pipeline.
The friction, caused by the negative buoyancy of the pipeline, may not be sufficient to withstand adequately the tendency of the pipeline to "open" the bend and therefore connection with the passage route in the laying corridor becomes a problem.
An operating situation of this type occurred during the installation of 1st NORTH line in the Sicily Channel; a narrow corridor had to be approached at 5,000 m. radius bend, at one third of the route from CAPE BON to SICILY, in 500 m. water depth.
The fact that it was not possible to make the pipeline follow the expected route, checked during the operating phase and justified a posteriori by an erroneus evaluation of the friction coefficient, resulted in the use of a series of gravel and bitumen "mattresses" (12 mattresses, 7 tonnes each), forming an artificial bearing abutment on which it was intended that the pipe would rest, when tending to open the curve, Fig. 2.2. //1//
This intervention was successful although a subsequent check showed that the pipeline rested on the first stoppers only, thus leaving a

significant part of the abutment inactive.
Similar intervention works, together with the laying equipment on stand-by, have highly significant economic implications, especially in short crossings. Consequently, it is important to have an adequate model for forecasting pipeline bend stability in order to allow the following:

- the contractor can intervene in good time and modify the lay barge route according to the mcvements of the touch down point;
- the designer can intervene, if the tensioning device on the lay barge so allows, by using gunite coated pipe sections to increase the stabilizing action of friction on the bends;
- the designer can try to estimate adequately the construction of the artificial abutment in order to avoid the very expensive stand-by of laying equipment.

b) Formulation of the numerical model

A preliminary check on pipeline bend stability can be performed according to the scheme in Fig. 2.3, where:

- homogeneous soil is assumed and the interaction between the soil and pipe is modelled according to the "Coulomb" model;
- a displacement, assigned to a pipeline length, is imposed;
- a check is performed to verify if the friction succeds in maintaining the displaced configuration.

As a marker of facts; the first pratical problem consists in cheeking the stability of the pipeline after being laid on a prefixed route. In this case the pipe configuration is known and therefore the equilibrium between the external and internal actions can easily be expressed. Should the designer need to simulate the behaviour of the pipe when moving from its original position, the problem comes more complicated. In this case it is necessary to take into account the actual configuration of the pipe when being laid. The problem //6, 7// must therefore be modelled and solved numerically.
Going back to the modelling concept, the first problem that must be solved, concerns the choice of the route the lay barge must follow once the laying configuration is established (laying ramp geometry and pulling at the tensioning device), so that the pipeline follows the designed route, previously checked from the stability point of view, as closely as possible.
The aforementioned aspects need the development of a model which simulates the laying phase so that single laying operations can be followed step, by step and the lay barge movements can be identified sc as to mantain the touch down point of the pipeline according to the designed route and oriented so that the line is tangential to the curve. The simulation can be split into two phases, (Fig. 2.4), as follows:

- an external incremental procedure where the curved-route profile is divided into steps corresponding to each single laying step.
The direction of lay barge movement is calculated using an iterative method to ensure that the pipeline follows the designed curvilinear route;
- an internal iterative procedure, which pursues the equilibrium configuration achieved by the pipeline under the action of lateral forces induced by the movements of the lay barge.

In practice, the on-bottom pulling force acting on the pipeline, residual from the laying operation in "S" mode, has a component normal to the pipe axis, the one bending the pipe, because of the "Z" movements of the lay barge on the sea water surface.

The moment effect due to pipeline rotation, imposed by the lay barge, is neglected because of the high flexibility in the pipeline suspended span during laying.

The internal iterative procedure is based on:
- a finite element model,in which the stiffness matrix is expressed using the analytical solution of an inflected beam under a pulling action which tends to stiffen the system (see Sect. 2.b.);
- a friction model, COULOMB type, in which the frictional reactions are concentrated on the nodes which separate the finite segments and furthermore, have an opposite direction to node displacement on the seabed plane.

Fig. 2.5 summarizes the iterative calculation flow which defines the equilibrium configuration.

The proposed technique to define the frictional reaction, (i.e. the "SECANT" Method, commonly used for the analysis of foundation piles under lateral loads) speeds up the convergence of the iterative process remarkably.

Obviously the proposed model has some limits namely:
- in the description of the real situation, in proximity to the touch-down point, where the following is neglected:
. dynamical effects;
. 3-dimentional effects; in particular the formation, in the vertical plane, of "flexural" waves typical of a beam on an elastic soil.
- in the assumption of a friction coefficient that has to be univocally accepted.

However, field experience has permitted positive validation of the instruments used for the simulation, in particular:
- when designing a curvilinear pipeline route, it is possible to gain a sensitivity on the most characteristic parameters, which either identify the tolerances for safe maneouvring of the lay barge during installation, or ponder the need of a visual monitoring of the operations, using

the R.C.V. system;
- when checking on the pipeline's curvilinear route, it is possible to individuate the stability margin of the configuration, considering the subsequent hydrostatic test and start-up phases as well.

3. CROSSING UNEVEN SEABED AREAS

a) Introduction

The equilibrium configuration //4, 5// of a pipeline laid on uneven seabed generally consists of a sequence of free spans, separated by different lengths of pipeline sections where the pipe rests on the seabed, Fig. 3.1.
The discontinuity in support points induces bending stresses on the pipeline which may be unacceptable when static and fatigue strengths are considered.
In particular, sea-bottom unevenness has to be within limits which permit the pipeline to be laid (in empty condition) without exceeding the allowable bending stress (note, that when necessary the seabed shall be prepared considering the above).
When the pipeline has been laid, the following must be considered:
- the equilibrium configuration may not be acceptable considering the static condition if, during the hydrostatic test/commissioning phase of the pipeline, the weight of the internal fluid plus the test pressure, stresses the support points of the pipeline with bending moments which are beyond the allowable;
- the equilibrium configuration may not be acceptable from a dynamic point of view, if bottom currents acting across the pipeline are such as to induce hydro-elastic resonant oscillations (synchronization of vortex shedding on one of the natural frequencies of the elastic configuration) of such an amplitude so as to jeopardize the fatigue life of the pipeline.

The field experience acquired during the numerous kilometers crossed underwater in areas that were morphologically difficult, permitted the acquisition of a particular technology consisting of remedial works for the critical situations mentioned above.
The remedial works include:
- levelling of uneven seabed along the pipeline route, using explosive charges to eliminate peaks and/or filling-in depressions with excavated material;

- intervention works to reduce free spans, which occur on the equilibrium configuration after laying, using artificial supports.

Two peculiar aspects of the technology mentioned, are pointed out hereunder:

- on one hand the opportunity to carry out intervention works to reduce free spans, considering the static strength of the pipeline, is easily definable, whilst on the other, several dcubts persist when defining the critical length over which it is necessary to reduce the free spans, when the fatigue life of the pipeline is considered. The above is either due to the often aleatory nature of vortex shedding synchronization on one of the natural frequencies of the free span, or due to the often unidentifiable dependence of the parameters which define the fluid-dynamic field on the fluid-dynamic field itself, in which the phenomenon develops.
- the remedial works, consisting of seabed preparation and reduction of free span, are very costly. Unfortunately, this is typical of all sea works which require mobilization of particular equipment and the use of sophisticated technology. The above also depends on the limited availability of this equipment.

On one hand a real-time analysis (analysis carried-out at the same time as the construction phase) of the critical situation makes it possible to take a prompt decision regarding the need, quality, planning and entity of the remedial work which have to be carried out; and on the other it makes it possible to plan availability and mobilization of particular equipment necessary for this type of operation, as well as to programme in due time the supply of materials needed for the intervention.

Therefore, it is very important to have analytical instruments which help the designer to forecast equilibrium configuration of the pipeline and the level of stress on the pipeline when laid on uneven seabed.

The formulation of the mathematical model by which the pipeline behaviour on an uneven seabed is simulated, is now introduced.

Two particular aspects concerning pipeline sizing are analyzed:

- the pipeline stability under pressure according to the model for EULER stability;
- the dynamic analysis of the self-induced oscillations, due to vortex shedding synchronization on the natural frequencies of a free span.

b) <u>The pipeline considered as a beam under large displacements; a finite element model.</u>

The sketch in Fig. 3.2 shows the kinematics //4, 5// of a deformed beam in 3-D, and it also shows the equilibrium relationship for the deformed O'A' element as a free body. The terms which couple the equilibrium

equations, usually negligeble in small displacement analysis, have been high-lighted.
The sketch in Fig. 3.3 illustrates a beam bent in the vertical plane; this 2-D model can be used to represent the equilibrium configuration of a pipeline resting on uneven sea-bottom. The elastic coupling between bending and axial behaviour of the beam, high-lighted through the underlined terms, gives rise to, in case of longer free spans occurring on a pipeline resting on uneven seabed, a significant contribution of the axial to the bending strength of the beam against lateral loads.
When long free spans have to be analyzed, techniques based on finite element model of the beam shall be used. In Fig. 3.4 the pipe finite element is shown.
In particular as far as the stiffness matrix is concerned, //6, 7, 8// two alternative solutions are proposed in order to consider the effect of the geometric stiffness of the element on the equilibrium of the pipeline under large displacements:
- the analytical alternative, resulting in a single stiffness matrix, distinguished on the basis of the tension (hyperbolic functions) or compression (circular functions), acting at the ends of the element, whose terms are obtained from the analytical solution of the differential equation of the beam under axial loads;
- the approximate alternative, resulting in a stiffness matrix, separated in two parts, the former relevant to the bending behaviour under small displacements (the elastic stiffness), the latter relevant to the axial coupling which arises under large displacements (the geometric stiffness).

Usually it is preferred to utilize the first alternative when the axial constraints do not cause axial-bending coupling such as to modify the tension/compression of the element.
However, the second alternative is preferred in cases in which the axial-bending coupling has to be analyzed, and furthermore to verify the contribution of the "catenary effect" on the bending strength of the beam.
Main aspects which make the problem extremely non-linear are pointed out below, without entering into detailed analysis:
- the equilibrium has to be imposed on the displaced configuration, which is unknown before (geometric non-linearity; large displacements);
- the bottom restraint is unilateral;
- axial movements are hindered by friction;
- the search for the equilibrium configuration is, therefore, an iterative procedure, the convergence of which is extremely dependent on the initial solution.

c) The Euler stability of a pressurized pipeline in free span configuration

The sketch in Fig. 3.5, even if semplified, is significant to introduce the buckling problem for a pressurized pipeline where the internal pressure is greater than the external.
When the pipe is considered "ideal" in the sense of perfect geometry and elasticity, it remains rectilinear under progressive pressurization (the bending contribution due to the weight being neglected).
The equilibrium configuration is considered stable, when a little transient bending action can be absorbed in such a manner that the pipeline, at the end of this action, attains by oscillating the original equilibrium position. There is, however, a critical pressure, called the "BUCKLING PRESSURE", over which the rectilinear configuration is no longer stable. The pipe, in conseguence of any bending disturbance, tends to assume one of the possible deflected configurations according to the boundary conditions and to the mode of the disturbance.
In the proposed scheme of buckling, the pipe under pressure differs from the compressed/deflected beam, due to the absence of compression stresses in the pipe wall.
The proposed model, for buckling of pipeline in a free span configuration, is not suitable to simulate a post-critical behaviour, in the elastic field, for the pressurized pipeline.
The sketch in Fig. 3.6 is more realistic than the previous one as it considers some very important components which contribute to the equilibrium of the elastic system under study.
Firstly the basic factors which help to increase the theoretical value of the buckling pressure are:
- the residual pulling, due to the laying operations, typical of "S" mode laying carried out by means of a stinger and tensioning device;
- the tensioning force resulting from the longitudinal contraction due to POISSON effect, restrained by friction.
Secondly, the basic factor which helps to guarantee a post-critical behaviour, in the elastic field, for the pressurized pipeline is the axial reaction which arises due to the bending deformation of the pipeline. It tends to increase the stiffness of the elastic configuration under lateral loads by means of a "catenary" action.
The model shown in Fig. 3.6 allows to roughly estimate the buckling pressure, where the degree of approximation obviously depends on the boundary conditions both for bending and axial behaviours.
The numerical model presented in para 3.b has a particular importance, since the knowledge of the real stretching level of a free span has sensible implications even in the hydro-elastic behaviour of the pipeline in free span configuration and therefore, for the necessary

remedial works to be carried out in order to guarantee the pipeline structural safety.

d) The forecast of the amplitude of resonant oscillations, when the vortex shedding synchronizes on the natural frequencies of the elastic configuration //14, 15//

As previously mentioned, there is a real difficulty in systematicly resolving this problem; however, the designer must have an adequate instrument in order to foresee the dynamic behaviour of a pipeline free span under the action of a steady transversal currents (the vortex shedding associated with the surface waves, are neglected in deep water conditions).
The above is mandatory in order to decide when and how to intervene on the pipeline free spans occurring on uneven seabed.
This kind of problem is usually solved using, Fig. 3.7:
- a structural model, to determine the natural frequencies and the modal shapes of the configuration in the elastic equilibrium configuration under analysis;
- a fluid-dynamic model, to determine the forces transmitted to the pipeline by the incoming current and by the consequent vortex shedding;
- a damage model to determine the pipeline's safe-life when a sequence of loads, foreseen by the designer, are applied to the pipeline.

d1) The structural model: //9, 10, 11//

Paragraph 3.b has exhaustively introduced the problems concerning the characterization of the equilibrium for a pipeline resting on uneven seabed. In particular, the importance of the catenary effect to resist lateral loads acting on the pipeline under pressure.
Fig. 3.8. shows an example for evaluating the natural frequencies of the free span, using an approximate model to investigate the axial constraints versus the pipeline axial movement, when the weight of the internal flow and pressure tend to deflect the line.

d2) The fluid dynamic model:

This is the most debated model and therefore requires a more detailed analysis than the one carried out for the previous point.
The sketch in Fig. 3.9 tryes to explain interaction between a cylinder and regular vortex shedding.
Briefly, there are two current velocity ranges in which the synchronization of the vortex shedding on the natural frequencies of the elastic body occurs. In the case under analysis, a pipeline free span, the self-induced resonant oscillations have such amplitude so as to

dangerously overstress the pipeline.
In the first range the synchronisation is in-line with the current and therefore parallel to the seabed.
In the second range the synchronization occurs perpendicularly to the current and therefore on a vertical plane (the bending plane).
The most debated aspect, from the design point of view, is the forecast of the amplitude of the oscillations, for which at this moment, no incontestable model is available.
In spite of numerous laboratory researches carried out in the past on this subject and proposed to date by various institutes of research, it is our opinion, also confirmed by field experiences, that only an instrumentation of a series of free spans on a laid pipeline can give significant information concerning the real implication that the phenomenon could give to the design of a submarine pipeline resting on uneven seabed. To this end, it is necessary to mention two research projects in which the test set-up approached very closely to the real situation:
- the ones //13// carried out by CIRIA-UEG at IMMINGHAM, whose results are taken as a reference by the international ruling bodies;
- the ones //2//carried out by SNAMPROGETTI in the VENICE LAGOON, whose results have been utilized for designing the TRANSMEDITERRANEAN PIPELINE.
However, it is important to point out that the self-induced oscillations of a pipeline free span in sea water are more critical than in air.
In Fig. 3.10. and 3.11. the two procedures //12, 14, 15// which are usually followed to foresee the amplitude of resonant oscillations for a free span under in-line/cross-flow lockin, are shown.
The first figure is based on CIRIA-UEG'S proposal, considering the results obtained from the tests performed at IMMINGHAN.
This procedure, suggested by D.N.V., is commonly used by the engineering companies involved in the design of submarine pipelines, and is usually required by clients.
However, from our experience, it has been noted that certain assumptions proposed, are too conservatives; especially as regards the forecast of the amplitude of oscillations, assumed dependent only on the stability parameter.
The second figure concerns the forecast of the cross-flow oscillations as a result from an analytical-heuristic model, the wake oscillator model. This model was introduced in the sixties, and improved by several researchers in the seventies; they tried to analytically simulate the self-induced oscillations in a condition of cross-flow synchronization.
Fig. 3.12 tries to explain the significance of the analytical-heuristic model; in particular, it must be pointed out that experimental tests

have to be carried out in order to determine the heuristic parameters which link the experimental findings and the theorethical solution.

d3) The damage model //3, 17//

Cyclic actions due to hydro-dynamic loads damage the pipe expecially where both geometry and welding have produced dangerous stress concentrations.
The elastic energy accumulated in these particular areas of the material, can be released, sometimes, increasing the defect size.
The above tends to propagate up to values for which the residual strength is no longer capable of with standing external loads and, consequently, the defect propagates with an unstable trend until the structure collapses.
It is now usual to perform a durability analysis on submarine pipelines using fracture mechanics, for which the flow diagram is shown in Fig. 3.13.
From points d.1, d.2, and d.3, it results that it is necessary in design activities to define, for each route section, the critical length of the free span over which it is necessary to intervene to reduce the span length (by means of artificial supports and/or by using a filling-in technique).
The above is actually proposed by the most advanced Engineering Companies operating in the submarine pipeline field.

CONCLUSIONS

The Transmediterranean Pipeline project represented a huge step forward for deep water pipeline technology. Not all of the problems have been completely sòlved, but the way is clear up to depths of 1000 m.
The technology is currently pushing ahead in two directions:
- pipeline installation in very deep water, > 1000 m, with various installation techniques (the"J" mode);
- the design and construction of submarine pipelines in arctic seas.

There are however some aspects of this technology and these pipelines, that can now be referred to as conventional, which still require special attention; particularly the development of efficient equipment for inspection, maintenance and repair.
It should not be forgotten that the feasibility of a project is also linked to the possibility of being able to detect and intervene in the case of accidents that had not been anticipated. Furthermore, the seabed holds a certain number of unknowns that can be solved by deepening our knowledge of certain phenomena, but there must be a margin for intervention should something unexpected occur.

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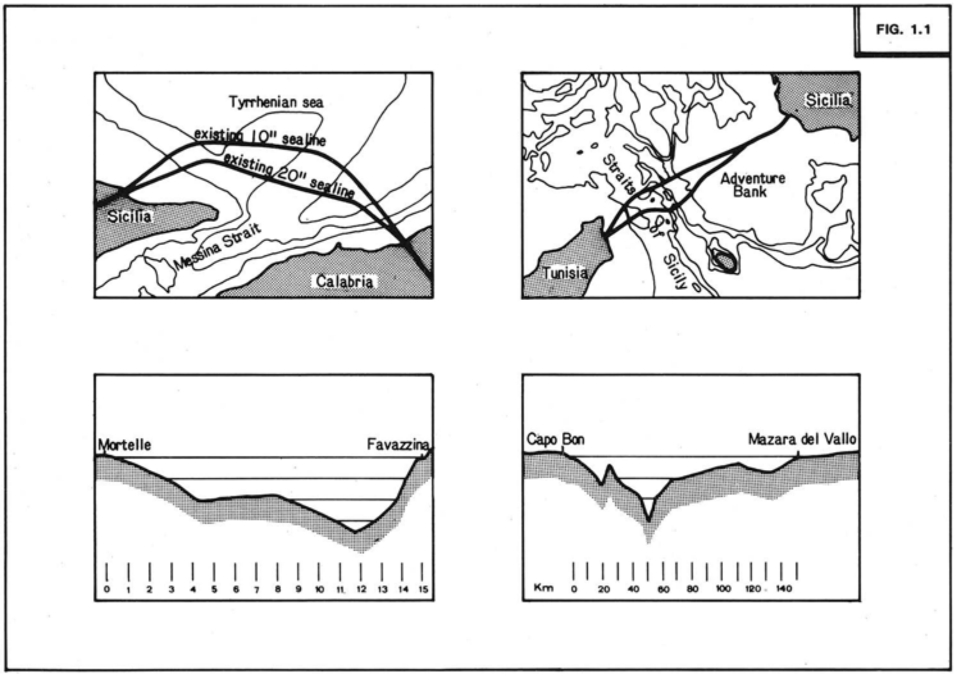


FIG. 1.1

TAB. 1

TRANSMEDITERRANEAN PIPELINE

MAIN DATA	SICILY CHANNEL			STRAIT OF MESSINA		
	1st SOUTH	2nd SOUTH	1st NORTH	1st	2nd	3rd
LENGTH (M)	154812	154868	154515	14028	13960	13896
MAXIMUM DEPTH (M)	610	610	580	370	330	330
N° OF PIPE JOINTS	12684	12812	12318	1126	1143	1140
CONCRETE COATING (M)	122000	122000	120000	1920	13960	13896
N° OF BUCKLE ARRESTORS	126	127	226	-	-	-
N° OF ANODES	1272	1280	1282	289	124	124
SEABED LEVELLING (M)	2500	2500	3000	-	-	-
GRAVEL FILLING-IN (Tonnes)	18000	9600	21500	10125	6750	10125
N° OF ARTIFICIAL SUPPORTS	36	39	26	20	13	16
N° OF MATTRESSES	119	93	115	13	22	31
MATERIAL	API 5LX X65			API 5LX X65 (X52)		
O.D.(inches) W.T. (mm)	20"/20.62 (19.05)			20"/20.62 (17.48,23.83)		
TEMPERATURE (°C)	60			55		
MAX.OPER.PRESS. (bar)	207			73.44		

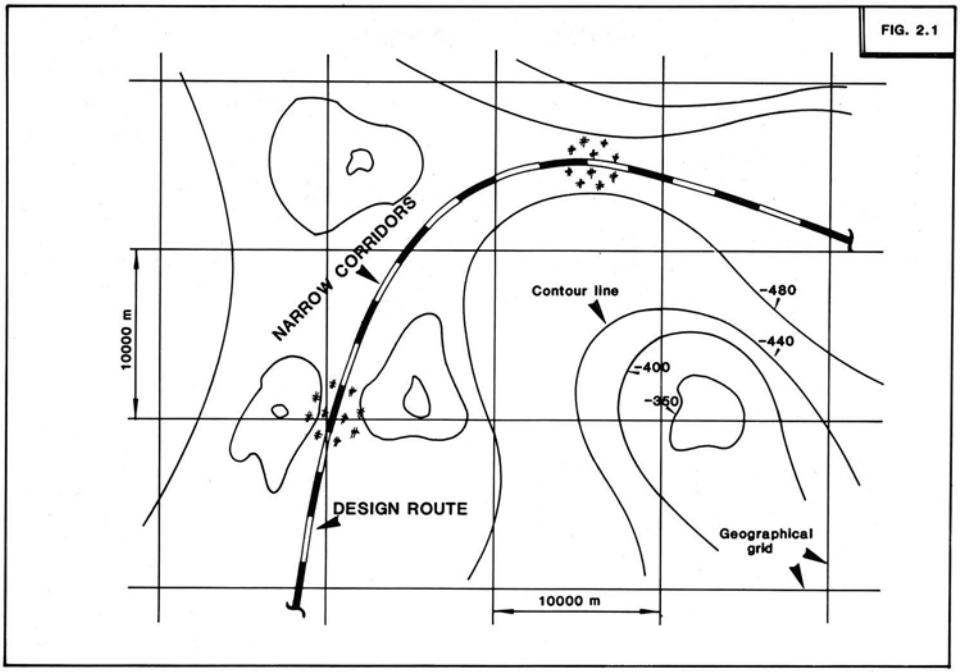


FIG. 2.1
NARROW CORRIDORS
DESIGN ROUTE
Contour line
-480
-440
-400
-350
10000 m
10000 m
Geographical
grid

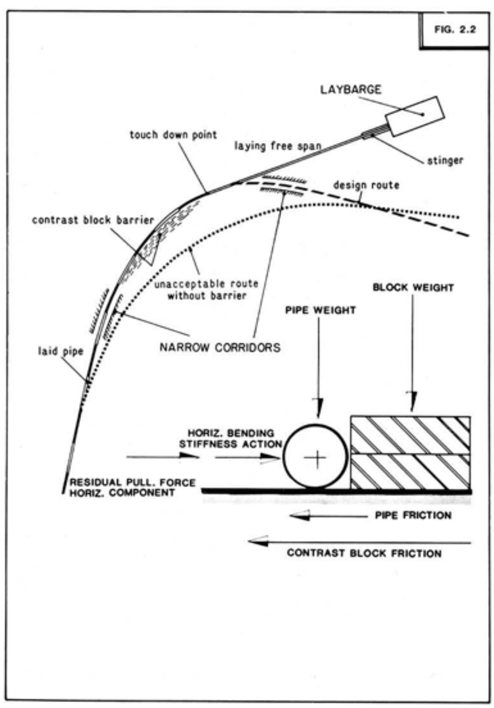


FIG. 2.2
LAYBARGE
touch down point
laying free span
stinger
design route
contrast block barrier
unacceptable route
without barrier
BLOCK WEIGHT
PIPE WEIGHT
laid pipe
NARROW CORRIDORS
HORIZ. BENDING
STIFFNESS ACTION
RESIDUAL PULL. FORCE
HORIZ. COMPONENT
PIPE FRICTION
CONTRAST BLOCK FRICTION

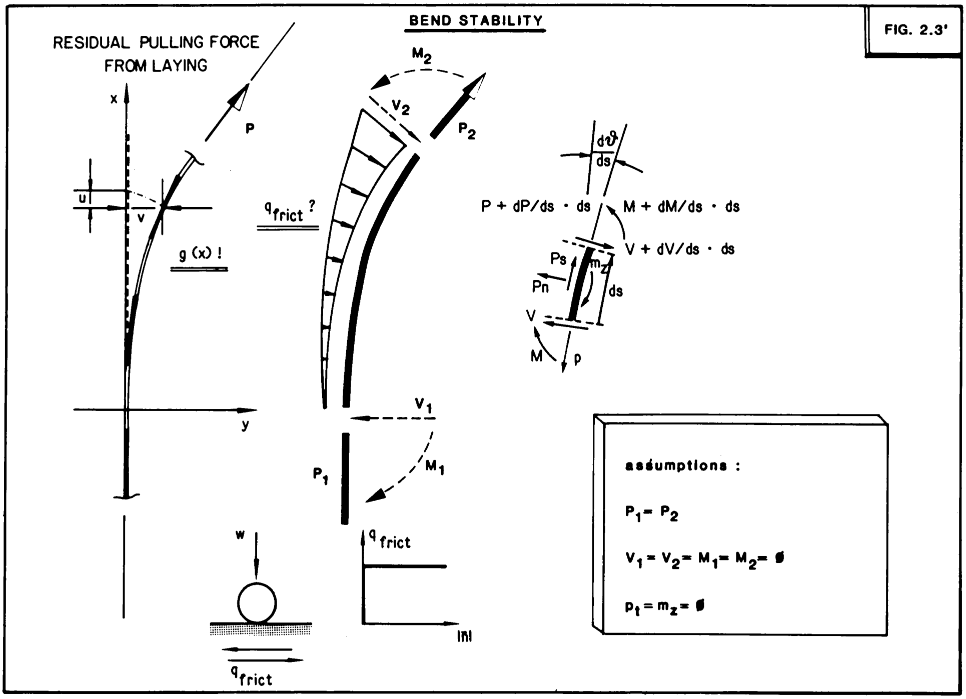


FIG. 2.3''

1. EQUILIBRIUM EQUATIONS

$$\text{transl} \,.\, \bar{s} : dP/ds + p_s = \emptyset$$

$$\text{transl} \,.\, \bar{n} : - dV/ds + p_n + P d\theta/ds = \emptyset$$

$$\text{rotat} \,.\, \bar{z} : dM/ds - V + m_z = \emptyset$$

$$(p_s = m_z = \emptyset;\ P = \text{const.})$$

2. CURVATURE vs. LATERAL DISPLACEMENT

$$1/R = d\theta : ds = d^2 v/ds^2 / \left[1 + (dv/ds)^2 \right]^{3/2}$$

3. CURVATURE vs. BENDING MOMENT

$$1/R = M/EJ$$

4. THE DIFFERENTIAL EQUATION OF BEAMS

$$EJ\, d^2 (1/R)/ds^2 - P (1/R) = - p_m$$

(internal reaction) (external action)

BEND STABILITY

$$|p_n| < \mu \cdot w \cdot \bar{n} / |\bar{n}|$$

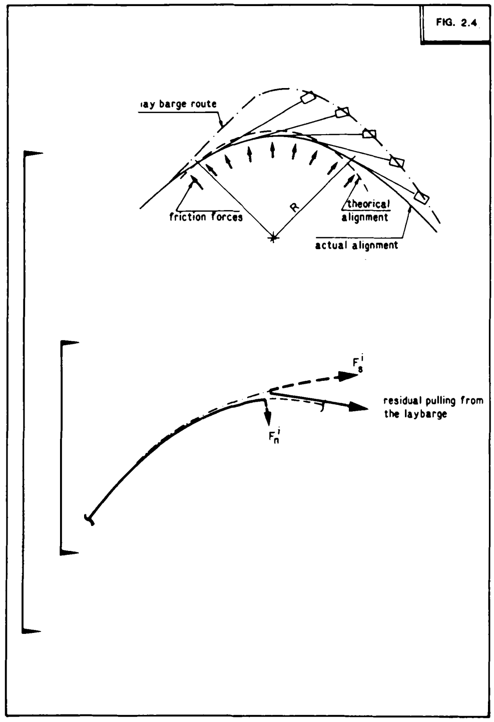

lay barge route
friction forces
R
theorical alignment
actual alignment
residual pulling from the laybarge

FIG. 2.4

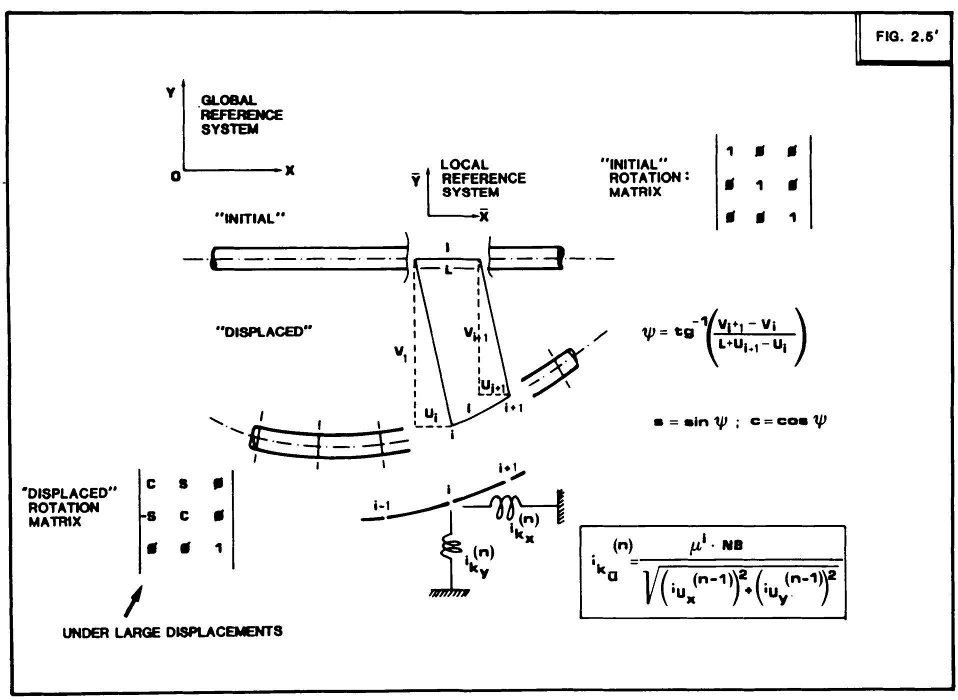

GLOBAL REFERENCE SYSTEM
LOCAL REFERENCE SYSTEM
"INITIAL"
"DISPLACED"
"INITIAL" ROTATION MATRIX
"DISPLACED" ROTATION MATRIX
UNDER LARGE DISPLACEMENTS
s = sin ψ ; c = cos ψ

FIG. 2.5'

FIG. 2.5″

$\underline{\underline{K}}^{E}$: STIFFNESS MATRIX FOR THE PIPELINE RESTING ON THE SEABED

$\underline{\underline{K}}^{F}$: STIFFNESS MATRIX FOR FRICTION – EQUIVALENT SPRINGS ($k_{ij} = \phi$, $i \neq j$).

1^{st} step : $(\underline{\underline{K}}^{E} + \underline{\underline{K}}^{F})\ \underline{s} = \underline{p}$

CONVERGENCE:

FRICTIONAL RESTRAINT – μ. NEGATIVE BUOYANCY $< \varepsilon$

2^{nd} step : $\underline{\underline{K}}^{F} = \dfrac{\mu\text{. NEGATIVE BUOYANCY}}{|\ s_{ii}\ |}$

($k_{ij} = \phi$, $i = j$)

Go to 1^{st} step

FIG. 3.1

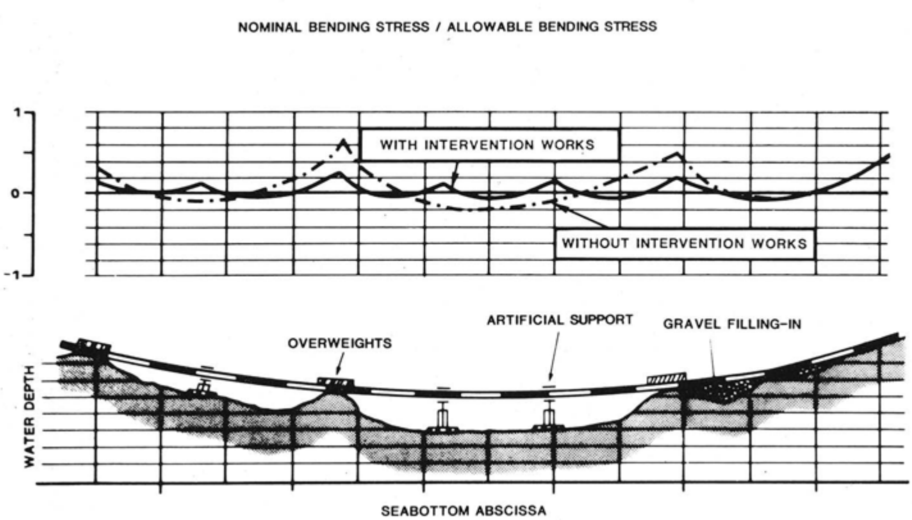
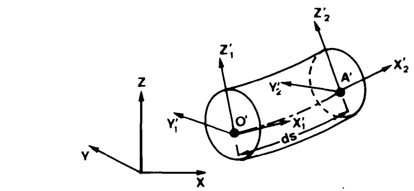
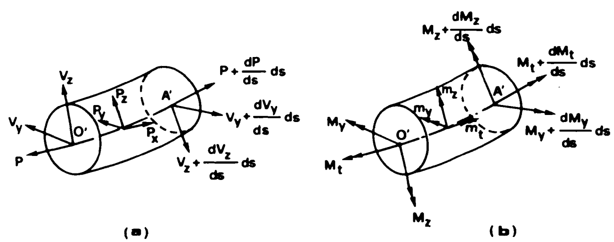


FIG. 3.2'



LOCAL AXIS OF THE DEFORMED BEAM AT O' AND A'

	X'_2	Y'_2	Z'_2
X'_1	1	$-d\theta_z$	$d\theta_y$
Y'_1	$d\theta_z$	1	$-d\varphi$
Z'_1	$-d\theta_y$	$d\varphi$	1



FREE-BODY DIAGRAMS OF DEFORMED-BEAM SEGMENT O' A'.

(a) APPLIED FORCES PER UNIT LENGTH AND FORCE STRESS RESULTANTS

(b) APPLIED MOMENTS PER UNIT LENGTH AND MOMENT STRESS RESULTANTS

FIG. 3.2''

1. EQUILIBRIUM EQUATIONS, 3-D BEAM

$$
\begin{array}{lll}
\text{transl. } x: & dP/ds + V_y \cdot d\theta_z/ds + V_z \cdot d\theta_y/ds + p_x & = 0 \\
\quad '' \quad y: & -dV_y/ds + P \cdot d\theta_z/ds + V_z \cdot d\varphi/ds + p_y & = 0 \\
\quad '' \quad z: & -dV_z/ds + P \cdot d\theta_y/ds - V_y \cdot d\varphi/ds + p_z & = 0 \\
\text{rotat. } x: & dM_t/ds + M_y \cdot d\theta_z/ds - M_z \cdot d\theta_y/ds + m_t & = 0 \\
\quad '' \quad y: & -dM_y/ds - M_z \cdot d\varphi/ds + M_t \cdot d\theta_z/ds + m_y + V_z & = 0 \\
\quad '' \quad z: & dM_z/ds - M_y \cdot d\varphi/ds + M_t \cdot d\theta_y/ds + m_z - V_y & = 0
\end{array}
$$

2. CURVATURE OF THE PROJECTION OF THE DEFLECTED AXIS OF THE BEAM INTO THE xy, xz PLANES

$$d\theta_z/ds = 1/R_y = d^2v/dx^2 / \left[1 + (dv/dx)^2 \right]^{3/2}$$

$$d\theta_y/ds = 1/R_z = d^2w/dx^2 / \left[1 + (dw/dx)^2 \right]^{3/2}$$

ASSUMPTION OF SMALL DEFLECTIONS, IN THE SENSE OF:

$dv^2/dx^2,\ d^2w/dx^2 \ll 1.$

$V_y/R_y,\ V_z/R_z,\ V_y\ d\varphi/ds,\ V_z\ d\varphi/ds \ll 1$

3. SIMPLIFIED EQUILIBRIUM EQUATIONS, 3-D BEAM

$$
\begin{array}{lll}
\text{transl. } x: & dP/dx = -p_x & \\
\quad '' \quad y: & dV_y/dx = p_y & + P \cdot d^2v/dx^2 \\
\quad '' \quad z: & dV_z/dx = p_z & + P \cdot d^2w/dx^2 \\
\text{rotat. } x: & dM_t/dx = -m_t & - M_y \cdot d^2v/dx^2 + M_z \cdot d^2w/dx^2 \\
\quad '' \quad y: & dM_y/dx = m_y + V_z & - M_z \cdot d\varphi/dx + M_t \cdot d^2v/dx^2 \\
\quad '' \quad z: & dM_z/dx = -m_z + V_y & + M_y \cdot d\varphi/dx - M_t \cdot d^2w/dx^2
\end{array}
$$

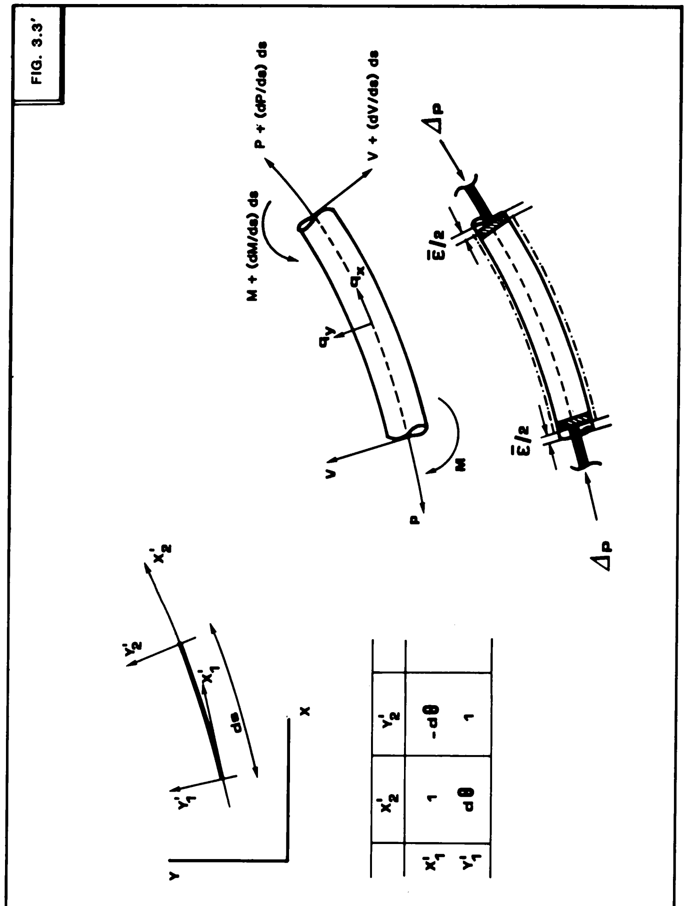
4. THE EQUATIONS WHICH THE DISPLACEMENTS MUST SATISFY TO ASSURE EQUILIBRIUM OF THE DEFORMED 3-D BEAM

$$\text{transl. } x: \ \frac{d}{dx}\left(EA\frac{du}{dx}\right) + \frac{1}{2}\frac{d}{dx}\left(EA\left(\left(\frac{dv}{dx}\right)^2 + \left(\frac{dw}{dx}\right)^2\right)\right) = -p_x$$

$$\begin{array}{l}\text{transl. } y \\ \text{rotat. } z\end{array}: \ \frac{d^2}{dx^2}\left(EJ\frac{d^2v}{dx^2}\right) - \frac{d}{dx}\left(My\frac{d\varphi}{dx}\right) + \frac{d}{dx}\left(Mt\frac{d^2w}{dx^2}\right) - P\frac{d^2v}{dx^2} = p_y - \frac{dm_z}{dx}$$

$$\begin{array}{l}\text{transl. } z \\ \text{rotat. } y\end{array}: \ \frac{d^2}{dx^2}\left(EJ\frac{d^2v}{dx^2}\right) - \frac{d}{dx}\left(Mz\frac{d\varphi}{dx}\right) - \frac{d}{dx}\left(Mt\frac{d^2v}{dx^2}\right) - P\frac{d^2w}{dx^2} = p_z + \frac{dm_y}{dx}$$

$$\text{rotat. } x: \ \frac{d}{dx}\left(GJ\frac{d\varphi}{dx}\right) = -m_t - My\frac{d^2v}{dx^2} + Mz\frac{d^2w}{dx^2}$$



	x'_2	y'_2
x'_1	1	$-d\theta$
y'_1	$d\theta$	1

FIG. 3.3'

FIG. 8,3''

a. THE DIFFERENTIAL EQUATION OF A 2-D PIPE UNDER AXIAL LOADS

1. STRAIN LAW : $\varepsilon_{xx} - \bar{\varepsilon}_{xx} = \dfrac{du}{dx} + \dfrac{1}{2}\left[\left(\dfrac{dv}{dx}\right)^2\right]$
2. STRESS LAW : $\sigma_{xx} = \dfrac{P}{A}$
3. HOOKE LAW : $\sigma_{xx} = E\left(\varepsilon_{xx} - \bar{\varepsilon}_{xx}\right)$
4. EQ.TRAN. X : $\dfrac{dP}{dx} + p_x = \emptyset$

WHERE :

5. INIT.STRAIN : $\bar{\varepsilon}_{xx} = \alpha \cdot \Delta T - \gamma\ \sigma_H/E$
6. HOOP STRESS : $\sigma_H = p \cdot R/t$

b. THE DIFFERENTIAL EQUATION OF A 2-D PIPE UNDER LATERAL LOADS

1. STRAIN LAW : $\varepsilon_{xx} = -\dfrac{d^2v}{dx^2}$
2. STRESS LAW : $\sigma_{xx} = (M/J) \cdot y$
3. HOOKE LAW : $\sigma_{xx} = E\left(y \cdot \dfrac{d^2v}{dx^2}\right)$
4. EQ.TRAN. Y : $\dfrac{dV}{dx} = q_y + (P - \Delta p) \cdot \dfrac{d^2v}{dx^2}$
5. EQ.ROTA. Z : $\dfrac{dM}{dx} = V$

WHERE:

6. PRESS.EFF. : $\Delta p = p_{INT.} \cdot A_{INT.} - p_{EST.} \cdot A_{EST.}$

ELASTIC COUPLING

$$E\,A\left[\frac{d^2u}{dx^2} \boxed{+ \frac{1}{2}\frac{d}{dx}\left(\frac{dv}{dx}\right)^2}\right] = -q_x + E\,A\,\frac{d\bar{\varepsilon}}{dx}$$

$$E\,J\,\frac{d^4v}{dx^4} \boxed{-(P - \Delta p)\frac{d^2v}{dx^2}} = q_y$$

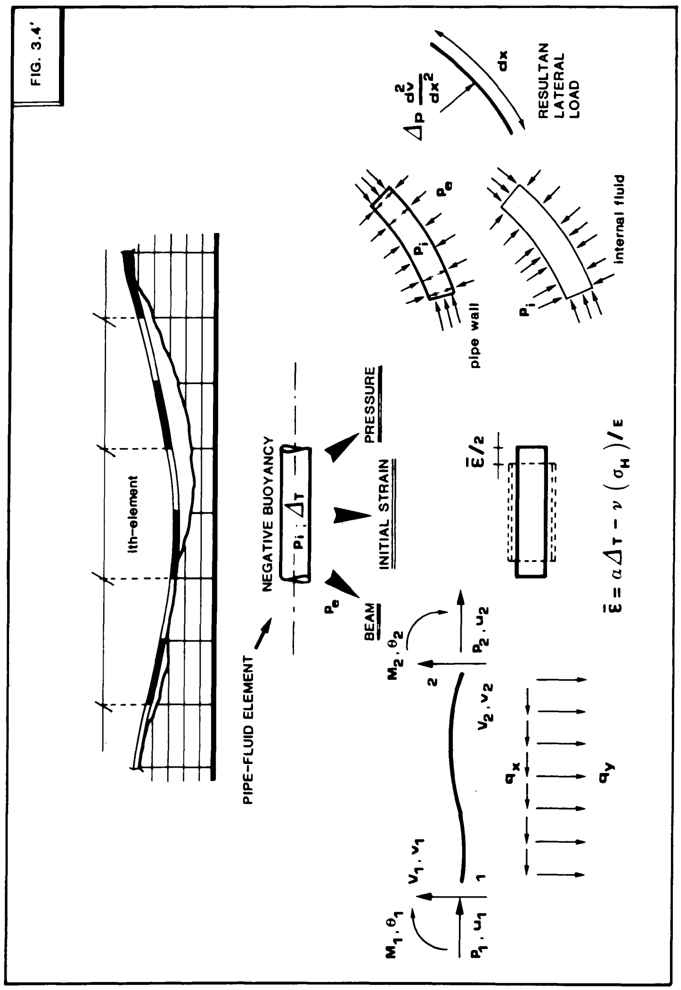


FIG. 3.4'
Ith-element
PIPE-FLUID ELEMENT
NEGATIVE BUOYANCY
p_i ; Δ_T
p_e
BEAM
INITIAL STRAIN
PRESSURE
M_1, θ_1
P_1, u_1
V_1, v_1
1
2
M_2, θ_2
P_2, u_2
V_2, v_2
q_x
q_y
$\bar{\varepsilon}/2$
$\bar{\varepsilon} = \alpha \Delta_T - \nu (\sigma_H) / E$
pipe wall
p_i
p_e
internal fluid
$\Delta p \frac{d^2 v}{dx^2}$
dx
RESULTAN
LATERAL
LOAD

FIG. 3.4''

STIFFNESS MATRIX OF A PIPE SEGMENT

a. Analytical formulation:

$$\mathbf{K} = f \cdot \begin{vmatrix} \mathbf{K}aa & \mathbf{K}ab \\ \mathbf{K}ba & \mathbf{K}bb \end{vmatrix}$$

a1. Axial compression

$\alpha = |P| / EJ \quad ; \quad \beta = EA / L$

$s = \sin(\alpha L) \quad ; \quad c = \cos(\alpha L)$

$f = EJ / (2 - 2c - \alpha L s)$

$$\mathbf{K}aa = \begin{vmatrix} \beta & & \\ 0 & \alpha^3 s & \\ 0 & -\alpha^2(1-c) & \alpha(s-\alpha Lc) \end{vmatrix}$$

$$\mathbf{K}bb = \begin{vmatrix} \beta & & \\ 0 & \alpha^3 s & \\ 0 & \alpha^2(1-c) & \alpha(s-\alpha Lc) \end{vmatrix}$$

$$\mathbf{K}ab = \begin{vmatrix} -\beta & 0 & 0 \\ 0 & -\alpha^3 s & -\alpha^2(1-c) \\ 0 & \alpha^2(1-c) & \alpha(\alpha L-s) \end{vmatrix}$$

$\mathbf{K}ba = \mathbf{K}ab$

a2. Axial tension

$\alpha = P / EJ \quad ; \quad \beta = EA / L$

$s' = \sinh(\alpha L) \quad ; \quad c' = \cosh(\alpha L)$

$f = EJ / (2 - 2c' + \alpha L s')$

$$\mathbf{K}aa = \begin{vmatrix} \beta & & \\ 0 & \alpha^3 s' & \\ 0 & -\alpha^2(c-1) & \alpha(\alpha Lc'-s') \end{vmatrix}$$

$$\mathbf{K}bb = \begin{vmatrix} \beta & & \\ 0 & \alpha^3 s' & \\ 0 & \alpha^2(c'-1) & \alpha(\alpha Lc'-c') \end{vmatrix}$$

$$\mathbf{K}ab = \begin{vmatrix} -\beta & 0 & 0 \\ 0 & -\alpha^3 s' & -\alpha^2(c'-1) \\ 0 & \alpha^2(c'-1) & \alpha(s'-\alpha L) \end{vmatrix}$$

$\mathbf{K}ba = \mathbf{K}ab$

b. Approximate formulation:

$$\mathbf{K} = \frac{EJ}{L^3} \cdot \begin{vmatrix} \mathbf{K}^e_{aa} & \mathbf{K}^e_{ab} \\ \mathbf{K}^e_{ba} & \mathbf{K}^e_{bb} \end{vmatrix} + \frac{P}{L} \begin{vmatrix} \mathbf{K}^g_{aa} & \mathbf{K}^g_{ab} \\ \mathbf{K}^g_{ba} & \mathbf{K}^g_{bb} \end{vmatrix}$$

b1. Stiffness matrix of a beam under small displacement

$\beta' = EA / L / \gamma \quad ; \quad \gamma = EJ / L^3$

$$\mathbf{K}^e_{aa} = \begin{vmatrix} \beta' & & \\ 0 & 12 & \\ 0 & -6\,L & 4\,L^2 \end{vmatrix}$$

$$\mathbf{K}^e_{bb} = \begin{vmatrix} \beta' & & \\ 0 & 12 & \\ 0 & 6\,L & 4\,L^2 \end{vmatrix}$$

$$\mathbf{K}^e_{ab} = \begin{vmatrix} -\beta' & 0 & 0 \\ 0 & -12 & -6\,L \\ 0 & 6\,L & 2\,L^2 \end{vmatrix}$$

$\mathbf{K}^e_{ba} = \mathbf{K}^e_{ab}$

b2. Incremental (geometric, differential) stiffness of a beam under large displacement

$$\mathbf{K}^g_{aa} = \begin{vmatrix} 0 & & \\ 0 & 6/5 & \\ 0 & -L/10 & 2L^2/15 \end{vmatrix}$$

$$\mathbf{K}^g_{bb} = \begin{vmatrix} 0 & & \\ 0 & 6/5 & \\ 0 & L/10 & 2L^2/15 \end{vmatrix}$$

$$\mathbf{K}^g_{ab} = \begin{vmatrix} 0 & 0 & 0 \\ 0 & -6/5 & L/10 \\ 0 & -L/10 & -L^2/30 \end{vmatrix}$$

$\mathbf{K}^g_{ba} = \mathbf{K}^g_{ab}$

FIG. 3.5

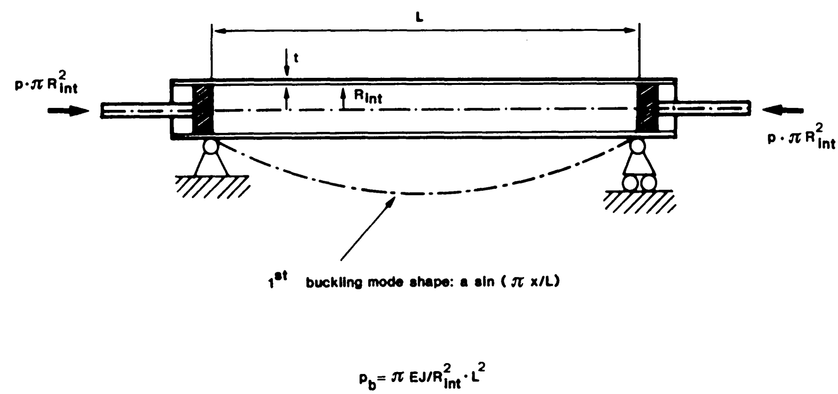

L
t
$p \cdot \pi R_{int}^2$
R_{int}
$p \cdot \pi R_{int}^2$
1st buckling mode shape: a sin (π x/L)
$p_b = \pi EJ/R_{int}^2 \cdot L^2$

FIG. 3.6

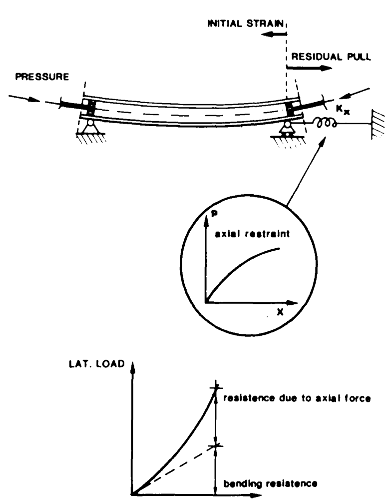

INITIAL STRAIN
RESIDUAL PULL
PRESSURE
K_x
p
axial restraint
x
LAT. LOAD
resistence due to axial force
bending resistence
LAT. DISPL.

FIG. 3.7'

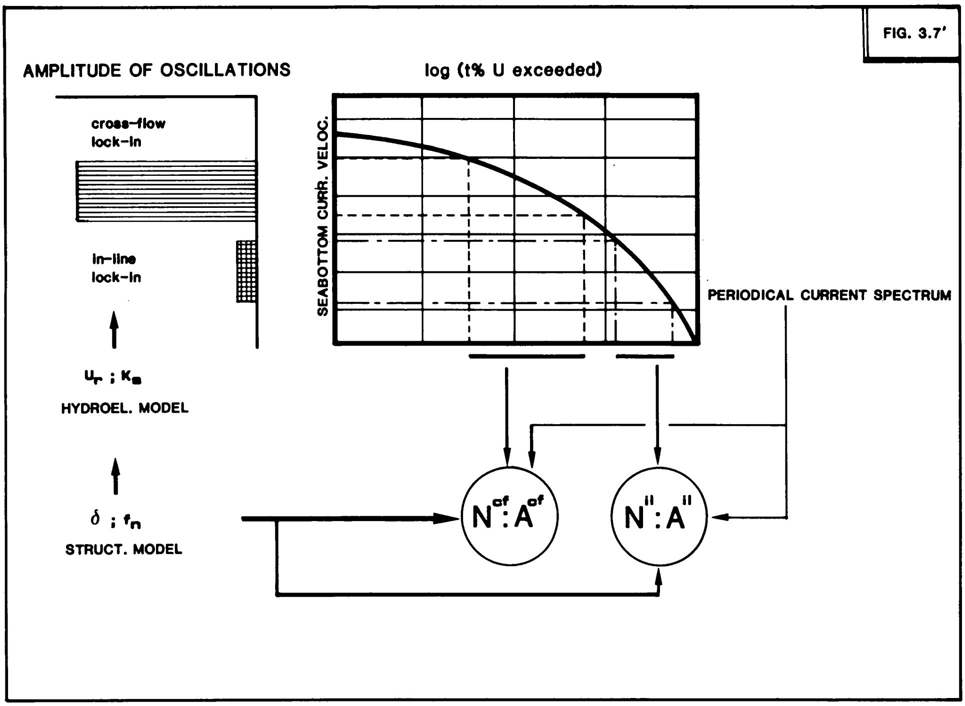
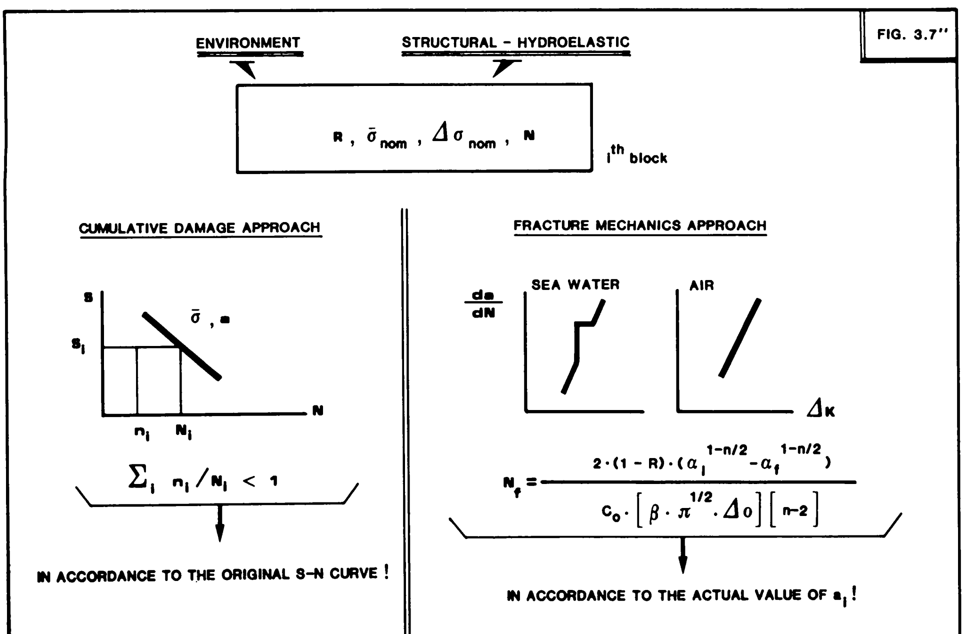


FIG. 3.7''



• The model of competitive processes (corrosion and fatigue) by Walker and May //19//

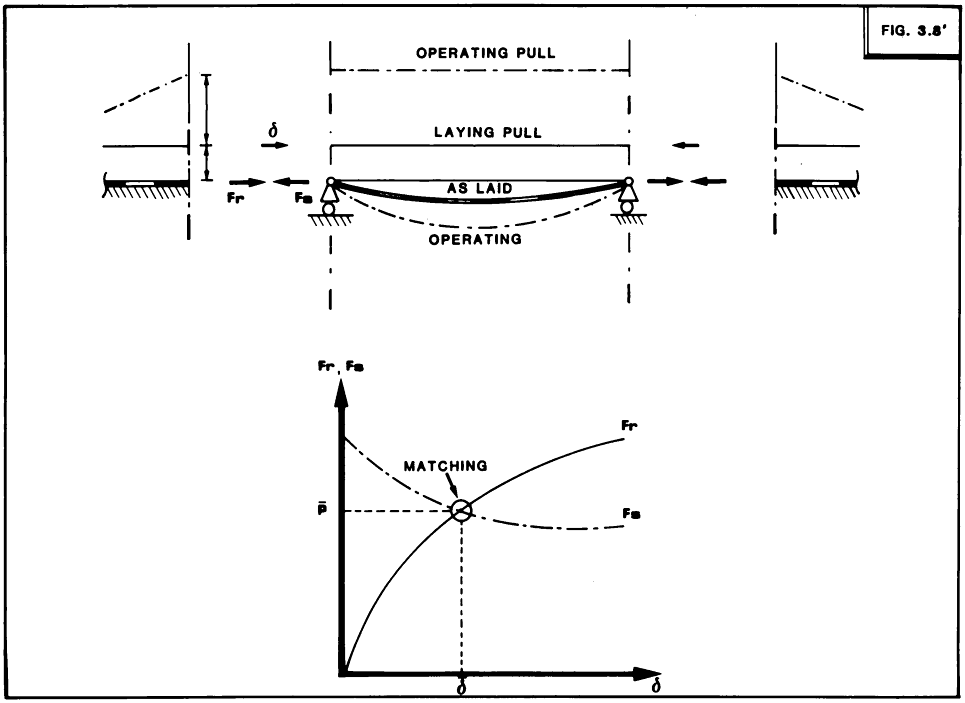


FIG. 3.8'
OPERATING PULL
LAYING PULL
δ
AS LAID
OPERATING
Fr
Fs
Fr, Fs
MATCHING
P̄
δ̄
δ

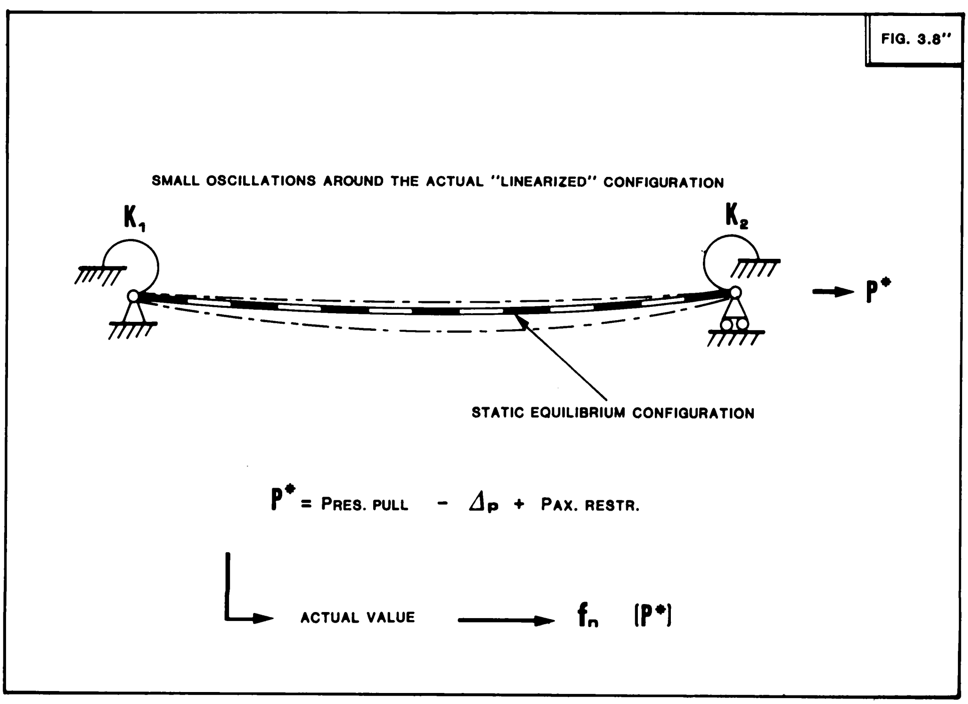


FIG. 3.8''
SMALL OSCILLATIONS AROUND THE ACTUAL "LINEARIZED" CONFIGURATION
K1
K2
P*
STATIC EQUILIBRIUM CONFIGURATION
P* = PRES. PULL - Δp + PAX. RESTR.
ACTUAL VALUE
fn (P*)

FIG. 3.8'''

NATURAL FREQUENCIES FOR BEAMS SUPPORTING A UNIFORM AXIAL LOAD

1. NATURAL FREQUENCY PARAMETER:

$$\lambda_i = \lambda_i \left(\frac{P^* \, L}{E \;\; J} \text{, boundary conditions} \right)$$

$$\left(\text{if} \quad P^* = 0, \; f_n^o = \frac{\lambda_i^2}{2\pi \, L^2} \left(\frac{EJ}{m} \right)^{1/2} ; \; i = 1, n \right)$$

2. NATURAL FREQUENCIES, FOR PINNED-PINNED UNIFORM BEAMS:

$$f_n = i^2 \, \pi^2 \left[1 + \frac{P \cdot L^2}{EJ \cdot i^2 \, \pi^2} \right] \left(\frac{EJ}{m} \right)^{1/2}$$

(exact solution)

3. NATURAL FREQUENCIES, FOR OTHER BEAMS:

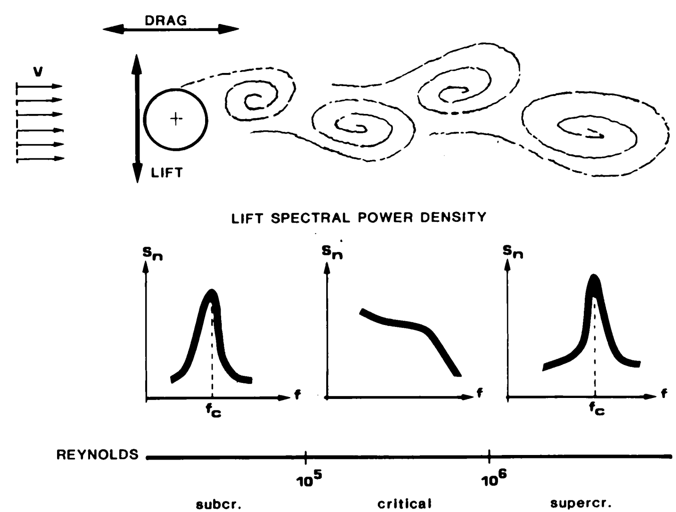
$$f_n = f_n^o \cdot \left[1 + \frac{P^*}{|P_b|} \left(\lambda_1^2 / \lambda_i^2 \right) \right]$$

$$P_b = \frac{-\pi^2 \cdot EJ}{(L_e)^2} \qquad (\, L_e = L_* \text{ wave parameter})$$

FIG. 3.9'

STATIONARY PIPE

VON KARMAN WAKE



STROUHAL RELATIONSHIP

$$f_{vs} = v/D \;\; St\,(Re)$$

FIG. 3.9''

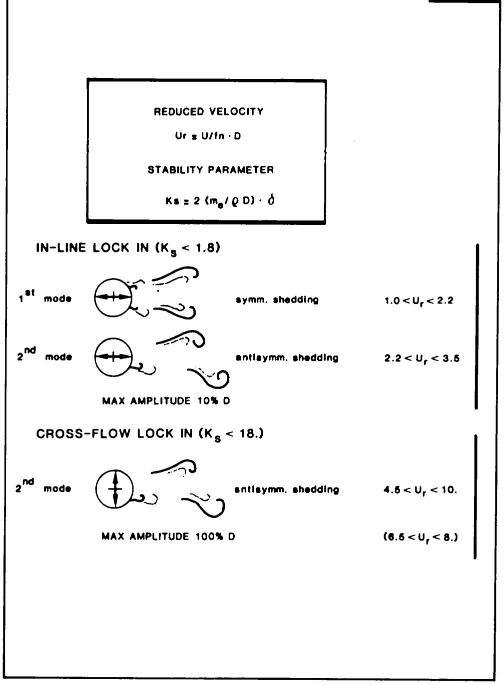

REDUCED VELOCITY
Ur = U/fn · D
STABILITY PARAMETER
Ks = 2 (me / ρ D) · δ
IN-LINE LOCK IN (Ks < 1.8)
1st mode
symm. shedding
1.0 < Ur < 2.2
2nd mode
antisymm. shedding
2.2 < Ur < 3.5
MAX AMPLITUDE 10% D
CROSS-FLOW LOCK IN (Ks < 18.)
2nd mode
antisymm. shedding
4.5 < Ur < 10.
MAX AMPLITUDE 100% D
(6.5 < Ur < 8.)

FIG. 3.9'''

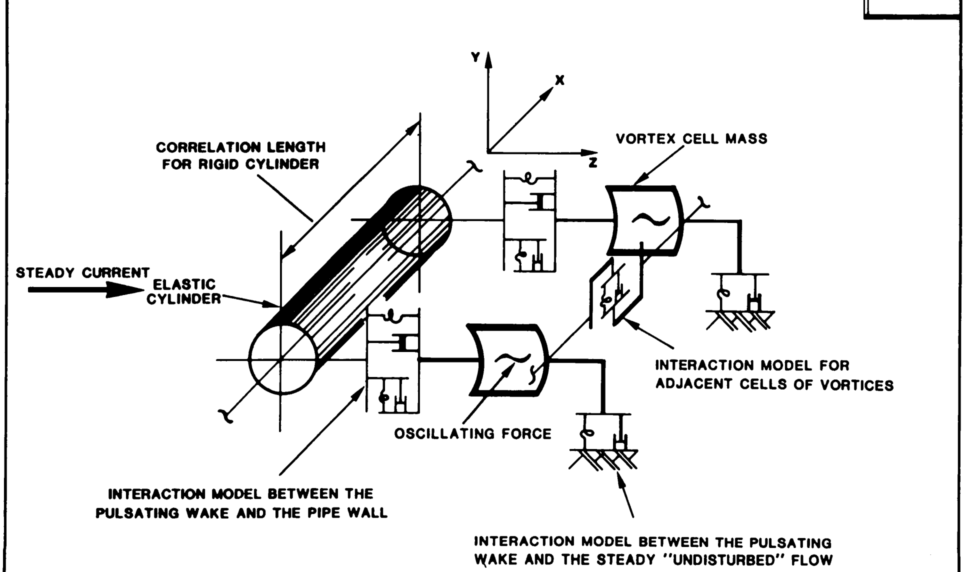

Y
X
Z
CORRELATION LENGTH
FOR RIGID CYLINDER
VORTEX CELL MASS
STEADY CURRENT
ELASTIC
CYLINDER
INTERACTION MODEL FOR
ADJACENT CELLS OF VORTICES
OSCILLATING FORCE
INTERACTION MODEL BETWEEN THE
PULSATING WAKE AND THE PIPE WALL
INTERACTION MODEL BETWEEN THE PULSATING
WAKE AND THE STEADY "UNDISTURBED" FLOW

FIG. 3.10'

THE "WAKE OSCILLATOR" APPROACH

STEP 1 : IDENTIFICATION OF VIBRATIONAL PROPERTIES OF FREE SPAN

a - NATURAL FREQUENCIES, f_n

b - MODE SHAPES , ψ_n

c - EQUIVALENT MASS , m_e

$$\left(m_e = \int_o^L m(x) \cdot \psi^2(x)\, dx \Big/ \int_o^{h=L} \psi^2(x)\, dx\right)$$

d - MODE SCALING FACTORS, I_{iiii}

$$\left(I_{iiii} = \int_o^L \psi_i^4\,(x)\, dx \Big/ \int_o^L \psi_i^2\,(x)\, dx\right)$$

STEP 2 : IDENTIFICATION OF MAIN PARAMETERS FOR VORTEX-INDUCED OSCILLATIONS

a - REDUCED VELOCITY , U_R

$$(U_R = U\infty \,/\, f_n \cdot D)$$

b - STABILITY PARAMETER, K_s

$$(K_s = 2 \cdot \underbrace{(m_e \,/\, \varrho D^2)}_{\text{mass ratio}} \cdot \underbrace{\delta}_{\text{logaritmic decrement of structural vibrations}})$$

CHECK ON SYNCHRONIZATION

IN-LINE : $1 < U_R < 3.5$ CROSS-FLOW : $4.5 < U_R < 10$

FIG. 3.10''

STEP 3: IDENTIFICATION OF MAXIMUM AMPLITUDE DUE TO RESONANT VORTEX-EXCITED OSCILLATIONS (CROSS-FLOW, IN-LINE)

REDUCED AMPLITUDE (:D)
VS.
STABILITY PARAMETER

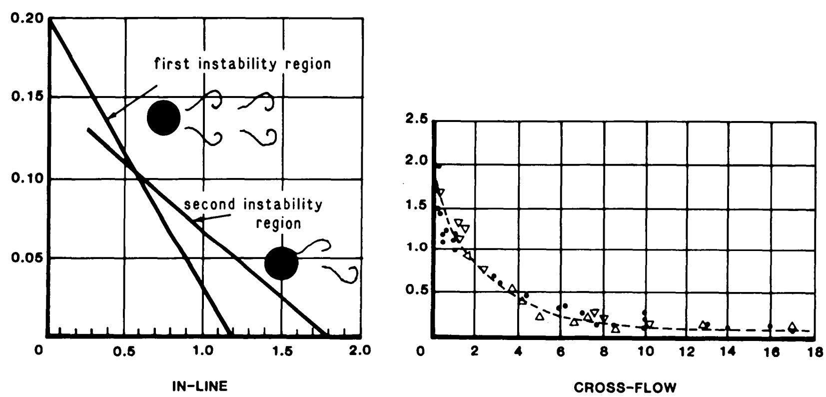


FIG. 3.11

THE "WAKE OSCILLATOR" APPROACH

STEP 1 : IDENTIFICATION OF VIBRATIONAL PROPERTIES OF FREE SPAN

a - NATURAL FREQUENCIES, f_n

b - MODE SHAPES, ψ_n

c - EQUIVALENT MASS, m_e

$$\left(m_e = \int_0^L m(x) \cdot \psi^2(x)\, dx \Big/ \int_0^{h=L} \psi^2(x)\, dx\right)$$

d - MODE SCALING FACTORS, I_{iiii}

$$\left(I_{iiii} = \int_0^L \psi_i^4(x)\, dx \Big/ \int_0^L \psi_i^2(x)\, dx\right)$$

STEP 2 : IDENTIFICATION OF MAIN PARAMETERS FOR VORTEX-INDUCED OSCILLATIONS

a - REDUCED VELOCITY, U_R

$(U_R = U\infty / f_n \cdot D)$

b - STABILITY PARAMETER, K_s

$$\left(K_s = 2 \cdot \underbrace{(m_e / \rho D^2)}_{\text{mass ratio}} \cdot \underbrace{\delta}_{\text{logaritmic decrement of structural vibrations}}\right)$$

CHECK ON SYNCHRONIZATION

IN-LINE : $1 < U_R < 3.5$ CROSS-FLOW : $4.5 < U_R < 10$

STEP 3 : IDENTIFICATION OF MAXIMUM AMPLITUDE DUE TO RESONANT VORTEX-EXCITED OSCILLATIONS

GRIFFIN, SKOP, RAMBERG : $$\frac{A}{D} = \frac{1.29\,\gamma}{\left[1 + 0.43\,(2\pi\, S_T^2 \cdot K_s)\right]^2}$$

BLEVINS : $$\frac{A}{D} = \frac{0.07\,\gamma}{(1.9 + K_s)\, S_T^2}\left[0.3 + \frac{0.72}{(1.9 + K_s) S_T^2}\right]$$

SARPKAYA : $$\frac{A}{D} = \frac{0.32\,\gamma}{\left[0.6 + (2\pi\, S_T^2 \cdot K_s)^2\right]^{1/2}}$$

(γ = dimensionless mode shape factor)

FIG. 3.12'

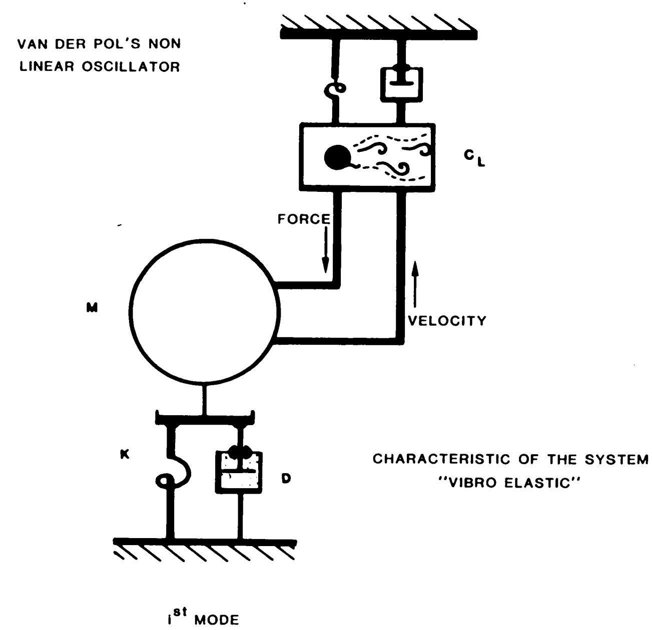


FIG. 3.12''

THE WAKE OSCILLATOR MODEL

a) THE ELASTIC BEAM

1. $EJ\, v^{IV} - P^* v^{II} = - m\, \ddot{v} + \text{LIFT}$
2. $v\,(x,t) = \Sigma_i\, q_i\,(t) \cdot \psi_i\,(x)$
3. $\ddot{q}_i + 2\,\omega_{i,n}\, \zeta_i\, q_i + \omega_{i,n}^2\, q_i =$

$$= L_i \,/\, M_i = \frac{\frac{1}{2}\, \rho U^2 D \int_0^1 C_L\,(x,t) \cdot \psi_i\,(x)\, dx}{\int_0^1 m\,(x) \cdot \psi_i^2\,(x)\, dx}$$

b) THE LIFT FORCE

1. $\ddot{C}_L + \omega_s^2\, C_L + N.L.T.\ (C_L,\, \dot{C}_L,\, \bar{h}_i) = \omega_s\, \bar{h}_o\, \dot{v} \,/\, D$
2. $C_L\,(x,t) = \Sigma_i\, Q_i\,(t)\, \psi_i\,(x)\, dx$
3. $\ddot{Q}_i + \omega_s^2\, Q_i + N.L.T.\ (Q,\, \dot{Q},\, \bar{h}_i,\, m,\, \psi) = \omega_s\, \bar{h}_o\, \dot{q} \,/\, D$

THE EQUATION OF MOTION

$$\ddot{q}_i \,/\, D + 2\,\omega_{i,n}\, \zeta_i\, \dot{q}_i \,/\, D + \omega_{i,n}^2\, q_i \,/\, D = \omega_s^2 \cdot \mu_{ii} \cdot Q_i$$

THE RESPONSE OF THE SYSTEM AT A FREQUENCY ω

3a, UNDAMPED : $q_i \propto Q_i \cdot \left[1 - (\omega / \omega_{i,n})^2\right]^{-1}$

3b, UNDAMPED : $Q_i \propto q_i \cdot \left[1 - (\omega / \omega_s)^2\right]^{-1}$

AT ω / ω_s , $\omega / \omega_{i,n} \simeq 1$, THE i^{th} VIBRATION MODE UNDERGOES A RESONANT TYPE BEHAVIOUR; THE REMAINING VIBRATION MODES REMAIN SMALL (ASSUMPTION: AN EQUIVALENT LINEAR SOLUTION)

At $\omega_s \simeq \omega_{i,n}$:

$q_i \,/\, D = A_i \sin \omega_{i,n}\, t ;\ q_j \,/\, D = \emptyset\ ,\ j \neq i$

$Q_i \,/\, C_{Lo} = B_i \cdot \sin\, (\omega_{i,n}\, t + \varphi_i)\ ;\ Q_j \,/\, C_{Lo} = \emptyset\ ,\ j \neq i$

A_i , B_i DEPEND ON HEURISTIC PARAMETERS $\bar{h}_o$, $\bar{h}_i$!

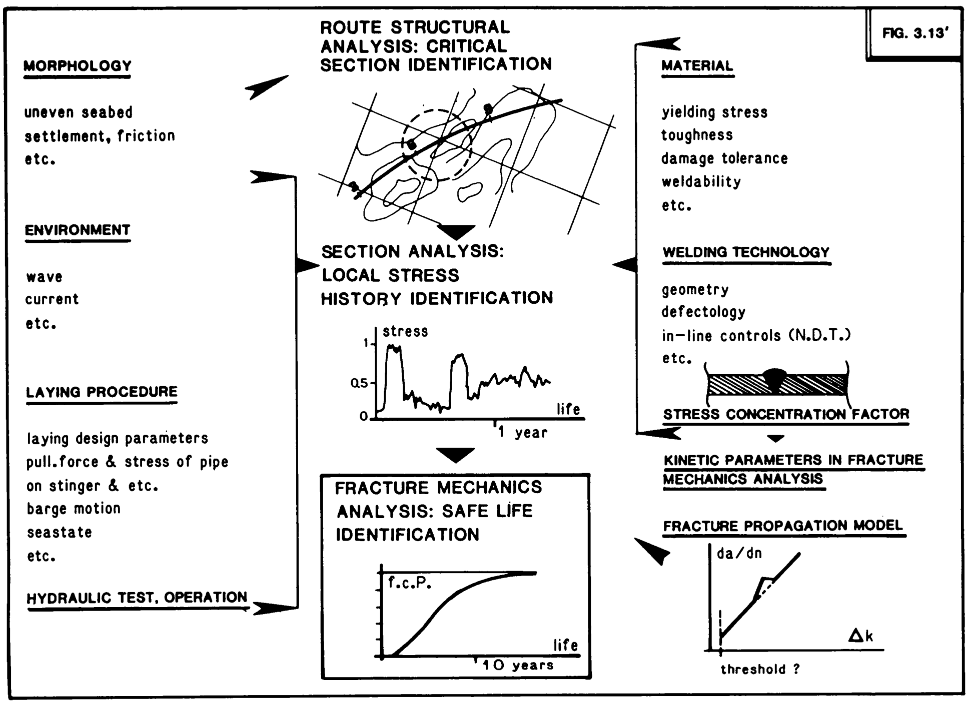


FIG. 3.13'
MORPHOLOGY
uneven seabed
settlement, friction
etc.
ENVIRONMENT
wave
current
etc.
LAYING PROCEDURE
laying design parameters
pull.force & stress of pipe
on stinger & etc.
barge motion
seastate
etc.
HYDRAULIC TEST, OPERATION
ROUTE STRUCTURAL
ANALYSIS: CRITICAL
SECTION IDENTIFICATION
SECTION ANALYSIS:
LOCAL STRESS
HISTORY IDENTIFICATION
stress
1
0.5
0
life
1 year
FRACTURE MECHANICS
ANALYSIS: SAFE LIFE
IDENTIFICATION
f.c.P.
life
10 years
MATERIAL
yielding stress
toughness
damage tolerance
weldability
etc.
WELDING TECHNOLOGY
geometry
defectology
in-line controls (N.D.T.)
etc.
STRESS CONCENTRATION FACTOR
KINETIC PARAMETERS IN FRACTURE
MECHANICS ANALYSIS
FRACTURE PROPAGATION MODEL
da/dn
Δk
threshold ?

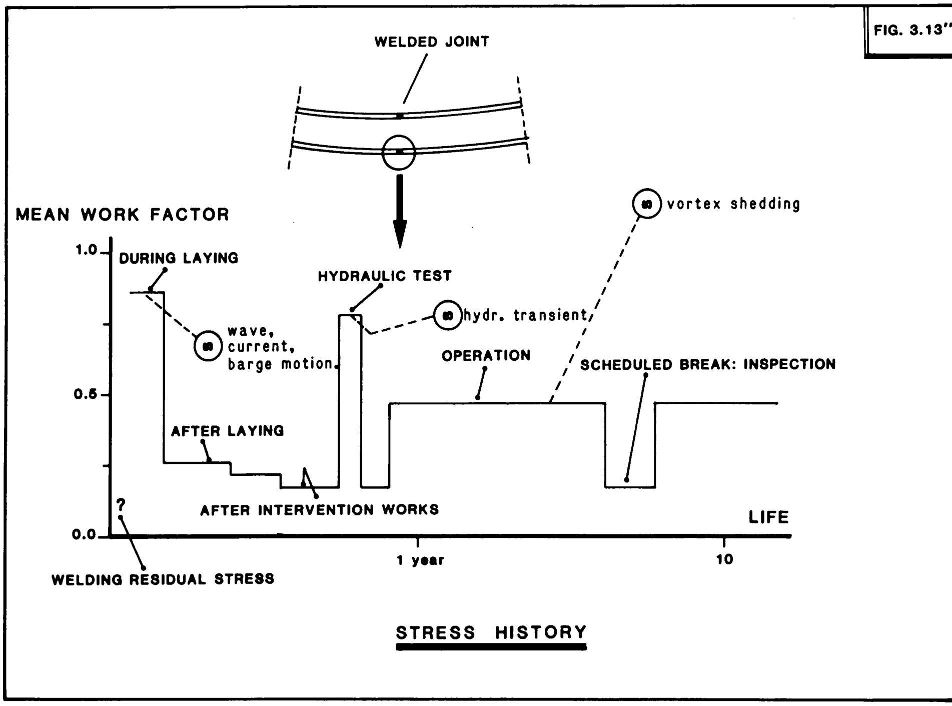


FIG. 3.13''

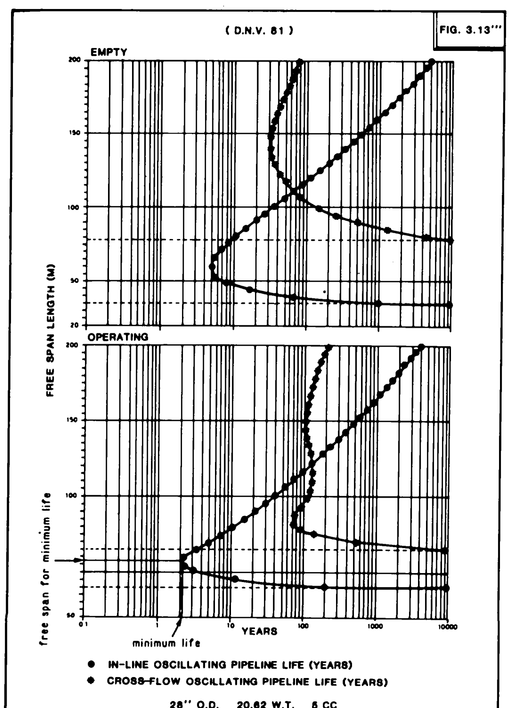


FIG. 3.13'''

LAYING OPERATIONS IN THE NORTH SEA DURING THE 1981 SEASON

V. Giardinieri

Saipem, Offshore Division

1. INTRODUCTION

Three BP projects - the Magnus to Ninian central platform oil export line (fig. 2), the West Sole gasline to shore (fig. 4) and the Magnus flowlines (fig. 3) have been executed by Saipem during the 1981 season. This paper presents their features of interest and describes them in some detail (fig. 1).

The Magnus platform - to which two of the projects are related - is by itself of special interest. Weighing 41,000 tons, it is the heaviest steel jacket installed in one piece today and the deepest tie-in (in 187 m. of water) of the pipeline laid by Saipem into the riser prefabricated with the jacket. This tie-in was performed in March 1983 by means of a hyperbaric welding.

The following are the main features of the three projects:

- a very thorough and reliable construction planning was required to insure their completion by the end of the season; special attention was given to procedures and techniques capable of decreasing the weather sensitivity of the operations (fig. 5);
- for the first time the new automatic welding system "Passo" has been adopted and validated on off-shore operations.

The system developed by Saipem and Arcos has been extensively proven on land and its main features are reported in appendix 1.

The customer BP approved the decision of Saipem after the positive fulfillment of a supplementary set of "ad hoc" tests.

The operational characteristics of "Passo" welding systems, associated with the excellent mechanical properties of the welded joint, make it extremely flexible and in particular the design concept of the oscillator motion has been selected with the purpose of obtaining first class weldments substantially free from lack of fusion defects.

This has been demonstrated by adopting this welding process in the construction of more than 1.000 Km. of on-shore/off-shore pipelines.

In the particular case of Magnus Project (line length = 91 Km.), which was the first off-shore application of this welding system, the total repair rate resulted to be 2 % and the maximum daily rate was 264 joints equal to 3332 m.

- In order to substantiate the requirements of planned, reliable implementation as independent as possible from prevailing meteomarine conditions, an unprecedented integration between the laying vessel and the multi-purpose diving vessel "Ragno Due" has been pursued and carried out, especially on the Magnus to Ninian trunkline and on Magnus flowlines.

The operativity of "Ragno Due" (fig. 6) has been very significant taking into consideration the meteorological conditions in the area (fig. 7).

Saipem was especially dedicated to these projects also because of previous cooperation with BP during the years 1973-75, when the laying operations of the Forties Field 32" pipeline were carried out in the North Sea.

At that time the project was unprecedented in terms of water depth (132 m.) for such a large pipeline, and it is still important to remember the performance of the "above water tie-in", a special operation in order to connect the two sections of line laid by two different contractors.

Previous cooperation with BP was especially gratifying for Saipem in terms of actual performances and technical advances in the early '70's whereby not only a successful pioneering effort was rewarded with the timely completion of the project, but a wealth of novel technology was attained with significant technological achievements and a couple of joint patents.

2. MAGNUS FIELD OIL EXPORT PIPELINE

2.1) The technical contents of the contract can be summarized as follows (fig. 8):

- the beginning of the laying operations called for the touch-down of the pipe initiation head in 187 m. water depth, in a target area 10 m. x 10 m.

  This requirement, particularly tight, was related to the accuracy requested for the subsequent Magnus platform installation;
- the laying termination was at the Ninian control platform, in a "congested area" that means with severe restrictions to the mooring of the lay barge, due to the presence of existing pipelines;
- the installation of the spool piece required a careful study from the point of view of both "structural analysis" and "operation". It consists of a piece of line, with the so-called "dog-leg" shape, having the function of expansion loop in the final connection between the sealine and the future riser. The lifting procedure and the movement sequence has been defined in order to avoid undue stresses during the lifting and "hydrostatic" instability when moving the spool through the sea surface.

  Each step of the operation has been checked with lay-out sketches;
- crossing of existing pipelines: this point will be discussed extensively in point 2.2).

2.2) One of the most significant operations of the project has been the crossing of the untrenched pipeline, performed through the following preliminary phases (fig. 9):

- laying of a transponder array in the crossing area and calibration;
- diving of the manned submersible for the following operations:

a) video inspection;

b) positioning of the existing pipeline;

c) identification of the crossing point by means of visual markers;

d) marking of the approach route of the pipeline to be laid by means of visual markers (fig. 11);

- computation of the distance between visual markers and the route of the pipeline to be laid;
- lowering of mattresses according to the sequence shown in fig. 10;
- lowering of concrete sleepers, to be put in contact with the lateral mattresses;
- final survey of the crossing area.

2.3) Subsequently the guided laying of the pipeline was performed, according to the following procedure:

- estimation of the touch-down point (T.D.P.) by means of a computer program, available on board of "Castoro Sei", for the stress analysis of the pipe deflection curve;
- when the T.D.P. was in correspondence with one of the markers previously located, we decided for the launching of the observation and manipulation bell;
- positioning of the bell on the relevant marker;
- measurement of the distance marker/pipeline by means of the bell side looking sonar;
- comparison with the theoretical distance figures;
- correction of the lay barge's position, as required by the previous point;
- the above said procedure has been repeated for each marker;

- repositioning of the bell over the actual crossing point for the visual monitoring.

  We point out that, in each of the three crossings, the positioning error has been less than one meter.

2.4) The laying start-up operations have been performed with a procedure including the following steps (fig. 12 and fig. 18):

- laying of a medium frequency transponder array;
- correlation with an existing array, operating in low frequency and then not suitable for the Saipem positioning equipment;
- completion of the calibration operations;
- laying of the dead-man anchor.A transponder T1 has been fitted on the anchor chain, in order to allow the right positioning;
- laying, across the target area, of the initiation cable, fitted with a transponder T2, and load testing of the cable;
- calibration of the T2 transponder;
- repositioning of the cable, changing the tension and/or moving the barge, in order to ensure that T2 is within the target area;
- computation of the distance between T2 and the center of the target area (C.T.A.);
- recovery on board of the cable;
- cutting of the cable in the section corresponding to the C.T.A.;
- connection with the cable of the initiation head, where a transponder T3 has been fitted;
- start-up of the laying operations;
- monitoring of the initiation head position when approaching the

sea bottom. Also in this case the positioning error of the initiation head with respect to the C.T.A. has been less than 1 m.

2.5) The problems to be overcome on the pipeline termination at Ninian platform (fig. 13) have been mainly due to the anchoring limitations in the so-called "congested area". Therefore the following procedure has been decided:

- positioning, by the submarine, of the transponder array as reference for the termination monitoring;
- positioning and calibration of supplementary transponder in order to determine with accuracy the existing obstacle; both the previous transponders have been used as reference for the T.D.P. guidance procedure performed by the bell;
- fitting with transponder to the pipeline during the final laying and calibration; in this way we could determine the pipeline position in respect of the target point; consequently the exact pipeline cut-out section was defined;
- finally, positioning of a transponder T1 on the lay-down head, likewise the laying start-up phase.

The operations for correcting the positioning error of the pipeline head have been critical, due to the difficulty of manoeuvring close to the platform. With the help of the four thrusters, the final positioning error has been very satisfactory in this case too; approximately 2 m.

3. MAGNUS FIELD FLOWLINES

3.1) Also for this contract a highly sophisticated laying procedure has been devised. The solution has been imposed by the dimensional requirements of the two target areas (10 m. x 31 m. in the S.W. area and 10 m. x 25 m. in the N.E. area).

The approach proposed by Saipem to the technical problems of the contract has been one of the qualifying factors in the awarding of the job.

The laying sequence of the seven flowlines has been defined in order to minimize the working times and this result has mainly been obtained by decreasing the number of move-in and move-out for the anchor pattern of "Castoro Sei".

3.2) The time scheduled for the laying operations of the flowlines (fig. 14) was at the end of the working season, and therefore a procedure in order to minimize the duration, mainly through a reduction of the number of mooring and unmooring operations,has been studied in details.

The technological innovation of our design has been represented by the connection, by means of a required cable length between the lay-down head of the first flowline and the initiation head of the subsequent.

Practically three operations of "laying through the platform" and one operation of normal laying for one flowline (fig. 16) have been performed.

The daily working sequence for each flowline is shown in fig. 17. This procedure called for high accuracy in the laying operations, due to the fact that a small alignment error could have involved as a consequence a greater misalignment of both flowline heads in their target areas.

Furthermore two other facts contributed to make the situation worse:

- the curved route of the flowlines;
- the sea bottom soil conditions, allowing a low friction factor.

Consequently:

- the pipes tended to slip, therefore it was very difficult to correct the misalignments in the P.L. route by means of the lay-barge manoeuvring, as usual;
- a continuous control and monitoring of the T.D.P. by means of medium frequency transponders has been decided, avoiding the uncertainties due to the operational limits, in terms of environmental conditions, of the submarine.

3.3) The following are the details of the operation "laying through the platform area" (fig. 15).

As a first step, a transponder array of five elements has been activated; three have been directly laid on the 24" pipeline for these reasons:

- to avoid obstacles on the sea bottom, that could limit the lay barge anchor pattern;
- to make the calibration and the repositioning easier, should a misfunction occur.

The performed laying procedure required the following steps:

- positioning of the transponders directly on the flowlines (about

200 m. distance from each other) and on the lay down head (TF);

- calibration of the transponders, as soon as they reach the sea bottom, in order to know the real route of the P.L.;
- final calibration, during the abandonment, of the transponder (TF), in order to check the real length of the flowlines;
- recovery of a required line section, so that the connection of the abandonment cable to the initiation head of the next flowline can be performed;
- positioning of a transponder (T1) on the initiation head; the laying operations continue;
- continuous monitoring of (TF) and (T1) positions, during this lowering phase requiring the highest accuracy. The lay barge route has to be promptly adjusted, so that the T.D.P. can follow the required pipeline route;
- final calibration of (TF) and (T1);
- the laying operations continue (flowline no. 2).

4. <u>WEST SOLE GAS COMPRESSION PROJECT</u>

In conclusion a few words on this project:

a) <u>Shore approach</u>:

This phase is typical for all the sealines leaving from the shore and it requires that we can move with the lay barge as close as possible to the shore, in order to minimize the pipe length to be pulled on the sea bottom. Practically the lay barge is positioned at the minimum allowable draught and then the pipe head is connected to the cable of a pulling winch, located on the shore (single or double line pull).

b) <u>Laying operations in very shallow waters</u>:

In our case this requirement brought as a consequence the reduction of the laying ramp from the usual configuration of 80 m. length (two sections, internal and external, 40 m. each) to 40 m. (internal section only).

We point out that:

- the internal ramp has been completed with a so-called "terminal kit". This additional structure has mainly the function of measuring the loads (both in the vertical and in the horizontal plane) transmitted from the pipe on the last support to the load cells. The kit has also the subsidiary function of facilitating the abandonment/recovery operations;
- the laying operations in very shallow waters of pipelines with a large O.D., as in this case, call for high figures in the tensioning force when we try to minimize the ramp length.

Consequently in the system pipe+barge+anchor cables the pipe behaves as a very stiff element, then the following problems arise:

- a small positioning error of the lay barge greatly increases the stress levels in the pipe. Therefore a great accuracy is required both in the "station-keeping" and in the "laying run" operations;
- the control of the pull applied by the tensioner, with an established band width at the set-point, becomes much more difficult to be maintained.

5. CONCLUSIONS

In both "Magnus Field" projects 16 special operations have been performed:

- 5 initiations
- 5 terminations
- 3 crossings
- 3 layings in "follow-through" mode

A laying accuracy in the range of 1 m. has been consistently achieved. These results have been possible because of:

a) careful planning of the operations, based on the environmental discrepancies among the various vessels and system components, with different operational limits;

b) adequate preparatory approaches to the following phases:

- preliminary thorough survey on the P.L. route and adjacent areas;
- continuous monitoring of the laying operations;
- choice of extremely accurate positioning systems;
- integration between the two surface and sea bottom reference systems;
- computer programs for the stress analysis of the pipe-laying, available on board for continuous "cross checks" with the actual operational data.

APPENDIX 1

The automatic welding system "Passo" has the following main features:

a) the welding method is the traditional M.A.G. (Metal Active Gas) based on the use of motor driven wire and CO_2 , or mixture of argon and CO_2 ;

b) one d.c. constant voltage welding generator, which provides a two station output via a control unit;

c) two welding heads, one right hand and one left hand supported by the guide-band;

d) main control box, which provides the main control including the wire feed speed and the travel speed control units of the welding heads, which operate simultaneously on the same girth point with a downhill technique;

e) service cables assemblage including electrical power connection cables, compressed air feeding system at 4-6 Kg./cm. required to drive oscillator and the clamping and unclamping of welding bugs onto the guide-band;

f) one guide-band supported on pins and locked in place onto the pipe before it is loaded into the line of the pipe string. Accurate alignment of the band relative to the pipe is ensured by the use of a gauge;

g) in connection with the above equipment an internal line-up clamp, similar to a conventional type but properly modified, is used to perform welding.

The line-up clamp incorporates a segmented copper backing bar, which is expanded on completion of line-up operation to give a close fit-up on the underside of the joint preparation.

The main operational properties of the system are described below and can be identified in the following significant points:

- the torch, previously manually operated by the welder, is now replaced by a welding head moving along the pipe circumference on a guide fixed to the pipe;
- the operator can adjust the main parameters (carriage speed, wire speed, voltage control) within the established range from a control box located on the welding head;
- a pneumatic oscillator allows a high flexibility performing all the welding pass (from the root pass to the cap pass).

FIGURA 1

FIRST REASON:		**The whole of the activities performed.**
SECOND "	:	**The technical contents of each contract.**
THIRD "	:	**The new technological aspects.**
FOURTH "	:	**The integration between the lay-barge and the M.S.V. for underwater works.**
FIFTH "	:	**The customer.**

FIGURA 2

MAGNUS FIELD/OIL EXPORT PIPELINE

AREA:	61°40'	(Northern North Sea)
LINE LENGTH:	91 Km.	
from:	Target area "Magnus" (Pltf. not installed at that time)	
to:	Ninian Central Pltf.	
MAX. WATER DEPTH:	186 m.	(620 ft.)
PIPE DIAMETER:	24" O.D.	
PIPE W.T.:	15,88 mm.	
MATERIAL:	X65	
START OF THE WORKS:	10.05.81	
END OF THE WORKS:	29.06.81	
DURATION:	51 days	

FIGURA 3

MAGNUS FIELD/FLOWLINES

AREA:	61°40' (N.N.S.)
N. OF LINES:	7
TOTAL LENGTH OF THE LINES:	34,658 m.
from:	well heads
to:	target area "Magnus" (Pltf. not installed at that time)
PIPE DIAMETER:	6"5/8 O.D.
PIPE W.T.:	14,27 mm.
MATERIAL:	X65
MAX. WATER DEPTH:	189 m.
START OF THE WORKS:	22.08.81
END OF THE WORKS:	18.09.81
DURATION:	28 days

FIGURA 4

WEST SOLE GAS COMPRESSION PROJECT

AREA:	53°45'	(North Sea)
TOTAL LENGTH OF THE LINES:	68 Km.	
from:	shore	
to:	Pltf.	
MAX. WATER DEPTH:	33 m.	
PIPE DIAMETER:	24" O.D.	
PIPE W.T.:	15,88 mm.	
MATERIAL:	X65	
START OF THE WORKS:	10.07.81	
END OF THE WORKS:	11.08.81	
DURATION:	33 days	

FIGURA 5

Technical contents of the contracts:

MAGNUS OIL EXPORT PIPELINE

- Initiation procedure for the laying.
- Termination procedure in the Ninian Central Pltf. area.
- Spool Piece Installation.
- Crossings.

MAGNUS FIELD FLOWLINES

- Laying procedure purposely designed.

WEST SOLE G.C.P.

- Shore approach.
- Laying operations in very shallow waters.

FIGURA 6

OPERATIVITY OF THE M.S.V. "RAGNO DUE"

1. **MAGNUS FIELD O.E.P.L.**

   - **Contractual days 100 (from 4/5 to 11/8)**
   - **Working days 84**
   - **Standby W.O.W. days 16**

2. **MAGNUS FIELD FLOWLINES**

   - **Contractual days 120 (from 12/8 to 8/12)**
   - **Working days 76**
   - **Standby W.O.W. days 44**

FIGURA 7

ENVIRONMENTAL CONDITIONS IN THE MAGNUS FIELD (SURVIVAL, ALL SEASONS, AS PER D.N.V. RULES)

- **SIGNIFICANT WAVE HEIGHT** $H_S =$ **16 mt.**
- **AVERAGE ZERO UPCROSSING WAVE PERIOD** $T_2 =$ **11÷14,5 SEC.**
- **WIND (1 HOUR AVERAGE WIND AT 10 MT. ABOVE S.L.)** $V =$ **41 M/SEC.**
- **CURRENT (TIDAL + WIND DRIVEN VELOCITY)** $V_C =$ **1,3 M/SEC.**

FIGURA 8

MAGNUS O.E.P.L.

1. Preparation of three crossings over existing pipelines.

   aa) Trenched DUNLIN/CORMORANT P.L.

   bb) Trenched BRENT/CORMORANT OIL P.L.

   cc) Untrenched BRENT/CORMORANT GAS P.L.

2. Spool Piece connection at Ninian Pltf.

SURVEY AND MONITORING OF THE PIPE POSITION

Operations performed during the activities:

- Laying start-up
- Laying termination
- Crossings

FIGURA 9

CROSSING PREPARATION

A) - LAYING OF A TRANSPONDER ARRAY

B) - DIVE OF THE SUBMERSIBLE

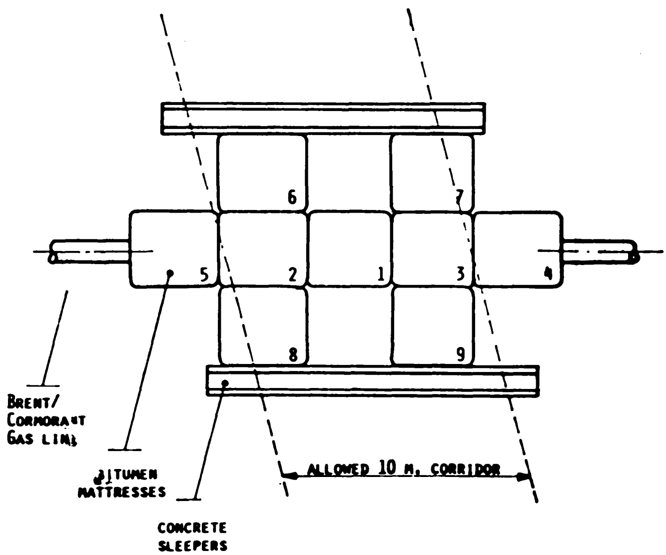
C) - CHECK OF THE DISTANCES

D) - LOWERING OF MATTRESSES

E) - LOWERING OF SLEEPERS

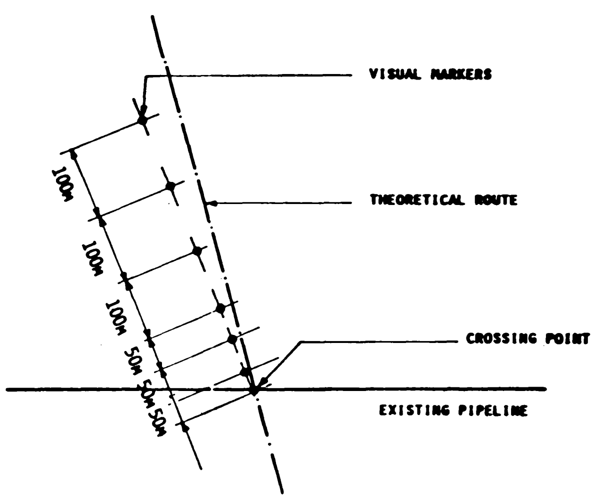
F) - FINAL SURVEY

FIGURA 10



- CROSSING PREPARATION

FIGURA 11



APPROACH TO CROSSING POINT

FIGURA 12

LAYING START-UP

1) - LAYING OF A TRANSPONDER ARRAY

2) - CORRELATION WITH AN EXISTING ARRAY

3) - FIRST CALIBRATION

4) - LAYING OF THE INITIATION ANCHOR

5) - LAYING OF THE INITIATION CABLE

6) - SECOND CALIBRATION

7) - REPOSITIONING OF THE CABLE

8) - CHECK OF THE DISTANCES

9) - RECOVERY OF THE CABLE

10) - CUTTING THE CABLE

11) - CONNECTION OF THE INITIATION HEAD

12) - LAYING START-UP

13) - MONITORING OF THE APPROACHING OPERATION

FIGURA 13

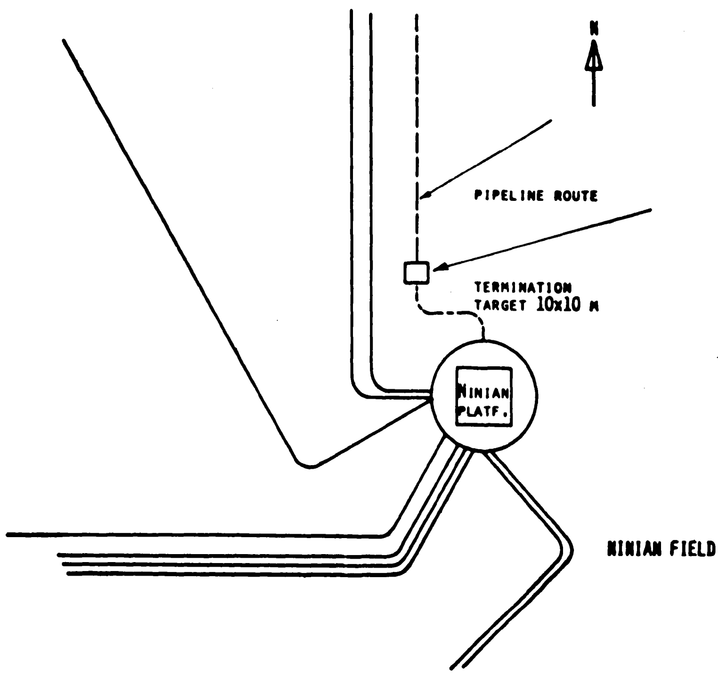
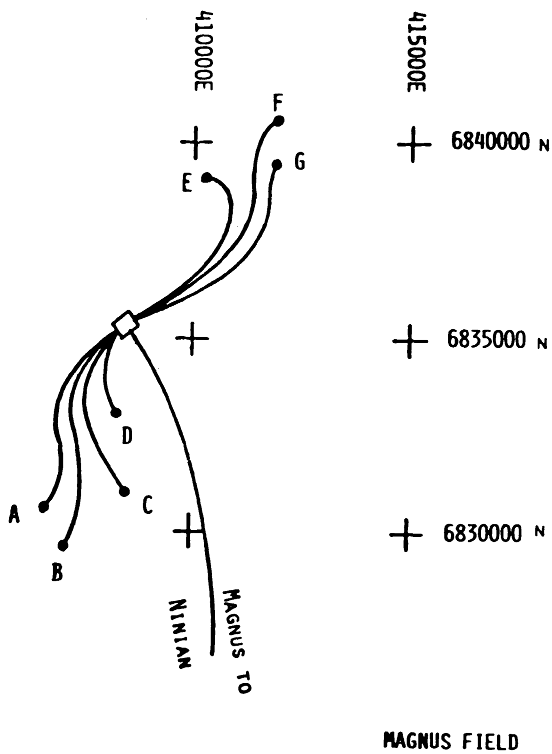
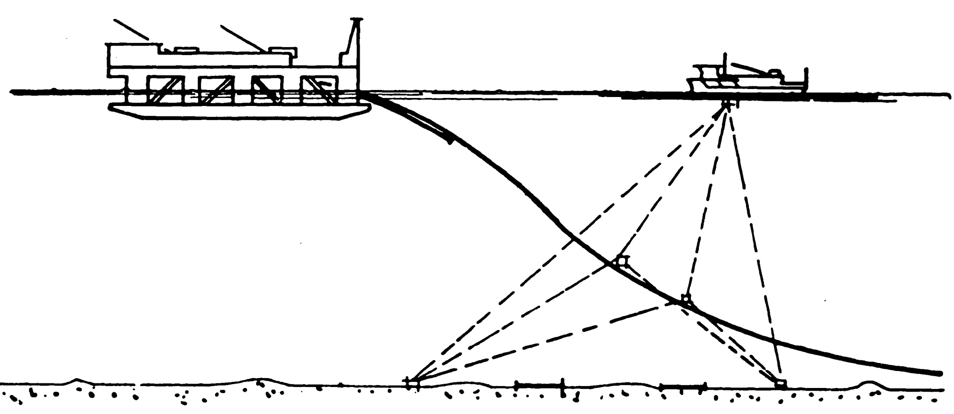

N
PIPELINE ROUTE
TERMINATION
TARGET 10x10 M
NINIAN
PLATF.
NINIAN FIELD

FIGURA 14



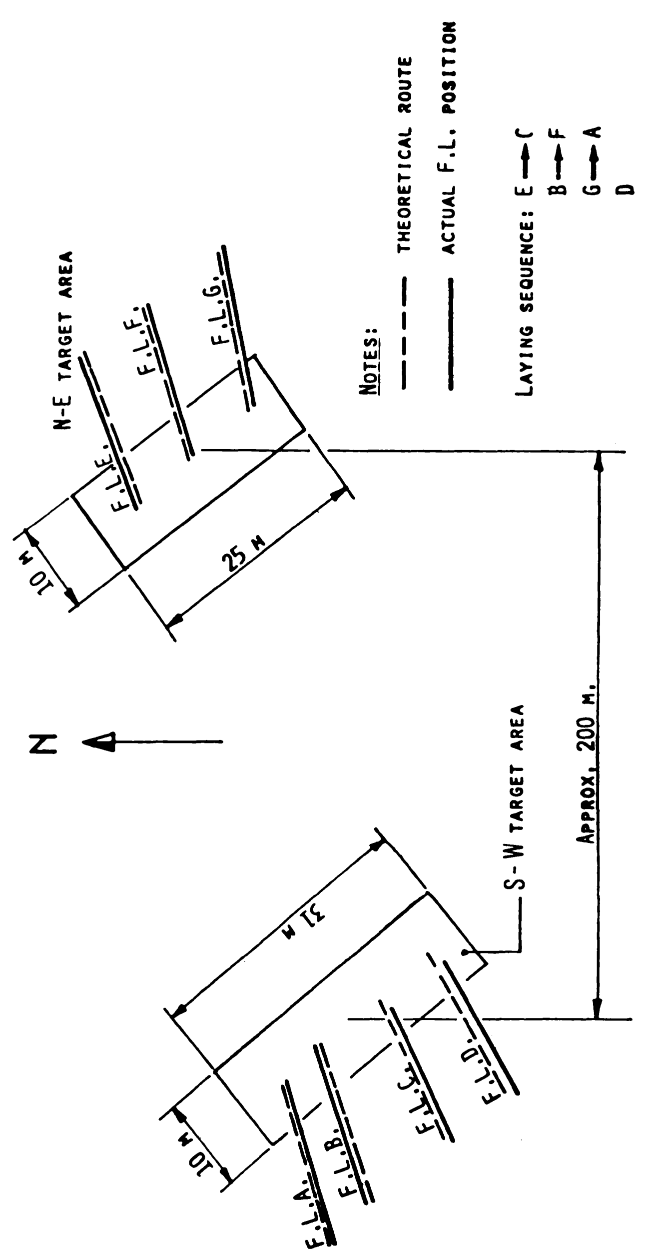
- MAGNUS FIELD FLOW LINES

FIGURA 15



- LAYING IN "FOLLOW THROUGH"

FIGURA 16



- FINAL FLOW LINES POSITION IN PLATFORM TARGET AREAS

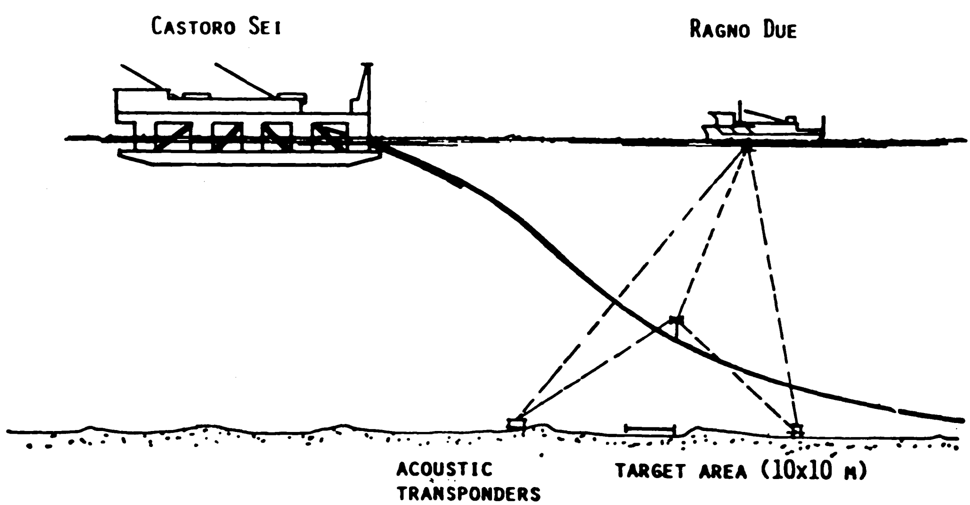
FIGURE 17

FLOWLINES PROJECT

DAILY WORKING SEQUENCE

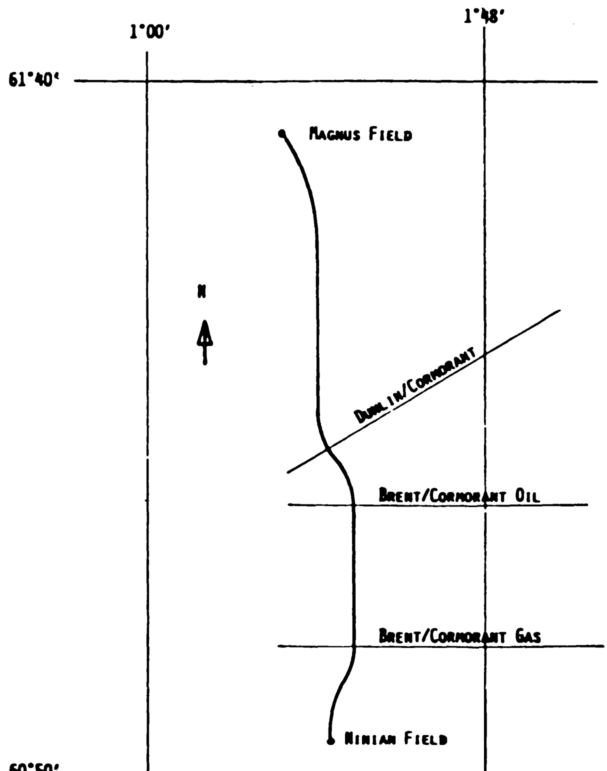
	FROM	TO	TOTAL
FLOWLINE E =	22/8	24/8	3 DAYS
FLOWLINE C =	26/8	28/8	3 DAYS
FLOWLINE B =	30/8	2/9	3½DAYS
FLOWLINE F =	2/9	5/9	3½DAYS
FLOWLINE G =	9/9	13/9	4½DAYS
FLOWLINE A =	13/9	15/9	2½DAYS
FLOWLINE D =	16/9	18/9	3 DAYS

FIGURE 18



- INITIATION

FIGURA 19



- MAGNUS/NINIAN MAIN LINE

A CASE HISTORY OF A MARINE TERMINAL

Y. Eprim
Polytecna Harris S.p.A.

INTRODUCTION

The designer of a marine terminal, although dealing with structures in relatively shallow waters (usually less than 30 m), is frequently confronted with a series of challenging problems and difficulties which he has to resolve or overcome, and which have become even more challenging in the recent years when operating companies experience a continuous reduction in their construction investment budgets and a steady alarming increase in the cost of materials and labour.

Although an engineering solution is always meant to be optimal, technically and economically, during the fifties and early sixties, in the marine field and particularly in the petroleum industry, little attention was given to developping solutions which were both technically and economically (as construction costs are concerned) attractive. The main objective was to put the plant on-stream as early as possible even if this caused, on one hand, cutting down on the planning and designing efforts, and on the other hand, adopting a straight forward heavy construction type structure. Naturally this led to overdesign of the principal sections with huge construction costs, but in some cases it also involved neglecting some serious engineering problems such as secondary stresses, fatigue and corrosion phenomena, and foundation problems.

As it is well known, very heavy construction does not necessarily overcome such problems.

The type of construction and engineering approach have undergone substantial changes in the recent years, due to lower investment budgets and due to the new achievements in the field of marine/offshore engineering such as a better and deeper understanding of the meteo-marine conditions, and the development of sophisticated computerized programmes specifically elaborated for analysing marine structures.

The case history herewith presented consists of the Abu-Dhabi Gas Project marine terminal at Ruwais which was constructed and successfully completed by SAIPEM in 1981.

GENERAL DESCRIPTION

The terminal consists of a jetty approx. 3 km long, products loading and auxiliary platforms, and four breasting and six mooring dolphins, with interconnecting catwalks and walkways.

Because cryogenic liquids were to be handled, two auxiliary platforms, located at a safe distance from the product loading area, were required to support the blowdown tanks and equipment and the utilities plant.

Due to the flat sea bottom profile in the area, the berthing line had to be located at some 3 km distance from the shoreline to provide the required water depth at the berth and in the manoeuvring area.

In order to moor the ship heading into the resultant direction of prevailing wind and wave action, and in order to keep the length of the jetty to the minimum necessary, the berthing line resulted inclined 55 degrees with respect to the longitudinal axis of the jetty instead of the usual "T" at 90 degrees.

ENGINEERING APPROACH

The design of the terminal was based on a number of conditions and parameters dictated in part by the Client and in part by the current engineering practice in the marine/offshore field.

In addition to the Client's general and particular requirements, the following main parameters were rigorously analyzed, and integrated to form the basis of the design :

- terminal duty
- primary function of each structure or element
- sea bottom soil properties and expected foundation behaviour
- governing meteomarine conditions and consequential forces
- method of construction and choice of materials.

While the effect of the first two parameters on the design was clear and straight-forward, the last three parameters were interrelated and therefore were analyzed in parallel in order to achieve an optimum solution.

The terminal duty was basically to provide the necessary facilities for loading propane and butane on gas carriers varying in size up to

120.000 cum, and pentane on tankers of up to 65.000 DWT in size.

The effect of the terminal duty on the design can be summarized as follows :

- higher safety requirements because gas carriers are handled;
- larger areas for piping required because of the necessity to keep all the product lines under direct visual inspection and to allow easy access for repair and maintenance work;
- additional service platforms required in a safe area for housing the auxiliary equipment connected with the LPG loading system.

In the following three chapters, the governing meteomarine conditions, the sea bottom soil properties and the method of construction, are given.

The primary function of each structure and the effect of the other four parameters on the design are introduced in the last chapters, whare each main structure is separately described.

GOVERNING METEOMARINE CONDITIONS

Besides wave and wind actions, the tidal excursion, being significant, had to be given special attention. Wave action on structures was analyzed using computer programmes; wave loads on each structural element were computed and the induced stresses calculated.
The relatively strong tidal excursion required that the platforms and jetty decks be placed on high levels even if the wave action by itself was not so strong.

The sea bottom profile was not favourable for construction. The approx. 600 m long reef zone characterizing the near shore surface morphology had a direct influence on the structural dimensioning and method of construction of the jetty in that area.

SEA BOTTOM SOIL PROPERTIES AND PILE BEHAVIOUR

According to the results of the soil investigation campaign commissioned by the client prior to the tendering period and made part of the tender documents the sea bottom soil consisted of a thin layer of sand overlying a weak weathered rock formation basically siltstone. Although this formation offered good bearing capacity for compression loads, it did not guarantee sufficient tension resistence.

Moreover the results of unconfined compression tests carried out on the rock samples indicated strengths not superior to 40 kg/cm2. It was therefore concluded, based on dynamic analysis of pile driving, that a Delmag 55 diesel hammer would be capable of driving the piles to the required penetration.

Because of the expected heterogeneous nature of the weak rock formations, a second soil investigation campaign was later on commissioned. The results of this investigation showed that in some cases the unconfined compression strength of the rock formations exceeded 80 kg/cm2, and that it may not be possible to drive the piles by hammering only to the required penetration using the Delmag 55 hammer. It was therefore decided to use a more powerful hammer, and take into consideration the possibility of having to drill from inside the pile to assist in achieving the required rock penetration.

Because of the inherent difficulty in predicting pile behaviour in such rock formations, a series of pile driving and load tests were carried out to establish the most appropriate piling procedure so as to satisfy the compression, tension, and bending moment requirements. The latter parameter required that piles be embedded in the rock formations to a minimum depth of 4.5 m near the shoreline and to 6.0 m further offshore.

The testing confirmed that in some cases the minimum pile penetration could not be achieved by driving only and that drilling from inside the piles would have to be carried out to assist in driving the piles to the minimum required rock penetration. Based on the results of the dynamic analysis of pile driving correlated with the results of the pile load tests, two blow-count levels were established: one which defined the minimum blow-count which must be reached provided the minimum penetration in rock was achieved; and one which defined the refusal and represented the blow-count not to be exceeded to avoid overstressing the pile and/or damaging the hammer.

When during driving the refusal blow-count was reached before the required minimum penetration in rock had been achieved, pile driving was interrupted and the following procedure adopted :

<u>For piles under compression load</u> :

- Piles were cleaned internally by air lifting system
- Pilot holes 15 cm smaller than the pile diameter, were drilled to the required penetration
- Piles were redriven until their tips reached the bottom of the pilot hole or deeper and the minimum blow-count was achieved.

In some cases the limiting refusal blow count was reached before achieving at least the bottom of the pilot hole.
In these cases (provided pile penetration in rock was at least 2.5 m) the following procedure was adopted :

- The inside of the pile and pilot hole were thoroughly cleaned.
- The pile and pilot hole were filled with tremmie concrete up to a level of 2.5 m above the pile tip.
- Before setting of concrete took place, a pipe smaller than the pilot hole was inserted with its tip down at the hole bottom and its top projecting few centimeters above the concrete level.

The purpose of the pin pile was to absorb the residual shear and bending moment, while the main pile supported the vertical compression load.

If the main pile stopped short of 2.5 m rock penetration, then the following procedure had to be adopted :

- pile and pilot hole were cleaned and the hole enlarged to 10 cm more than the main pile diameter by underreaming down to the required penetration.

- the hole was filled with tremmie concrete up to a level of 50 cm above the pile tip
- before concrete setting, the pile was driven into the under-reamed hole until the minimum required blow count was reached.

During all these phases of this procedure, the pile was to be held in position by an appropriate holding device. The holding device which had to be connected to the pile by means of a clamp ensured stability during driving/drilling operations, and held the pile firm in position during the setting time of the concrete.

For piles under tension load

The tension piles which were basically the piles driven into the jacket structures, were planned to be installed with the following procedure :

- After positioning the jacket, the pile would be inserted into the template and driven to a penetration of about 1 m in the rock.
- The inside of the pile cleaned by air-lifting.
- An under-reamed hole (15 cm larger than the pile outside diameter) drilled to 6.5 to 9.5 m below the pile tip level (the 9.5 m penetration was required for the dolphin structures which carried higher tension loads).
- The hole filled with tremmie concrete.
- The pile driven through the fresh concrete down to the bottom of the hole.

The depth of the under-reamed hole was determined assuming a grout/steel ultimate bond stress of 5 kg/cm2 and was verified by tension load tests.

However, because of the difficulty in drilling underreamed holes in such formations, and the inherent doubt in the tension carrying capacity of the system, it was later on decided to drill pilot holes, fill the hole and part of the pile with tremmie concrete, and insert a smaller pipe inside the pile down to the required depth.

In addition it was required that the pin pile, and the main pile were permanently connected by welding to ensure the tension carrying capacity of the structure. This procedure involved complicated operations but was imposed by the client based on the assumption that the bond strength between the pin-pile and the grout, and between the grout and the rock formation did not give sufficient guarantee for high tension load bearing capacity.

METHOD OF CONSTRUCTION AND CHOICE OF MATERIAL

The method of construction was based on the hypothesis of prefabricating abroad as much as possible, and further limiting the amount of field work by adopting large pre-assembled units and by reducing the number of piles as much as possible. Because of a rather long reef zone which did not permit floating equipment to work near the shoreline, a temporary causeway was built from shore to the -5 m bathymetric line.

In this area, the jetty piles and bridge elements were to be installed with land based equipment working from the causeway. The remaining major portion of the jetty was planned to be constructed using the heavier barge mounted equipment.

Because of the difficulty of finding locally suitable aggregates and cement and considering local conditions, steel was preferred to concrete. The choice of steel was also favoured because of the intention to prefabricate abroad entire units or major structural parts and transport them to site ready for erection.

THE JETTY

The function of the jetty is to support the products transfer pipelines, utility lines and a vehicle carriage way. It was designed to meet the following particular requirements.

- Pipelines to be laid out, arranged and supported so as to permit direct inspection and easy access and to avoid high stresses due to own weight and thermal variations.
- Long bridge spans to reduce the number of piles which were expected to be costly and time consuming.
- Notwithstanding the above requirements, the weight of the bridges had to be compatible with the capacity of the handling and lifting equipment and with the method of transportation.

Analysis and weighing of these requirements yielded in rather wide trussed bridge elements most of which were 43.4 m in length and 96 tons in weight.

Because the method of construction for the near shore part of the jetty was to use the causeway to drive the piles and install the bridges, the span in this area was reduced to 24.4 meters so that these could be handled by land based transporting and lifting equipment.

The bridge trusses were composed of rectangular sections as the main chords, and tubular sections as diagonal, vertical, and horizontal bracing elements.

Because of the large width of the bridge trusses, three bearing supports had to be used at each end. This caused some difficulty in the erection of the jetty elements as it reduced considerably erection tolerances. This difficulty was further increased because of the effect of ambient temperature variations on the structures which rendered the setting up work of the theoretical positions of the bearing plates very tedious.

In order to reduce the friction forces due to pipelines and structures expansion and contraction, all bearing supports were designed with teflon plates. However, right at the start of the works, it was observed that the upper teflon plate (attached to the underside of the top bearing plate) came off the steel plate at the edges. It was found out that the cause was the type of the teflon material used which was not resistant to the ultraviolet rays.

Having changed the type of the teflon, the supports held quite satisfactorily, and inspection made during the commissioning phase revealed that the supports functioned properly.

THE LOADING AND AUXILIARY PLATFORMS

The main function of the loading platform was to carry the bank of six 12" loading arms, fire fighting monitor, and ship access gangway. The structure had to be solid with no appreciable deflections even when exposed to the wind and wave dynamic actions.

Vertical piles supporting the deck structure were therefore excluded, and because it was expected that drilling would be necessary to drive the piles to the required penetration, a jacket type structure was preferred which provided lateral rigidity to the platform and facilitated the piling operations.

The jacket structure was supported on and anchored to the underlying rock formation by eight 30 inch tubular steel piles. The jacket legs had a diameter of 36 inch.

The jacket supporting the deck was designed and prefabricated in two separate parts, each part of four legs only, to facilitate transportation and handling. The two jacket elements incorporated trunks of interconnecting elements which were permanently connected by telescopic joints after installation. A similar type of structure on four piles was used for the Blow-Down platform.

THE MOORING AND BREASTING DOLPHINS

Since the early seventies, in certain cases, these structures have been designed as flexible monopile structures having diameters as large as 3.0 meters and weighing more than 100 tonnes. The monopile type is usually fast to execute and involves a very simple superstructure. In this case, however, the monopile solution was not very attractive because of the sea-bottom soil characteristics. The limited thin sand layer and the frequently strong and superficial rock layers might not have permitted driving large diameter pile deep enough to develop sufficient stability to allow drilling from inside the pile to take place, unless costly platforms were used to hold the pile and support the drilling rig.

Consequently the choice of the jacket type dolphin became more attractive. Four leg jackets were used for the breasting as well as the mooring dolphins.

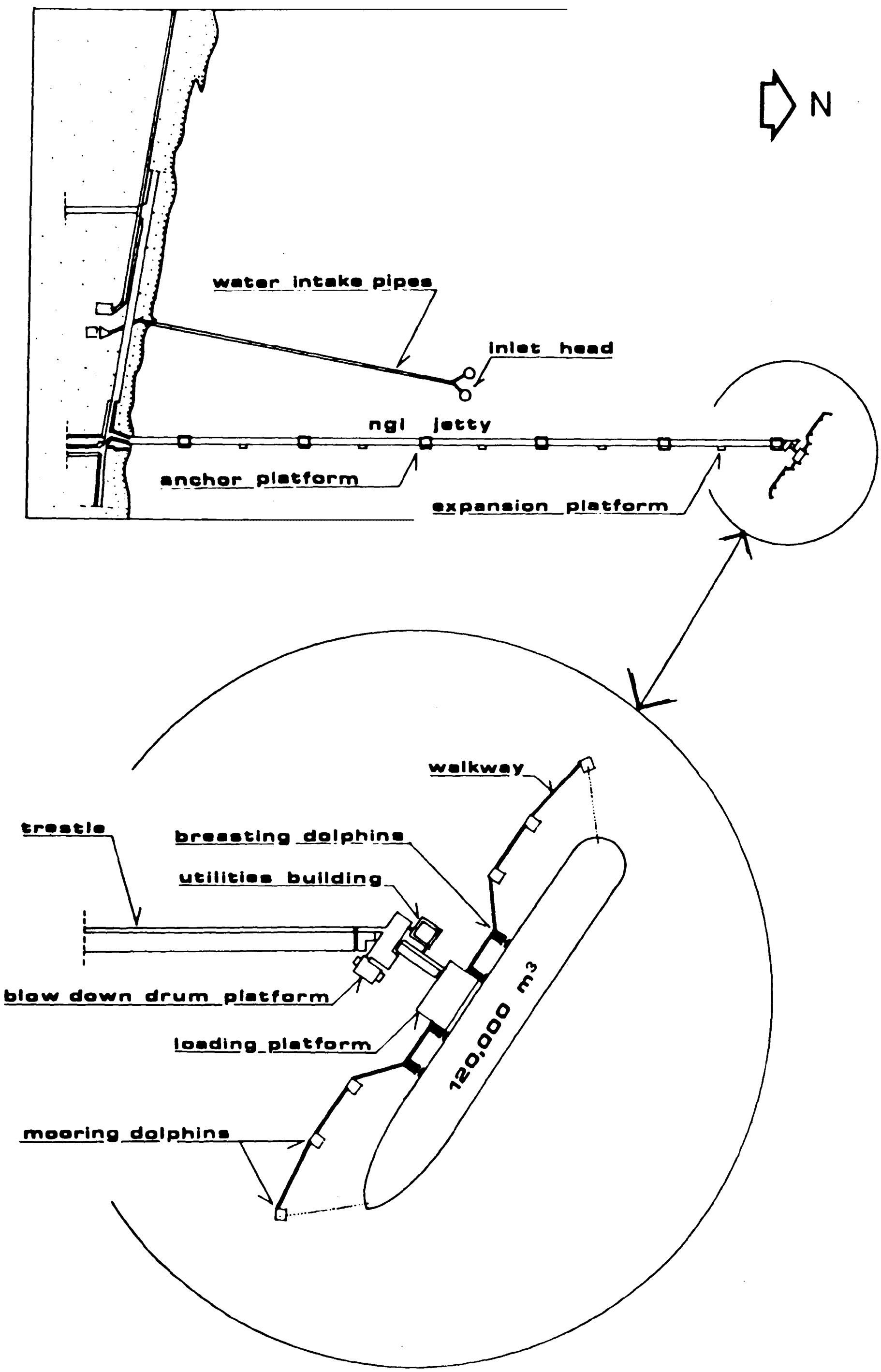
Since the breasting dolphins were of the rigid type structure, the berthing energy was totally absorbed by rubber fenders topped by a protector panel covered with very low friction coefficient resin boards.

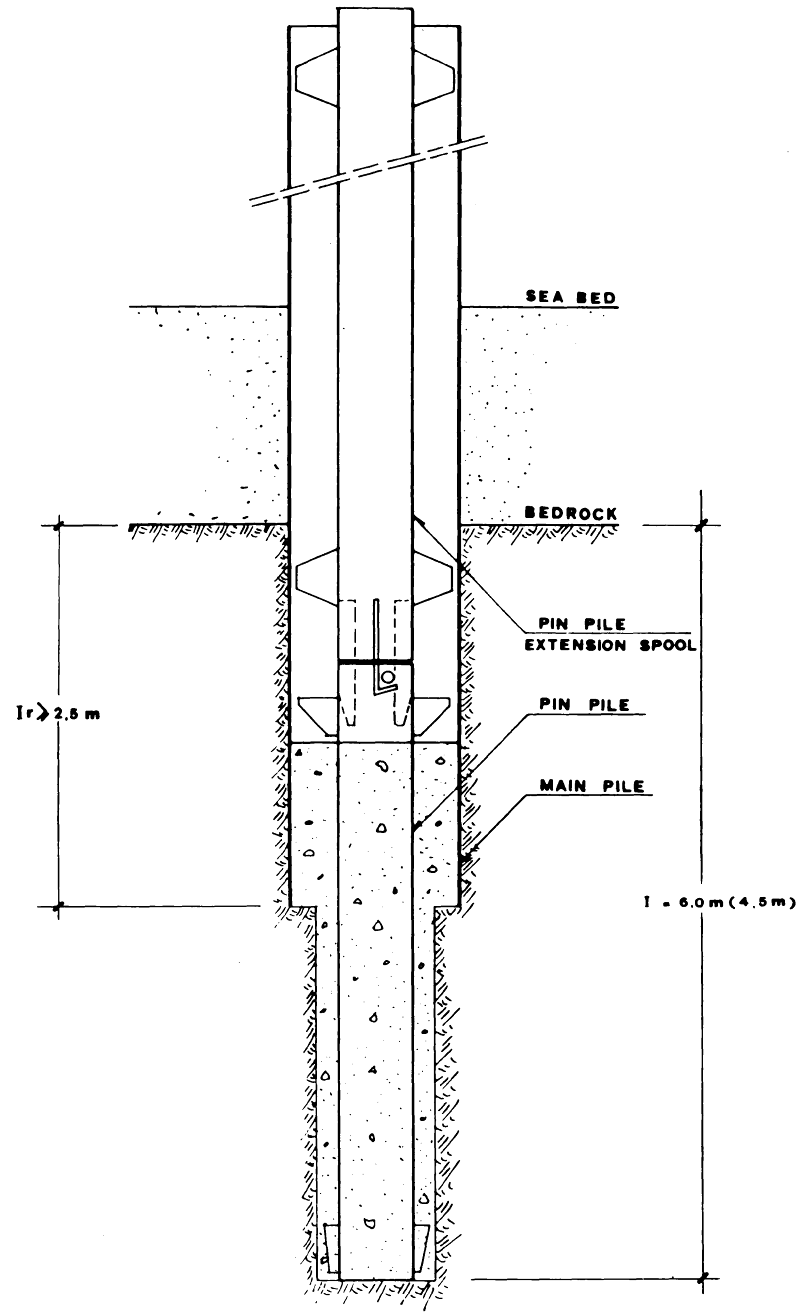
CONCLUSIONS

It can never be sufficiently emphasized that it is of utmost importance to collect and obtain extensive data on the site conditions in the early stages of project development.

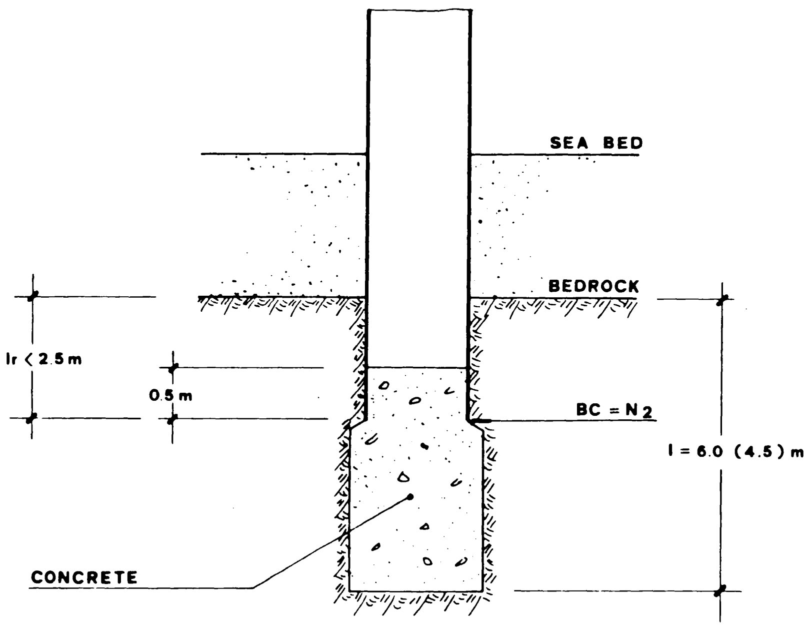
A lot of time and money can be saved by clients and contractors if the meteomarine and soil conditions are properly investigated and defined prior to the signature of construction contracts.

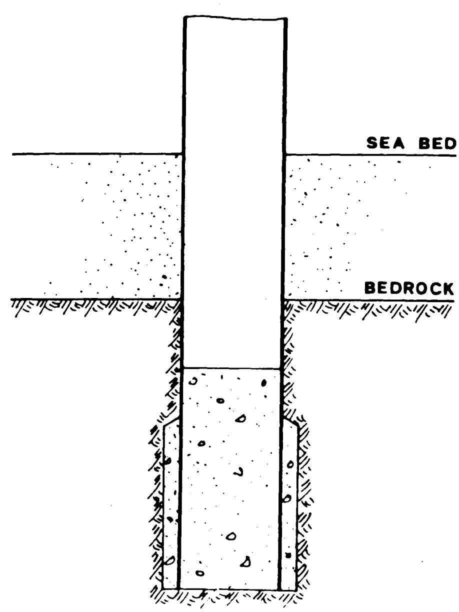
Even with the F.I.D.I.C. rules and conditions, (not frequently applied in the Middle East and North African countries), serious controversies between clients and contractors on foundation works have occurred in more than one marine project.


N
water intake pipes
inlet head
ngl jetty
anchor platform
expansion platform
walkway
trestle
breasting dolphins
utilities building
blow down drum platform
loading platform
120,000 m3
mooring dolphins

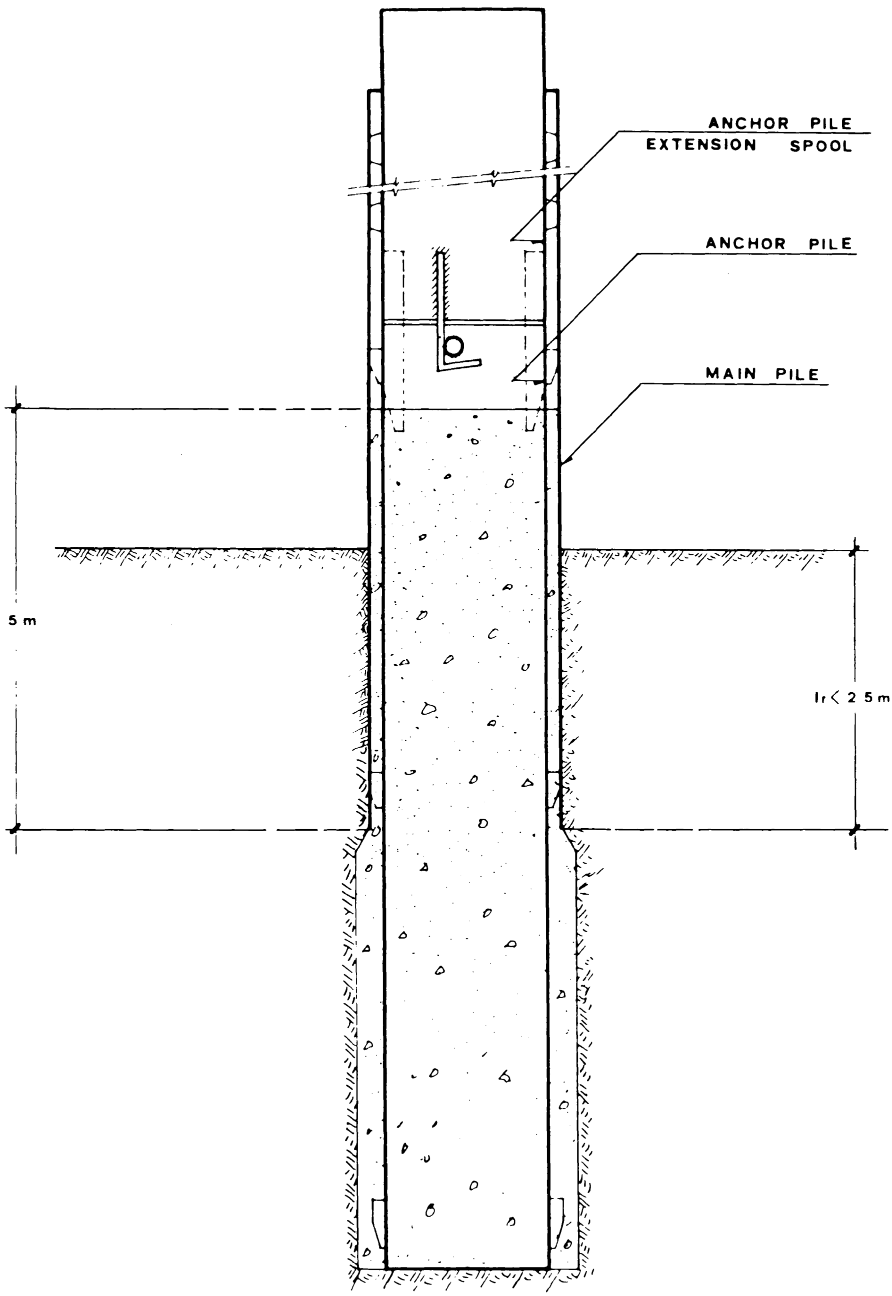


DRIVING TO AT LEAST 2,5m IN ROCK
THEN PILOT HOLE AND PIN PILE

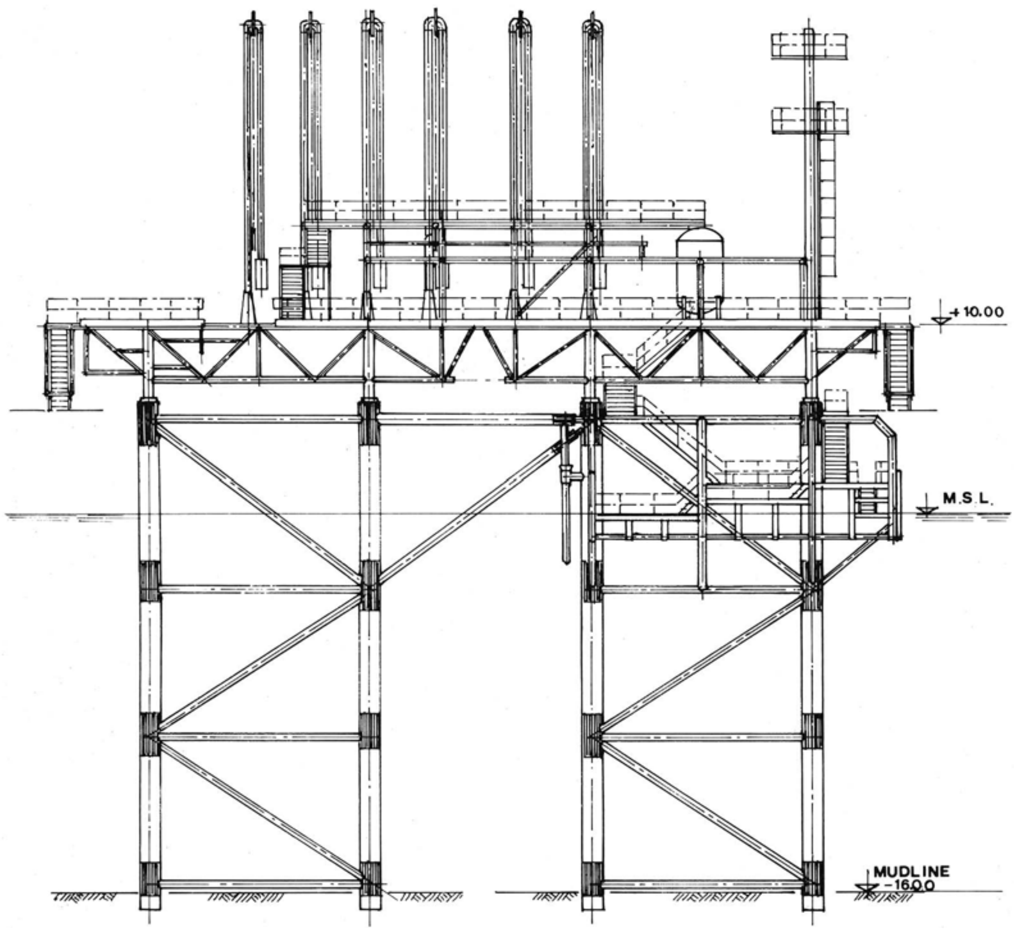




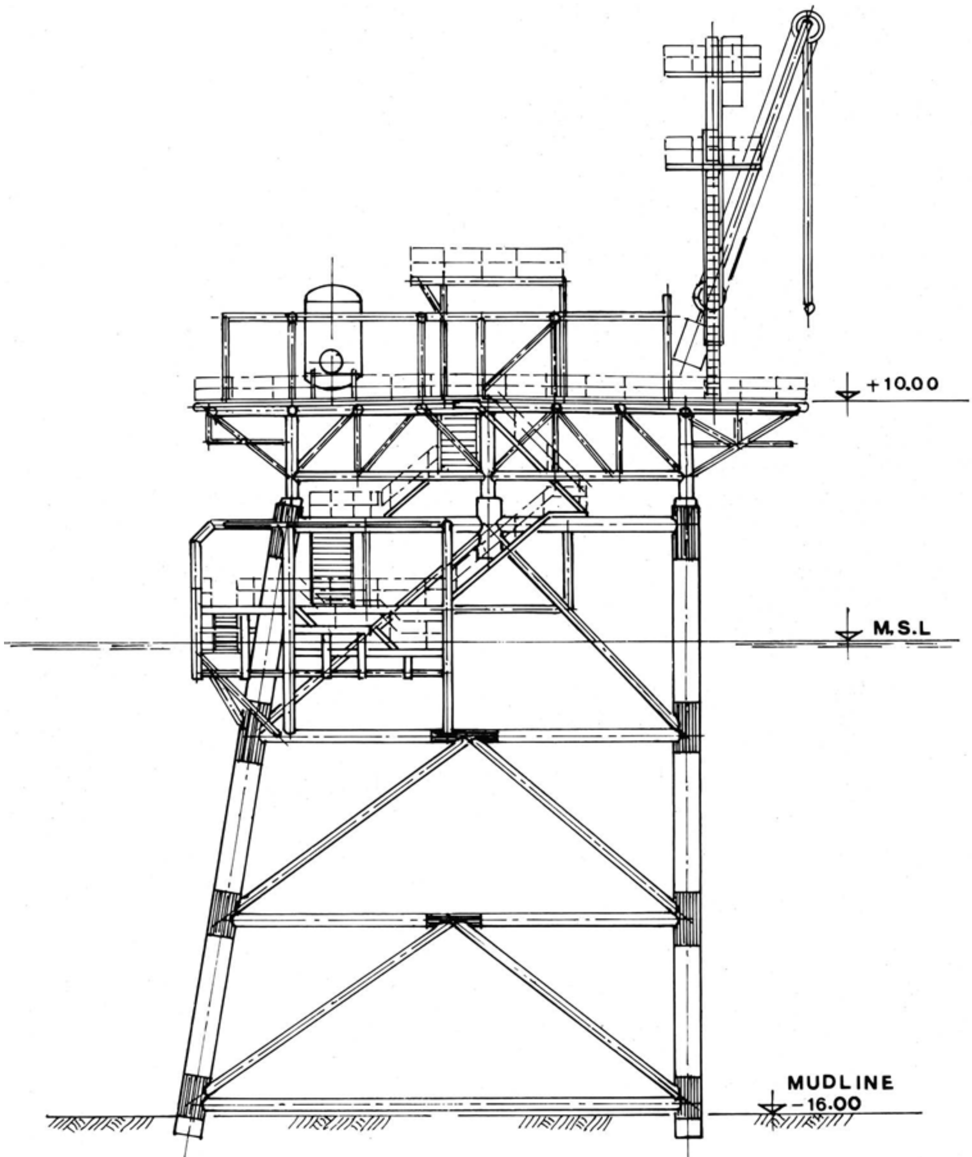
DRIVING TO LESS THAN 2,5m IN ROCK THEN UNDER-REAMING AND DRIVING THE PILE TO BOTTOM OF HOLE



ANCHOR PILE
EXTENSION SPOOL
ANCHOR PILE
MAIN PILE
2 5 m
lr < 2 5 m

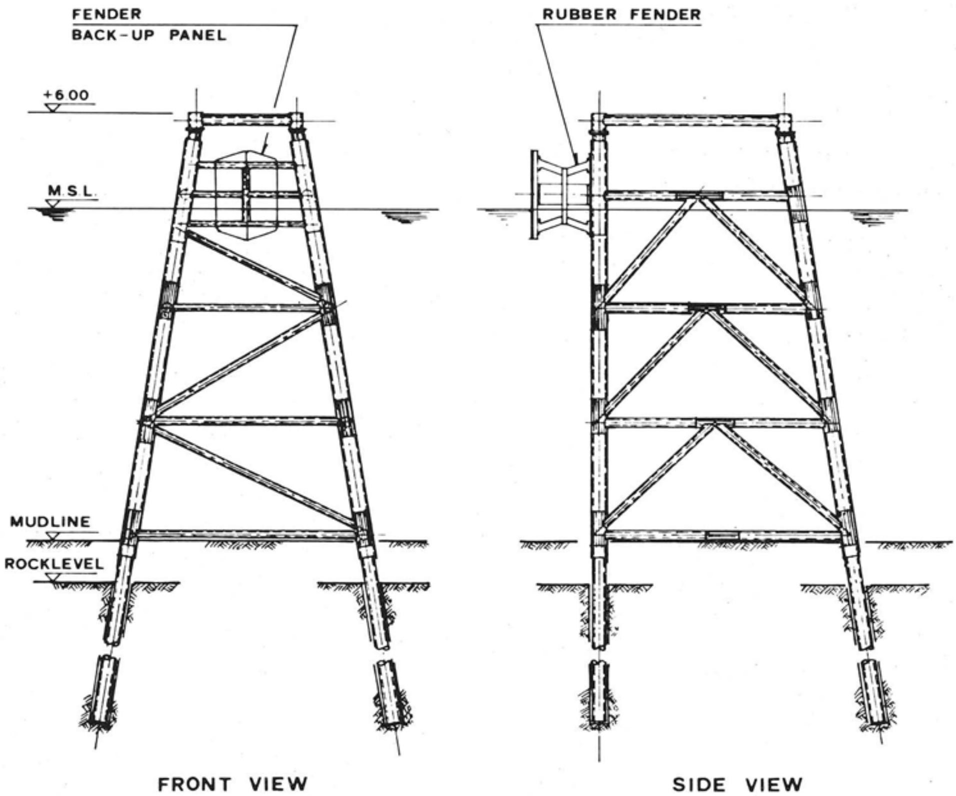


LOADING PLATFORM
FRONT VIEW

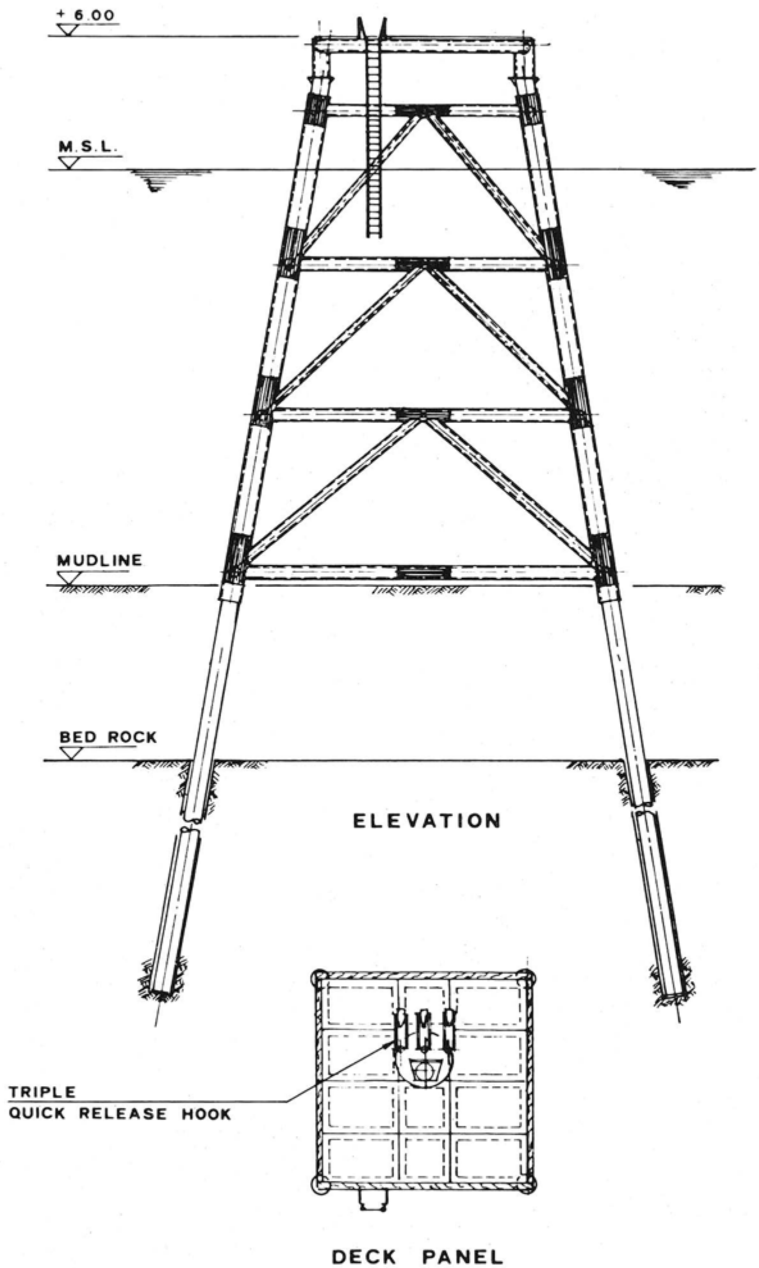


LOADING PLATFORM

SIDE VIEW



BREASTING DOLPHIN



TYPICAL MOORING DOLPHIN

DEVELOPMENT IN
INSPECTION AND MONITORING
OF OFFSHORE STRUCTURES

G. Sebastiani

Tecnomare S.p.A.

INTRODUCTION

Requirements for Platform Inspections

Requirements and recommendations for underwater inspections are generated by:

- National Authority
- Classification Society
- Oil Company/Platform Operator
- Vehicle Operator/Inspection Company.

North Sea is today the sole area in the world where a complete set of inspection regulations is well defined.
These regulations have been issued mainly by DnV and Lloyd's.

General requirement for underwater inspection is to obtain the same inspection standard as above water. Normally the Oil Company, soon after the installation of a platform, presents an Inspection Program to the

Certification Body for Approval.
The Certification Society requires the approval of, not only the whole inspection program, but also some specific items like:

- inspection personnel
- underwater inspection equipment
- inspection method
- reporting procedure.

Inspection Classification

Det Norske Veritas considers the following three types of inspections for steel platforms:

GREEN INSPECTION

A general visual survey using divers or ROV or manned submersible to find obvious damages.
The working tasks are normally:

- drive ROV along route inside or outside jacket
- general TV inspection
- cathodic protection measurements.

BLUE INSPECTION

A survey to detect hidden damages, where cleaning is required.
It includes, normally, the following working tasks (and relevant documentation):

- drive ROV along route inside or outside jacket
- move ROV close to node
- remove debris to gain access to inspection location
- remove marine growth and clean to bare metal
- general TV inspection
- close TV inspection

- still photography
- cathodic protection measurements.

RED INSPECTION

A BLUE Inspection, requiring non-destructive testing.
It includes normally the following working tasks (and relevant documentation):

- all tasks of BLUE Inspection
- grinding of steel structure
- material thickness measurements
- crack detection through NDT.

Inspection Program (DnV)

It normally consists of a Long-Term Inspection Program with Periodical Survey and Special Survey when required.

PERIODICAL SURVEY (normally ANNUAL)

- General GREEN Inspection
- BLUE and RED Inspection on local areas (selected joints, zones, members, and components)
- Inspection, as needed, of the foundation (scouring, etc.).

LONG TERM SURVEY

The whole structure should be covered in a period of 5 years, i.e. before renewal of Certification of Approval.

SPECIAL SURVEY

In the event of accident or discovery of damage that may affect the short-term safety of the structure, a special survey may be required. This is approved and surveyed by the Certification Society.

Typical Annual North Sea Platform Inspection Program and Cost (Source: BP)

		TIME (Days)	COST (Mil.Lit.)
1) AIR DIVING	- Cleaning - Visual Inspection - N.D.T.	30	750
2) R O V	- General Survey	10	25
3) SATURATION DIVING	- Standard and detailed inspection	15	930
	- Demob	5	310
	TOTAL COST		2,000

Development in Underwater Structural Inspections

There is today a growing demand for new technology in offshore inspection activity, mainly devoted to reducing or substituting the work of the diver in underwater complex tasks.

Research and development in this area follow two different lines:

- Development of advanced vehicles capable of performing complex inspection works
- Development of structural monitoring systems, capable of giving information on structural integrity and for a rational and cost-effective inspection program.

In the following sections present status and future trend in these two sectors of activities will be summarized.

VEHICLES FOR STRUCTURAL INSPECTIONS

At present many Companies have already developed different types of underwater manned or unmanned vehicles. But most of them are general purpose (for different surveys or works) vehicles, and they have no capability of performing tasks so specific and so complex as required in standard structural inspection (BLUE and RED inspections of the nodes). Some existing manned and unmanned vehicles are shown in Figs 1 and 2. Typical operational approach is shown in Fig.3.

Today the unmanned approach is considered the most promising (in respect to cost-effectiveness).
The ROVs (Remotely Operated Vehicles) include two main categories:

- tetherless ROVs
- tethered ROVs.

In respect to the tethered vehicle, the tetherless has the following advantages/disadvantages:

Advantages : reduced hydrodynamic forces and no risk of cable entranglement

Disadvantages : control and sensor data must be transmitted through the water and an autonomous power supply must be carried on board.

Certainly today the tetherless approach presents many more unknown factors than the tethered one. In any case, to obtain a ROV capable of performing inspection work at industrial level (i.e. capable of substituting the diver) the following basic requirements are envisaged:

- to specialize the vehicle for the specific detailed task (shape, navigation, positioning, manipulation, etc.)
- to give to the vehicle a right level of autonomy (robotics) so the man-machine interaction will be done at a sufficiently high level.

The most critical areas for the development of such a vehicle are:

- navigation to the target
- docking system and procedure
- cable handling if tethered
- power and hydroacoustic communication if tetherless
- cleaning the welding area to bare metal
- execution of the NDT.

An accurate systematic analysis of the mission and of the environment in which the vehicle shall operate should be made beforehand to establish the ROV configuration. Complexity of the structure geometry, marine growths of various nature unforeseen obstacles must be carefully taken into consideration in the early design phase.

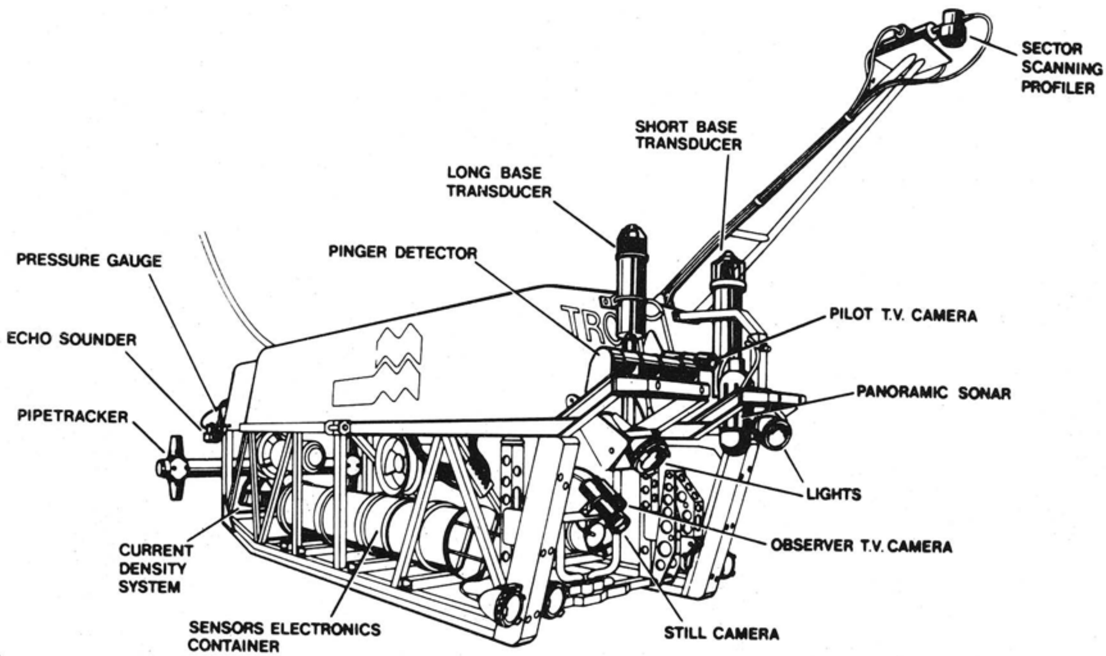


FIGURE 1 : TYPICAL REMOTELY OPERATED VEHICLE
(by courtesy of Intersub)

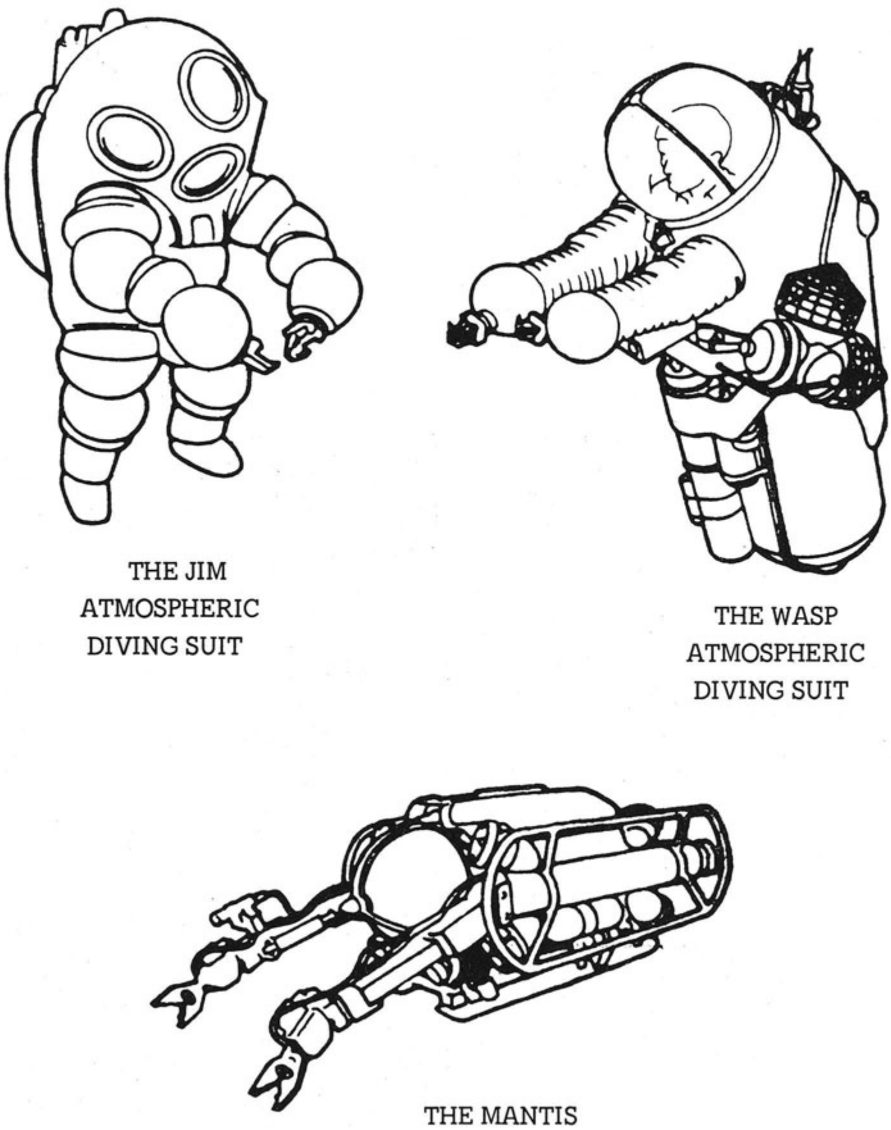


FIGURE 2 : EXAMPLES OF MANNED VEHICLES

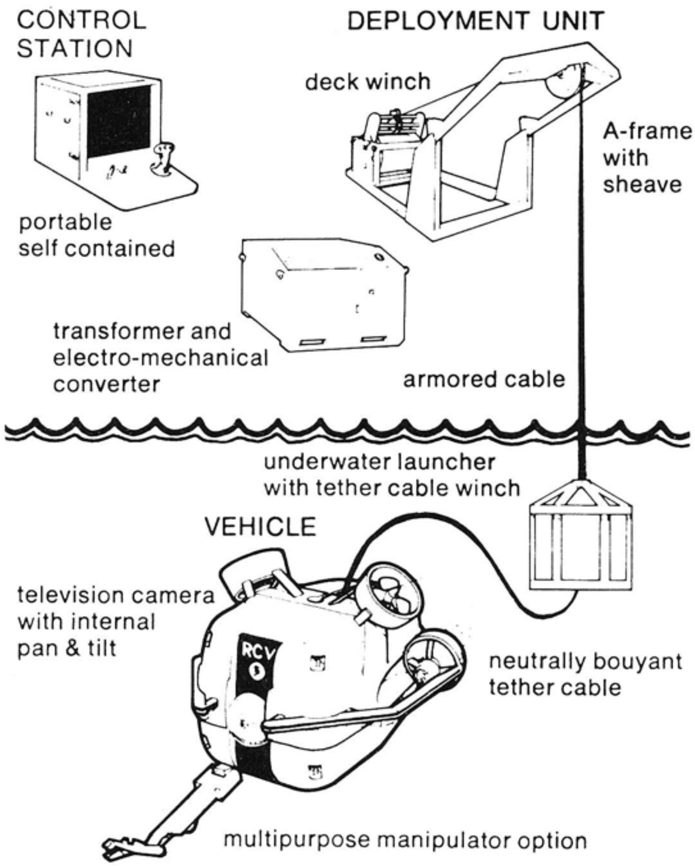


FIGURE 3 : OPERATIONAL SCHEME (by courtesy of Hydro Products)

STRUCTURAL MONITORING

General

The objectives of a Structural Monitoring system (SM) are:

- to check in field if the structure and its foundation behave in agreement with the design assumptions and calculations
- to give information on the structure integrity during the life
- to account the actual fatigue damage on the structure
- to give updated information for efficient and optimized inspection planning.

A typical SM includes one or more of the most promising detection methods, i.e.:

- vibration monitoring
- acoustic emission
- fatigue account.

From operational point of view, there are two cathegories of SM:

- permanent SM
- transportable SM for periodical or occasional checks.

In the following sections some considerations on the actual status and new developments of vibration monitoring will be given.
This method includes three different approaches:

- global monitoring

- flexibility monitoring
- local monitoring.

Global Vibration Monitoring

This is based on an accelerometric unit located on the deck and uses the ambient as excitation.
Advantages of this approach are in the low costs and very easy operational requirements.
Disadvantages are in the low sensitivity/resolution in terms of natural frequency variations versus cracks; this means that only major damages (i.e. primary structural elements failure) can be detected.

One interesting application of permanent SM based on global vibration monitoring is that of BARBARA platform in Adriatic Sea, in 70 m w.d.
The analysis of the collected data (Fig. 4) shows that 4 natural frequencies have been detected even if they are out of the range of the wave frequencies.

Flexibility Monitoring

This new method, proposed by Dr S.Rubin, is based on one accelerometric unit on each bay and ambient excitation.
The detection method is based on the analysis of the mode shape modifications when the structure is damaged.
This method is much more sensible than the global vibration approach and it is able to identify the zone of the platform where the damage has occurred.
In Fig. 5 the performance of the flexibility method is shown, in comparison with the conventional global vibration method, based on the natural frequency shift.

Local Vibration Monitoring

A local vibration monitoring is based on the analysis of the local vibration modes and their modification due to possible damages.
Normally the system comprises an underwater exciter and one or more underwater accelerometric units.
Generally the system is transportable and is used for periodical or occasional integrity checks on existing structures.

Two different approaches have been developed up to now:

- the first is based on member by member approach
- the second based on mode by mode approach; a special system called VIBRACHECK has been developed by Tecnomare for this purpose.

Vibracheck System (Fig. 6)

This system has been developed by Tecnomare and DnV as part of research project DMOS (Diagnostic Methods of Offshore Structures), sponsored by AGIP S.p.A. and NORSK AGIP A/S.

The method adopted is based on the underwater monitoring of vibrations induced by local excitation.
The system which has been developed is composed mainly of three underwater triaxial accelerometer heads and electronics for data acquisition, an underwater electro-hydraulic exciter, a surface container equipped with data recording and control electronics.

The choice of forced local vibration monitoring is derived by theoretical and experimental analysis.
A systematic analysis of all branches of vibration monitoring (global/local behaviour of the structure under natural/forces excitation) and extended tests on a jacket model showed that local vibration monitoring may be used to detect cracks above a threshold of about a quarter of the member circumference.

Some results of the experimental tests carried out in the DnV laboratories are shown in Figs 7 and 8.

An in-house computer analysis of the structure using modal analysis techniques is performed in order to select the best excitation and measurement points (Fig. 9).

On the basis of this selection, field measurements are carried out.
The measurement campaign is completed with an exclusive in-house diagnostic computer analysis to identify size and location of possible damage.

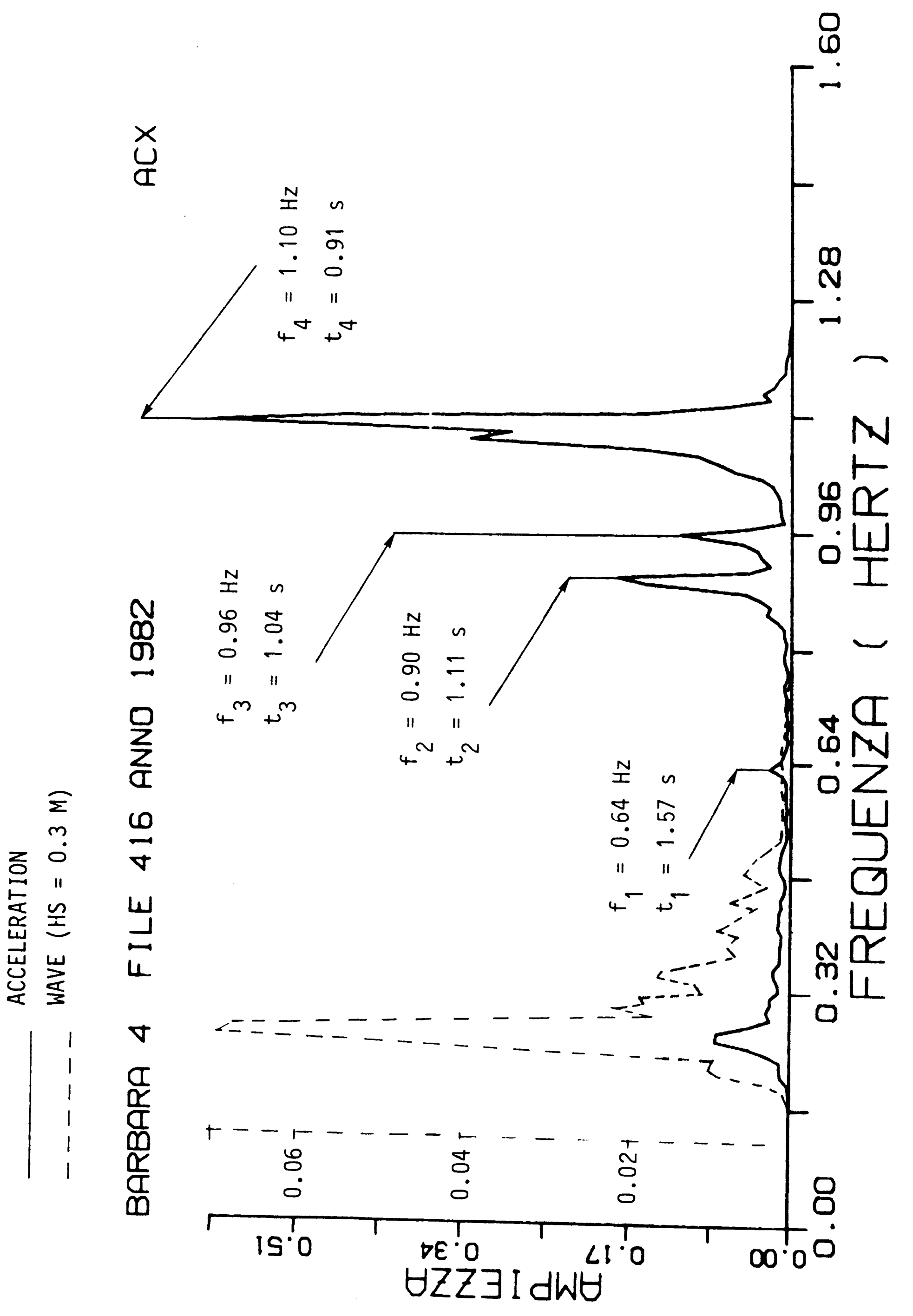
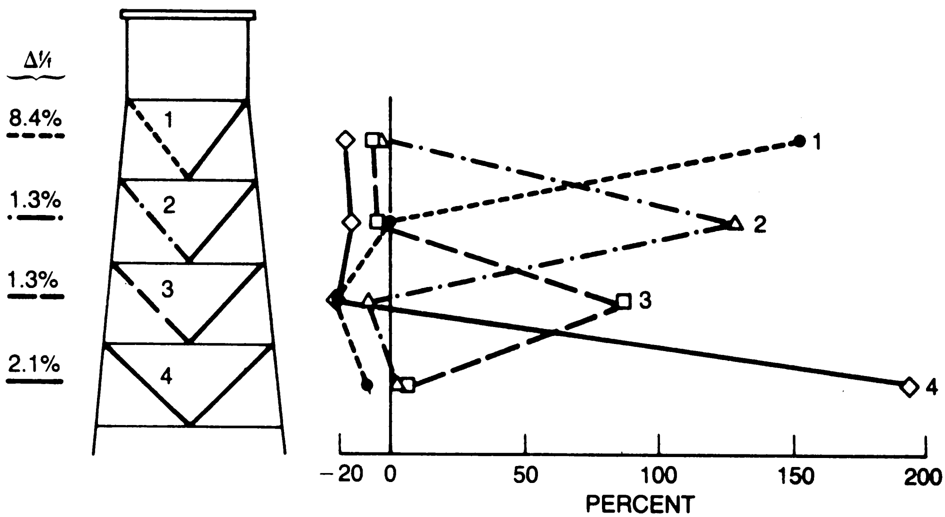


FIGURE 4 : DYNAMIC RESPONSE OF BARBARA PLATFORM



—Indicated flexibility changes in first sway mode

FIGURE 5 : FLEXIBILITY MONITORING PRINCIPLE

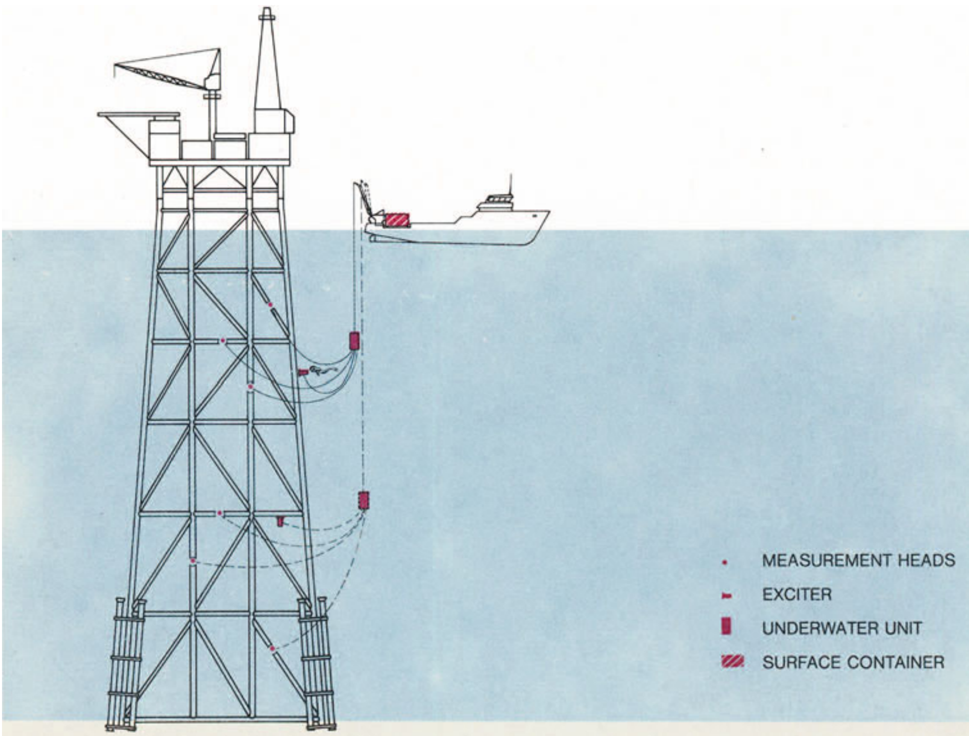
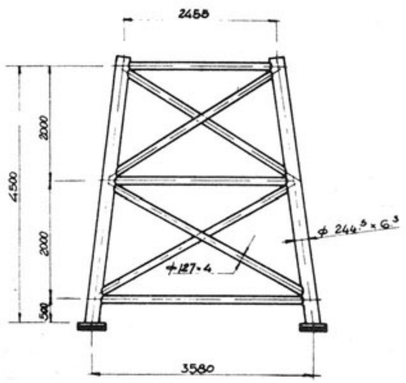
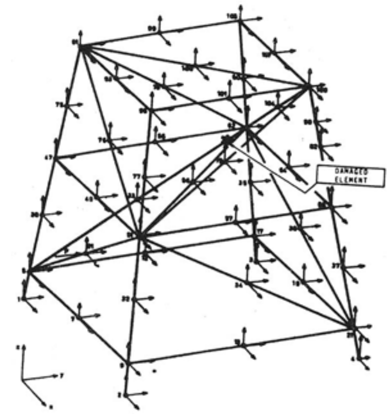


FIGURE 6 : VIBRACHECK SYSTEM

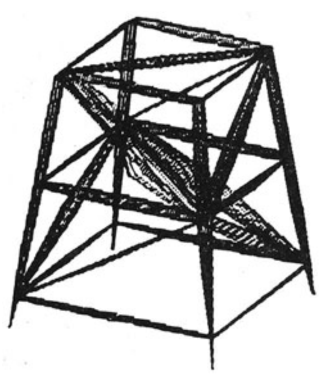


SIDE VIEW OF DMOS TEST STRUCTURE

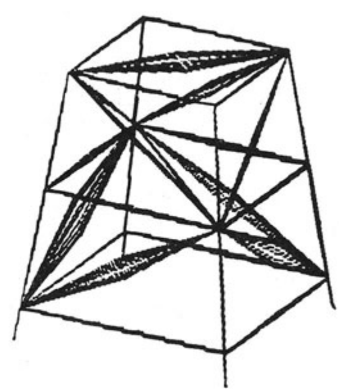


DMOS TEST STRUCTURE - MEASUREMENT POINTS FOR MODE SHAPE ESTIMATION

GLOBAL MODE NO. 1, 29.254 Hz
(UNDAMAGED CONDITION)

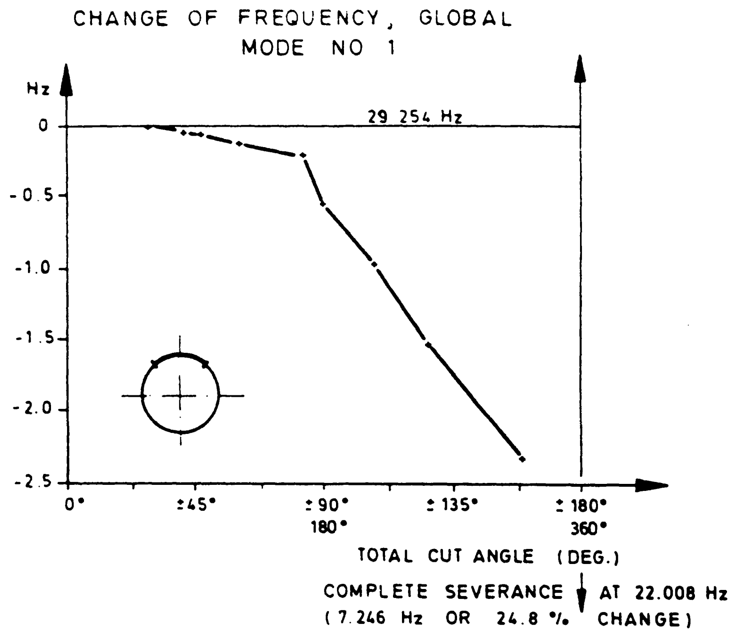


LOCAL MODE NO. 17, 54.295 Hz
(UNDAMAGED CONDITION)



DMOS TEST STRUCTURE MODAL ANALYSIS - EXAMPLES OF MODE SHAPES

FIGURE 7 : JACKET MODEL EXPERIMENTAL MODAL ANALYSIS



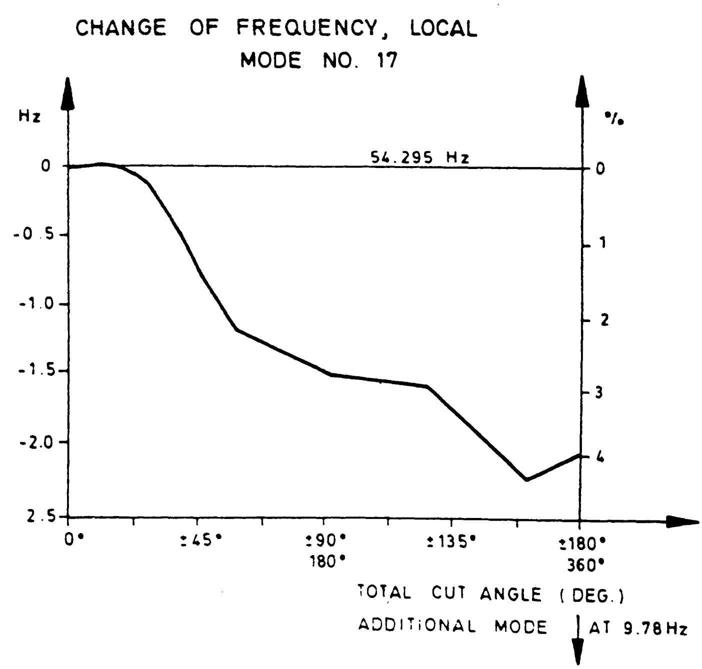


FIGURE 8 : EFFECTS OF CRACK ANGLE ON MODAL FREQUENCY SHIFTS

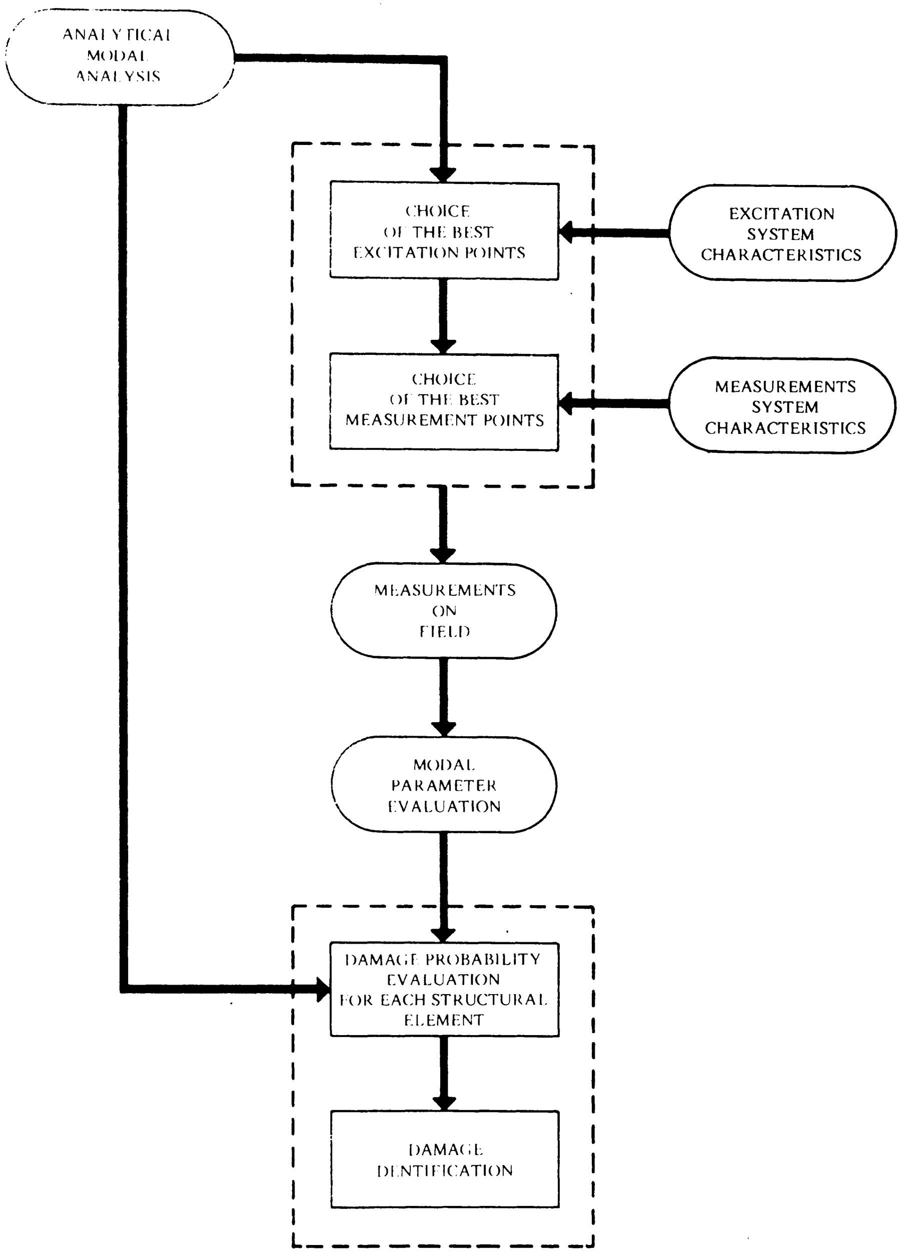


FIGURE 9 : STRUCTURAL DIAGNOSTIC PROCEDURE BASED ON FORCED VIBRATION MONITORING